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Structural Steel Designer's Handbook

Fourth Edition

AISC, AASHTO, AISI, ASTM, AREMA, and ASCE-07 Design Standards

- Unified ASD and LRFD design
- AISC building design specifications
- ASCE-07 standard loadings data
- AASHTO bridge design specifications

Roger L. Brockenbrough · Frederick S. Merritt

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CHAPTER 1

PROPERTIES OF STRUCTURAL STEELS AND EFFECTS OF STEELMAKING AND FABRICATION

Roger L. Brockenbrough, P.E.
President
R. L. Brockenbrough & Associates, Inc.
Pittsburgh, Pennsylvania

This chapter presents and discusses the properties of structural steels that are of importance in design and construction. Designers should be familiar with these properties so that they can select the most economical combination of suitable steels for each application and use the materials efficiently and safely.

In accordance with contemporary practice, the steels described in this chapter are given the names of the corresponding specifications of ASTM, 100 Barr Harbor Dr., West Conshohocken, PA 19428. For example, all steels covered by ASTM A588, "Specification for High-Strength Low-Alloy Structural Steel," are called A588 steel. Most of them can also be furnished to a metric designation such as A588M.

1.1 STRUCTURAL STEEL SHAPES AND PLATES

Steels for structural uses may be classified by chemical composition, tensile properties, and method of manufacture as carbon steels, high-strength low-alloy (HSLA) steels, heat-treated carbon steels, and heat-treated constructional alloy steels. A typical stress-strain curve for a steel in each classification is shown in Fig. 1.1 to illustrate the increasing strength levels provided by the four classifications of steel. The availability of this wide range of specified minimum strengths, as well as other material properties, enables the designer to select an economical material that will perform the required function for each application.

Some of the most widely used steels in each classification are listed in Table 1.1 with their specified strengths in shapes and plates. These steels are weldable, but the welding materials and procedures for each steel must be in accordance with approved methods. Welding information for each of the steels is available in publications of the American Welding Society.

1.1.1 Carbon Steels

A steel may be classified as a carbon steel if (1) the maximum content specified for alloying elements does not exceed the following: manganese—1.65%, silicon—0.60%, copper—0.60%; (2) the specified minimum for copper does not exceed 0.40%; and (3) no minimum content is specified for other elements added to obtain a desired alloying effect.

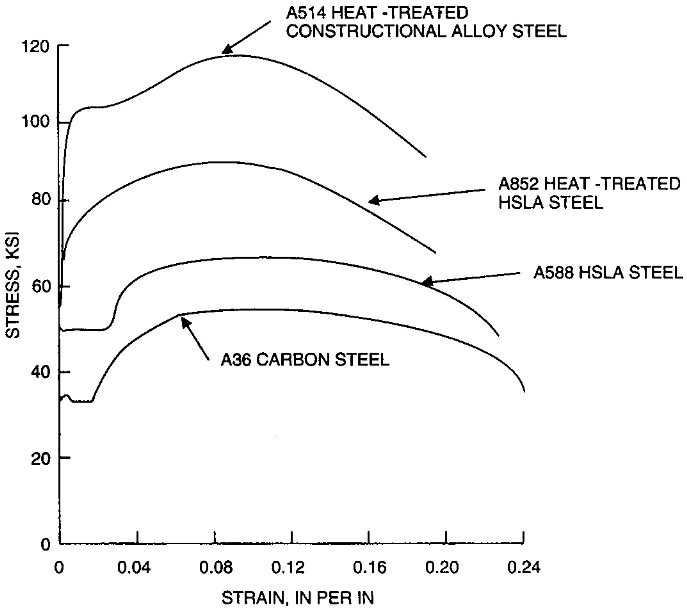


FIGURE 1.1 Typical stress-strain curves for structural steels. (Curves have been modified to reflect minimum specified properties.)

A36 steel has been the principal carbon steel for bridges, buildings, and many other structural uses. This steel provides a minimum yield point of 36 ksi in all structural shapes and in plates up to 8 in thick. In structural steel framing for building construction, A36 steel has been largely replaced by the higher-strength A992 steel (Art. 1.1.2).

A529 is a carbon-manganese steel for general structural purposes, available in shapes and plates of a limited size range. It can be furnished with a specified minimum yield point of either 50 ksi (Grade 50) or 55 ksi (Grade 55).

A573, another carbon steel listed in Table 1.1, is available in three strength grades for plate applications in which improved notch toughness is important.

1.1.2 High-Strength Low-Alloy Steels

Those steels which have specified minimum yield points greater than 40 ksi and achieve that strength in the hot-rolled condition, rather than by heat treatment, are known as HSLA steels. Because these steels offer increased strength at moderate increases in price over carbon steels, they are economical for a variety of applications.

A242 steel is a **weathering steel,** used where resistance to atmospheric corrosion is of primary importance. Steels meeting this specification usually provide a resistance to atmospheric corrosion at least four times that of structural carbon steel. However, when required, steels can be selected to provide a resistance to atmospheric corrosion of five to eight times that of structural carbon steels. A specified minimum yield point of 50 ksi can be furnished in plates up to $^3/_4$ in thick and the lighter structural shapes. It is available with a lower yield point in thicker sections, as indicated in Table 1.1.

A588 is the primary weathering steel for structural work. It provides a 50-ksi yield point in plates up to 4 in thick and in all structural sections; it is available with a lower yield point in thicker plates. Several grades are included in the specification to permit use of various compositions developed by

TABLE 1.1 Specified Minimum Properties for Structural Steel Shapes and Plates*

ASTM designation	Plate thickness range, in	Structural shape flange or leg thickness range, in	Yield stress, ksi[†]	Tensile strength, ksi[†]	Elongation, %	
					In 2 in[‡]	In 8 in
A36	8 maximum	All	36	58–80	23–21	20
	Over 8	All	32	58–80	23	20
A529						
Grade 50	1 maximum	1½ max	50	70–100	21	18
Grade 55	1 maximum	1½ max	55	70–100	20	17
A573						
Grade 58	1½ maximum	¶	32	58–71	24	21
Grade 65	1½ maximum	¶	35	65–77	23	20
Grade 70	1½ maximum	¶	42	70–90	21	18
High-strength low-alloy steels						
A242	¾ maximum	1½ max	50	70	21	18
	Over ¾ to 1½ max	Over 1½ to 2	46	67	21	18
	Over 1½ to 4 max	Over 2	42	63	21	18
A588	4 maximum	All	50	70	21	18
	Over 4 to 5 max	All	46	67	21	—
	Over 5 to 8 max	All	42	63	21	—
A572						
Grade 42	6 maximum	All	42	60	24	20
Grade 50	4 maximum	All	50	65	21	18
Grade 55	2 maximum	All	55	70	20	17
Grade 60	1¼ maximum	2 max	60	75	18	16
Grade 65	1¼ maximum	2 max	65	80	17	15
A992	¶	All	50–65	65	21	18
Heat-treated carbon and HSLA steels						
A633						
Grade A	4 maximum	¶	42	63–83	23	18
Grade C, D	2½ maximum	¶	50	70–90	23	18
	Over 2½ to 4 max	¶	46	65–85	23	18
Grade E	4 maximum	¶	60	80–100	23	18
	Over 4 to 6 max	¶	55	75–95	23	18
A678						
Grade A	1½ maximum	¶	50	70–90	22	—
Grade B	2½ maximum	¶	60	80–100	22	—
Grade C	¾ maximum	¶	75	95–115	19	—
	Over ¾ to 1½ max	¶	70	90–110	19	—
	Over 1½ to 2 max	¶	65	85–105	19	—
Grade D	3 maximum	¶	75	90–110	18	—
A852	4 maximum	¶	70	90–110	19	—
A913	¶	All	50	65	21	18
	¶	All	60	75	18	16
	¶	All	65	80	17	15
	¶	All	70	90	16	14

(*Continued*)

TABLE 1.1 Specified Minimum Properties for Structural Steel Shapes and Plates* (*Continued*)

ASTM designation	Plate thickness range, in	Structural shape flange or leg thickness range, in	Yield stress, ksi[†]	Tensile strength, ksi[†]	Elongation, % In 2 in[‡]	Elongation, % In 8 in
Heat-treated constructional alloy steels						
A514	$2^1/_2$ maximum	¶	100	110–130	18	—
	Over $2^1/_2$ to 6 max	¶	90	100–130	16	—

*The following are approximate values for all the steels:
Modulus of elasticity—29 × 10^3 ksi.
Shear modulus—11 × 10^3 ksi.
Poisson's ratio—0.30.
Yield stress in shear—0.57 times yield stress in tension.
Ultimate strength in shear—$^2/_3$ to $^3/_4$ times tensile strength.
Coefficient of thermal expansion—6.5 × 10^{-6} in per in per °F for temperature range −50 to +150°F.
Density—490 lb/ft^3.

[†] Where two values are shown for yield stress or tensile strength, the first is minimum and the second is maximum.

[‡] The minimum elongation values are modified for some thicknesses in accordance with the specification for the steel. Where two values are shown for the elongation in 2 in, the first is for plates and the second for shapes.

¶ Not applicable.

steel producers to obtain the specified properties. This steel provides about four times the resistance to atmospheric corrosion of structural carbon steels.

These relative corrosion ratings are determined from the slopes of corrosion-time curves and are based on carbon steels not containing copper. (The resistance of carbon steel to atmospheric corrosion can be doubled by specifying a minimum copper content of 0.20%.) Typical corrosion curves for several steels exposed to industrial atmosphere are shown in Fig. 1.2.

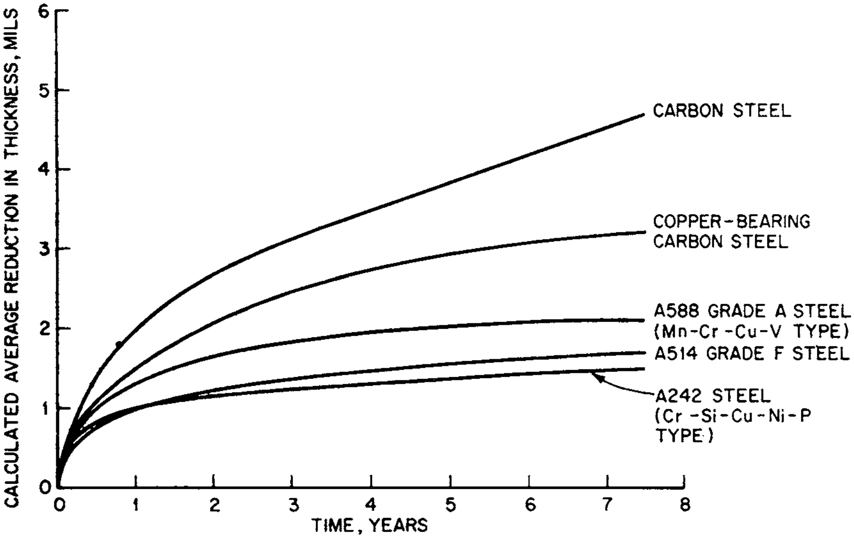


FIGURE 1.2 Corrosion curves for structural steels in an industrial atmosphere. (*From R. L. Brockenbrough and B. G. Johnston*, USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission.*)

For methods of estimating the atmospheric corrosion resistance of low-alloy steels based on their chemical composition, see ASTM Guide G101. The A588 specification requires that the resistance index calculated according to Guide 101 shall be 6.0 or higher.

A588 and A242 steels are called **weathering steels** because, when subjected to alternate wetting and drying in most bold atmospheric exposures, they develop a tight oxide layer that substantially inhibits further corrosion. They are often used bare (unpainted) where the oxide finish that develops is desired for aesthetic reasons or for economy in maintenance. Bridges and exposed building framing are typical examples of such applications. Designers should investigate potential applications thoroughly, however, to determine whether a weathering steel will be suitable. Information on bare-steel applications is available from steel producers.

A572 specifies columbium-vanadium HSLA steels in five grades with minimum yield points of 42 to 65 ksi. Grade 42 in thicknesses up to 6 in and Grade 50 in thicknesses up to 4 in are used for welded bridges. All grades may be used for bolted construction and for welded construction in most applications other than bridges.

A992 steel, introduced in 1998, is now the main specification for rolled wide flange shapes for building framing. All other hot-rolled shapes, such as channels and angles, can be furnished to A992. It provides a minimum yield point of 50 ksi, a maximum yield point of 65 ksi, and a maximum yield to tensile ratio of 0.85. These maximum limits are considered desirable attributes, particularly for seismic design. To enhance weldability, a maximum carbon equivalent is also included, equal to 0.47% or 0.45%, depending on thickness. A supplemental requirement can be specified for an average Charpy V-notch toughness of 40 ft · lb at 70°F.

1.1.3 Heat-Treated Carbon and HSLA Steels

Both carbon and HSLA steels can be heat treated to provide yield points in the range of 50 to 75 ksi. This provides an intermediate strength level between the as-rolled HSLA steels and the heat-treated constructional alloy steels.

A633 is a normalized HSLA plate steel for applications where improved notch toughness is desired. Available in four grades with different chemical compositions, the minimum yield point ranges from 42 to 60 ksi depending on grade and thickness.

A678 includes quenched-and-tempered plate steels (both carbon and HSLA compositions) with excellent notch toughness. It is also available in four grades with different chemical compositions; the minimum yield point ranges from 50 to 75 ksi, depending on grade and thickness.

A852 is a quenched-and-tempered HSLA plate steel of the weathering type. It is intended for welded bridges and buildings and similar applications where weight savings, durability, and good notch toughness are important. It provides a minimum yield point of 70 ksi in thickness up to 4 in. The resistance to atmospheric corrosion is typically four times that of carbon steel.

A913 is a high-strength low-allow steel for structural shapes, produced by the quenching and self-tempering (QST) process. It is intended for the construction of buildings, bridges, and other structures. Four grades provide a minimum yield point of 50 to 70 ksi. Maximum carbon equivalents to enhance weldability are included as follows: Grade 50, 0.38%; Grade 60, 0.40%; Grade 65, 0.43%; and Grade 70, 0.45%. Also, the steel must provide an average Charpy V-notch toughness of 40 ft · lb at 70°F.

1.1.4 Heat-Treated Constructional Alloy Steels

Steels that contain alloying elements in excess of the limits for carbon steel and are heat treated to obtain a combination of high strength and toughness are termed **constructional alloy steels.** Having a yield strength of 100 ksi, these are the strongest steels in general structural use.

A514 includes several grades of quenched and tempered steels, to permit use of various compositions developed by producers to obtain the specified strengths. Maximum thickness ranges from 1¼ to 6 in depending on the grade. Minimum yield strength for plate thicknesses over 2½ in is 90 ksi.

Steels furnished to this specification can provide a resistance to atmospheric corrosion up to four times that of structural carbon steel depending on the grade.

Constructional alloy steels are also frequently selected because of their ability to resist abrasion. For many types of abrasion, this resistance is related to hardness or tensile strength. Therefore, constructional alloy steels may have nearly twice the resistance to abrasion provided by carbon steel. Also available are numerous grades that have been heat treated to increase the hardness even more.

TABLE 1.2 Charpy V-Notch Toughness for A709 Bridge Steels[a]

				Test temperature, °F		
Grade	Maximum thickness, in, inclusive	Joining/ fastening method	Minimum average energy, ft · lb	Zone 1	Zone 2	Zone 3
			Non-fracture-critical members			
36T	4	Mech./weld.	15	70	40	10
50T,[b]	2	Mech./weld.	15			
50WT[b], 50ST						
	2 to 4	Mechanical	15	70	40	10
	2 to 4	Welded	20			
70WT[c]	2½	Mech./weld.	20			
	2½ to 4	Mechanical	20	50	20	−10
	2½ to 4	Welded	25			
100T, 100WT	2½	Mech./weld.	25			
	2½ to 4	Mechanical	25	30	0	−30
	2½ to 4	Welded	35			
HPS50WT	4	Mech./weld.	20	10	10	10
HPS50WT	4	Mech./weld.	25	−10	−10	−10
			Fracture-critical members			
36F	4	Mech./weld.[d]	25	70	40	10
50F,[b] 50WF[b]	2	Mech./weld.[d]	25	70	40	10
	2 to 4	Mechanical[d]	25	70	40	10
	2 to 4	Welded[e]	30	70	40	10
70WF[c]	2½	Mech./weld.[e]	30	50	20	−10
	2½ to 4	Mechanical[e]	30	50	20	−10
	2½ to 4	Welded[f]	35	50	20	−10
100F, 100WF	2½	Mech./weld.[f]	35	30	0	−30
	2½ to 4	Mechanical[f]	35	30	0	−30
	2½ to 4	Welded[g]	45	30	0	NA
HPS50WF	4	Mech./weld.	30	10	10	10
HPS50WF	4	Mech./weld.	35	−10	−10	−10

[a] Minimum **service** temperatures:
Zone 1, 0°F; Zone 2, below 0 to −30°F; Zone 3, below −30 to −60°F.

[b] If yield strength exceeds 65 ksi, reduce test temperature by 15°F for each 10 ksi above 65 ksi.

[c] If yield strength exceeds 85 ksi, reduce test temperature by 15°F for each 10 ksi above 85 ksi.

[d] Minimum test value energy is 20 ft-lb.

[e] Minimum test value energy is 24 ft-lb.

[f] Minimum test value energy is 28 ft-lb.

[g] Minimum test value energy is 36 ft-lb.

1.1.5 Bridge Steels

Steels for application in bridges are covered by A709, which includes steel in several of the categories mentioned above. Under this specification, grades 36, 50, 70, and 100 are steels with yield strengths of 36, 50, 70, and 100 ksi, respectively. Similar AASHTO grades are designated M270.

The grade designation is followed by the letter W, indicating whether ordinary or high atmospheric corrosion resistance is required. An additional letter, T or F, indicates that Charpy V-notch impact tests must be conducted on the steel. The T designation indicates that the material is to be used in a non-fracture-critical application as defined by AASHTO; the F indicates use in a fracture-critical application. There is also a Grade 50S, where the S indicates the steel must be killed.

A trailing numeral, 1, 2, or 3, indicates the testing zone, which relates to the lowest ambient temperature expected at the bridge site. (See Table 1.2.) As indicated by the first footnote in the table, the service temperature for each zone is considerably less than the Charpy V-notch impact-test temperature. This accounts for the fact that the dynamic loading rate in the impact test is more severe than that to which the structure is subjected. The toughness requirements depend on fracture criticality, grade, thickness, and method of connection.

High-performance steels (HPS) are the newest additions to the family of bridge steels. They are being used increasingly to improve reliability and reduce cost, with approximately 200 bridges in service in 2005. The initial grade, HPS70W, with a specified minimum yield stress of stress of 70 ksi, has been used most. HPS50W, with a specified minimum yield stress of 50 ksi, has also become popular. HPS100W, with a specified minimum yield stress of stress of 100 ksi, is available to reduce thickness where members are highly loaded.

1.2 STEEL-QUALITY DESIGNATIONS

Steel plates, shapes, sheetpiling, and bars for structural uses—such as the load-carrying members in buildings, bridges, ships, and other structures—are usually ordered to the requirements of ASTM A6 and are referred to as **structural-quality steels.** (A6 does not indicate a specific steel.) This specification contains general requirements for delivery related to chemical analysis, permissible variations in dimensions and weight, permissible imperfections, conditioning, marking and tension and bend tests of a large group of structural steels. (Specific requirements for the chemical composition and tensile properties of these steels are included in the specifications discussed in Art. 1.1.) All the steels included in Table 1.1 are structural-quality steels.

Steel plates for pressure vessels are usually furnished to the general requirements of ASTM A20 and are referred to as **pressure-vessel-quality steels.** Generally, a greater number of mechanical-property tests and additional processing are required for pressure-vessel-quality steel.

1.3 STEEL SHEET AND STRIP FOR STRUCTURAL APPLICATIONS

Steel sheet and strip are used for many structural applications, particularly for cold-formed structural members for residential and light commercial building construction (Chap. 9). The facade of many high-rise structures is supported by cold-formed sheet steel systems and interior partitions are often built with steel C-sections. The stressed skin of transportation equipment is another application of such material. Tensile properties of several sheet steels are presented in Table 1.3. Many of them are available in several strength levels, with a specified minimum yield point from 25 to 80 ksi. Some grades may not be suitable for all applications, depending on the ratio of tensile strength to yield point and other considerations (Chap. 9).

ASTM A606 covers high-strength low-alloy, hot- and cold-rolled steel sheet and strip with enhanced corrosion resistance. This material, available in cut lengths or coils, is intended for structural and other uses where savings in weight and improved durability are important. It may be ordered as Type 2 or Type 4, with atmospheric corrosion resistance approximately two or four times,

TABLE 1.3 Specified Minimum Mechanical Properties for Steel Sheet and Strip for Structural Applications

ASTM designation	Type of product	Grade	Yield point, ksi	Tensile strength, ksi	Elongation in 2 in, %[a]	F_u/F_y
A606	Hot rolled (as rolled)	—	50	70	22	1.40
	Hot rolled (annealed or normalized)	—	45	65	22	1.44
	Cold rolled	—	45	65	22	1.44
A653[b]	Galvanized or galvannealed	SS 33	33	45	20	1.36
		SS 37	37	52	18	1.41
		SS 40	40	55	16	1.38
		SS 50, Cl 1	50	65	12	1.30
		SS 50, Cl 3	50	70	12	1.40
		SS 80	80	82	—	1.03
		HSLAS 40	40	50	22–24	1.25
		HSLAS 50	50	60	20–22	1.20
		HSLAS 60	60	70	16–18	1.17
		HSLAS 70	70	80	12–14	1.14
		HSLAS 80	80	90	10–12	1.12
A792	55% aluminum-zinc alloy coated	SS 33	33	45	20	1.36
		SS 37	37	52	18	1.41
		SS 40	40	55	16	1.38
		SS 50, Cl 1	50	65	12	1.30
		SS 50, Cl 4	50	60	12	1.20
		SS 80	80	82	—	1.03
A1003	Sheet for framing members	ST33H	33	—	10	1.08[c]
		ST37H	37	—	10	1.08[c]
		ST40H	40	—	10	1.08[c]
		ST50H	50	—	10	1.08[c]
		ST33L	33	—	3	—
		ST37L	37	—	3	—
		ST40L	40	—	3	—
		ST50L	50	—	3	—
A1008	Cold rolled	SS 25	25	42	26	1.68
		SS 30	30	45	24	1.50
		SS 33, Type 1	33	48	22	1.45
		SS 33, Type 2	33	48	22	1.45
		SS 40, Type 1	40	52	20	1.30
		SS 40, Type 2	40	52	20	1.30
		SS 80	80	82	—	1.03
		HSLAS 45, Cl 1	45	60	22	1.33
		HSLAS 50, Cl 1	50	65	20	1.30
		HSLAS 55, Cl 1	55	70	18	1.27
		HSLAS 60, Cl 1	60	75	16	1.25
		HSLAS 65, Cl 1	65	80	15	1.23
		HSLAS 70, Cl 1	70	85	14	1.21
		HSLAS 45, Cl 2	45	55	22	1.22
		HSLAS 50, Cl 2	50	60	20	1.20
		HSLAS 55, Cl 2	55	65	18	1.18
		HSLAS 60, Cl 2	60	70	16	1.17
		HSLAS 65, Cl 2	65	75	15	1.15
		HSLAS 70, Cl 2	70	80	14	1.14
		HSLAS-F 50	50	60	22	1.20

(Continued)

TABLE 1.3 Specified Minimum Mechanical Properties for Steel Sheet and Strip for Structural Applications (*Continued*)

ASTM designation	Type of product	Grade	Yield point, ksi	Tensile strength, ksi	Elongation in 2 in, %[a]	F_u/F_y
A1008 (*cont.*)	Cold rolled	HSLAS-F 60	60	70	18	1.17
		HSLAS-F 70	70	80	16	1.14
		HSLAS-F 80	80	90	14	1.12
A1011	Sheet	SS 30	30	49	25–21	1.63
		SS 33	33	52	23–18	1.62
		SS 36, Type 1	36	53	22–17	1.47
		SS 36, Type 2	36	58/80	21–16	1.61
		SS 40	40	55	21–15	1.38
		SS 45	45	60	19–13	1.33
		SS 50	50	65	17–11	1.30
		SS 55	55	70	15–9	1.27
		SS 60	60	75	14–13	1.25
		SS 70	70	85	13–12	1.21
		SS 80	80	95	12–11	1.19
		HSLAS 45, Cl 1	45	60	25–23	1.33
		HSLAS 50, Cl 1	50	65	22–20	1.30
		HSLAS 55, Cl 1	55	70	20–18	1.27
		HSLAS 60, Cl 1	60	75	18–16	1.25
		HSLAS 65, Cl 1	65	80	16–14	1.23
		HSLAS 70, Cl 1	70	85	14–12	1.21
		HSLAS 45, Cl 2	45	55	25–23	1.22
		HSLAS 50, Cl 2	50	60	22–20	1.20
		HSLAS 55, Cl 2	55	65	20–18	1.18
		HSLAS 60, Cl 2	60	70	18–16	1.17
		HSLAS 65, Cl 2	65	75	16–14	1.15
		HSLAS 70, Cl 2	70	80	14–12	1.14
		HSLAS-F 50	50	60	24–22	1.20
		HSLAS-F 60	60	70	22–20	1.17
		HSLAS-F 70	70	80	20–18	1.14
		HSLAS-F 80	80	90	18–16	1.12

[a] Modified for some thicknesses in accordance with the specification. For A653, where two values are given, the first is for Type A and the second for Type B. For A1011, specified value varies with thickness range.

[b] Also available as A875 with zinc-5% aluminum alloy coating.

[c] For ASTM A1003, this is a specified minimum ratio.

respectively, that of plain carbon steel. Where properly exposed to the atmosphere, Type 4 can be used in the bare (unpainted) condition for many applications.

A653 covers steel sheet, zinc coated (galvanized) or zinc-iron alloy coated (galvannealed) by the hot-dip process, in coils and cut lengths. Included are several grades based on yield strength in both structural steel (SS) and high strength low alloy (HSLA). HSLA sheets are available as Type A, where improved formability is required, and Type B, where even better formability is required.

A792 covers 55% aluminum-zinc alloy coated steel sheet in coils and cut lengths, coated by the hot-dip process. The aluminum-zinc alloy composition is nominally 55% aluminum, 1.6% silicon, and the balance zinc. The product is intended for applications requiring corrosion resistance or heat resistance. Aluminum-zinc alloy coated sheet is available in various designations, including commercial steel, forming steel, drawing steel, and high-temperature steel, as well as structural steel (SS).

A875 covers steel sheet, in coils and cut lengths, metallic coated by the hot-dip process, with zinc-5% aluminum alloy coating. The Zn-5Al alloy coating also contains small amounts of elements other than zinc and aluminum, which are intended to improve processing and other characteristics. The material is intended for applications requiring corrosion resistance, formability, and paintability. It is produced in a number of designations, types, grades, and classes for differing application requirements. The coating is produced as two types—zinc-5% aluminum-mischmetal alloy (Type I) and zinc-5% aluminum-0.1% magnesium alloy (Type II)—in two coating structures (classes), and in several coating weight designations. Mechanical properties are generally similar to those of A653.

A1003 covers coated steel sheet used in the manufacture of cold-formed framing members, such as, but not limited to, studs, joists, purlins, girts, and track. The sheet steel used for cold-formed framing members includes metallic coated, painted metallic coated, and painted nonmetallic coated. The grade designations use the following suffix indicators: *H*, high ductility; *L*, low ductility; and *NS*, nonstructural. H and L are associated with structural or load-bearing applications, and NS with nonstructural or non-load-bearing applications.

A1008 covers cold-rolled structural steel (SS), high-strength low-alloy steel (HSLAS), and high-strength low-alloy steel with improved formability (HSLAS-F), in coils and cut lengths. The steel is fully deoxidized, made to fine-grain practice, and includes microalloying elements such as columbium, vanadium, and zirconium. The steel may be treated to achieve inclusion control. Cold-rolled steel sheet is supplied for either exposed or unexposed applications.

A1011 covers hot-rolled sheet and strip, in coils and cut lengths. The product is produced in a number of designations, including SS, HSLAS, and HSLAS-F. The steel is fully deoxidized, made to fine-grain practice, and includes microalloying elements such as columbium, vanadium, and zirconium. The steel may be treated to achieve inclusion control.

1.4 TUBING FOR STRUCTURAL APPLICATIONS

Structural tubing is being used more frequently in modern construction. Commonly referred to as hollow structural sections (HSS), it is often preferred to other steel members when resistance to torsion is required and when a smooth, closed section is aesthetically desirable. In addition, structural tubing may be the economical choice for compression members subjected to moderate to light loads. Square and rectangular tubing is manufactured either by cold or hot forming welded or seamless round tubing in a continuous process. A500 cold-formed carbon-steel tubing (Table 1.4) is produced in four strength grades in each of two product forms, shaped (square or rectangular) or round. A minimum

TABLE 1.4 Specified Minimum Mechanical Properties of Structural Tubing

ASTM designation	Product form	Yield point, ksi	Tensile strength, ksi	Elongation in 2 in, %
A500	Shaped			
Grade A		39	45	25
Grade B		46	58	23
Grade C		50	62	21
Grade D		36	58	23
A500	Round			
Grade A		33	45	25
Grade B		42	58	23
Grade C		46	62	21
Grade D		36	58	23
A501	Round or shaped	36	58	23
A618	Round or shaped			
Grades Ia, lb, II				
Walls ≤ $^{3}/_{4}$ in		50	70	22
Walls > $^{3}/_{4}$ to $1^{1}/_{2}$ in		46	67	22
Grade III		50	65	20
A847	Round or shaped	50	70	19

yield point of up to 50 ksi is available for shaped tubes and up to 46 ksi for round tubes. A500 Grade B and Grade C are commonly specified for building construction applications and are available from producers and steel service centers. A500 tubing may not be suitable for dynamically loaded elements in welded structures where low-temperature notch-toughness properties are important.

A 501 tubing is a hot-formed carbon-steel product available as hot rolled or hot dip galvanized. It provides a yield point equal to that of A36 steel in tubing having a wall thickness of 1 in or less.

A618 tubing is a hot-formed HSLA product that provides a minimum yield point of up to 50 ksi. The three grades all have enhanced resistance to atmospheric corrosion. Grades Ia and Ib can be used in the bare condition for many applications when properly exposed to the atmosphere.

A847 tubing covers cold-formed HSLA tubing and provides a minimum yield point of 50 ksi. It also offers enhanced resistance to atmospheric corrosion and, when properly exposed, can be used in the bare condition for many applications.

1.5 STEEL CABLE FOR STRUCTURAL APPLICATIONS

Steel cables have been used for many years in bridge construction and are occasionally used in building construction for the support of roofs and floors. The types of cables used for these applications are referred to as **bridge strand** or **bridge rope.** In this use, **bridge** is a generic term that denotes a specific type of high-quality strand or rope.

A **strand** is an arrangement of wires laid helically about a center wire to produce a symmetrical section. A **rope** is a group of strands laid helically around a core composed of either a strand or another wire rope. The term **cable** is often used indiscriminately in referring to wires, strands, or ropes. Strand may be specified under ASTM A586, wire rope, under A603.

During manufacture, the individual wires in bridge strand and rope are generally galvanized to provide resistance to corrosion. Also, the finished cable is prestretched. In this process, the strand or rope is subjected to a predetermined load of not more than 55% of the breaking strength for a sufficient length of time to remove the "structural stretch" caused primarily by radial and axial adjustment of the wires or strands to the load. Thus, under normal design loadings, the elongation that occurs is essentially elastic and may be calculated from the elastic-modulus values given in Table 1.5.

Strands and ropes are manufactured from cold-drawn wire and do not have a definite yield point. Therefore, a working load or design load is determined by dividing the specified minimum breaking strength for a specific size by a suitable safety factor. The breaking strengths for selected sizes of bridge strand and rope are listed in Table 1.5.

TABLE 1.5 Mechanical Properties of Steel Cables

Minimum breaking strength, kips,* of selected cable sizes			Minimum modulus of elasticity, ksi,* for indicated diameter range	
Nominal diameter, in	Zinc-coated strand	Zinc-coated rope	Nominal diameter range, in	Minimum modulus, ksi
$\frac{1}{2}$	30	23	Prestretched zinc-coated strand	
$\frac{3}{4}$	68	52		
1	122	91.4	$\frac{1}{2}$ to $2\frac{9}{16}$	24,000
$1\frac{1}{2}$	276	208	$2\frac{5}{8}$ and over	23,000
2	490	372	Prestretched zinc-coated rope	
3	1076	824		
4	1850	1460	$\frac{3}{8}$ to 4	20,000

*Values are for cables with Class A zinc coating on all wires. Class B or C can be specified where additional corrosion protection is required.

1.6 TENSILE PROPERTIES

The tensile properties of steel are generally determined from tension tests on small specimens or coupons in accordance with standard ASTM procedures. The behavior of steels in these tests is closely related to the behavior of structural-steel members under static loads. Because, for structural steels, the yield points and moduli of elasticity determined in tension and compression are nearly the same, compression tests are seldom necessary.

Typical tensile stress-strain curves for structural steels are shown in Fig. 1.1. The initial portion of these curves is shown at a magnified scale in Fig. 1.3. Both sets of curves may be referred to for the following discussion.

Strain Ranges. When a steel specimen is subjected to load, an initial **elastic range** is observed in which there is no permanent deformation. Thus, if the load is removed, the specimen returns to its original dimensions. The ratio of stress to strain within the elastic range is the **modulus of elasticity,** or **Young's modulus** E. Since this modulus is consistently about 29×10^3 ksi for all the structural steels, its value is not usually determined in tension tests, except in special instances.

The strains beyond the elastic range in the tension test are termed the **inelastic range.** For as-rolled and high-strength low-alloy (HSLA) steels, this range has two parts. First observed is a **plastic range,** in which strain increases with no appreciable increase in stress. This is followed by a **strain-hardening range,** in which strain increase is accompanied by a significant increase in stress. The curves for heat-treated steels, however, do not generally exhibit a distinct plastic range or a large amount of strain hardening.

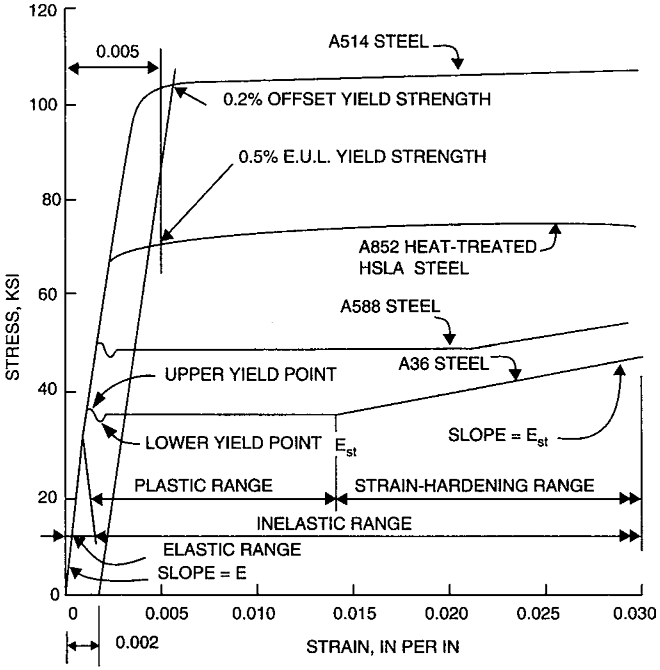


FIGURE 1.3 Partial stress-strain curves for structural steels strained through the plastic region into the strain-hardening range. (*From R. L. Brockenbrough and B. G. Johnston*, USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission*.)

The strain at which strain hardening begins (ϵ_{st}) and the rate at which stress increases with strain in the strain-hardening range (the strain-hardening modulus E_{st}) have been determined for carbon and HSLA steels. The average value of E_{st} is 600 ksi, and the length of the yield plateau is 5 to 15 times the yield strain. (T. V. Galambos, "Properties of Steel for Use in LRFD," *Journal of the Structural Division, American Society of Civil Engineers*, Vol. 104, No. ST9, 1978.)

Yield Point, Yield Strength, and Tensile Strength. As illustrated in Fig. 1.3, carbon and HSLA steels usually show an upper and lower yield point. The upper yield point is the value usually recorded in tension tests and thus is simply termed the **yield point.**

The heat-treated steels in Fig. 1.3, however, do not show a definite yield point in a tension test. For these steels it is necessary to define a **yield strength,** the stress corresponding to a specified deviation from perfectly elastic behavior. As illustrated in the figure, yield strength is usually specified in either of two ways: For steels with a specified value not exceeding 80 ksi, yield strength is considered as the stress at which the test specimen reaches a 0.5% extension under load (0.5% EUL) and may still be referred to as the yield point. For higher-strength steels, the yield strength is the stress at which the specimen reaches a strain 0.2% greater than that for perfectly elastic behavior.

Since the amount of inelastic strain that occurs before the yield strength is reached is quite small, yield strength has essentially the same significance in design as yield point. These two terms are sometimes referred to collectively as **yield stress.**

The maximum stress reached in a tension test is the tensile strength of the steel. After this stress is reached, increasing strains are accompanied by decreasing stresses. Fracture eventually occurs.

Proportional Limit. The proportional limit is the stress corresponding to the first visible departure from linear-elastic behavior. This value is determined graphically from the stress-strain curve. Since the departure from elastic action is gradual, the proportional limit depends greatly on individual judgment and on the accuracy and sensitivity of the strain-measuring devices used. The proportional limit has little practical significance and is not usually recorded in a tension test.

Ductility. Ductility is an important property of structural steels. It allows redistribution of stresses in continuous members and at points of high local stresses, such as those at holes or other discontinuities.

In a tension test, ductility is measured by percent elongation over a given gage length or percent reduction of cross-sectional area. The percent elongation is determined by fitting the specimen together after fracture, noting the change in gage length and dividing the increase by the original gage length. Similarly, the percent reduction of area is determined from cross-sectional measurements made on the specimen before and after testing.

Both types of ductility measurements are an index of the ability of a material to deform in the inelastic range. There is, however, no generally accepted criterion of minimum ductility for various structures.

Poisson's Ratio. The ratio of transverse to longitudinal strain under load is known as **Poisson's ratio ν.** This ratio is about the same for all structural steels—0.30 in the elastic range and 0.50 in the plastic range.

True-Stress–True-Strain Curves. In the stress-strain curves shown previously, stress values were based on original cross-sectional area, and the strains were based on the original gage length. Such curves are sometimes referred to as **engineering-type stress-strain curves.** However, since the original dimensions change significantly after the initiation of yielding, curves based on instantaneous values of area and gage length are often thought to be of more fundamental significance. Such curves are known as **true-stress–true-strain curves.** A typical curve of this type is shown in Fig. 1.4.

The curve shows that when the decreased area is considered, the true stress actually increases with increase in strain until fracture occurs instead of decreasing after the tensile strength is reached, as in the engineering stress-strain curve. Also, the value of true strain at fracture is much greater than the engineering strain at fracture (though until yielding begins, true strain is less than engineering strain).

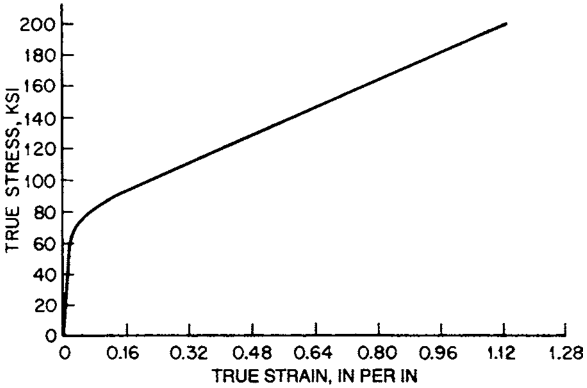


FIGURE 1.4 Curve shows the relationship between true stress and true strain for 50-ksi-yield-point HSLA steel.

1.7 PROPERTIES IN SHEAR

The ratio of shear stress to shear strain during initial elastic behavior is the **shear modulus** G. According to the theory of elasticity, this quantity is related to the modulus of elasticity E and Poisson's ratio v by

$$G = \frac{E}{2(1+v)} \tag{1.1}$$

Thus a minimum value of G for structural steels is about 11×10^3 ksi. The yield stress in shear is about 0.57 times the yield stress in tension. The shear strength, or shear stress at failure in pure shear, varies from two-thirds to three-fourths of the tensile strength for the various steels. Because of the generally consistent relationship of shear properties to tensile properties for the structural steels, and because of the difficulty of making accurate shear tests, shear tests are seldom performed.

1.8 HARDNESS TESTS

In the Brinell hardness test, a small spherical ball of specified size is forced into a flat steel specimen by a known static load. The diameter of the indentation made in the specimen can be measured by a micrometer microscope. The **Brinell hardness number** may then be calculated as the ratio of the applied load, in kilograms, to the surface area of the indentation, in square millimeters. In practice, the hardness number can be read directly from tables for given indentation measurements.

The Rockwell hardness test is similar in principle to the Brinell test. A spheroconical diamond penetrator is sometimes used to form the indentation and the depth of the indentation is measured with a built-in, differential depth-measurement device. This measurement, which can be read directly from a dial on the testing device, becomes the **Rockwell hardness number.**

In either test, the hardness number depends on the load and type of penetrator used; therefore, these should be indicated when listing a hardness number. Other hardness tests, such as the Vickers tests, are also sometimes used. Tables are available that give approximate relationships between the different hardness numbers determined for a specific material.

Hardness numbers are considered to be related to the tensile strength of steel. Although there is no absolute criterion to convert from hardness numbers to tensile strength, charts are available that give approximate conversions (see ASTM A370). Because of its simplicity, the hardness test is widely used in manufacturing operations to estimate tensile strength and to check the uniformity of tensile strength in various products.

1.9 EFFECT OF COLD WORK ON TENSILE PROPERTIES

In the fabrication of structures, steel plates and shapes are often formed at room temperatures into desired shapes. These cold-forming operations cause inelastic deformation, since the steel retains its formed shape. To illustrate the general effects of such deformation on strength and ductility, the elemental behavior of a carbon-steel tension specimen subjected to plastic deformation and subsequent tensile reloadings will be discussed. However, the behavior of actual cold-formed structural members is more complex.

As illustrated in Fig. 1.5, if a steel specimen is unloaded after being stressed into either the plastic or strain-hardening range, the unloading curve follows a path parallel to the elastic portion of the stress-strain curve. Thus a residual strain, or **permanent set,** remains after the load is removed. If the specimen is promptly reloaded, it will follow the unloading curve to the stress-strain curve of the virgin (unstrained) material.

If the amount of plastic deformation is less than that required for the onset of strain hardening, the yield stress of the plastically deformed steel is about the same as that of the virgin material. However, if the amount of plastic deformation is sufficient to cause strain hardening, the yield stress of the steel is larger. In either instance, the tensile strength remains the same, but the ductility, measured from the point of reloading, is less. As indicated in Fig. 1.5, the decrease in ductility is nearly equal to the amount of inelastic prestrain.

A steel specimen that has been strained into the strain-hardening range, unloaded, and allowed to age for several days at room temperature (or for a much shorter time at a moderately elevated temperature)

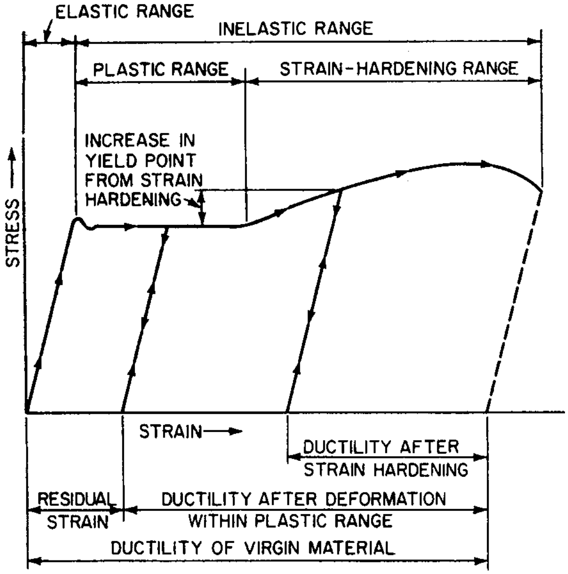


FIGURE 1.5 Stress-strain diagram (not to scale) illustrating the effects of strain-hardening steel. (*From R. L. Brockenbrough and B. G. Johnston*, USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission.*)

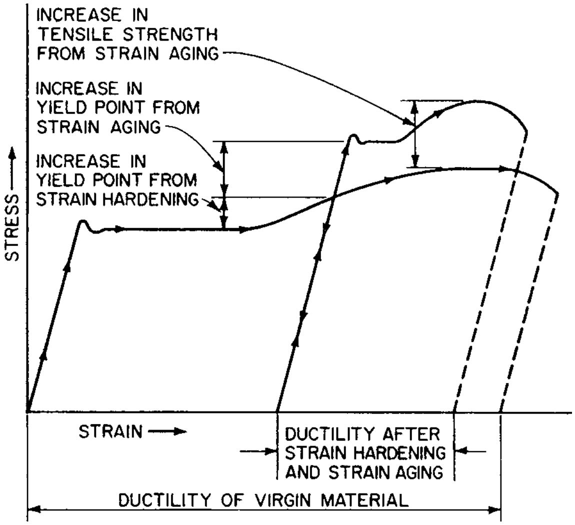


FIGURE 1.6 Effects of strain aging are shown by stress-strain diagram (not to scale). (*From R. L. Brockenbrough and B. G. Johnston*, USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission*.)

usually shows the behavior indicated in Fig. 1.6 during reloading. This phenomenon, known as **strain aging,** has the effect of increasing yield and tensile strength while decreasing ductility.

Most of the effects of cold work on the strength and ductility of structural steels can be eliminated by thermal treatment, such as stress relieving, normalizing, or annealing. However, such treatment is not often necessary.

(G. E. Dieter, Jr., *Mechanical Metallurgy*, 3d ed., McGraw-Hill, New York.)

1.10 EFFECT OF STRAIN RATE ON TENSILE PROPERTIES

Tensile properties of structural steels are usually determined at relatively slow strain rates to obtain information appropriate for designing structures subjected to static loads. In the design of structures subjected to high loading rates, such as those caused by impact loads, however, it may be necessary to consider the variation in tensile properties with strain rate.

Figure 1.7 shows the results of rapid tension tests conducted on a carbon steel, two HSLA steels, and a constructional alloy steel. The tests were conducted at three strain rates and at three temperatures to evaluate the interrelated effect of these variables on the strength of the steels. The values shown for the slowest and the intermediate strain rates on the room-temperature curves reflect the usual room-temperature yield stress and tensile strength, respectively. (In determination of yield stress, ASTM E8 allows a maximum strain rate of $^1/_{16}$ in per in per mm, or 1.04×10^{-3} in per in per sec. In determination of tensile strength, E8 allows a maximum strain rate of 0.5 in per in per mm, or 8.33×10^{-3} in per in per sec.)

The curves in Fig. 1.7*a* and *b* show that the tensile strength and 0.2% offset yield strength of all the steels increase as the strain rate increases at −50°F and at room temperature. The greater increase in tensile strength is about 15%, for A514 steel, whereas the greatest increase in yield strength is about 48%, for A515 carbon steel. However, Fig. 1.7*c* shows that at 600°F, increasing the strain rate

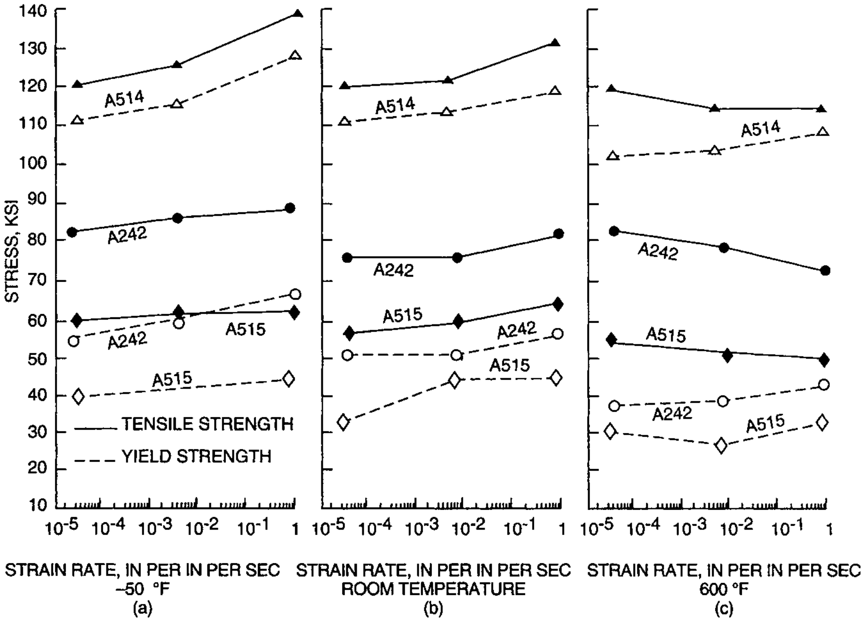


FIGURE 1.7 Effects of strain rate on yield and tensile strengths of structural steels at low, normal, and elevated temperatures. (*From R. L. Brockenbrough and B. G. Johnston*, USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission.*)

has a relatively small influence on the yield strength. But a faster strain rate causes a slight decrease in the tensile strength of most of the steels.

Ductility of structural steels, as measured by elongation or reduction of area, tends to decrease with strain rate. Other tests have shown that modulus of elasticity and Poisson's ratio do not vary significantly with strain rate.

1.11 EFFECT OF ELEVATED TEMPERATURES ON TENSILE PROPERTIES

The behavior of structural steels subjected to short-time loadings at elevated temperatures is usually determined from short-time tension tests. In general, the stress-strain curve becomes more rounded and the yield strength and tensile strength are reduced as temperatures are increased. The ratios of the elevated-temperature value to room-temperature value of yield and tensile strengths typical for structural steels are shown in Fig. 1.8*a*.

Modulus of elasticity decreases with increasing temperature, as shown in Fig. 1.8*b*. The relationship shown is typical for structural steels. The variation in shear modulus with temperature is similar to that shown for the modulus of elasticity. But Poisson's ratio does not vary over this temperature range.

Ductility of structural steels, as indicated by elongation and reduction-of-area values, decreases with increasing temperature until a minimum value is reached. Thereafter, ductility increases to a value much greater than that at room temperature. The exact effect depends on the type and thickness of steel. The initial decrease in ductility is caused by strain aging and is most pronounced in the temperature range of 300 to 700°F. Strain aging also causes an increase in tensile strength in this temperature range shown for some steels.

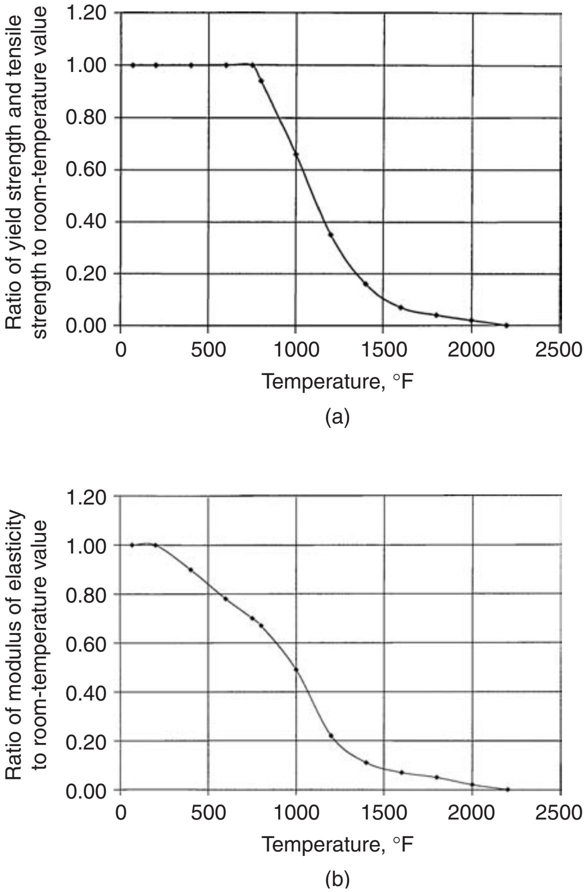


FIGURE 1.8 Effect of temperature on (*a*) yield strength and tensile strength and (*b*) modulus of elasticity of structural steels. (*Adapted from data in AISC "Specification for Structural Steel Buildings," 2005.*)

Under long-time loadings at elevated temperatures, the effects of creep must be considered. When a load is applied to a specimen at an elevated temperature, the specimen deforms rapidly at first but then continues to deform, or creep, at a much slower rate. A schematic creep curve for a steel subjected to a constant tensile load and at a constant elevated temperature is shown in Fig. 1.9. The initial elongation occurs almost instantaneously and is followed by three stages. In stage 1, elongation increases at a decreasing rate. In stage 2, elongation increases at a nearly constant rate. And in stage 3, elongation increases at an increasing rate. The failure, or creep-rupture, load is less than the load that would cause failure at that temperature in a short-time loading test.

Table 1.6 indicates typical creep and rupture data for a carbon steel, an HSLA steel, and a constructional alloy steel. The table gives the stress that will cause a given amount of creep in a given time at a particular temperature.

For special elevated-temperature applications in which structural steels do not provide adequate properties, special alloy and stainless steels with excellent high-temperature properties are available.

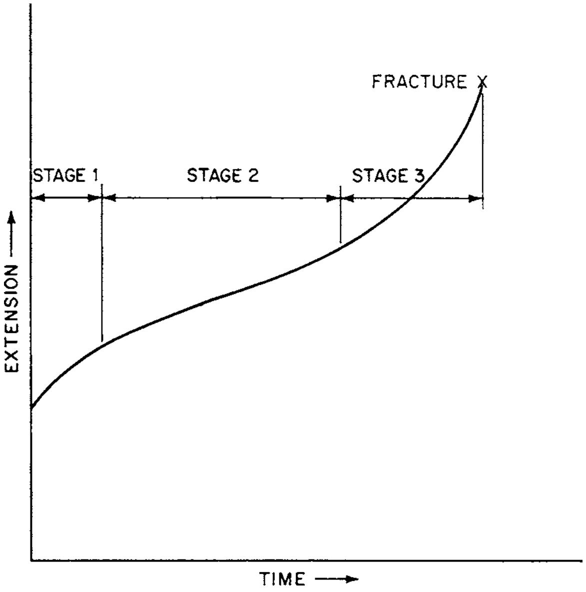


FIGURE 1.9 Creep curve for structural steel in tension (schematic). (*From R. L. Brockenbrough and B. G. Johnston,* USS Steel Design Manual, *R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission.*)

TABLE 1.6 Typical Creep Rates and Rupture Stresses for Structural Steels at Various Temperatures

	Stress, ksi, for creep rate of		Stress, ksi, for rupture in		
Test temperature, °F	0.0001% per h*	0.00001% per h[†]	1000 h	10,000 h	100,000 h
		A36 steel			
800	21.4	13.8	38.0	24.8	16.0
900	9.9	6.0	18.5	12.4	8.2
1000	4.6	2.6	9.5	6.3	4.2
		A588 Grade A steel[†]			
800	34.6	29.2	44.1	35.7	28.9
900	20.3	16.3	28.6	22.2	17.3
1000	11.4	8.6	17.1	12.0	8.3
1200	1.7	1.0	3.8	2.0	1.0
		A514 Grade F steel[†]			
700	—	—	101.0	99.0	97.0
800	81.0	74.0	86.0	81.0	77.0

*Equivalent to 1% in 10,000 h.

[†] Equivalent to 1% in 100,000 h.

[‡] Not recommended for use where temperatures exceed 800°F.

1.12 FATIGUE

A structural member subjected to cyclic loadings may eventually fail through initiation and propagation of cracks. This phenomenon is called **fatigue** and can occur at stress levels considerably below the yield stress.

Extensive research programs conducted to determine the fatigue strength of structural members and connections have provided information on the factors affecting this property. These programs included studies of large-scale girder specimens with flange-to-web fillet welds, flange cover plates, stiffeners, and other attachments. The studies showed that the **stress range** (algebraic difference between maximum and minimum stress) and **notch severity** of details are the most important factors. Yield point of the steel had little effect. The knowledge developed from these programs has been incorporated into specifications of the American Institute of Steel Construction, American Association of State Highway and Transportation Officials, and the American Railway Engineering and Maintenance-of-Way Association, which offer detailed provisions for fatigue design.

1.13 BRITTLE FRACTURE

Under sufficiently adverse combinations of tensile stress, temperature, loading rate, geometric discontinuity (notch), and restraint, a steel member may experience a brittle fracture. All these factors need not be present. In general, a **brittle fracture** is a failure that occurs by cleavage with little indication of plastic deformation. In contrast, a **ductile fracture** occurs mainly by shear, usually preceded by considerable plastic deformation.

Design against brittle fracture requires selection of the proper grade of steel for the application and avoiding notchlike defects in both design and fabrication. An awareness of the phenomenon is important so that steps can be taken to minimize the possibility of this undesirable, usually catastrophic, failure mode.

An empirical approach and an analytical approach directed toward selection and evaluation of steels to resist brittle fracture are outlined below. These methods are actually complementary and are frequently used together in evaluating material and fabrication requirements.

Charpy V-Notch Test. Many tests have been developed to rate steels on their relative resistance to brittle fracture. The most commonly used is the Charpy V-notch test, which specifically evaluates notch toughness, that is, the resistance to fracture in the presence of a notch. In this test, a small square bar with a specified-size V-shaped notch at its mid-length (Type A impact-test specimen of ASTM A370) is simply supported at its ends as a beam and fractured by a blow from a swinging pendulum. The amount of energy required to fracture the specimen or the appearance of the fracture surface is determined over a range of temperatures. The appearance of the fracture surface is usually expressed as the percentage of the surface that appears to have fractured by shear.

A **shear fracture** is indicated by a dull or fibrous appearance. A shiny or crystalline appearance is associated with a **cleavage fracture.**

The data obtained from a Charpy test are used to plot curves, such as those in Fig. 1.10, of energy or percentage of shear fracture as a function of temperature. The temperature near the bottom of the energy-temperature curve, at which a selected low value of energy is absorbed, often 15 ft · lb, is called the **ductility transition temperature** or the **15-ft · lb transition temperature.** The temperature at which the percentage of shear fracture decreases to 50% is often called the **fracture-appearance transition temperature.** These transition temperatures serve as a rating of the resistance of different steels to brittle fracture. The lower the transition temperature, the greater is the notch toughness.

Of the steels in Table 1.1, A36 steel generally has about the highest transition temperature. Since this steel has an excellent service record in a variety of structural applications, it appears likely that any of the structural steels, when designed and fabricated in an appropriate manner, could be used for similar applications with little likelihood of brittle fracture. Nevertheless, it is important to avoid unusual temperature, notch, and stress conditions to minimize susceptibility to brittle fracture.

In applications where notch toughness is considered important, the minimum Charpy V-notch value and test temperature should be specified, because there may be considerable variation in toughness

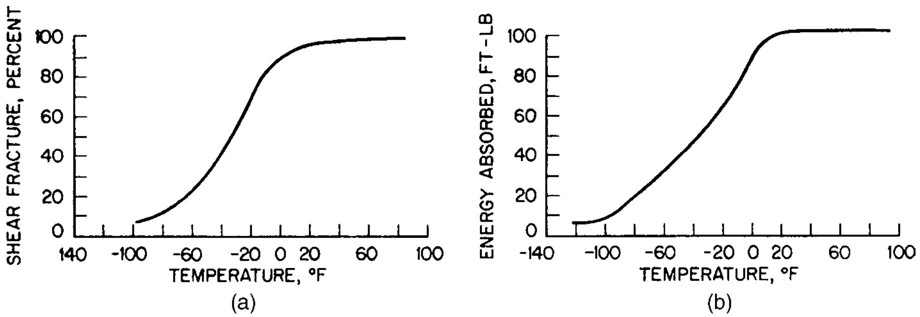


FIGURE 1.10 Transition curves from Charpy-V notch impact tests. (*a*) Variation of percent shear fracture with temperature, (*b*) Variation of absorbed energy with temperature.

within any given product designation unless specifically produced to minimum requirements. The test temperature may be specified higher than the lowest operating temperature to compensate for a lower rate of loading in the anticipated application. (See Art. 1.1.5.)

It should be noted that as the thickness of members increases, the inherent restraint increases and tends to inhibit ductile behavior. Thus special precautions or greater toughness, or both, is required for tension or flexural members comprised of thick material. (See Art. 1.16.)

Fracture-Mechanics Analysis. Fracture mechanics offers a more direct approach for prediction of crack propagation. For this analysis, it is assumed that a **crack,** which may be defined as a flat, internal defect, is always present in a stressed body. By linear-elastic stress analysis and laboratory tests on a precracked specimen, the defect size is related to the applied stress that will cause crack propagation and brittle fracture, as outlined below.

Near the tip of a crack, the stress component *f* perpendicular to the plane of the crack (Fig. 1.11*a*) can be expressed as

$$f = \frac{K_I}{\sqrt{2\pi r}} \tag{1.2}$$

where *r* is distance from tip of crack and K_I is a stress-intensity factor related to geometry of crack and to applied loading. The factor K_I can be determined from elastic theory for given crack geometries and

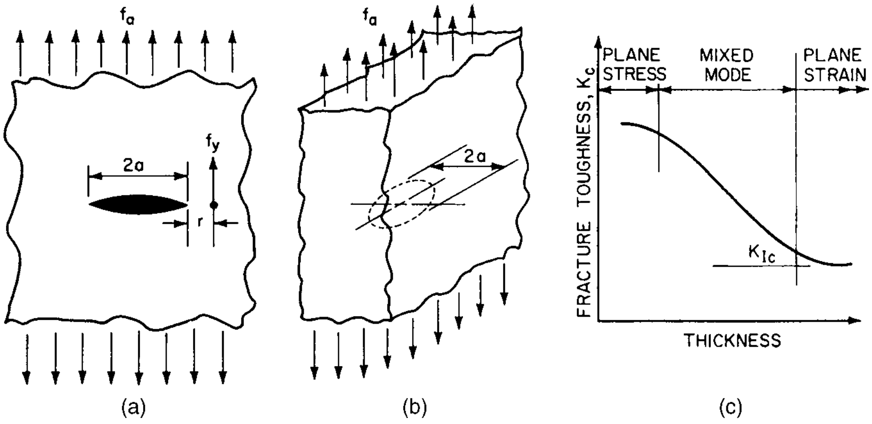


FIGURE 1.11 Fracture mechanics analysis for brittle fracture. (*a*) Sharp crack in a stressed infinite plate. (*b*) Disk-shaped crack in an infinite body. (*c*) Relation of fracture toughness to thickness.

loading conditions. For example, for a through-thickness crack of length 2*a* in an infinite plate under uniform stress (Fig. 1.11*a*),

$$K_l = f_a\sqrt{\pi a} \tag{1.3}$$

where f_a is the nominal applied stress. For a disk-shaped crack of diameter 2*a* embedded in an infinite body (Fig. 1.11*b*), the relationship is

$$K_l = 2f_a\sqrt{\frac{a}{\pi}} \tag{1.4}$$

If a specimen with a crack of known geometry is loaded until the crack propagates rapidly and causes failure, the value of K_l at that stress level can be calculated from the derived expression. This value is termed the **fracture toughness** K_c.

A precracked tension or bend-type specimen is usually used for such tests. As the thickness of the specimen increases and the stress condition changes from plane stress to plane strain, the fracture toughness decreases to a minimum value, as illustrated in Fig. 1.11*c*. This value of plane-strain fracture toughness, designated K_{lc}, may be regarded as a fundamental material property.

Thus, if K_{lc} is substituted for K_l, for example, in Eq. (1.3) or (1.4) a numerical relationship is obtained between the crack geometry and the applied stress that will cause fracture. With this relationship established, brittle fracture may be avoided by determining the maximum-size crack present in the body and maintaining the applied stress below the corresponding level. The tests must be conducted at or correlated with temperatures and strain rates appropriate for the application, because fracture toughness decreases with temperature and loading rate. Correlations have been made to enable fracture toughness values to be estimated from the results of Charpy V-notch tests.

Fracture-mechanics analysis has proven quite useful, particularly in critical applications. Fracture-control plans can be established with suitable inspection intervals to ensure that imperfections, such as fatigue cracks, do not grow to critical size.

(J. M. Barsom and S. T. Rolfe, *Fracture and Fatigue Control in Structures; Applications of Fracture Mechanics*, Prentice-Hall, Englewood Cliffs, N.J.)

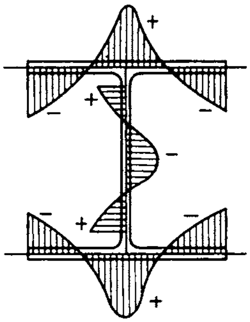
1.14 RESIDUAL STRESSES

Stresses that remain in structural members after rolling or fabrication are known as **residual stresses.** The magnitude of the stresses is usually determined by removing longitudinal sections and measuring the strain that results. Only the longitudinal stresses are usually measured. To meet equilibrium conditions, the axial force and moment obtained by integrating these residual stresses over any cross section of the member must be zero.

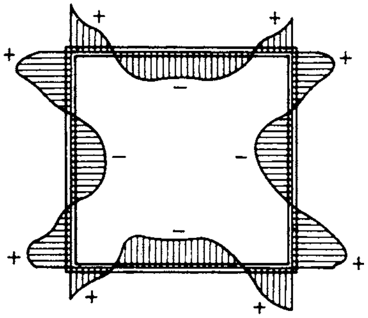
In a hot-rolled structural shape, the residual stresses result from unequal cooling rates after rolling. For example, in a wide-flange beam, the center of the flange cools more slowly and develops tensile residual stresses that are balanced by compressive stresses elsewhere on the cross section (Fig. 1.12*a*). In a welded member, tensile residual stresses develop near the weld and compressive stresses elsewhere provide equilibrium, as shown for the welded box section in Fig. 1.12*b*.

For plates with rolled edges (UM plates), the plate edges have compressive residual stresses (Fig. 1.12*c*). However, the edges of flame-cut plates have tensile residual stresses (Fig. 1.12*d*). In a welded I-shaped member, the stress condition in the edges of flanges before welding is reflected in the final residual stresses (Fig. 1.12*e*). Although not shown in Fig. 1.12, the residual stresses at the edges of sheared-edge plates vary through the plate thickness. Tensile stresses are present on one surface and compressive stresses on the opposite surface.

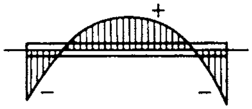
The residual-stress distributions mentioned above are usually relatively constant along the length of the member. However, residual stresses also may occur at particular locations in a member, because of localized plastic flow from fabrication operations, such as cold straightening or heat straightening.



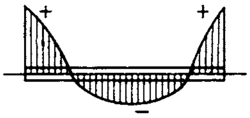
(a) HOT-ROLLED SHAPE



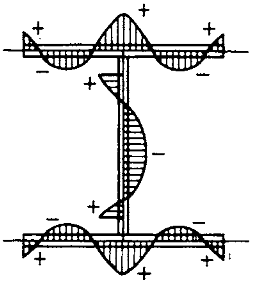
(b) WELDED BOX SECTION



(c) PLATE WITH ROLLED EDGES



(d) PLATE WITH FLAME-CUT EDGES



(e) BEAM FABRICATED FROM FLAME-CUT PLATES

FIGURE 1.12 Typical residual-stress distributions (+ indicates tension and − compression).

When loads are applied to structural members, the presence of residual stresses usually causes some premature inelastic action; that is, yielding occurs in localized portions before the nominal stress reaches the yield point. Because of the ductility of steel, the effect on strength of tension members is not usually significant, but excessive tensile residual stresses, in combination with other conditions, can cause fracture. In compression members, residual stresses decrease the buckling load from that of an ideal or perfect member. However, current design criteria in general use for compression members account for the influence of residual stress.

In bending members that have residual stresses, a small inelastic deflection of insignificant magnitude may occur with the first application of load. However, under subsequent loads of the same magnitude, the behavior is elastic. Furthermore, in "compact" bending members, the presence of residual stresses has no effect on the ultimate moment (plastic moment). Consequently, in the design of statically loaded members, it is not usually necessary to consider residual stresses.

1.15 LAMELLAR TEARING

In a structural steel member subjected to tension, elongation and reduction of area in sections normal to the stress are usually much lower in the through-thickness direction than in the planar direction. This inherent directionality is of small consequence in many applications, but it does become important in design and fabrication of structures with highly restrained joints because of the possibility of

lamellar tearing. This is a cracking phenomenon that starts underneath the surface of steel plates as a result of excessive through-thickness strain, usually associated with shrinkage of weld metal in highly restrained joints. The tear has a steplike appearance consisting of a series of terraces parallel to the surface. The cracking may remain completely below the surface or may emerge at the edges of plates or shapes or at weld toes.

Careful selection of weld details, filler metal, and welding procedure can restrict lamellar tearing in heavy welded constructions, particularly in joints with thick plates and heavy structural shapes. Also, when required, structural steels can be produced by special processes, generally with low sulfur content and inclusion control, to enhance through-thickness ductility.

The most widely accepted method of measuring the susceptibility of a material to lamellar tearing is the tension test on a round specimen, in which is observed the reduction in area of a section oriented perpendicular to the rolled surface. The reduction required for a given application depends on the specific details involved. The specifications to which a particular steel can be produced are subject to negotiations with steel producers.

(R. L. Brockenbrough, Chap. 1.2 in *Constructional Steel Design—An International Guide*, R. Bjorhovde et al., eds., Elsevier Science Publishers, New York.)

1.16 WELDED SPLICES IN HEAVY SECTIONS

Shrinkage during solidification of large welds in structural steel members causes, in adjacent restrained metal, strains that can exceed the yield-point strain. In thick material, triaxial stresses may develop because there is restraint in the thickness direction as well as in planar directions. Such conditions inhibit the ability of a steel to act in a ductile manner and increase the possibility of brittle fracture. Therefore, for members subject to primary tensile stresses due to axial tension or flexure in buildings, the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings imposes special requirements for welded splicing of either hot-rolled shapes with a flange thickness more than 2 in thick or of shapes built up by welding plates more than 2 in thick. The specifications include requirements for notch toughness, generous-sized weld-access holes, preheating for thermal cutting, and grinding and inspecting cut edges. Even for primary compression members, the same precautions should be taken for sizing weld access holes, preheating, grinding, and inspection.

Most heavy wide-flange shapes and tees cut from these shapes have regions where the steel has low toughness, particularly at flange-web intersections. These low-toughness regions occur because of the slower cooling there and, because of the geometry, the lower rolling pressure applied there during production. Hence, to ensure ductility and avoid brittle failure, bolted splices should be considered as an alternative to welding.

“Specification for Structural Steel Buildings,” American Institute of Steel Construction; R. L. Brockenbrough, Sec. 9 in *Standard Handbook for Civil Engineers*, 4th ed., McGraw-Hill, New York.)

1.17 k-AREA CRACKING

Wide flange sections are typically straightened as part of the mill production process. Often a rotary straightening process is used, although some heavier members may be straightened in a gag press. Some reports have indicated a potential for crack initiation at or near connections in the “*k*” area of wide flange sections that have been rotary straightened. The *k* area is the region extending from approximately the mid-point of the web-to-flange fillet, into the web for a distance approximately 1 to $1\frac{1}{2}$ in beyond the point of tangency. In some cases, this limited region had a reduced notch toughness due to cold working and strain hardening. Most of the incidents reported occurred at highly restrained joints with welds in the *k* area. However, the number of examples reported was limited and these occurred during construction or laboratory tests, with no evidence of difficulties with steel members in service.

Most of the concern was related to welding of continuity plates and doubler plates in beam-to-column connections. Recent research has shown that such cracking can be avoided if the continuity plates are fillet welded to both the web and the flange, with the cutout in the corners of the continuity plate at least 1.5 by 1.5 in, and the fillet welds stopped short by a weld length from the edges of the cutout. Groove welding is unnecessary. Similarly, tests also showed that web doubler plates should be fillet welded, and that they do not need to be in contact with the column web. Design details should follow the requirements of the AISC "Specification" and the recommendations given in its Commentary.

1.18 VARIATIONS IN MECHANICAL PROPERTIES

Tensile properties of structural steel may vary from specified minimum values. Product specifications generally require that properties of the material "as represented by the test specimen" meet certain values. ASTM specifications dictate only a limited number of tests per heat (in each strength level produced, if applicable). If the heats are very large, the test specimens qualify a considerable amount of product. As a result, there is a possibility that properties at locations other than those from which the specimens were taken will be different from those specified.

For plates, a test specimen is required by ASTM A6 to be taken from a corner. If the plates are wider than 24 in, the longitudinal axis of the specimen should be oriented transversely to the final direction in which the plates were rolled. For other products, however, the longitudinal axis of the specimen should be parallel to the final direction of rolling.

For structural shapes with a flange width of 6 in or more, test specimens should be selected from a point in the flange as near as practicable to two-thirds the distance from the flange centerline to the flange toe. Prior to 1997–1998, the specimens were taken from the web.

An extensive study commissioned by the American Iron and Steel Institute (AISI) compared yield points at various sample locations with the official product test. The studies indicated that the average difference at the check locations was −0.7 ksi. For the top and bottom flanges, at either end of beams, the average difference at check locations was −2.6 ksi.

Although the test value at a given location may be less than that obtained in the official test, the difference is offset to the extent that the value from the official test exceeds the specified minimum value. For example, a statistical study made to develop criteria for load and resistance factor design showed that the mean yield points exceeded the specified minimum yield point F_y (specimen located in web) as indicated below and with the indicated coefficient of variation (COV):

Flanges of rolled shapes:	$1.05F_y$, COV = 0.10
Webs of rolled shapes:	$1.10F_y$, COV = 0.11
Plates:	$1.10F_y$, COV = 0.11

Also, these values incorporate an adjustment to the lower "static" yield points.

For similar reasons, the notch toughness can be expected to vary throughout a product. (R. L. Brockenbrough, Chap. 1.2 in *Constructional Steel Design—An International Guide*, R. Bjorhovde et al., eds., Elsevier Science Publishers, New York.)

1.19 CHANGES IN CARBON STEELS ON HEATING AND COOLING*

As pointed out in Art. 1.11, heating changes the tensile properties of steels. Actually, heating changes many steel properties. Often, the primary reason for such changes is a change in structure brought about by heat. Some of these structural changes can be explained with the aid of an iron-carbon equilibrium diagram (Fig. 1.13).

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*Articles 1.19 through 1.27 are adapted from a previous edition written by Frederick S. Merritt, Consulting Engineer, West Palm Beach, Fla.

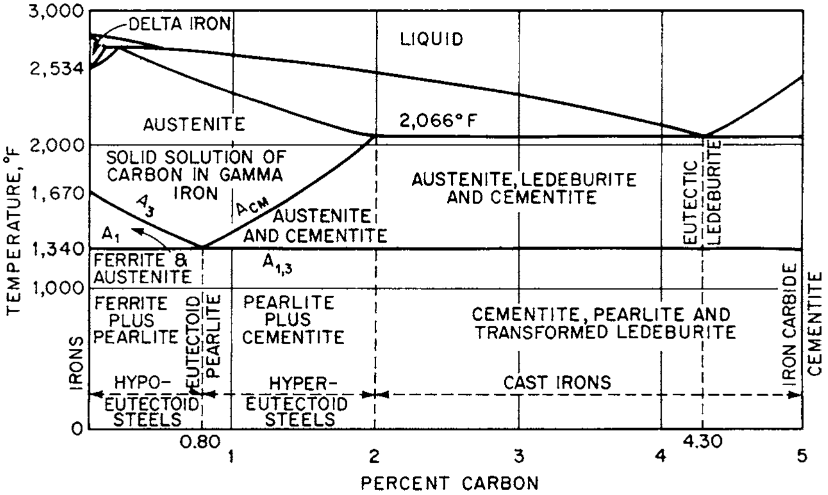

FIGURE 1.13 Iron-carbon equilibrium diagram.

The diagram maps out the constituents of carbon steels at various temperatures as carbon content ranges from 0 to 5%. Other elements are assumed to be present only as impurities, in negligible amounts.

If a steel with less than 2% carbon is very slowly cooled from the liquid state, a solid solution of carbon in gamma iron will result. This is called **austenite.** (Gamma iron is a pure iron whose crystalline structure is face-centered cubic.)

If the carbon content is about 0.8%, the carbon remains in solution as the austenite slowly cools, until the A_1 temperature (1340°F) is reached. Below this temperature, the austenite transforms to the eutectoid **pearlite.** This is a mixture of ferrite and **cementite** (iron carbide, Fe_3C). Pearlite, under a microscope, has a characteristic platelike, or lamellar, structure with an iridescent appearance, from which it derives its name.

If the carbon content is less than 0.8%, as is the case with structural steels, cooling austenite below the A_3 temperature line causes transformation of some of the austenite to **ferrite.** (This is a pure iron, also called **alpha iron,** whose crystalline structure is body-centered cubic.) Still further cooling to below the A_1 line causes the remaining austenite to transform to pearlite. Thus, as indicated in Fig. 1.13, low-carbon steels are **hypoeutectoid steels,** mixtures of ferrite and pearlite.

Ferrite is very ductile but has low tensile strength. Hence carbon steels get their high strengths from the pearlite present or, more specifically, from the cementite in the pearlite.

The iron-carbon equilibrium diagram shows only the constituents produced by slow cooling. At high cooling rates, however, equilibrium cannot be maintained. Transformation temperatures are lowered, and steels with microstructures other than pearlitic may result. Properties of such steels differ from those of the pearlitic steels. Heat treatments of steels are based on these temperature effects.

If a low-carbon austenite is rapidly cooled below about 1300°F, the austenite will transform at constant temperature into steels with one of four general classes of microstructure:

Pearlite, or lamellar, microstructure results from transformations in the range 1300 to 1000°F. The lower the temperature, the closer is the spacing of the platelike elements. As the spacing becomes smaller, the harder and tougher the steels become. Steels such as A36, A572, and A588 have a mixture of a soft ferrite matrix and a hard pearlite.

Bainite forms in transformations below about 1000°F and above about 450°F. It has an acicular, or needlelike, microstructure. At the higher temperatures, bainite may be softer than the pearlitic steels. However, as the transformation temperature is decreased, hardness and toughness increase.

Martensite starts to form at a temperature below about 500°F, called the M_s temperature. The transformation differs from those for pearlitic and bainitic steels in that it is not time dependent. Martensite occurs almost instantly during rapid cooling, and the percentage of austenite transformed to martensite depends only on the temperature to which the steel is cooled. For complete conversion to martensite, cooling must extend below the M_f temperature, which may be 200°F or less. Like bainite, martensite has an acicular microstructure, but martensite is harder and more brittle than pearlitic and bainitic steels. Its hardness varies with carbon content and to some extent with cooling rate. For some applications, such as those where wear resistance is important, the high hardness of martensite is desirable, despite brittleness. Generally, however, martensite is used to obtain tempered martensite, which has superior properties.

Tempered martensite is formed when martensite is reheated to a subcritical temperature after quenching. The tempering precipitates and coagulates carbides. Hence the microstructure consists of carbide particles, often spheroidal in shape, dispersed in a ferrite matrix. The result is a loss in hardness but a considerable improvement in ductility and toughness. The heat-treated carbon and HSLA steels and quenched and tempered constructional steels discussed in Art. 1.1 are low-carbon martensitic steels.

(Z. D. Jastrzebski, *Nature and Properties of Engineering Materials*, John Wiley & Sons, New York.)

1.20 EFFECTS OF GRAIN SIZE

As indicated in Fig. 1.13, when a low-carbon steel is heated above the A_1 temperature line, austenite, a solid solution of carbon in gamma iron, begins to appear in the ferrite matrix. Each island of austenite grows until it intersects its neighbor. With further increase in temperature, these grains grow larger. The final grain size depends on the temperature above the A_3 line to which the metal is heated. When the steel cools, the relative coarseness of the grains passes to the ferrite-plus-pearlite phase.

At rolling and forging temperatures, therefore, many steels grow coarse grains. Hot working, however, refines the grain size. The temperature at the final stage of the hot-working process determines the final grain size. When the finishing temperature is relatively high, the grains may be rather coarse when the steel is air-cooled. In that case, the grain size can be reduced if the steel is normalized (reheated to just above the A_3 line and again air-cooled). (See Art. 1.21.)

Fine grains improve many properties of steels. Other factors being the same, steels with finer grain size have better notch toughness because of lower transition temperatures (see Art. 1.13) than coarser-grained steels. Also, decreasing grain size improves bendability and ductility. Furthermore, fine grain size in quenched and tempered steel improves yield strength. And there is less distortion, less quench cracking, and lower internal stress in heat-treated products.

On the other hand, for some applications, coarse-grained steels are desirable. They permit deeper hardening. If the steels should be used in elevated-temperature service, they offer higher load-carrying capacity and higher creep strength than fine-grained steels.

Austenitic-grain growth may be inhibited by carbides that dissolve slowly or remain undissolved in the austenite or by a suitable dispersion of nonmetallic inclusions. Steels produced this way are called **fine grained.** Steels not made with grain-growth inhibitors are called **coarse grained.**

When heated above the critical temperature, 1340°F, grains in coarse-grained steels grow gradually. The grains in fine-grained steels grow only slightly, if at all, until a certain temperature, the coarsening temperature, is reached. Above this, abrupt coarsening occurs. The resulting grain size may be larger than that of coarse-grained steel at the same temperature. Note further that either fine-grained or coarse-grained steels can be heat-treated to be either fine-grained or coarse-grained (see Art. 1.21).

The usual method of making fine-grained steels involves controlled aluminum deoxidation (see also Art. 1.23). The inhibiting agent in such steels may be a submicroscopic dispersion of aluminum nitride or aluminum oxide.

(W. T. Lankford, Jr., ed., *The Making, Shaping and Treating of Steel*, Association of Iron and Steel Engineers, Pittsburgh, Pa.)

1.21 ANNEALING AND NORMALIZING

Structural steels may be annealed to relieve stresses induced by cold or hot working. Sometimes, also, annealing is used to soften metal to improve its formability or machinability.

Annealing involves austenitizing the steel by heating it above the A_3 temperature line in Fig. 1.13, then cooling it slowly, usually in a furnace. This treatment improves ductility but decreases tensile strength and yield point. As a result, further heat treatment may be necessary to improve these properties.

Structural steels may be normalized to refine grain size. As pointed out in Art. 1.20, grain size depends on the finishing temperature in hot rolling.

Normalizing consists of heating the steel above the A_3 temperature line, then cooling the metal in still air. Thus the rate of cooling is more rapid than in annealing. Usual practice is to normalize from 100 to 150°F above the critical temperature. Higher temperatures coarsen the grains.

Normalizing tends to improve notch toughness by lowering ductility and fracture transition temperatures. Thick plates benefit more from this treatment than thin plates. Requiring fewer roller passes, thick plates have a higher finishing temperature and cool slower than thin plates, thus have a more adverse grain structure. Hence the improvement from normalizing is greater for thick plates.

1.22 EFFECTS OF CHEMISTRY ON STEEL PROPERTIES

Chemical composition determines many characteristics of steels important in construction applications. Some of the chemicals present in commercial steels are a consequence of the steelmaking process. Other chemicals may be added deliberately by the producers to achieve specific objectives. Specifications therefore usually require producers to report the chemical composition of the steels.

During the pouring of a heat of steel, producers take samples of the molten steel for chemical analysis. These heat analyses are usually supplemented by product analyses taken from drillings or millings of blooms, billets, or finished products. ASTM specifications contain maximum and minimum limits on chemicals reported in the heat and product analyses, which may differ slightly.

Principal effects of the elements more commonly found in carbon and low-alloy steels are discussed below. Bear in mind, however, that the effects of two or more of these chemicals when used in combination may differ from those when each alone is present. Note also that variations in chemical composition to obtain specific combinations of properties in a steel usually increase cost, because it becomes more expensive to make, roll, and fabricate.

Carbon is the principal strengthening element in carbon and low-alloy steels. In general, each 0.01% increase in carbon content increases the yield point about 0.5 ksi. This, however, is accompanied by increase in hardness and reduction in ductility, notch toughness, and weldability, raising of the transition temperatures, and greater susceptibility to aging. Hence limits on carbon content of structural steels are desirable. Generally, the maximum permitted in structural steels is 0.30% or less, depending on the other chemicals present and the weldability and notch toughness desired.

Aluminum, when added to silicon-killed steel, lowers the transition temperature and increases notch toughness. If sufficient aluminum is used, up to about 0.20%, it reduces the transition temperature even when silicon is not present. However, the larger additions of aluminum make it difficult to obtain desired finishes on rolled plate. Drastic deoxidation of molten steels with aluminum or aluminum and titanium, in either the steelmaking furnace or the ladle, can prevent the spontaneous increase in hardness at room temperature called **aging.** Also, aluminum restricts grain growth during heat treatment and promotes surface hardness by nitriding.

Boron in small quantities increases hardenability of steels. It is used for this purpose in quenched and tempered low-carbon constructional alloy steels. However, more than 0.0005 to 0.004% boron produces no further increase in hardenability. Also, a trace of boron increases strength of low-carbon, plain molybdenum (0.40%) steel.

Chromium improves strength, hardenability, abrasion resistance, and resistance to atmospheric corrosion. However, it reduces weldability. With small amounts of chromium, low-alloy steels have higher creep strength than carbon steels and are used where higher strength is needed for elevated-temperature service. Also, chromium is an important constituent of stainless steels.

Columbium in very small amounts produces relatively larger increases in yield point but smaller increases in tensile strength of carbon steel. However, the notch toughness of thick sections is appreciably reduced.

Copper in amounts up to about 0.35% is very effective in improving the resistance of carbon steels to atmospheric corrosion. Improvement continues with increases in copper content up to about 1% but not so rapidly. Copper increases strength, with a proportionate increase in fatigue limit. Copper also increases hardenability, with only a slight decrease in ductility and little effect on notch toughness and weldability. However, steels with more than 0.60% copper are susceptible to precipitation hardening. And steels with more than about 0.5% copper often experience hot shortness during hot working, and surface cracks or roughness develop. Addition of nickel in an amount equal to about half the copper content is effective in maintaining surface quality.

Hydrogen, which may be absorbed during steelmaking, embrittles steels. Ductility will improve with aging at room temperature as the hydrogen diffuses out of the steel, faster from thin sections than from thick. When hydrogen content exceeds 0.0005%, flaking, internal cracks or bursts, may occur when the steel cools after rolling, especially in thick sections. In carbon steels, flaking may be prevented by slow cooling after rolling, to permit the hydrogen to diffuse out of the steel.

Manganese increases strength, hardenability, fatigue limit, notch toughness, and corrosion resistance. It lowers the ductility and fracture transition temperatures. It hinders aging. Also, it counteracts hot shortness due to sulfur. For this last purpose, the manganese content should be three to eight times the sulfur content, depending on the type of steel. However, manganese reduces weldability.

Molybdenum increases yield strength, hardenability, abrasion resistance, and corrosion resistance. It also improves weldability. However, it has an adverse effect on toughness and transition temperature. With small amounts of molybdenum, low-alloy steels have higher creep strength than carbon steels and are used where higher strength is needed for elevated-temperature service.

Nickel increases strength, hardenability, notch toughness, and corrosion resistance. It is an important constituent of stainless steels. It lowers the ductility and fracture transition temperatures, and it reduces weldability.

Nitrogen increases strength, but it may cause aging. It also raises the ductility and fracture transition temperatures.

Oxygen, like nitrogen, may be a cause of aging. Also, oxygen decreases ductility and notch toughness.

Phosphorus increases strength, fatigue limit, and hardenability, but it decreases ductility and weldability and raises the ductility transition temperature. Additions of aluminum, however, improve the notch toughness of phosphorus-bearing steels. Phosphorus improves the corrosion resistance of steel and works very effectively together with small amounts of copper toward this result.

Silicon increases strength, notch toughness, and hardenability. It lowers the ductility transition temperature, but it also reduces weldability. Silicon often is used as a deoxidizer in steelmaking (see Art. 1.23).

Sulfur, which enters during the steelmaking process, can cause hot shortness. This results from iron sulfide inclusions, which soften and may rupture when heated. Also, the inclusions may lead to brittle failure by providing stress raisers from which fractures can initiate. And high sulfur contents may cause porosity and hot cracking in welding unless special precautions are taken. Addition of manganese, however, can counteract hot shortness. It forms manganese sulfide, which is more refractory than iron sulfide. Nevertheless, it usually is desirable to keep sulfur content below 0.05%.

Titanium increases creep and rupture strength and abrasion resistance. It plays an important role in preventing aging. It sometimes is used as a deoxidizer in steelmaking (see Art. 1.23) and grain-growth inhibitor (see Art. 1.20).

Tungsten increases creep and rupture strength, hardenability and abrasion resistance. It is used in steels for elevated-temperature service.

Vanadium, in amounts up to about 0.12%, increases rupture and creep strength without impairing weldability or notch toughness. It also increases hardenability and abrasion resistance. Vanadium sometimes is used as a deoxidizer in steelmaking (see Art. 1.23) and as a grain-growth inhibitor (see Art. 1.20).

In practice, carbon content is limited so as not to impair ductility, notch toughness, and weldability. To obtain high strength, therefore, resort is had to other strengthening agents that improve these desirable properties or at least do not impair them as much as carbon. Often, the better these properties are required to be at high strengths, the more costly the steels are likely to be.

Attempts have been made to relate chemical composition to weldability by expressing the relative influence of chemical content in terms of **carbon equivalent.** One widely used formula, which is a supplementary requirement in ASTM A6 for structural steels, is

$$C_{eq} = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15} \tag{1.5}$$

where C = carbon content, %
Mn = manganese content, %
Cr = chromium content, %
Mo = molybdenum, %
V = vanadium, %
Ni = nickel content, %
Cu = copper, %

Carbon equivalent is related to the maximum rate at which a weld and adjacent plate may be cooled after welding, without underbead cracking occurring. The higher the carbon equivalent, the lower will be the allowable cooling rate. Also, use of low-hydrogen welding electrodes and preheating becomes more important with increasing carbon equivalent. (*Structural Welding Code—Steel*, American Welding Society, Miami, Fla.)

Though carbon provides high strength in steels economically, it is not a necessary ingredient. Very-high-strength steels are available that contain so little carbon that they are considered carbon-free.

Maraging steels, carbon-free iron-nickel martensites, develop yield strengths from 150 to 300 ksi, depending on alloying composition. As pointed out in Art. 1.19, iron-carbon martensite is hard and brittle after quenching and becomes softer and more ductile when tempered. In contrast, maraging steels are relatively soft and ductile initially but become hard, strong, and tough when aged. They are fabricated while ductile and later strengthened by an aging treatment. These steels have high resistance to corrosion, including stress-corrosion cracking.

(W. T. Lankford, Jr., ed., *The Making, Shaping and Treating of Steel*, Association of Iron and Steel Engineers, Pittsburgh, Pa.)

1.23 STEELMAKING METHODS

Structural steel is usually produced today by one of two production processes. In the traditional process, iron or "hot metal" is produced in a blast furnace and then further processed in a basic oxygen furnace to make the steel for the desired products. Alternatively, steel can be made in an electric arc furnace that is charged mainly with steel scrap instead of hot metal. In either case, the steel must be produced so that undesirable elements are reduced to levels allowed by pertinent specifications to minimize adverse effects on properties.

In a **blast furnace,** iron ore, coke, and flux (limestone and dolomite) are charged into the top of a large refractory-lined furnace. Heated air is blown in at the bottom and passed up through the bed of raw materials. A supplemental fuel such as gas, oil, or powdered coal is also usually charged. The iron is reduced to metallic iron and melted; then it is drawn off periodically through tap holes into transfer ladles. At this point, the molten iron includes several other elements (manganese, sulfur, phosphorus, and silicon) in amounts greater than permitted for steel, and thus further processing is required.

In a **basic oxygen furnace,** the charge consists of hot metal from the blast furnace and steel scrap. Oxygen, introduced by a jet blown into the molten metal, reacts with the impurities present to facilitate the removal or reduction in level of unwanted elements, which are trapped in the slag or in the gases produced. Also, various fluxes are added to reduce the sulfur and phosphorus contents to desired levels. In this batch process, large heats of steel may be produced in less than an hour.

An **electric-arc furnace** does not require a hot metal charge but relies mainly on steel scrap. The metal is heated by an electric arc between large carbon electrodes that project through the furnace roof into the charge. Oxygen is injected to speed the process. This is a versatile batch process that can be adapted to producing small heats where various steel grades are required, but it also can be used to produce large heats.

Ladle treatment is an integral part of most steelmaking processes. The ladle receives the product of the steelmaking furnace so that it can be moved and poured into either ingot molds or a continuous casting machine. While in the ladle, the chemical composition of the steel is checked, and alloying elements are added as required. Also, deoxidizers are added to remove dissolved oxygen. Processing can be done at this stage to reduce further sulfur content, remove undesirable nonmetallics, and change the shape of remaining inclusions. Thus significant improvements can be made in the toughness, transverse properties, and through-thickness ductility of the finished product. Vacuum degassing, argon bubbling, induction stirring, and the injection of rare earth metals are some of the many procedures that may be employed.

Killed steels usually are deoxidized by additions to both furnace and ladle. Generally, silicon compounds are added to the furnace to lower the oxygen content of the liquid metal and stop oxidation of carbon (block the heat). This also permits addition of alloying elements that are susceptible to oxidation. Silicon or other deoxidizers, such as aluminum, vanadium, and titanium, may be added to the ladle to complete deoxidation. Aluminum, vanadium, and titanium have the additional beneficial effect of inhibiting grain growth when the steel is normalized. (In the hot-rolled conditions, such steels have about the same ferrite grain size as semikilled steels.) Killed steels deoxidized with aluminum and silicon (**made to fine-grain practice**) often are used for structural applications because of better notch toughness and lower transition temperatures than semikilled steels of the same composition.

(W. T. Lankford, Jr., ed., *The Making, Shaping and Treating of Steel*, Association of Iron and Steel Engineers, Pittsburgh, Pa.)

1.24 CASTING AND HOT ROLLING

Today, the **continuous casting** process is used to produce semifinished products directly from liquid steel, thus eliminating the ingot molds and primary mills used previously. With continuous casting, the steel is poured from sequenced ladles to maintain a desired level in a tundish above an oscillating water-cooled copper mold (Fig. 1.14). The outer skin of the steel strand solidifies as it passes through the mold, and this action is further aided by water sprayed on the skin just after the strand exits the mold. The strand passes through sets of supporting rolls, curving rolls, and straightening rolls and is then rolled into slabs. The slabs are cut to length from the moving strand and held for subsequent rolling into finished product. Not only is the continuous casting process a more efficient method, but it also results in improved quality through more consistent chemical composition and better surfaces on the finished product.

Plates, produced from slabs or directly from ingots, are distinguished from sheet, strip, and flat bars by size limitations in ASTM A6. Generally, plates are heavier, per linear foot, than these other products. Plates are formed with straight horizontal rolls and later trimmed (sheared or gas cut) on all edges.

Slabs usually are reheated in a furnace and descaled with high-pressure water sprays before they are rolled into plates. The plastic slabs are gradually brought to desired dimensions by passage through a series of rollers. In the last rolling step, the plates pass through leveling, or flattening, rollers. Generally, the thinner the plate, the more flattening required. After passing through the leveler, plates are cooled uniformly, then sheared or gas cut to desired length, while still hot.

Some of the plates may be heat treated, depending on grade of steel and intended use. For carbon steel, the treatment may be annealing, normalizing, or stress relieving. Plates of HSLA or constructional alloy steels may be quenched and tempered. Some mills provide facilities for on-line heat treating or for thermomechanical processing (controlled rolling). Other mills heat treat off-line.

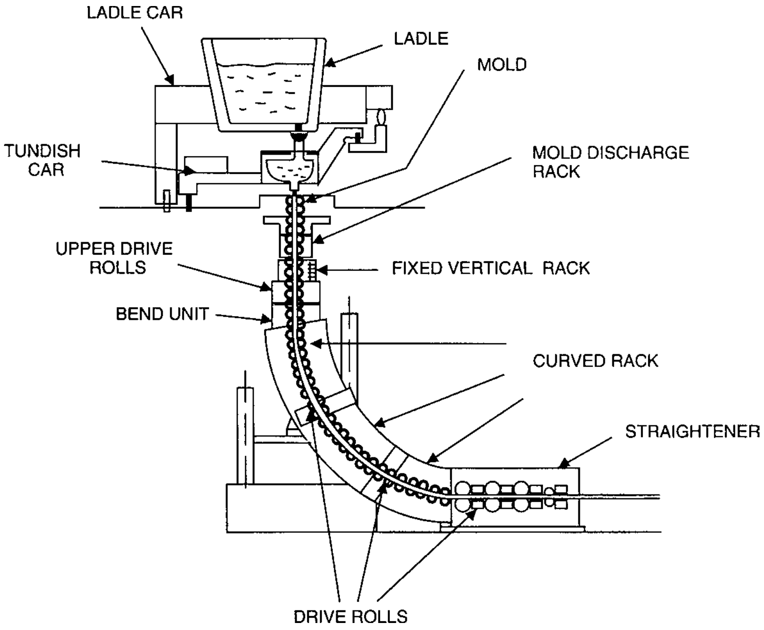

FIGURE 1.14 Schematic of slab caster.

Shapes are rolled from continuously cast beam blanks or from blooms that first are reheated to 2250°F. Rolls gradually reduce the plastic blooms to the desired shapes and sizes. The shapes then are cut to length for convenient handling, with a hot saw. After that, they are cooled uniformly. Next, they are straightened, in a roller straightener or in a gag press. Finally, they are cut to desired length, usually by hot shearing, hot sawing, or cold sawing. Also, column ends may be milled to close tolerances.

ASTM A6 requires that material for delivery "shall be free from injurious defects and shall have a workmanlike finish." The specification permits manufacturers to condition plates and shapes "for the removal of injurious surface imperfections or surface depressions by grinding, or chipping and grinding. . . ." Except in alloy steels, small surface imperfections may be corrected by chipping or grinding, then depositing weld metal with low-hydrogen electrodes. Conditioning also may be done on slabs before they are made into other products. In addition to chipping and grinding, they may be scarfed to remove surface defects.

Hand chipping is done with a cold chisel in a pneumatic hammer. Machine chipping may be done with a planer or a milling machine.

Scarfing, by hand or machine, removes defects with an oxygen torch. This can create problems that do not arise with other conditioning methods. When the heat source is removed from the conditioned area, a quenching effect is produced by rapid extraction of heat from the hot area by the surrounding relatively cold areas. The rapid cooling hardens the steel, the amount depending on carbon content and hardenability of the steel. In low-carbon steels, the effect may be insignificant. In high-carbon and alloy steels, however, the effect may be severe. If preventive measures are not taken, the hardened area will crack. To prevent scarfing cracks, the steel should be preheated before scarfing to between 300 and 500°F and, in some cases, postheated for stress relief. The hardened surface later can be removed by normalizing or annealing.

Internal structure and many properties of plates and shapes are determined largely by the chemistry of the steel, rolling practice, cooling conditions after rolling, and heat treatment, where used. Because the sections are rolled in a temperature range at which steel is austenitic (see Art. 1.19), internal structure is affected in several ways.

The final austenitic grain size is determined by the temperature of the steel during the last passes through the rolls (see Art. 1.20). In addition, inclusions are reoriented in the direction of rolling. As a result, ductility and bendability are much better in the longitudinal direction than in the transverse, and these properties are poorest in the thickness direction.

The cooling rate after rolling determines the distribution of ferrite and the grain size of the ferrite. Since air cooling is the usual practice, the final internal structure and, therefore, the properties of plates and shapes depend principally on the chemistry of the steel, section size, and heat treatment. By normalizing the steel and by use of steels made to fine-grain practice (with grain-growth inhibitors, such as aluminum, vanadium, and titanium), grain size can be refined and properties consequently improved.

In addition to the preceding effects, rolling also may induce residual stresses in plates and shapes (see Art. 1.14). Still other effects are a consequence of the final thickness of the hot-rolled material.

Thicker material requires less rolling, the finish rolling temperature is higher, and the cooling rate is slower than for thin material. As a consequence, thin material has a superior microstructure. Furthermore, thicker material can have a more unfavorable state of stress because of stress raisers, such as tiny cracks and inclusions, and residual stresses.

Consequently, thin material develops higher tensile and yield strengths than thick material of the same steel chemistry. ASTM specifications for structural steels recognize this usually by setting lower yield points for thicker material. A36 steel, however, has the same yield point for all thicknesses. To achieve this, the chemistry is varied for plates and shapes and for thin and thick plates. Thicker plates contain more carbon and manganese to raise the yield point. This cannot be done for high-strength steels because of the adverse effect on notch toughness, ductility, and weldability.

Thin material generally has greater ductility and lower transition temperatures than thick material of the same steel. Since normalizing refines the grain structure, thick material improves relatively more with normalizing than does thin material. The improvement is even greater with silicon-aluminum-killed steels.

(W. T. Lankford, Jr., ed., *The Making, Shaping and Treating of Steel*, Association of Iron and Steel Engineers, Pittsburgh, Pa.)

1.25 EFFECTS OF PUNCHING HOLES AND SHEARING

Excessive cold working of exposed edges of structural-steel members can cause embrittlement and cracking and should be avoided. Punching holes and shearing during fabrication are cold-working operations that can cause brittle failure in thick material.

Bolt holes, for example, may be formed by drilling, punching, or punching followed by reaming. Drilling is preferable to punching, because punching drastically coldworks the material at the edge of a hole. This makes the steel less ductile and raises the transition temperature. The degree of embrittlement depends on type of steel and plate thickness. Furthermore, there is a possibility that punching can produce short cracks extending radially from the hole. Consequently, brittle failure can be initiated at the hole when the member is stressed.

Should the material around the hole become heated, an additional risk of failure is introduced. Heat, for example, may be supplied by an adjacent welding operation. If the temperature should rise to the 400 to 850°F range, strain aging will occur in material susceptible to it. The result will be a loss in ductility.

Reaming a hole after punching can eliminate the short, radial cracks and the risks of embrittlement. For that purpose, the hole diameter should be increased from $^1/_{16}$ to $^1/_4$ in by reaming, depending on material thickness and hole diameter.

Shearing has about the same effects as punching. If sheared edges are to be left exposed, $^1/_{16}$ in or more material, depending on thickness, should be trimmed, usually by grinding or machining. Note also that rough machining, for example, with edge planers making a deep cut, can produce the same effects as shearing or punching.

(M. E. Shank, *Control of Steel Construction to Avoid Brittle Failure*, Welding Research Council, New York.)

1.26 EFFECTS OF WELDING

Failures in service rarely, if ever, occur in properly made welds of adequate design. If a fracture occurs, it is initiated at a notchlike defect. Notches occur for various reasons. The toe of a weld may form a natural notch. The weld may contain flaws that act as notches. A welding-arc strike in the base metal may have an embrittling effect, especially if weld metal is not deposited. A crack started at such notches will propagate along a path determined by local stresses and notch toughness of adjacent material.

Preheating before welding minimizes the risk of brittle failure. Its primary effect initially is to reduce the temperature gradient between the weld and adjoining base metal. Thus, there is less likelihood of cracking during cooling and there is an opportunity for entrapped hydrogen, a possible source of embrittlement, to escape. A consequent effect of preheating is improved ductility and notch toughness of base and weld metals, and lower transition temperature of weld.

Rapid cooling of a weld can have an adverse effect. One reason that arc strikes that do not deposit weld metal are dangerous is that the heated metal cools very fast. This causes severe embrittlement. Such arc strikes should be completely removed. The material should be preheated, to prevent local hardening, and weld metal should be deposited to fill the depression.

Welding processes that deposit weld metal low in hydrogen and have suitable moisture control often can eliminate the need for preheat. Such processes include use of low-hydrogen electrodes and inert-arc and submerged-arc welding.

Pronounced segregation in base metal may cause welds to crack under certain fabricating conditions. These include use of high-heat-input electrodes and deposition of large beads at slow speeds, as in automatic welding. Cracking due to segregation, however, is rare for the degree of segregation normally occurring in hot-rolled carbon-steel plates.

Welds sometimes are peened to prevent cracking or distortion, although special welding sequences and procedures may be more effective. Specifications often prohibit peening of the first and last weld passes. Peening of the first pass may crack or punch through the weld. Peening of the last pass makes inspection for cracks difficult. Peening considerably reduces toughness and impact properties of the weld metal. The adverse effects, however, are eliminated by the covering weld layer (last pass).

(M. E. Shank, *Control of Steel Construction to Avoid Brittle Failure*, Welding Research Council, New York; R. D. Stout and W. D. Doty, *Weldability of Steels*, Welding Research Council, New York.)

1.27 EFFECTS OF THERMAL CUTTING

Fabrication of steel structures usually requires cutting of components by thermal cutting processes such as oxyfuel, air carbon arc, and plasma arc. Thermal cutting processes liberate a large quantity of heat in the kerf, which heats the newly generated cut surfaces to very high temperatures. As the cutting torch moves away, the surrounding metal cools the cut surfaces rapidly and causes the formation of a heat-affected zone analogous to that of a weld. The depth of the heat-affected zone depends on the carbon and alloy content of the steel, the thickness of the piece, the preheat temperature, the cutting speed, and the postheat treatment. In addition to the microstructural changes that occur in the heat-affected zone, the cut surface may exhibit a slightly higher carbon content than material below the surface.

The detrimental properties of the thin layer can be improved significantly by using proper preheat, or postheat, or decreasing cutting speed, or any combination thereof. The hardness of the thermally cut surface is the most important variable influencing the quality of the surface as measured by a bend test. Plate chemistry (carbon content), Charpy V-notch toughness, cutting speed, and plate temperature are also important. Preheating the steel prior to cutting, and decreasing the cutting speed, reduce the temperature gradients induced by the cutting operation, thereby serving to (1) decrease the migration of carbon to the cut surface, (2) decrease the hardness of the cut surface, (3) reduce distortion, (4) reduce or give more favorable distribution to the thermally induced stresses, and (5) prevent the formation of quench or cooling cracks. The need for preheating increases with increased carbon and alloy content of the steel, with increased thickness of the steel, and for cuts having geometries that act as high stress raisers. Most recommendations for minimum preheat temperatures are similar to those for welding.

The roughness of thermally cut surfaces is governed by many factors such as (1) uniformity of the preheat, (2) uniformity of the cutting velocity (speed and direction), and (3) quality of the steel. The larger the nonuniformity of these factors, the larger is the roughness of the cut surface. The roughness of a surface is important because notches and stress raisers can lead to fracture. The acceptable roughness for thermally cut surfaces is governed by the job requirements and by the magnitude and fluctuation of the stresses for the particular component and the geometrical detail within the component. In general, the surface roughness requirements for bridge components are more stringent than for buildings. The desired magnitude and uniformity for surface roughness can be achieved best by using automated thermal cutting equipment where cutting speed and direction are easily controlled. Manual procedures tend to produce a greater surface roughness that may be unacceptable for primary tension components. This is attributed to the difficulty in controlling both the cutting speed and the small transverse perturbations from the cutting direction.

(R. L. Brockenbrough and J. M. Barsom, Metallurgy, Chap. 1.1 in *Constructional Steel Design—An International Guide*, R. Bjorhovde et al., eds., Elsevier Science Publishers, New York.)

CHAPTER 2

FABRICATION AND ERECTION*

Thomas Schlafly
Director of Research
American Institute of Steel Construction, Inc.
Chicago, Illinois

Designers of steel-framed structures must be familiar with fabrication and erection practices to provide designs that are practical and cost efficient. Awareness of the process and limits of routine practices will facilitate orderly construction of the project with a minimum of problems and lead to economical design.

2.1 ESTIMATES, MATERIAL ORDERS, AND SHOP DRAWINGS

Structural steel fabricators may be classified as general industry firms. They participate in the construction industry as suppliers, but also share many attributes with manufacturers. They operate fixed facilities with full-time employees hired on a permanent basis, not just for the project. While the successful fabricator considers the flexibility necessary to produce the variety of members anticipated for the type of project furnished, much planning time is spent on setting up the shop for efficient production. Issues such as information flow, material flow and handling, cost reduction of routine tasks, and taking advantage of repetition are fundamental to daily operations of a fabrication shop. Perhaps unusual in general industry is the size of projects in terms of annual sales, the physical size of pieces, and the amount of variation between pieces and projects, along with other conditions involved in construction projects. These all affect the balance of risk and cost against revenue and success.

Successful fabricators strive to distinguish themselves from others with good records of performance, experience with particular types of work, ideas to save money or time, or other attributes to make themselves the preferred provider in their market. An experienced contractor will recognize and reward companies that offer extra attributes of value, but price is usually one of the key factors in selecting a fabricator.

2.1.1 Estimates

One of the needs encountered is the ability to establish the proper cost for a project. The estimating department is the first group in a fabrication firm that considers a project in detail. Realistic estimates are fundamental to initiating successful projects.

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*Revised; originally authored by Charles Peshek, Consulting Engineer, Naperville, Ill., and Richard W. Marshall, Vice President, American Steel Erectors, Inc., Allentown, Pa.

At various stages in the development of a project, a fabricator can provide estimates based on different levels of precision. During the early development stages some fabricators will be willing to give a conceptual estimate using basic statistics about the project. In most cases, the final estimate will be based on a precise **take-off** or listing of the material, a take-off of the work to be done, a calculation of the labor costs to perform that work, and an evaluation of the conditions of the project. A structural steel estimate will include the cost of materials, fasteners, purchased items such as deck and joists, preparation of **detail drawings,** shop labor, inbound and outbound freight, and overheads.

Costs of material will depend on whether mill quantities can be purchased or the material must be purchased from a service center at a higher price. Wide flange shapes are supplied from mills in **bundle quantities** and usually in standard lengths between 40 and 60 ft. Sizes ordered in small quantities or lengths that cannot be obtained economically from standard lengths may increase material costs. The standard material specification for wide flange shapes in building construction, published by the American Society for Testing and Materials as ASTM A992 steel, provides a 50-ksi specified minimum yield stress. The standard material for other shapes and detail plate, ASTM A36 steel, provides a 36-ksi specified minimum yield stress. Where special grades or supplementary requirements must be specified, material costs will be affected.

Time is usually not included in the estimating process to check design dimensions, evaluate each connection against fabrication limitations, and to find and eliminate interferences. Time should be included for unusual pieces and details that demand special attention.

2.1.2 Material Orders

Schedule is usually a primary consideration in steel fabrication. The steel frame is on the critical path of most projects, and there is rarely extra time in the schedule. A steel fabricator starts a project with two major items on the critical path: material acquisition and preparation of shop drawings.

In most cases, a fabricator will generate an advance **bill of material** starting almost immediately after award of the contract. Advance bills of material are even more precise take-offs of the material required for the project than was created for the estimate. Drafters generate the advance bills and send them to the purchasing department. Purchasing sorts the advance bills, grouping like sections and assembling piece sizes into economical sizes for purchase. Material orders are assembled and placed with suppliers that can provide the material economically and on time. This is where small quantities of a size will force the use of higher-price material from a service center. Also, deviations from sizes in stock and unusual grades, or supplementary requirements, may result in the mill supplying material on an extended schedule.

2.1.3 Shop Drawings

At the same time that some drafters are working on the advance bills, others begin the process of creating shop drawings. The more sophisticated designers and drafters of building structures generate design information by creating a three-dimensional model using advanced design software. The information is downloaded to detailers, who use these electronic files with detailing software to generate shop fabrication information.

Neutral file formats are available for data transmission that permit design software to generate information in a format that can be used by detailing packages. The detailing software not only generates drawings, it is also capable of generating numerical control code to operate saws, drills, punches, and thermal cutting and coping machines in the shop. The benefits of this method of design and detailing are time saved, economic effectiveness, skill set requirements that are better suited to the current workforce, and a reduction in errors associated with manual drafting.

Other fabricators and people working with other types of structures may generate shop drawings by hand or use a combination of manual and automated calculation and drafting.

Detailers may be employees of the fabricator or independent contractors. Most fabricators employ some detailers but use independent detailing firms to level the in-house workload. The detailer works from structural design drawings and specifications to obtain member sizes and grades of material, controlling dimensions and all information pertinent to fabrication and erection of the structural frame.

After the detail drawings have been completed, they are checked by an experienced employee (a checker) before being submitted to the engineer for approval. Drawings generated manually should have virtually every depiction and dimension checked. Drawings generated by computer may be checked mainly for input information and selected detail dimensions to assure accuracy. After approval, the drawings are released to the shop for fabrication.

2.2 REQUIREMENTS FOR DRAWINGS

There are essentially two types of detail drawings, erection drawings and shop working drawings. Erection drawings are used by the erector in the field. They consist of line diagrams showing the location and orientation of each member or assembly, called **shipping pieces,** which will be shipped to the construction site. Each shipping piece is identified by a piece mark, which is painted on the member and shown in the erection drawings on the corresponding member. Erection drawings should also show enough of the connection details to guide field forces in their work.

Shop working drawings, simply called **details,** are prepared for every member of a steel structure. All information necessary for fabricating the piece is shown clearly on the detail. The size and location of all holes are shown, as well as the type, size, and length of welds. While shop detail drawings are absolutely imperative in fabrication of structural steel, they are used also by inspectors to ascertain that members are being made as detailed. In addition, the details have lasting value to the owner of the structure in that they show exactly what was constructed, should future alterations or additions be required.

Design and detail drawings may be considered as a complex but important form of communication. The design drawings need to communicate clearly to the detailer, to avoid delays inherent in requests for information, to avoid revisions necessitated by approval comments, and to avoid errors.

The most critical details are usually for connections. Connection design requires knowledge of design loads, how forces are transferred through the structure, and calculated resistance of elements, fasteners, and welds. It is important to know shop capability, limitations, and potential for fabrication economy. Many fabricators can economically provide connections suited to the equipment and practices in their shops. Others desire as much detail information as possible. It is considered best practice to provide general configuration and loads for common connection types and precise detail for unusual or difficult connections. Awareness of the connection requirements is valuable for the designer because there are cases in which member selection should be adjusted to provide room for connections. In any case, the fabricator may request adjustment to accommodate shop limitations or economic improvements. Where seismic loads are involved, the designer must provide all the necessary detail to assure the building meets code requirements, including sizes of connection elements.

The following is a guide to information that should be provided by the designer on design drawings.

Simple Beam Connections. Reactions should be shown. Defining the reaction as a function of the capacity of the beam causes problems when beams are selected for reasons other than strength, such as stiffness, uniformity with other members, fitting detail, and attachment. If reactions are defined generically in terms of beam capacity, reactions greater than that standard must be given, and those significantly lower should be also. Horizontal forces (longitudinal transfer or drag forces) must be given as well.

Moment Connections. Relative to simple connections, moment connections require extra labor, either in the shop or the field or both. Moment connections can be made by welding the flanges to the columns (and welding the webs to the columns for seismic loads), by bolting to welded connection plates, or with flange-welded end plates. In any of these configurations, large flange forces are resisted by welds or bolts or both. Proper selection of the connection configuration depends on the geometry of the frame, the size of the members, and regional practices and skill sets. Forces and moments must be shown unless all connection details are shown.

Braced Connections. Forces in the braces and the beams must be known in order to size connections for bracing. Unnecessary work can be required if the connection designer does not know how loads are transferred between beams and braces, and this is not always evident when only the maximum loads from all of the load cases are shown. Therefore, it may be necessary to show the maximum loads from each critical load case.

Welds. The American Institute of Steel Construction's "Specification for Structural Steel Buildings," particularly Chap. J, and the American Welding Society's "Structural Welding Code—Steel," AWS D1.1, both contain provisions governing welding in structural frames. AWS D1.1 contains many provisions that require input from the engineer. Section 1 contains a list of engineer responsibilities and Sec. 2 defines contract plan and specification requirements. Plans and specifications include complete information regarding base-metal specification, location, type, size and extent of all welds. If the engineer requires certain welds to be performed in the field, they must be designated. Special requirements such as nondestructive testing and notch toughness must be shown. If the engineer sizes any welds, the filler-metal strength classification must be shown. Alternatively, if reactions are given, the detailer will be able to develop weld sizes and configurations.

Fasteners. The fastener specification must be shown on design drawings. When specifying high-strength bolts, designers must also indicate the type of connection to be used: snug-tight bolts (the typical and most economical choice), fully tightened bearing bolts, or slip-critical bolts. Any connections specifically intended to slip, such as slotted or oversize holes, must also be indicated.

Tolerances Defined. Tolerances and provisions for adjustment should be considered together to result in a structure that meets the user's needs. Provisions for adjustment may entail extra costs, but may still be less costly than special tight tolerances, and may be implemented more reliably. Standard tolerances for building structures are defined in the AISC "Code of Standard Practice for Buildings and Bridges." For information on tolerances for highway bridge construction, refer to the "LRFD Bridge Design Specifications" and "Standard Specifications for Highway Bridges," both published by the American Association of State Highway and Transportation Officials (AASHTO). Structural steel tolerances provide for particular construction processes involved. Factors that contribute to tolerances include mill rolling (defined in ASTM A6, "General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use"), welding (defined in AWS D1.1 or D1.5), and anchor placement, fabrication and, erection (defined in the AISC "Code of Standard Practice").

Control of Tolerances. Control demands planning, expertise, and adjustment of manufacturing processes. Construction loads, residual stresses from rolling, welding shrinkage stresses, ambient temperature effects, and construction loads may all have an effect on final fit and shape. Control of tolerances may depend on the type of shape, size of the pieces, number of connections on a piece, amount of welding, stiffness of the pieces, and whether there is adjustment in the erected structure. For example, a wide-flange shape is easily connected to attachments on both ends, whereas a fabricated box section having end connections and intermediate connections will present issues. The box section may exhibit weld distortion and will be stiffer in torsion than a wide-flange shape by orders of magnitude. Control of tolerances of a fabricated box will demand special fabrication practices such as constraints in fabrication or milling of attachment points. Controlling welding sequences can improve tolerances but is difficult to predict. Provision for adjustment is often a more practical method to accommodate fabrication and erection tolerances. Other issues to consider with regard to tolerances include the relationship between theoretical points on different floors (the AISC "Code of Standard Practice" directs the contractor to establish reference points on each floor); movement of the structure due to temperature and construction loads (it is not unusual for points in a large structure to move $1\frac{1}{2}$ inches in a day); and possibly large numbers of pieces between points, which demand a controlled relationship.

Special Material Requirements. There are occasions when special material requirements are appropriate, and there are provisions in specifications for these requirements. ASTM material specifications include supplementary requirements. When such special requirements are necessary, they must be shown on contract documents. The AISC "Specification for Structural Steel Buildings" requires alternate core toughness defined in ASTM A6, "Supplementary Requirement S30," when the pieces have flanges more than 2 in thick ($1\frac{1}{2}$ in for seismic) and are connected with complete-joint-penetration (CJP) welds fusing through the thickness of the flange. Also, frame members exposed to low temperature and subject to large live loads may demand an enhanced level of toughness (see ASTM A6, "Supplemental Requirement S5").

Clearances and Interferences. In addition to dimensioning pieces to fit, pieces must be designed and detailed with consideration of a path for assembly in the field and with access for fasteners and tools necessary to make connections. Welds and bolt holes should be visible. Bolts require a clear length equal to the length of the bolt on one side of the connection for entry, and room for the wrench on one side. When tension control bolts are used, room for bolt entry and the wrench have to be on opposite sides of the connection. Access for welds should be provided so that the welder can aim the electrode normal to the surface of the completed weld. Steel erection is very labor intensive, and a clear path permitting the placement of pieces with a simple movement and at least $\frac{1}{2}$ in clearance is essential to efficient work. Tucking ends of pieces between flanges and reversing direction to final placement is time consuming. When stiffeners or other details block entry of a connection, the connection should be extended beyond the interfering detail so the mating piece can be erected without performing dangerous maneuvers in the air.

Shop Drawing Approval. When shop drawings have been completed and checked, they are submitted to the engineer for review or approval. Responsibility for specific information on shop drawings is the subject of ongoing controversy. In general, the engineer reviews the shop drawings to see that the drafter has interpreted the design requirements properly.

2.3 FABRICATION PRACTICES AND PROCESSES: MATERIAL PREPARATION

Steel fabrication shops are most commonly organized into departments such as receiving, detail material, main material cut and preparation, assembly fit and fasten, and shipping. Many shops also have paint departments. Material is received by trucks or by rail, off-loaded, compared to order requirements, and stored by project or by size and grade. Material is received from the mill or warehouse as individual pieces or in bundles. The pieces or bundles are identified with the size specification grade and heat number. Identity of the material in storage is maintained by segregating the material by grade, keeping the material in identified bundles or stacks, or marking the material. When the material is cut to be processed into a specific piece in the structure, it is marked with a piece mark and may be further labeled with the specification and grade. Any material left after cutting is marked with the grade and returned to the receiving yard. Material handling is a major consideration in the structural shop, and organized storage is key to reducing handling.

Detail material consists of the usually small parts that the shop attaches to main pieces to become shipping pieces. To keep the flow of work in the shop consistent, the detail material is prepared just before the main material, so it is ready for assembly when the large pieces are ready for assembly. Where it is feasible, fabricators create standard pieces such as standard clip angles to enhance repetition and permit work to be performed in reasonable groups. Angle shapes are usually processed in machines that automatically feed, punch holes or slots in both legs, and shear the angle in a single operation. Methods for processing plate vary with the size and shape of the final piece and the equipment in the shop. When large numbers of plates of the same narrow width are to be made, the fabricator buys bar to avoid extra cutting. Plates can be sawn but are most often sheared or thermal cut.

Thermal cutting offers the ability to create plate of any useful shape, including reentrant corners. It also offers the ability to bevel the edges as for groove welds and cut holes. Thermal cutting includes flame and plasma cutting. Laser cutting is also included but is not yet prevalent in structural fabrication shops. Lasers offer good cut-surface profiles and little thermal distortion but are currently expensive and limited in the thicknesses they can handle. Flame cutting may be manual or mechanically guided or automated. AISC encourages the use of a guide where practical. A guide can be as simple as a bar clamped to the work surface. Automation in flame and plasma cutting include torches mounted on self-propelled buggies and mounted on tracks. Tracks can be rigid for straight cuts or flexible for curves. Cutting tables are used to strip long lengths of plate to the needed width. Tables can be fitted with six or more torches to cut one plate into many pieces at one time. Tables can be fitted with devices that trace templates and copy the pattern onto the plate. Much more common in recent times are tables of various sizes that are numerically controlled. These burning tables can be coordinated with numerically controlled punches or drills.

In the flame-cutting process, the torch burns a mixture of oxygen and gas to bring the steel at the point where the cut is to be made to a preheat temperature of about 1600°F. At that temperature, the steel has a great affinity for oxygen. The torch then releases pure oxygen under pressure through the cutting tip. This oxygen combines immediately with the steel. As the torch moves along the cut line, the oxidation, coupled with the erosive force of the oxygen stream, produces a cut about $^1/_8$ in wide. Once cutting begins, the heat of oxidation helps to heat the material. Structural steel of certain grades and thicknesses may require additional preheat. In those cases, flame is directed to the metal ahead of the cut.

In such operations as stripping plate-girder flange plates, it is desirable to flame-cut both edges of the plate simultaneously. This limits distortion by imposing shrinkage stresses of approximately equal magnitude in both edges of the plate. For this reason, plates to be supplied by a mill for multiple cutting are ordered with sufficient width to allow a flame cut adjacent to the mill edges. It is not uncommon to strip three flange plates at one time using four torches.

Plasma-arc cutting is an alternative process for steel fabrication. A tungsten electrode may be used, but hafnium is preferred because it eliminates the need for expensive inert shielding gases. Advantages of this method include faster cutting, easy removal of dross, and lower operating cost. Disadvantages include higher equipment cost, limitation of thickness of cut to $1^1/_2$ in, slightly beveled edges, and a wider kerf. Plasma is advantageous for stainless steels that cannot be cut with oxyfuel torches.

Shearing is used in the fabricating shop to cut certain classes of plain material to size. Several types of shears are available. **Guillotine-type shears** are used to cut plates of moderate thickness. Some plate shears, called **rotary-plate shears,** have a rotatable cutting head that allows cutting on a bevel. **Angle shears** are used to cut both legs of an angle with one stroke. **Rotary-angle shears** can produce beveled cuts.

Cutting to length of main material shapes can be done by the steel producer, a service center, or a processor, but is most often done by the fabricator. Steel mills cut shapes with high-speed friction saws that are fast enough to cut the strand into separate lengths as the strand comes off the rolls. The cut surface is suitable for some applications but not for others. Structural fabrication shops cut main material shapes to length with thermal cutting or with saws. Band saws and cold saws are most common, but machine hacksaws and friction saws can be used. The choice of saw depends on the size being cut and the application requirements. Band and cold saws can produce cuts suitable and accurate enough for most applications, including bearing surfaces of columns, without further machining. The suitability of a cut for a particular application depends on maintenance of blades and the way the saw is set up.

Bolt holes can be drilled, punched, or, in some cases, cut thermally. The three methods each have different effects on the surrounding material. Strength limit states are established for structures subject to static loads and seismic loads such that any hole-forming method can be used. AASHTO requires drilled or subpunched and reamed holes in main member connections. The selection of the hole-forming method to be used depends on the thickness of the material and the hole size, the number of pieces with identical hole patterns, and the other operations that have to be performed on the material. In the abstract general case, punching is fast but is limited in the thickness that can be punched. AISC used to limit punching to the diameter of the hole plus $^1/_8$ in. While that limit no longer exists in the specification, because of advances in capabilities of available equipment, it is still a practical rule of thumb for routine use. Practically, punching is limited by the capacity of the machine and the punch itself relative to the ultimate strength of the material at the hole perimeter. Punching is a form of shearing whereby the material is sheared in a ductile fashion until the punch reaches a place where the remaining material fractures. A punched hole is slightly conical in shape, with dimensions of the punch on one side and of the die on the other.

On an individual basis, drilling is much slower than punching but leaves a smooth surface and is not limited by thickness. Another advantage of drilling is that it can be done on multiple thicknesses of material. This is done when there are many pieces with the same hole pattern, or to assure uniformity of the hole pattern between mating surfaces. In high-production environments, both drills and punches exist that form many holes at once. Automated lines feed shapes and drill or punch holes in both flanges and the web at the same time. Punches can be set up to form many holes at once when there are many pieces with the same pattern. Thermal cutting of holes was not permitted for many years but is now allowed when the hole is formed accurately and with an appropriate surface quality. Holes in base plates for anchor rods are commonly cut thermally because base plates are thick and holes are larger than those for bolts.

Camber is a curvature of a piece in its strong direction or the direction in which the primary load is applied. **Sweep** is a curvature in the weak direction. The term *camber* refers to the curvature of a piece as it is delivered, or to the curvature induced to compensate for deflection under applied loads. Hot-rolled shapes are air cooled without physical restraint. When a hot-rolled shape cools, it bends in both the strong and weak direction, twists, and distorts locally. ASTM A6 limits the amount of sweep and camber acceptable in a rolled shape, and producers straighten the product to meet those tolerances. Straightening is done with a rotary straightener or in a press. The delivered product usually has some camber and sweep within the A6 tolerances. When no camber is specified in rolled shapes fabricated into beams, the shape is fitted in fabrication so that the natural camber is up in the piece as erected. In long-span floors typical of commercial buildings, camber is a design parameter and is intentionally induced in the beam. Camber is designed as a percentage of the dead-load deflection or a percentage of the dead- and live-load deflection of the beam. Natural mill camber is acceptable if the design camber is $^3/_4$ in or less. The decision to camber and how much to camber is made by the designer.

Where camber is required, the fabrication shop cambers the shape after it is cut and punched or drilled, and before detail material is attached. Camber can be achieved with local application of heat, and this is typically done for heavy sections. For most beams, however, camber is induced by cold bending with hydraulic jacks in a cambering machine. Cold camber is achieved by inducing plastic tensile and compressive strain of elements of the shape. The curvature should be limited so that the induced strain is limited to a reasonable percentage, based on the minimum elongation requirements of the material specification. Also, the strain should be induced uniformly over a length of the section using multiple jacks.

In girders, webs may be cut thermally with a curve calculated to achieve the required camber when the flanges have been welded to the web. Variations do occur, and camber adjustments are made using local application of heat after the piece is assembled. Large bridge and roof trusses are cambered by detailing and fabricating the elements to calculated lengths such that the desired camber is achieved when the trusses are assembled. In other words, each member is fabricated to its geometric length in the cambered position.

Heat is used to induce camber or sweep in some cases and to adjust camber or straighten pieces in other cases. There are a variety of specific techniques used to heat-camber beams, but in all of them the side to be shortened is heated with a torch. As the part is heated, it tries to elongate. Because it is restrained by unheated material, the heated part with reduced yield stress is forced to upset (increase inelastically in thickness) to relieve its compressive stress. Since the increase in thickness is inelastic, the part will not return to its original thickness upon cooling. When the part is allowed to cool, therefore, it must shorten to return to its original volume. The heated flange thus experiences a net shortening that produces the camber. Heat cambering is generally slow and expensive and is typically used in sections larger than the capacity of available equipment. Heat can also be used to straighten or eliminate warping from parts. Some of these procedures are quite complex and intuitive, demanding experience on the part of the operator.

Research has shown that the residual stresses remaining in a beam after cambering are little different from those due to differential cooling rates of the elements of the shape after it has been produced by hot rolling. Strength limit states in design specifications include the effect of residual stresses where relevant.

2.4 FABRICATION PRACTICES AND PROCESSES: ASSEMBLY, FITTING, AND FASTENING

The work in a well-run shop should flow from one activity or work station to the next. The main material is prepared substantially in groups that are either efficient to build or that form erectable units, divisions, or sequences. When the main material is ready, it is sent to fabrication stations where the detail material is ready and in place for assembly to the shipping piece. Columns, beams, and braces in which a rolled shape is the main piece are individually fit and fastened. Repetitive similar trusses are assembled in jigs or fixtures that speed assembly and ensure uniformity. Built-up sections such as boxes and three-plate girders are assembled using spreaders, braces, and stiffeners that are sometimes left as part of the piece. Shipping pieces are assembled by a skilled worker called a **fitter** or **fabricator.** Bolting is usually performed by the fabricator. Welding may be done by the fabricator or by a separate welder.

2.4.1 Bolting

Most field connections are made by bolting, either with high-strength bolts (ASTM A325 or A490) or with ordinary machine bolts (A307 bolts), depending on strength requirements. Shop connections frequently are welded but may use these same types of bolts. When high-strength bolts are used, the connections should satisfy the requirements of the "Specification for Structural Joints Using ASTM A325 or A490 Bolts," approved by the Research Council on Structural Connections (RCSC) of the Engineering Foundation. Joints with high-strength bolts are generally designed either as bearing-type or slip-critical connections (see Chap. 3 and Art. 5.9). Some joints may be specially designed to slip, such as by using oversized holes or slots.

Bearing-type connections have a higher allowable load or design strength than slip-critical connections. Slip-critical connections always must be fully tightened to specified minimum values. Bearing-type connections may be either "snug tight" or fully tightened, depending on the type of connection and service conditions. Snug tight is defined as the tightness attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the connected plies into firm contact. AISC specifications for structural steel buildings require fully tensioned high-strength bolts (or welds) for certain connections (see Art. 5.9.1). The AASHTO specifications require slip-critical joints in bridges where slippage would be detrimental to the serviceability of the structure, including joints subjected to fatigue loading or significant stress reversal. In all other cases, connections may be made with "snug-tight" high-strength bolts or A307 bolts, as may be required to develop the necessary strength.

Pretensioned bolts are tightened to 70% of the tensile strength of the bolt (see Art. 5.9.5). The RCSC recognizes four methods of tightening bolts: turn-of-the-nut tightening, use of tension-control bolts, use of direct tension indicators, and calibrated wrench tightening. All of the methods depend on the installer first bringing the bolts to the snug-tight condition and then tightening them in a pattern from the most rigid to the least rigid part of the connection. This procedure is intended to prevent initially tightened bolts from coming loose when subsequent bolts are tightened.

Turn-of-the-nut tightening requires the installer to install the bolts to the snug condition and then rotate the nut relative to the bolt by an amount specified by the RCSC. In critical connections, the nut and end of the bolts are marked in the snug condition and the installer and inspector can confirm visually that the nut has been rotated the proper amount after snug tightening.

Tension-control (TC) bolts are manufactured with a spline on the end. The installation tool applies a torque to the bolt using the spline and the nut. A groove between the spline and the threaded part of the bolt is calibrated to shear off when the bolt achieves the proper tension. Since the performance of the bolt depends on the correlation between the torque on the bolt and the tension in the bolt, the condition of the bolt and nut are important. TC bolts, as with the other types, must be stored to prevent degradation of the threads or lubricant.

Direct tension indicators (DTIs), sometimes referred to as load-indicating washers or LIWs, are manufactured with profiles that deform under bolt tension. The geometry of the deformed LIW can be measured to assure that the bolt has reached the required tension.

Calibrated wrench tightening depends on establishment of a daily project torque and using impact wrenches adjusted to that torque.

Inspection of structural bolting is done by observation during installation. Inspection of bolt installation after the work is done is considered unreliable and tends to cause conflict.

2.4.2 Welding

Welding in the fabrication and erection of building structures is governed by the AISC "Specification for Structural Steel Buildings" and the AWS "Structural Welding Code—Steel" (AWS D1.1). Welding of bridges is governed by the Bridge Welding Code, a joint publication of AASHTO and AWS (AASHTO/AWS D1.5). Other codes that may be used in steel structures are AWS D1.3 for sheet, AWS D1.4 for reinforcing bars, and AWS D1.6 for stainless steel. Owners and engineers can supplement the requirements of AWS to meet specific needs and experience.

The number of variables included, the skill levels necessary, and the quality demands on welded joints require that welding be done as a controlled process. The fundamental scheme for that control is to have qualified personnel use materials, joint designs, and procedure variables that have been proven effective in combination, through tests. All welds must be inspected visually. Tests are expensive and many of the material, joint, and procedure combinations have been used extensively and are therefore considered prequalified. In summary, welders who have been qualified by test use weld procedures that have been qualified by test or are prequalified. However, there are no prequalified procedures in the "Bridge Welding Code," or for reinforcing bar.

The use of prequalified procedures is desirable and prevalent in the fabrication of buildings. Procedures are prequalified for four welding processes: shielding metal arc welding (SMAW), flux-cored arc welding (FCAW), gas metal arc welding (GMAW), and submerged arc welding (SAW). Other processes such as electroslag welding (ESW) can be used, but the procedures must be qualified by test. Procedure prequalification demands the use of certain materials and a limited set of joint geometries and electrical variables. The complete requirements are described in Chap. 3 of AWS D1.1. The AISC "Manual of Steel Construction" also includes the joint geometries. Materials approved for use by the AISC are included by the AWS in the list of prequalified materials. Filler metals meeting the requirements of the appropriate AWS A5 series specifications and listed in Chap. 3 of AWS D1.1, and Chap. A of the AISC "Specification for Structural Steel Buildings" can be used. Electrical variables must be within the filler-metal manufacturer's recommendations. Weld procedure specifications must be written, whether they are qualified by test or prequalified.

Shielded metal arc welding (SMAW) produces coalescence, or fusion, by the heat of an electric arc struck between a coated metal electrode and the material being joined, or **base metal.** The electrode supplies filler metal for making the weld, gas for shielding the molten metal, and flux for refining this metal. This process is commonly known also as **manual, hand,** or **stick welding.** Pressure is not used on the parts to be joined.

When an arc is struck between the electrode and the base metal, the intense heat forms a small molten pool on the surface of the base metal. The arc also decomposes the electrode coating and melts the metal at the tip of the electrode. The electron stream carries this metal in the form of fine globules across the gap and deposits and mixes it into the molten pool on the surface of the base metal. (Since deposition of electrode material does not depend on gravity, arc welding is feasible in various positions, including overhead.) The decomposed coating of the electrode forms a gas shield around the molten metal that prevents contact with the air and absorption of impurities. In addition, the electrode coating promotes electrical conduction across the arc, helps stabilize the arc, adds flux, slag-forming materials, to the molten pool to refine the metal, and provides materials for controlling the shape of the weld. In some cases, the coating also adds alloying elements. As the arc moves along, the molten metal left behind solidifies in a homogeneous deposit, or weld.

The electric power used with shielded metal arc welding may be direct or alternating current. With direct current, either straight or reverse polarity may be used. For straight polarity, the base metal is the positive pole and the electrode is the negative pole of the welding arc. For reverse polarity, the base metal is the negative pole and the electrode is the positive pole. Electrical equipment with a welding-current rating of 400 to 500 A is usually used for structural steel fabrication. The power source may be portable, but the need for moving it is minimized by connecting it to the electrode holder with relatively long cables.

The size of electrode (core wire diameter) depends primarily on joint detail and welding position. Electrode sizes of $^1/_8$, $^5/_{32}$, $^3/_{16}$, $^7/_{32}$, $^1/_4$, and $^5/_{16}$ in are commonly used. Small-size electrodes are 14 in long, and the larger sizes are 18 in long. Deposition rate of the weld metal depends primarily on welding current. Hence, use of the largest electrode and welding current consistent with good practice is advantageous.

About 57% to 68% of the gross weight of the welding electrodes results in weld metal. The remainder is attributed to spatter, coating, and stub-end losses.

Shielded metal arc welding is widely used for manual welding of low-carbon steels, such as A36, and HSLA steels, such as A572 and A588. Though stainless steels, high-alloy steels, and nonferrous metals can be welded with this process, they are more readily welded with the gas metal arc process.

Submerged arc welding (SAW) produces coalescence by the heat of an electric arc struck between a bare metal electrode and the base metal. The weld is shielded by flux, a blanket of granular fusible material placed over the joint. Pressure is not used on the parts to be joined. Filler metal is obtained either from the electrode or from a supplementary welding rod.

The electrode is pushed through the flux to strike an arc. The heat produced by the arc melts adjoining base metal and flux. As welding progresses, the molten flux forms a protective shield above the molten metal. On cooling, this flux solidifies under the unfused flux as a brittle slag that can be removed easily. Unfused flux is recovered for future use. About 1.5 lb of flux is used for each pound of weld wire melted.

Submerged arc welding requires high currents. The current for a given cross-sectional area of electrode often is as much as 10 times as great as that used for manual welding. Consequently, the deposition rate and welding speeds are greater than for manual welding. Also, deep weld penetration results. Consequently, less edge preparation of the material to be joined is required for submerged-arc welding than for manual welding. For example, material up to $^3/_8$ in thick can be groove-welded, without any preparation or root opening, with two passes, one from each side of the joint. Complete fusion of the joint results.

Submerged arc welding may be done with direct or alternating current. Conventional welding power units are used but with larger capacity than those used for manual welding. Equipment with current ratings up to 4000 A is used.

The process may be completely automatic or semiautomatic. In the semiautomatic process, the arc is moved manually. One-, two-, or three-wire electrodes can be used in automatic operation, two being the most common. Only one electrode is used in semiautomatic operation.

Submerged arc welding is widely used for welding low-carbon steels and HSLA steels. Though stainless steels, high-alloy steels, and nonferrous metals can be welded with this process, they are generally more readily welded with the gas-shielded metal-arc process.

Gas metal arc welding (GMAW) produces coalescence by the heat of an electric arc struck between a filler-metal electrode and base metal. Shielding is obtained from a gas or gas mixture (which may contain an inert gas) or a mixture of a gas and flux.

This process is used with direct or alternating current. Either straight or reverse polarity may be employed with direct current. Operation may be automatic or semiautomatic. In the semiautomatic process, the arc is moved manually.

As in the submerged arc process, high current densities are used, and deep weld penetration results. Electrodes range from 0.020 to $^1/_8$ in diameter, with corresponding welding currents of about 75 to 650 A.

Practically all metals can be welded with this process. It is superior to other presently available processes for welding stainless steels and nonferrous metals. For these metals, argon, helium, or a mixture of the two gases is generally used for the shielding gas. For welding of carbon steels, the shielding gas may be argon, argon with oxygen, or carbon dioxide. Gas flow is regulated by a flowmeter. A rate of 25 to 50 ft^3/h of arc time is normally used.

Flux-cored arc welding (FCAW) is similar to the GMAW process except that a flux-containing tubular wire is used instead of a solid wire. The process is classified into two subprocesses, self-shielded and gas-shielded. Shielding is provided by decomposition of the flux material in the wire.

In the gas-shielded process, additional shielding is provided by an externally supplied shielding gas fed through the electrode gun. The flux performs functions similar to the electrode coatings used for SMAW. The self-shielded process is particularly attractive for field welding because the shielding produced by the cored wire does not blow off in normal ambient conditions and heavy gas supply bottles do not have to be moved around the site.

Electroslag welding (ESW) produces fusion with a molten slag that melts filler metal and the surfaces of the base metal. The weld pool is shielded by this molten slag, which moves along the entire cross section of the joint as welding progresses. The electrically conductive slag is maintained in a molten condition by its resistance to an electric current that flows between the electrode and the base metal.

The process is started much like the submerged arc process by striking an electric arc beneath a layer of granular flux. When a sufficiently thick layer of hot molten slag is formed, arc action stops. The current then passes from the electrode to the base metal through the conductive slag. At this point, the process ceases to be an arc welding process and becomes the electroslag process. Heat generated by resistance to flow of current through the molten slag and weld puddle is sufficient to melt the edges at the joint and the tip of the welding electrode. The temperature of the molten metal is in the range of 3500°F. The liquid metal coming from the filler wire and the molten base metal collect in a pool beneath the slag and slowly solidify to form the weld. During welding, since no arc exists, no spattering or intense arc flash occurs.

Because of the large volume of molten slag and weld metal produced in electroslag welding, the process is generally used for welding in the vertical position. The parts to be welded are assembled with a gap 1 to 1¼ in wide. Edges of the joint need only be cut squarely, by either machine or flame.

Water-cooled copper shoes are attached on each side of the joint to retain the molten metal and slag pool and to act as a mold to cool and shape the weld surfaces. The copper shoes automatically slide upward on the base-metal surfaces as welding progresses.

Preheating of the base metal is usually not necessary in the ordinary sense. Since the major portion of the heat of welding is transferred into the joint base metal, preheating is accomplished without additional effort.

The electroslag process can be used to join plates from 1¼ to 18 in thick. The process cannot be used on heat-treated steels without subsequent heat treatment. AWS and other specifications prohibit the use of ESW for welding quenched-and-tempered steel or for welding dynamically loaded structural members subject to tensile stresses or to reversal of stress. However, research results currently being introduced on joints with narrower gaps should lead to acceptance in cyclically loaded structures.

Electrogas welding (EGW) is similar to electroslag welding in that both are automatic processes suitable only for welding in the vertical position. Both utilize vertically traveling, water-cooled shoes to contain and shape the weld surface. The electrogas process differs in that once an arc is established between the electrode and the base metal, it is continuously maintained. The shielding function is performed by helium, argon, carbon dioxide, or mixtures of these gases continuously fed into the weld area. The flux core of the electrode provides deoxidizing and slagging materials for cleansing the weld metal. The surfaces to be joined, preheated by the shielding gas, are brought to the proper temperature for complete fusion by contact with the molten slag. The molten slag flows toward the copper shoes and forms a protective coating between the shoes and the faces of the weld. As weld metal is deposited, the copper shoes, forming a weld pocket of uniform depth, are carried continuously upward.

The electrogas process can be used for joining material from ½ to more than 2 in thick. The process cannot be used on heat-treated material without subsequent heat treatment. AWS and other specifications prohibit the use of EGW for welding quenched-and-tempered steel or for welding dynamically loaded structural members subject to tensile stresses or to reversal of stress.

Stud welding produces coalescence by the heat of an electric arc drawn between a metal stud or similar part and another work part. When the surfaces to be joined are properly heated, they are brought together under pressure. Partial shielding of the weld may be obtained by surrounding the stud with a ceramic ferrule at the weld location.

Stud welding usually is done with a device, or gun, for establishing and controlling the arc. The operator places the stud in the chuck of the gun with the flux end protruding. Then the operator places the ceramic ferrule over this end of the stud. With timing and welding-current controls set, the operator holds the gun in the welding position, with the stud pressed firmly against the welding surface, and presses the trigger. This starts the welding cycle by closing the welding-current contactor. A coil is activated to lift the stud enough to establish an arc between the stud and the welding surface. The heat melts the end of the stud and the welding surface. After the desired arc time, a control releases a spring that plunges the stud into the molten pool.

Direct current is used for stud welding. A high current is required for a very short time. For example, welding currents up to 2500 A are used with arc time of less than 1 sec for studs up to 1 in diameter.

(O. W. Blodgett, *Design of Welded Structures*, James F. Lincoln Arc Welding Foundation, Cleveland, Ohio.) See also Arts. 5.9.1 to 5.9.4.

2.5 SHOP ASSEMBLY

When the principal operations on a main member, such as punching, drilling, and cutting, are completed, and when the detail pieces connecting to it are fabricated, all the components are brought together to be fitted up, that is, temporarily assembled with fit-up bolts, clamps, or tack welds. At this time, the member is inspected for dimensional accuracy, squareness, and, in general, conformance with shop detail drawings. Misalignment of attachments and holes should be detected at this time and corrections made before the piece is completed and shipped.

The foregoing type of shop assembly and fit-up is an ordinary shop practice, performed routinely in virtually all work. There are other classes of fit-up, however, that may be performed on some work. Sequential "lay downs," discussed below, or vertical, three-dimensional shop assembly may be required on some large complex pieces. This type of assembly usually includes drilling of holes for the mating connections in the assembled position. These assembly methods may be performed at the option of the fabricator but are not routine practices. If the designer or contractor deems that one of these assembly methods is necessary, it must be included in the contract documents. These assembly methods are commonly required on large bridge girders.

Assembly is helpful for pieces that are too large to "drift" during erection, where pieces are so large that normal shop tolerances might accumulate to inhibit connection on the site, where pieces are large enough to prevent the use of routine erection practices when minor fit-up problems occur, and where elastic deflections during erection can make "pinning" of connections difficult. When those conditions exist, as with large bridge girders, lay-down or vertical assembly in the shop may be specified.

Shop assembly is expensive and requires the extended use of valuable space. Accuracy of hole placement within a hole pattern, which at one time was a cause for assembly, is less of an issue with the automated drilling equipment used in contemporary shops. However, the distortion in large pieces or the complexity of pieces that mate at many points may still demand the extra precaution provided by shop assembly.

Where assembly and drilling in position are required, the following guidelines apply.

Splices in bridge girders are commonly drilled assembled. Alternatively, the abutting ends and splice material may be drilled to templates independently.

Lay-downs are the assembly of three or more pieces that will mate in the field into position relative to one another but are laid flat on supports on the shop floor. Splice materials are positioned between the connecting pieces, and the holes are drilled through the splice material and the main members while the pieces are in this position. Lay-downs are used where the main members will be in or near a plane in their final position, such as girders in a line. Where the main pieces are not in or near a plane, such as with a cross girder mating to stringers, a vertical assembly is used. In this case, pieces are placed in position relative to each other and usually nearly in their correct final orientation. Vertical assemblies can be reoriented to minimize blocking in the shop and keep the pieces accessible for work.

For reaming truss connections, three methods are in use in fabricating shops. The particular method to be used on a job is dictated by the project specifications or the designer.

Associated with the reaming methods for trusses is the method of cambering trusses. Highway and railroad bridge trusses are cambered by increasing the geometric (loaded) length of each compression member and decreasing the geometric length of each tension member by the amount of axial deformation it will experience under load (see Art. 2.13).

Method 1 (RT, or Reamed-Template, Method). All members are reamed to geometric angles (angles between members under load) and cambered (no-load) lengths. Each chord is shop-assembled and reamed. Web members are reamed to metal templates. The procedure is as follows:

With the bottom chord assembled in its loaded position (with a minimum length of three abutting sections), the field connection holes are reamed. (**Section,** as used here and in methods 2 and 3, means fabricated member. A chord section, or fabricated member, usually is two panels long.)

With the top chord assembled in its loaded position (with a minimum length of three abutting sections), the field connection holes are reamed.

The end posts of heavy trusses are normally assembled and the end connection holes reamed, first for one chord and then for the other. The angles between the end post and the chords will be the geometric angles. For light trusses, however, the end posts may be treated as web members and reamed to metal templates.

The ends of all web members and their field holes in gusset plates are reamed separately to metal templates. The templates are positioned on the gusset plates to geometric angles. Also, the templates are located on the web members and gusset plates so that when the unloaded member is connected, the length of the member will be its cambered length.

Method 2 (Gary or Chicago Method). All members are reamed to geometric angles and cambered lengths. Each chord is assembled and reamed. Web members are shop-assembled and reamed to each chord separately. The procedure is as follows:

With the bottom chord assembled in its geometric (loaded) alignment (with a minimum number of three abutting sections), the field holes are reamed.

With the top chord assembled in its geometric position (with a minimum length of three abutting sections), the holes in the field connections are reamed.

The end posts and all web members are assembled and reamed to each chord separately. All members, when assembled for reaming, are aligned to geometric angles.

Method 3 (Fully Assembled Method). The truss is fully assembled, then reamed. In this method, the bottom chord is assembled and blocked into its cambered (unloaded) alignment, and all the other members are assembled to it. The truss, when fully assembled to its cambered shape, is then reamed. Thus the members are positioned to cambered angles, not geometric angles.

When the extreme length of trusses prohibits laying out the entire truss, method 3 can be used sectionally. For example, at least three abutting complete sections (top and bottom chords and connecting web members) are fully assembled in their cambered position and reamed. Then complete sections are added to and removed from the assembled sections. The sections added are always in their cambered position. There should always be at least two previously assembled and reamed sections in the layout. Although reaming is accomplished sectionally, the procedure fundamentally is the same as for a full truss assembly.

In methods 1 and 2, field connections are reamed to cambered lengths and geometric angles, whereas in method 3, field connections are reamed to cambered lengths and angles. To illustrate the effects of these methods on an erected and loaded truss, Fig. 2.1*a* shows by dotted lines the shape of a truss that has been reamed by either method 1 or 2 and then fully connected, but without load. As the members are fitted up (pinned and bolted), the truss is forced into its cambered position. Bending stresses are induced into the members because their ends are fixed at their geometric (not cambered) angles. This bending is indicated by exaggerated S curves in the dotted configuration. The configuration shown in solid lines in Fig. 2.1*a* represents the truss under the load for which the truss was cambered. Each member now is strained; the fabricated length has been increased or decreased to

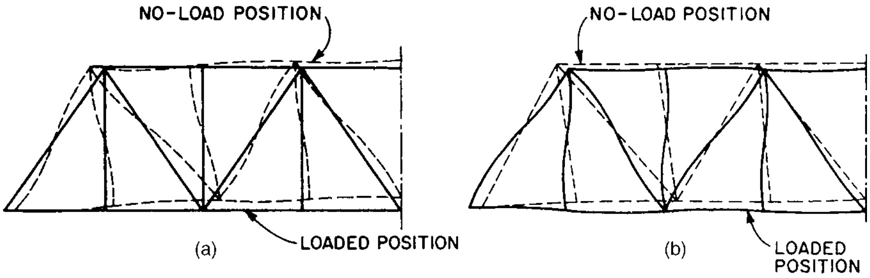


FIGURE 2.1 Effects of reaming methods on truss assembly. (*a*) Truss configurations produced in methods 1 and 2. (*b*) Truss shapes produced in method 3.

the geometric length. The angles that were set in geometric position remain geometric. Therefore, the S curves induced in the no-load assembly vanish. Secondary bending stresses, for practical purposes, have been eliminated. Further loading or a removal of load, however, will produce some secondary bending in the members.

Figure 2.1*b* illustrates the effects of method 3. Dotted lines represent the shape of a truss reamed by method 3 and then fully connected, but without load. As the members are fitted up (pinned and bolted), the truss takes its cambered position. In this position, as when they were reamed, members are straight and positioned to their cambered angles; hence, they have no induced bending. The solid lines in Fig. 2.1*b* represent the shape of the truss under the load for which the truss was cambered. Each member now is strained; the fabricated length has been increased or decreased to its geometric length. The angles that were set in the cambered (no-load) position are still in that position. As a result, S curves are induced in the members, as indicated in Fig. 2.1*b* by exaggerated S curves in solid lines. Secondary stresses due to bending, which do not occur under camber load in methods 1 and 2, are induced by this load in method 3. Further loading will increase this bending and further increase the secondary stresses.

Bridge engineers should be familiar with the reaming methods and see that design and fabrication are compatible.

2.6 ROLLED SECTIONS

Hot-rolled sections produced by rolling mills and delivered to the fabricator include the following designations: W shapes (wide-flange shapes with essentially parallel flange surfaces), S shapes (American Standard beams with slope of 16⅔% on inner flange surfaces), HP shapes (bearing-pile shapes similar to W shapes but with flange and web thicknesses equal), M shapes (miscellaneous shapes that are similar to W, S, or HP but do not meet that classification), C shapes (American Standard channel shape with slope of 16⅔% on inner flange surfaces), MC shapes (miscellaneous channels similar to C), L shapes or angles, and ST (structural tees cut from W, M, or S shapes). Such material, as well as plates and bars, is referred to collectively as **plain material.**

To fulfill the needs of a particular contract, some of the plain material may be purchased from a local warehouse or may be taken from the fabricator's own stock. The major portion of plain material, however, is ordered directly from a mill to specific properties and dimensions. Each piece of steel on the order is given an identifying mark through which its origin can be traced. Mill test reports, when required, are furnished by the mill to the fabricator to certify that the requirements specified have been met.

Steel shapes, such as beams, columns, and truss chords, that constitute main material for a project are often ordered from the mill to approximately their final length. The exact length ordered is usually a 4- to 6-in increment in excess of the required final dimension. Economies are achieved by limiting the number of lengths shipped, and current practice is to supply material grouped in length increments of 4 to 6 in.

Wide-flange shapes used as columns are ordered with an allowance for finishing the ends. Items such as angles for bracing or truss-web members, detail material, and light members in general are ordered in long pieces from which several members can be cut.

Plate material such as that for use in plate-girder webs is generally ordered to required dimensions plus additional amounts for trim and camber.

Plate material such as that for use in plate-girder flanges or built-up column webs and flanges is generally ordered to the required length plus trim allowance but in multiple widths for flame cutting or stripping to required widths.

The dimensions in which standard sections are ordered, i.e., multiple widths, multiple lengths, etc., are given careful consideration by the fabricator because the mill unit prices for the material depend on dimensions as well as on physical properties and chemistry. Computers are often used to optimize ordering of material.

ASTM A36, A572, A588, A913, A992, and A709 define the mechanical properties, chemistry, and permissible production methods for the materials commonly used in structural steel for buildings and bridges. The common production requirements for shapes, plate, and bar material are defined in ASTM A6. This standard includes requirements on what testing is required, what is to be included in test reports, quality requirements such as surface imperfection limits, and tolerances on physical dimensions. A6 also contains a list of shape designations with their associated dimensions. Not all shapes defined in A6 are produced by a mill at any given time. While most of the shapes listed are available from more than one domestic or foreign mill, some shapes may not be available at all, or may be available only in mill quantities (anywhere from 20 to 200 tons) or may be available only with long lead times. The AISC publishes information on the availability of shapes periodically. When rolled shapes are not available to suit a given requirement, shapes can be built in the fabricating shop.

Fabrication of standard sections entails several or all of the following operations: template making, layout, punching and drilling, fitting up and reaming, bolting, welding, finishing, inspection, cleaning, painting, and shipping.

2.7 BUILT-UP SECTIONS

Built-up sections are made up by a fabricator from two or more shapes or plates. Examples of common built-up sections are shown in Fig. 2.2. Built-up members are specified by the designer when the desired properties or configuration cannot be obtained in a single hot-rolled section. Built-up sections can be bolted or welded. Welded members, in general, are less expensive because much less handling is required in the shop and because of more efficient utilization of material. The clean lines of welded members also produce a better appearance.

Cover-plated rolled beams are used when the required bending capacity is not available in a rolled standard beam or when depth limitations preclude use of a deeper rolled beam or plate girder. Cover-plated beams are also used in composite construction to obtain the efficiency of a nonsymmetrical section.

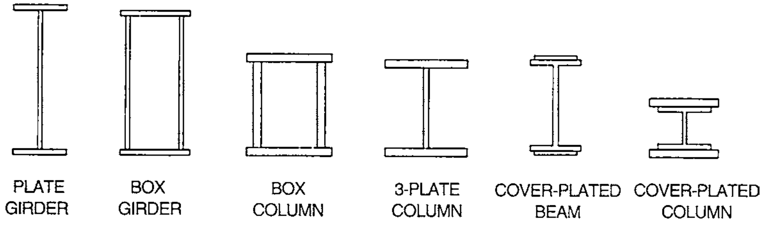


FIGURE 2.2 Typical built-up structural sections.

Cover-plate material is ordered to multiple widths for flame cutting or stripping to the required width in the shop. For this reason, when several different design conditions exist in a project, it is good practice, as well as good economy, for the designer to specify as few different cover-plate thicknesses as possible and to vary the width of plate for the different members.

Plate girders are specified when the moment capacity, stiffness, or on occasion, web shear capacity cannot be obtained in a rolled beam. They usually are fabricated by welding.

Welded plate girders consist of a web plate, a top flange plate, a bottom flange plate, and stiffener plates. Web material is ordered from the mill to the width between flange plates plus an allowance for trim and camber, if required. Flange material is ordered to multiple widths for stripping to the desired widths in the shop.

When an order consists of several identical girders having shop flange splices, fabricators usually first lay the flange material end to end in the ordered widths and splice the abutting ends with the required groove welds. The long, wide plates thus produced are then stripped to the required widths. For this procedure, the flanges should be designed to a constant width over the length of the girder. This method is advantageous for several reasons: Flange widths permit groove welds sufficiently long to justify use of automatic welding equipment. Run-out tabs for starting and stopping the welds are required only at the edges of the wide, unstripped plate. All plates can be stripped from one setup. And much less finishing is required on the welds.

After web and flange plates are cut to proper widths, they are brought together for fit-up and final welding. The web-to-flange welds, usually fillet welds, are positioned for welding with maximum efficiency. For relatively small welds, such as $^1/_4$- or $^5/_{16}$-in fillets, a girder may be positioned with web horizontal to allow welding of both flanges simultaneously. The girder is then turned over, and the corresponding welds are made on the other side. When relatively large fillet welds are required, the girder is held in a fixture with the web at an angle of about 45° to allow one weld at a time to be deposited in the flat position. In either method, the web-to-flange welds are made with automatic welding machines that produce welds of good quality at a high rate of deposition. For this reason, fabricators would prefer to use continuous fillet welds rather than intermittent welds, though an intermittent weld may otherwise satisfy design requirements.

After web-to-flange welds are made, the girder is trimmed to its detailed length. This is not done earlier because of the difficulty of predicting the exact amount of girder shortening due to shrinkage caused by the web-to-flange welds.

If holes are required in web or flange, the girder is drilled next. This step requires moving the whole girder to the drills. Hence, for economy, holes in main material should be avoided because of the additional amount of heavy-load handling required. Instead, holes should be located in detail material, such as stiffeners, which can be punched or drilled before they are welded to the girder.

The next operation applies the stiffeners to the web. Stiffener-to-web welds often are fillet welds. They are made with the web horizontal. The welds on each side of a stiffener may be deposited simultaneously with automatic welding equipment. For this equipment, many fabricators prefer continuous welds to intermittent welds. When welds are large, however, the girder may be positioned for flat, or downhand, welding of the stiffeners.

Variation in stress along the length of a girder permits reductions in flange material. For minimum weight, flange width and thickness might be decreased in numerous steps. But a design that optimizes material seldom produces an economical girder. Each change in width or thickness requires a splice. The cost of preparing a splice and making a weld may be greater than the cost of material saved to avoid the splice. Therefore, designers should hold to a minimum flange splices made solely to save material. Sometimes, however, the length of piece that can be handled may make splices necessary.

Welded crane girders differ from ordinary welded plate girders principally in that the upper surface of the top flange must be held at constant elevation over the span. A step at flange splices is undesirable. Since lengths of crane girders usually are such that flange splices are not made necessary by available lengths of material, the top flange should be continuous. In unusual cases where crane girders are long and splices are required, the flange should be held to a constant thickness. (It is not desirable to compensate for a thinner flange by deepening the web at the splice.) Depending on other elements that connect to the top flange of a crane girder, such as a lateral-support system or horizontal girder, holding the flange to a constant width also may be desirable.

The performance of crane girders is quite sensitive to the connection details used. Care must be taken in design to consider the effects of wheel loads, out-of-plane bending of the web, and permitting the ends of the girders to rotate as the crane travels along the length of the girder. The American Iron and Steel Engineers and the AISC both provide information concerning appropriate details.

Horizontally curved plate girders for bridges constitute a special case. Two general methods are used in fabricating them. In one method, the flanges are cut from a wide plate to the prescribed curve. Then the web is bent to this curve and welded to the flanges. In the second method, the girder is fabricated straight and then curved by application of heat to the flanges. This method, which is recognized by the AASHTO specifications, is preferred by many fabricators because less scrap is generated in cutting flange plates, savings may accrue from multiple welding and stripping of flange plates, and the need for special jigs and fittings for assembling a girder to a curve is avoided.

("Fabrication Aids for Continuously Heat-Curved Girders" and "Fabrication Aids for Girders Curved with V-Heats," American Institute of Steel Construction, Chicago, Ill.)

Procedures used in fabricating other built-up sections, such as box girders and box columns, are similar to those for welded girders.

Columns generally require the additional operation of end finishing for bearing. For welded columns, all the welds connecting main material are made first, to eliminate uncertainties in length due to shrinkage caused by welding. After the ends are finished, detail material, such as connection plates for beams, is added.

The selection of connection details on built-up sections has an important effect on fabrication economy. If the pieces making up the section are relatively thick, welded details can provide bolt holes for connections and thereby eliminate punching the thick material. On the other hand, fabricators that trim sections at the saw after assembly may choose to drill holes using a combination drill-saw line, thus avoiding manual layout for welded detail material.

2.8 CLEANING AND PAINTING

The AISC "Specification for Structural Steel Buildings" provides that, in general, steelwork to be concealed within the building need not be painted and that steel encased in concrete should not be painted. Inspection of old buildings has revealed that the steel withstands corrosion virtually the same whether painted or not.

Paint is expensive to apply, creates environmental concerns in the shop, and can create a slip hazard for erectors. Environmental requirements vary by region. Permitting flexibility in coating selection may lead to savings. When paint is required, a shop coat is often applied as a primer for subsequent field coats. It is intended to protect the steel for only a short period of exposure.

Many fabricators have invested in the equipment and skills necessary to apply sophisticated coatings when required. Compared with single-coat, surface-tolerant primers used in normal applications, these multiple-coat or special systems are sensitive to cleaning and applicator skill. While these sophisticated coating systems are expensive, they can be useful when life cycle costs are considered in very-long-term exposures or aggressive environments.

Steel which is to be painted must be thoroughly cleaned of all loose mill scale, loose rust, dirt, and other foreign matter. Cleaning can be done by hand tool, power tool, and a variety of levels of abrasive blasting. Abrasive blasting in most fabrication shops is done with centrifugal wheel blast units. The various surface preparations are described in specifications by the Society for Protective Coatings. Unless the fabricator is otherwise directed, cleaning of structural steel is ordinarily done with a wire brush. Sophisticated paint systems require superior cleaning, usually abrasive blast cleaning and appropriate quality systems. Knowledge of the coating systems, equipment maintenance, surface preparation, and quality control are all essential.

Treatment of structural steel that will be exposed to close public view varies somewhat from that for steel in unexposed situations. Since surface preparation is the most important factor affecting performance of paint on structural steel surfaces, it is common for blast cleaning to be specified for removing all mill scale on steel that is to be exposed. Mill scale that forms on structural steel after hot rolling protects the steel from corrosion, but only as long as this scale is intact and adheres firmly to the steel. Intact mill scale, however, is seldom encountered on fabricated steel because of weathering during storage and shipment and because of loosening caused by fabricating operations. Undercutting of mill scale, which can lead to paint failure, is attributable to the broken or cracked condition of mill scale at the time of painting. When structural steel is exposed to view, even small amounts of mill scale lifting and resulting rust staining will likely detract from the appearance of a building. On industrial buildings, a little rust staining might not be objectionable. But where appearance is of paramount importance, descaling by blast cleaning is the preferred way of preparing the surface of architecturally exposed steel for painting.

Steels are available which can be exposed to the weather and can be left unpainted, such as A588 steel. This weathering steel forms a tight oxide coating that will retard further atmospheric corrosion under common outdoor exposures. Many bridge applications are suited to this type of steel. Where the steel would be subjected to salts around expansion devices, owners often choose to paint that area. The steel that is to be left unpainted is generally treated in one of two ways, depending on the application.

For structures where appearance is not important and minimal maintenance is the prime consideration, the steel may be erected with no surface preparation at all. While it retains mill scale, the steel will not have a uniform color. But when the scale loses its adherence and flakes off, the exposed metal will form the tightly adherent oxide coating characteristic of this type of steel, and eventually, a uniform color will result.

Where uniform color of bare, unpainted steel is important, the steel must be freed of scale by blast cleaning. In such applications, extra precautions must be exercised to protect the blasted surfaces from scratches and staining.

Steel may also be prepared by grinding or blasting to avoid problems with welding through heavy scale or to achieve greater nominal loads or allowable loads in slip-critical bolted joints.

(*Steel Structures Painting Manual,* vol. I, *Good Painting Practice*, vol. II, *Systems and Specifications*, Society for Protective Coatings, 40 24th St., Pittsburgh, PA 15222.)

2.9 FABRICATION TOLERANCES

Variations from theoretical dimensions occur in hot-rolled structural steel because of the routine production process variations and the speed with which they must be rolled, wear and deflection of the rolls, human differences between mill operators, and differential cooling rates of the elements of a section. Also, mills cut rolled sections to length while they are still hot. Tolerances that must be met before structural steel can be shipped from mill to fabricator are listed in ASTM A6, “General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use.”

Tolerances are specified for the dimensions and straightness of plates, hot-rolled shapes, and bars. For example, flanges of rolled beams may not be perfectly square with the web and may not be perfectly centered on the web. There are also tolerances on surface quality of structural steel.

Specifications covering fabrication of structural steel do not, in general, require closer tolerances than those in A6, but rather extend the definition of tolerances to fabricated members. Tolerances for the fabrication of structural steel, both hot-rolled and built-up members, can be found in standard codes, such as the AISC “Specification for Structural Steel Buildings”; the AISC “Code of Standard Practice for Steel Buildings and Bridges”; AWS D1.1, “Structural Welding Code-Steel”; AWS D1.5, “Bridge Welding Code”; and AASHTO specifications.

The tolerance on length of material as delivered to the fabricator is one case where the tolerance as defined in A6 may not be suitable for the final member. For example, A6 allows wide flange beams 24 in or less deep to vary (plus or minus) from ordered length by $^3/_8$ in plus an additional $^1/_{16}$ in for each additional 5-ft increment over 30 ft. The AISC specification for length of fabricated steel, however, allows beams to vary from detailed length only $^1/_{16}$ in for members 30 ft or less long and $^1/_8$ in for members longer than 30 ft. For beams with framed or seated end connections, the fabricator can tolerate allowable variations in length by setting the end connections on the beam so as to not exceed the overall fabrication tolerance of $\pm^1/_{16}$ or $\pm^1/_8$ in. Members that must connect directly to other members, without framed or seated end connections, must be ordered from the mill with a little additional length to permit the fabricator to trim them to within $\pm^1/_{16}$ or $\pm^1/_8$ in of the desired length.

The AISC "Code of Standard Practice for Steel Buildings and Bridges" defines the clause "Architecturally Exposed Structural Steel" (AESS) with more restrictive tolerances than on steel not designated as AESS. The AESS section states that "permissible tolerances for out-of-square or out-of-parallel, depth, width and symmetry of rolled shapes are as specified in ASTM Specification A6. No attempt to match abutting cross-sectional configurations is made unless specifically required by the contract documents. The as-fabricated straightness tolerances of members are one-half of the standard camber and sweep tolerances in ASTM A6." It must be recognized that the requirements of the AESS section of the Code of Standard Practice entail special shop processes and costs and they are not required on all steel exposed to public view. Therefore, members that are subject to the provisions of AESS must be designated on design drawings.

Designers should be familiar with the tolerances allowed by the specifications covering each job. If they require more restrictive tolerances, they must so specify on the drawings and must be prepared for possible higher costs of fabrication.

While restrictive tolerances may be one way to make parts of a structure fit, they often are not a simple matter of care and are not practical to achieve. A steel beam can be fabricated at 65°F and installed at 20°F. If it is 50 ft in fabrication, it will be about ⅛ in short during installation. While ⅛ in may not be significant, a line of three or four of these beams in a row may produce unacceptable results. The alternative to restrictive tolerances may be adjustment in the structural steel or the parts attaching to it. Some conditions deserving consideration include parts that span vertically one or more stories, adjustment to properly set expansion joints, camber in cantilever pieces, and members that are supported some distance from primary columns.

2.10 STEEL FRAME ERECTION

Fabricated pieces are usually received at an erection site from fabrication plants on over-the-road, railroad, or floating carriers. The pieces are unloaded into a receiving (shake-out) area, where they are temporarily stored and sorted to comply with a planned installation sequence.

In some cases, pieces are preassembled into larger lift units or modules, to improve installation efficiency or to provide stability. There are also occasions when transport of each piece has to be scheduled to arrive from a fabrication plant at a predetermined time to be lifted directly from the carrier for placement in the structure. Such a situation occurs when erection is done in a congested location or where there is no viable shake-out area.

Forces on frame members during construction vary significantly from those in the completed structure. In most cases these forces are not critical to the design of the members, but in some cases they are. The erector analyzes member strengths and connections in load conditions related to the sequence of erection. If the construction loads are critical, the erector advises the engineer and/or fabricator and arranges appropriate design modification.

2.11 ERECTION EQUIPMENT

Lifting equipment is the single most critical item in the erector's inventory. Hook speed, reach, and capacity control the progress of erecting. Rigging, primarily wire rope slings and hardware, has to be designed for efficient attachment and removal. Steel buildings and bridges are generally erected with cranes, derricks, or specialized units. Mobile cranes include crawler cranes, rubber-tired rough terrain cranes, and truck cranes; stationary cranes include tower cranes and climbing cranes. Stiffleg derricks and guy derricks are generally considered stationary hoisting machines, but they may be mounted on mobile platforms. Guy derricks can be used where they are jumped from floor to floor. A catenary high line is an example of a specialized unit. Rubber-tired gantry-type cranes are often used in shipping and shake-out yard operations. These various types of erection equipment used for steel construction are also used for precast and cast-in-place concrete construction.

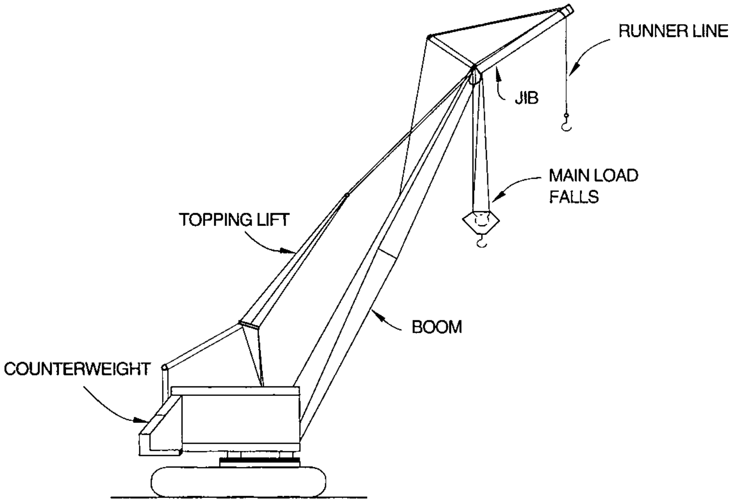


FIGURE 2.3 Crawler crane.

One of the most common machines for steel erection is the crawler crane (Fig. 2.3). Self-propelled, such cranes are mounted on a mobile base having endless tracks or crawlers for propulsion. The base of the crane contains a turntable that allows 360° rotation. Crawlers come with booms up to 540 ft high and capacities up to 1000 tons. Self-contained counterweights move the center of gravity of the loaded crane to the rear to increase the lift capacity of the crane. Crawler cranes can also be fitted with counterweights on attached mobile carriages or ring attachments to increase their capacity.

Truck cranes (Fig. 2.4) are similar in many respects to crawler cranes. The principal difference is that truck cranes are mounted on rubber tires and are therefore much more mobile on hard surfaces.

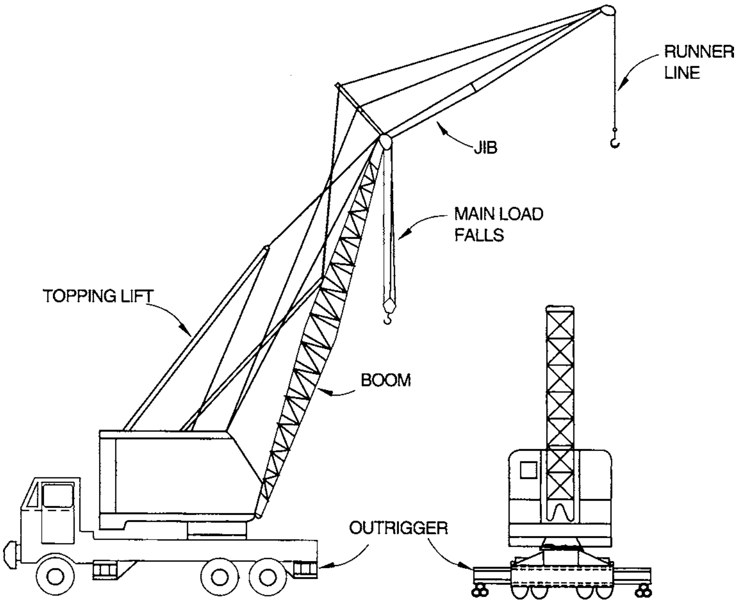


FIGURE 2.4 Truck crane.

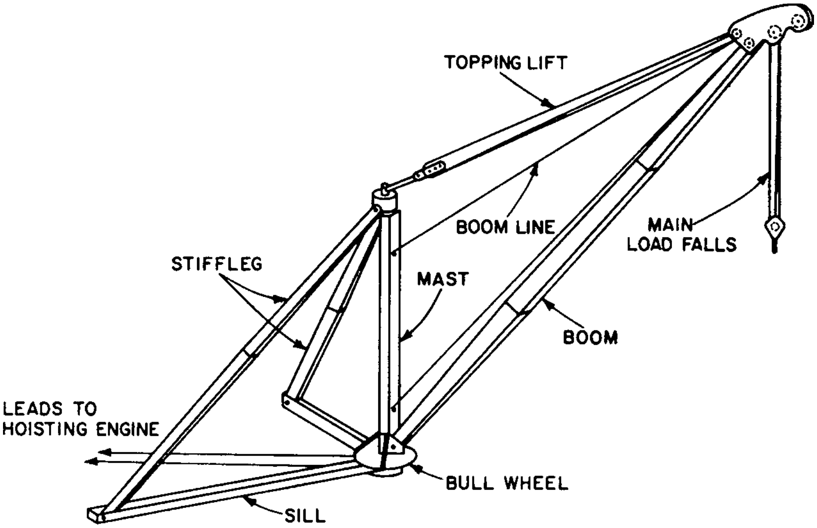


FIGURE 2.5 Stiffleg derrick.

Truck cranes can be used with booms up to 500 ft long and have capacities up to 750 tons. Rough-terrain cranes have hydraulic booms and are also highly mobile. Truck cranes and rough-terrain cranes have outriggers to provide stability.

A stiffleg derrick (Fig. 2.5) consists of a boom and a vertical mast rigidly supported by two legs. The two legs are capable of resisting either tensile or compressive forces, hence the name **stiffleg.** Stiffleg derricks are extremely versatile in that they can be used in a permanent location as yard derricks or can be movable for use as a traveler in bridge erection. A stiffleg derrick also can be mounted on a device known as a **creeper** and thereby lift itself vertically on a structure as it is being erected. Stiffleg derricks can range from small, 5-ton units to large, 250-ton units, with 80-ft masts and 180-ft booms.

A guy derrick (Fig. 2.6) is commonly associated with the erection of tall multistory buildings. It consists of a boom and a vertical mast supported by wire-rope guys which are attached to the structure being erected. Although a guy derrick can be rotated 360°, the rotation is handicapped by the presence of the guys. To clear the guys while swinging, the boom must be shorter than the mast and must be brought up against the mast. The guy derrick has the advantage of being able to climb vertically (jump) under its own power, such as illustrated for the construction of a building in Fig. 2.7. Guy derricks have been used with booms up to 160 ft long and with capacities up to 250 tons.

Tower cranes in various forms are used extensively for erection of buildings and bridges. Several manufacturers offer accessories for converting conventional truck or crawler cranes into tower cranes. Such a tower crane (Fig. 2.8) is characterized by a vertical

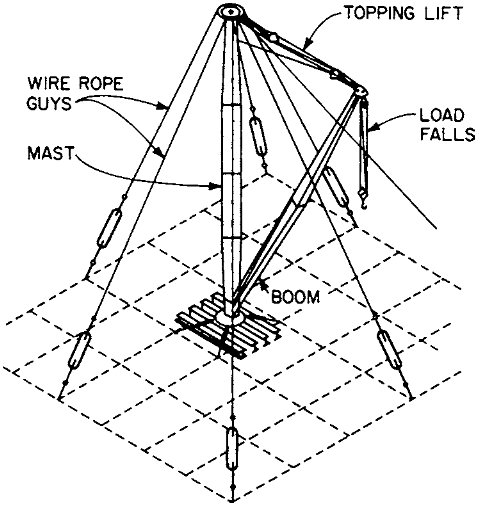


FIGURE 2.6 Guy derrick.

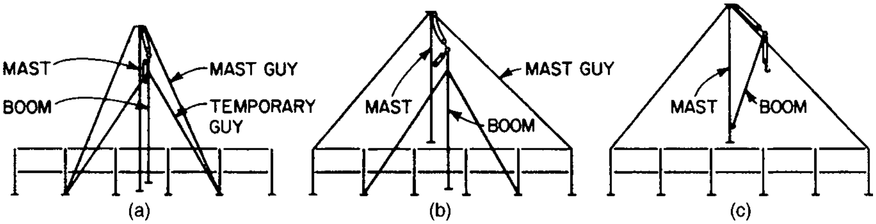


FIGURE 2.7 Steps in jumping a guy derrick. (*a*) Removed from its seat with the topping lift falls, the boom is revolved 180° and placed in a temporary jumping shoe. The boom top is temporarily guyed. (*b*) The load falls are attached to the mast above its center of gravity. Anchorages of the mast guys are adjusted and the load falls unhooked. (*c*) The temporary guys on the boom are removed. The mast raises the boom with the topping lift falls and places it in the boom seat, ready for operation.

tower, which replaces the conventional boom, and a long boom at the top that can usually accommodate a jib as well. With the main load falls suspended from its end, the boom is raised or lowered to move the load toward or away from the tower. The cranes are counterweighted in the same manner as conventional truck or crawler cranes. Capacities of these tower cranes vary widely depending on the machine, tower height, and boom length and angle. Such cranes have been used with towers 320 ft high and booms 240 ft long. They can usually rotate 360°.

Other types of tower cranes with different types of support are shown in Fig. 2.9*a* through *c*. The type selected will vary with the type of structure erected and erection conditions. Each type of support

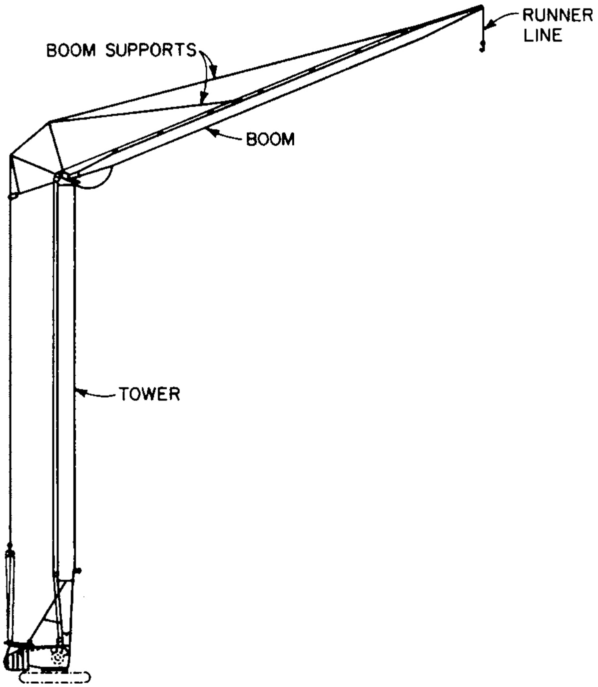


FIGURE 2.8 Tower crane on crawler-crane base.

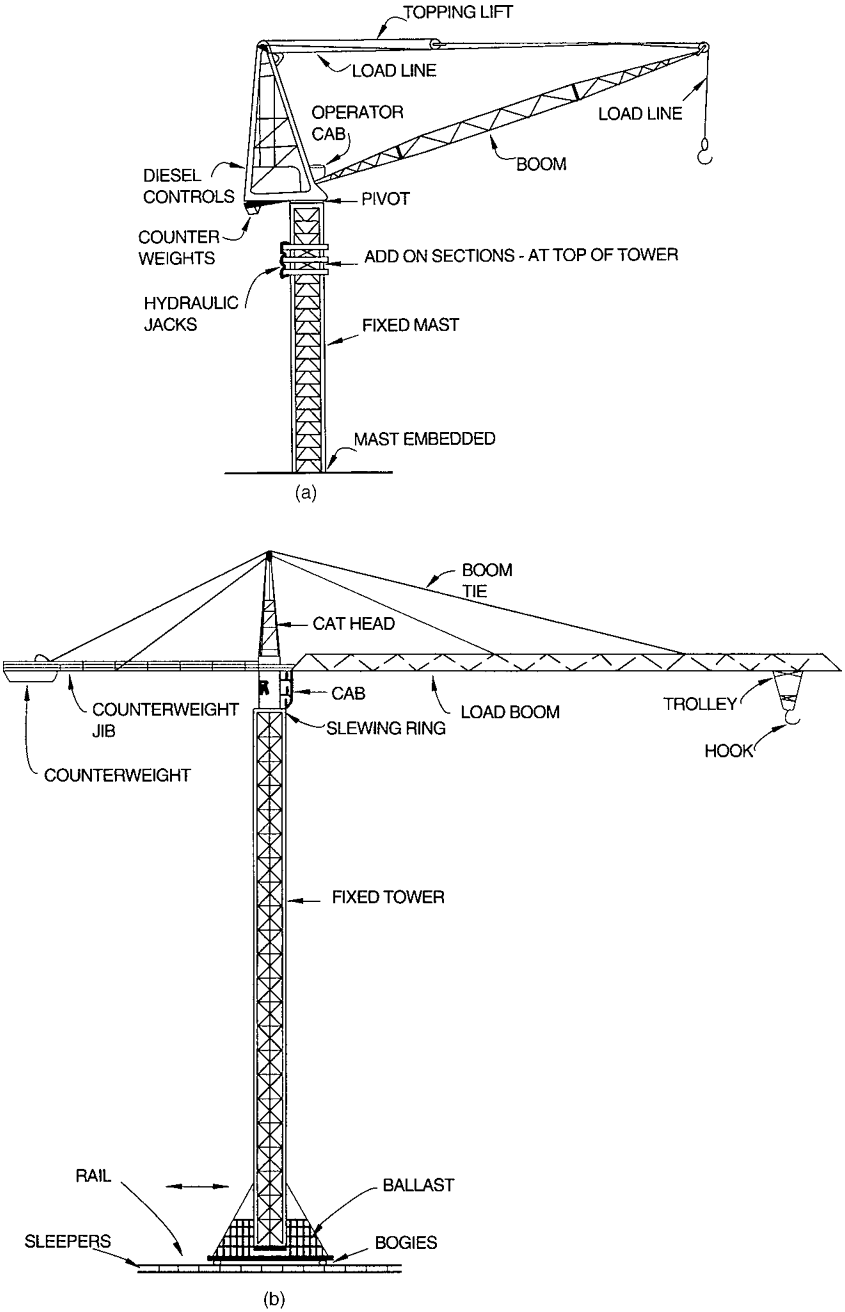


FIGURE 2.9 Variations of the tower crane: (*a*) kangaroo; (*b*) hammerhead; (*c*) climbing crane.

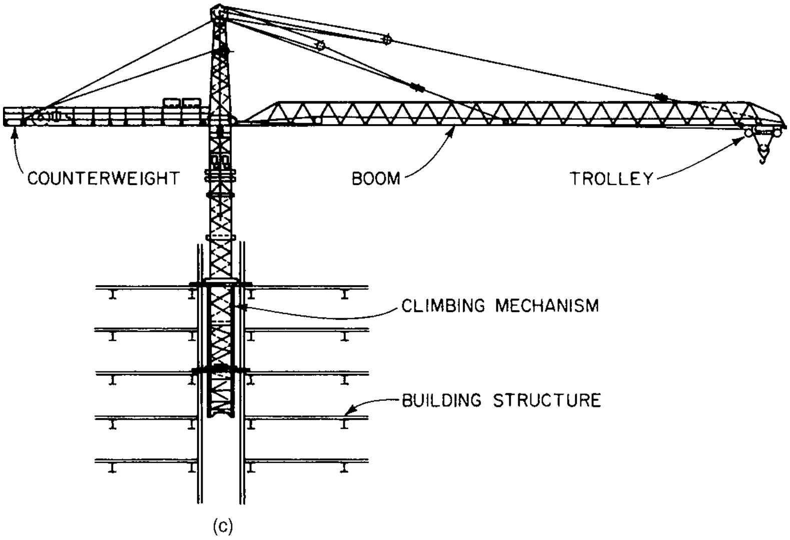


FIGURE 2.9 (*Continued*)

shown may have either the kangaroo (topping lift) or the hammerhead (horizontal boom) configuration. Kangaroo and hammerhead type cranes often have moveable counterweights that move back as the load is boomed out to keep the crane balanced. These cranes are sophisticated and expensive, but are often economical because they are usually fast and may be the only practical way to bring major building components to the floor they are needed. Crane time is a key asset on high-rise construction projects.

Jacking is another method used to lift major assemblies. Space frames that can be assembled on the ground, and suspended spans on bridges that can be assembled on shore, can be economically put together where there is access and then jacked into their final location. Jacking operations require specialized equipment, detailing to provide for final connections, and analysis of the behavior of the structure during the jacking.

For marine installations with adequate water depths, such as bridges over waterways or off-shore petroleum industry platforms, the magnitude of erecting lifts and reaches increases dramatically. Floating carriers transport large quantities and/or assemblies (modules) to the site for erection. Barge- and ship-mounted revolving or shear-leg cranes do the heavy lifting. These cranes are capable of reaching twice as far with as much as five times as much load as land-bound cranes. Wire-rope slings used for these lifts are comparably sized and specially configured for handling flexibility.

2.12 ERECTION METHODS FOR BUILDINGS

The determination of how to erect a building depends on many variables that must be studied by the erection engineer long before steel begins to arrive at the erection site. It is normal and prudent to have this erection planning developed on drawings and in written procedures. Such documents outline the equipment to be used, methods of supporting the equipment, conditions for use of the equipment, and sequence of erection. In many areas, such documents are required by law. The work plan that evolves from them is valuable because it can result in economies in the costly field work. Special types of structures require extensive planning to ensure stability of the structure during erection.

Mill buildings, warehouses, shopping centers, and low-rise structures that cover large areas usually are erected with truck or crawler cranes. Selection of the equipment to be used is based on site conditions, weight and reach for the heavy lifts, and availability of equipment. Preferably, erection of such building frames starts at one end, and the crane backs away from the structure as erection progresses. The underlying consideration at all times is that an erected member should be stable before it is released from the crane. High-pitched roof trusses, for example, are often unstable under their own weight without top-chord bracing. If roof trusses are long and shipped to the site in several sections, they are often spliced on the ground and lifted into place with one or two cranes.

Multistory structures, or portions of multistory structures that lie within reach and capacity limitations of crawler cranes, are usually erected with crawler cranes. For tall structures, a crawler crane places steel it can reach and then erects the guy derrick (or derricks), which will continue erection. Alternatively, tower crawler cranes (see Fig. 2.8) and climbing tower cranes (Fig. 2.9) are used extensively for multistory structures. Depending on height, these cranes can erect a complete structure. They allow erection to proceed vertically, completing floors or levels for other trades to work on before the structure is topped out.

Use of any erecting equipment that loads a structure requires the erector to determine that such loads can be adequately withstood by the structure or to install additional bracing or temporary erection material that may be necessary. For example, guy derricks impart loads at guys, and at the base of the boom a horizontal thrust that must be provided for. On occasion, floorbeams located between the base of the derrick and guy anchorages must be temporarily laterally supported to resist imposed compressive forces. Considerable temporary bracing is required in a multistory structure when a climbing crane is used. This type of crane imposes horizontal and vertical loads on the structure or its foundation. Loads are also imposed on the structure when the crane is jumped to the next level. Usually, these cranes jump about six floors at a time.

The sequence of placing the members of a multistory structure is, in general, columns, girders, bracing, and beams. The exact order depends on the erection equipment and type of framing. Planning must ensure that all members can be erected and that placement of one member does not prohibit erection of another.

Structural steel is erected by “ironworkers” who perform a multitude of tasks. The ground crew selects the proper members to hook onto the crane and directs crane movements in delivering the piece to the “connectors.” The connectors direct the piece into its final location, place sufficient temporary bolts for stability, and unhitch the crane. Regulations generally require a minimum of two bolts per connection or equivalent, but more should be used if required to support heavy pieces or loads that may accumulate before the permanent connection is made.

A “plumbing-up” (fitting-up crew), following the connectors, aligns the beams, plumbs the columns, and installs whatever temporary wire-rope bracing is necessary to maintain alignment. Following this crew are the gangs who make the permanent connection. This work, which usually follows several stories behind member erection, may include tightening high-strength bolts or welding connections. An additional operation may involve placing and welding joists and metal deck to furnish a working floor surface for subsequent operations. Safety codes require planking surfaces 25 to 30 ft (usually two floors) below the erection work above. For this reason, deck is often spread on alternate floors, stepping back to spread the skipped floor after the higher floor is spread, thus allowing the raising gang to move up to the next tier. This is one reason why normal columns are two floors high.

In field-welded multistory buildings with continuous beam-to-column connections, the procedure is slightly different from that for bolted work. The difference is that the welded structure is not in its final alignment until beam-to-column connections are welded because of shrinkage caused by the welds. To accommodate the shrinkage, the joints must be opened up or the beams must be detailed long so that, after the welds are made, the columns are pulled into plumb. It is necessary, therefore, to erect from the more restrained portion of the framing to the less restrained. If a structure has a braced center core, that area will be erected first to serve as a reference point, and steel will be erected toward the perimeter of the structure. If the structure is totally unbraced, an area in the center will be plumbed and temporarily braced for reference. Welding of column splices and beams is done after the structure is plumbed. The deck is attached for safety as it is installed, but final welding of deck and installation of studs and closures is completed after the tier is plumbed.

2.13 ERECTION PROCEDURE FOR BRIDGES

Bridges are erected by a variety of methods. The choice of method in a particular case is influenced by type of structure, length of span, site conditions, manner in which material is delivered to the site, and equipment available. Bridges over navigable waterways are sometimes limited to erection procedures that will not inhibit traffic flow; for example, falsework may be prohibited.

Regardless of erection procedure selected, there are two considerations that override all others. The first is the security and stability of the structure under all conditions of partial construction, construction loading, and wind loading that will be encountered during erection. The second consideration is that the bridge must be erected in such a manner that it will perform as intended. For example, in continuous structures, this can mean that jacks must be used on the structure to effect the proper stress distribution. These considerations will be elaborated upon later as they relate to erection of particular types of bridges.

Simple-beam bridges are often erected with a crawler or truck crane. Bridges of this type generally require a minimal amount of engineering and are put up routinely by an experienced erector. One problem that does occur with beam spans, however, and especially composite beam spans, arises from lateral instability of the top flange during lifting or before placement of permanent bracing. Beams or girders that are too limber to lift unbraced require temporary compression-flange support, often in the form of a stiffening truss. Lateral support also may be provided by assembling two adjacent members on the ground with their bracing or cross members and erecting the assembly in one piece. Beams that can be lifted unbraced but are too limber to span alone also can be handled in pairs. It may be necessary to hold them with the crane until bracing connections can be made.

Continuous-beam bridges are erected in much the same way as simple-beam bridges. One or more field splices, however, will be present in the stringers of continuous beams. With bolted field splices, the holes in the members and connection material have been reamed in the shop to insure proper alignment of the member. With a welded field splice, it is generally necessary to provide temporary connection material to support the member and permit adjustment for alignment and proper positioning for welding. For economy, field splices should be located at points of relatively low bending moment. It is also economical to allow the erector some options regarding splice location, which may materially affect erection cost. The arrangement of splices in Fig. 2.10*a*, for example, will require, if falsework is to be avoided, that both end spans be erected first, then the center spans. The splice arrangement shown in Fig. 2.10*b* will allow erection to proceed from one

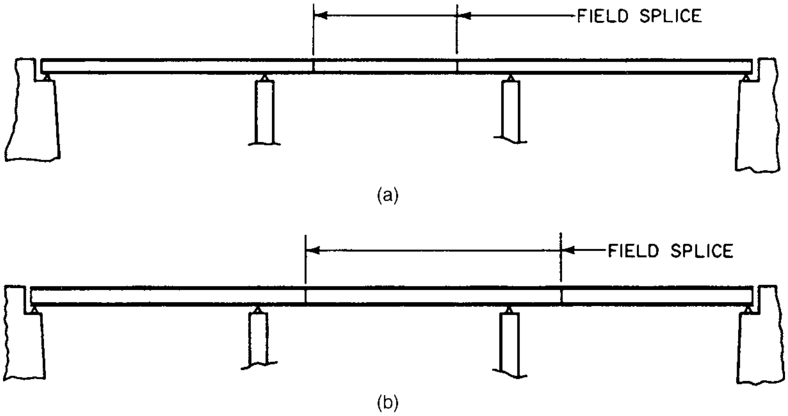


FIGURE 2.10 Field splices in girder bridges.

end to the other. While both arrangements are used, one may have advantages over the other in a particular situation.

Horizontally curved girder bridges are similar to straight-girder bridges except for torsional effects. If use of falsework is to be avoided, it is necessary to resist the torques by assembling two adjacent girders with their diaphragms and temporary or permanent lateral bracing and erect the assembly as a stable unit. Diaphragms and their connections must be capable of withstanding end moments induced by girder torques.

Truss bridges require a vast amount of investigation to determine the practicability of a desired erection scheme or the limitations of a necessary erection scheme. The design of truss bridges, whether simple or continuous, generally assumes that the structure is complete and stable before it is loaded. The erector, however, has to impose dead loads, and often live loads, on the steel while the structure is partly erected. The structure must be erected safely and economically in a manner that does not overstress any member or connection.

Erection stresses may be of opposite sign and of greater magnitude than the design stresses. When designed as tension members but subjected to substantial compressive erection stresses, the members may be braced temporarily to reduce their effective length. If bracing is impractical, they may be made heavier. Members designed as compression members but subjected to tensile forces during erection are investigated for adequacy of area of net section where holes are provided for connections. If the net section is inadequate, the member must be made heavier.

Once an erection scheme has been developed, the erection engineer analyzes the structure under erection loads in each erection stage and compares the erection stresses with the design stresses. At this point, the engineer plans for reinforcing or bracing members, if required. The erection loads include the weights of all members in the structure in the particular erection stage and loads from whatever erection equipment may be on the structure. Wind loads are added to these loads.

In addition to determining member stresses, the erection engineer usually calculates reactions for each erection stage, whether they be reactions on abutments or piers or on falsework. Reactions on falsework are needed for design of the falsework. Reactions on abutments and piers may reveal a temporary uplift that must be provided for, by counterweighting or use of tie-downs. Often, the engineer also computes deflections, both vertical and horizontal, at critical locations for each erection stage to determine stroke and capacity of jacks that may be required on falsework or on the structure.

When all erection stresses have been calculated, the engineer prepares detailed drawings showing falsework, if needed, necessary erection bracing with its connections, alterations required for any permanent member or joint, installation of jacks and temporary jacking brackets, and bearing devices for temporary reactions on falsework. In addition, drawings are made showing the precise order in which individual members are to be erected.

Figure 2.11 shows the erection sequence for a through-truss cantilever bridge over a navigable river. For illustrative purpose, the scheme assumes that falsework is not permitted in the main channel between piers and that a barge-mounted crane will be used for steel erection. Because of the limitation on use of falsework, the erector adopts the cantilever method of erection. The plan is to erect the structure from both ends toward the center.

Note that top chord U13–U14, which is unstressed in the completed structure, is used as a principal member during erection. Note also that in the suspended span all erection stresses are opposite in sign to the design stresses.

As erection progresses toward the center, a negative reaction may develop at the abutments (panel point *LO*). The uplift may be counteracted by tie-downs to the abutment.

Hydraulic jacks, which are removed after erection has been completed, are built into the chords at panel points U13, L13, and U13′. The jacks provide the necessary adjustment to allow closing of the span. The two jacks at U13 and L13 provide a means of both horizontal and vertical movement at the closing panel point, and the jack at U13′ provides for vertical movement of the closing panel point only.

Other bridge types are also encountered as variations of the bridge types shown above. There are also distinct different types of bridges, such as suspension, cable-stayed, and movable bridges, each requiring erection planning and equipment especially suited to configuration and location.

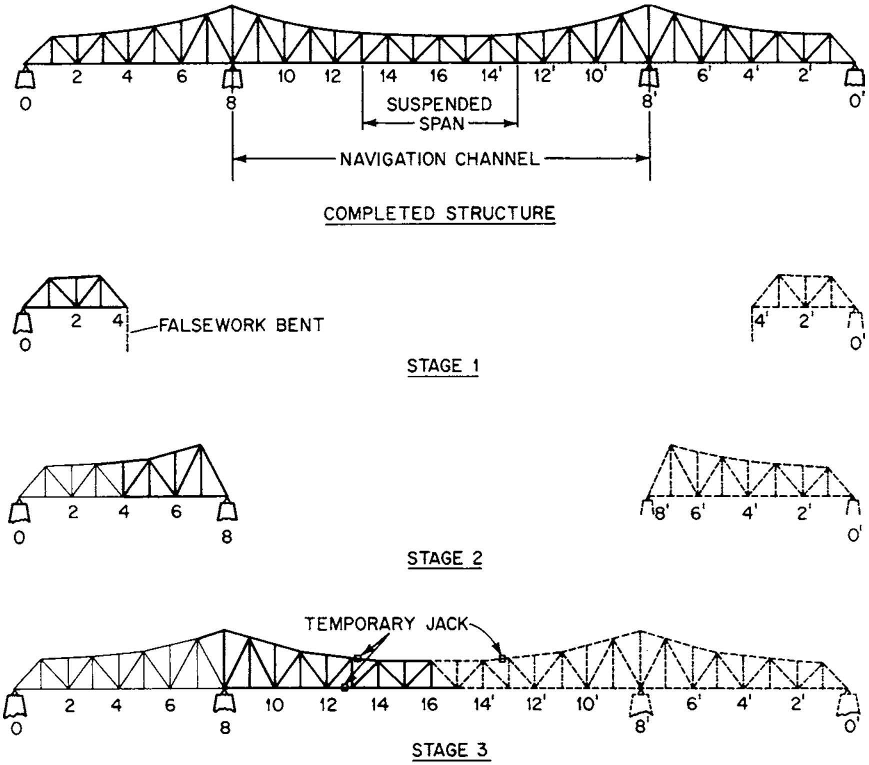


FIGURE 2.11 Erection stages for a cantilever-truss bridge. In Stage 1, with falsework at panel point 4, the portion of the truss from the abutment to that point is assembled on the ground and then erected on the abutment and the falsework. The operations are duplicated at the other end of the bridge. In Stage 2, members are added by cantilevering over the falsework, until the piers are reached. Panel points 8 and 8′ are landed on the piers by jacking down at the falsework, which then is removed. In Stage 3, main-span members are added by cantilevering over the piers, until midspan is reached. Jacks are inserted at panel points L13, U13, and U13′. The main span is closed by jacking. The jacks then are unloaded to hang the suspended span and finally are removed.

2.14 FIELD TOLERANCES

Permissible variations from theoretical dimensions of an erected structure are specified in the AISC "Code of Standard Practice for Steel Buildings and Bridges." It states that variations are within the limits of good practice or erected tolerance when they do not exceed the cumulative effect of permissible rolling and fabricating and erection tolerances. These tolerances are restricted in certain instances to total cumulative maximums.

The AISC "Code of Standard Practice" has a descriptive commentary that fully outlines and explains the application of the mill, fabrication, and erection tolerances for a building or bridge. Also see Art. 2.9 for specifications and codes that may require special or more restrictive tolerances for a particular type of structure.

An example of tolerances that govern the plumbness of a multistory building is the tolerance for columns. In multistory buildings, columns are considered to be plumb if the error does not exceed 1:500, except for columns adjacent to elevator shafts and exterior columns, for which additional limits are imposed. The tolerances governing the variation of columns, as erected, from their theoretical centerline are sometimes wrongfully construed to be lateral-deflection (drift) limitations on the completed structure when, in fact, the two considerations are unrelated. Measurement of tolerances requires experience. Structural steel is not static but moves due to varying ambient conditions and changing loads imposed during the construction process. Ambient conditions can be so extreme as to require final plumbing and span closing during nighttime hours. Making all components and attachments fit takes skill and experience on the part of designers and craftsmen.

(“Manual of Steel Construction,” American Institute of Steel Construction, Chicago, Ill.)

2.15 COORDINATION AND CONSTRUCTABILITY

The three shop assembly methods described in Art. 2.5 result in different geometries. Some make the pieces fit in the erected condition and others make the pieces fit in the dead-load condition. Builders have to choose which method is right for the project. Similar decisions may have to be made for other types of members. Project coordination is necessary in projects with complex pieces and is most effective if done prior to assigning contracts. Methods to accommodate distortion and construction loads and weld shrinkage, as well as use of erection aids, need to be considered and planned. All members of the construction team are affected and need to address these items in initial project stages.

2.16 SAFETY CONCERNS

Safety is the prime concern of steel erectors. Erectors tie-off above regulated heights, install perimeter cable around elevated work sites, and where necessary, install static lines. Lines for tying off have different requirements than perimeter cable, so perimeter cable cannot be used as a horizontal lifeline. Erectors are concerned with welding safety, protection around openings, and working over other trades. Stability of the structure during construction and of each piece as it is lifted are considered by the erector. Pieces that are laterally supported and under a positive moment in service, will frequently be unsupported and under a negative moment when they are raised, so precautions must be taken. Clearances for moving parts of lift equipment have to be monitored continually. Crane access and operating areas need to be capable of supporting superimposed loads.

Small changes in member proportions can lead to significant changes in the way an erector has to work. Long slender members may have to be raised with a spreader beam. Others may have to be braced before the load line is released. Erection aids such as column lifting hitches must be designed and provided such that they will afford temporary support and allow easy access for assembly. Full-penetration column splices are seldom necessary except on seismic moment frames, but require special erection aids when encountered. Construction safety is regulated by the federal Office of Safety and Health Administration (OSHA). Steel erector safety regulations are listed in Code of Federal Regulations (CFR) 1926, Subpart R. As well, American National Standards Institute (ANSI) issues standard A10 related to construction safety.

CHAPTER 3

CONNECTIONS

Larry S. Muir, P.E.
Chief Engineer, Cives Steel Company
President, Cives Engineering Corporation
Roswell, Georgia

William A. Thornton, Ph.D., P.E.
Corporate Consultant, Cives Corporation
Roswell, Georgia

In this chapter, the term **connection** is used in a general sense to include all types of joints in structural steel made with fasteners or welds. Emphasis is placed on the more commonly used connections, such as shear connections, beam-to-column moment connections, and axial force connections including main-member splices, bracing connections, and truss connections.

Recommendations apply to buildings that are not subject to special detailing requirements due to seismic loading. This material is generally based on the American Institute of Steel Construction (AISC), "Specification for Structural Steel Buildings," 2005, referred to herein as the **AISC Specification.** This new unified specification includes both load and resistance factor design (LRFD) and allowable strength design (ASD), with common expressions for nominal strength. All examples in this chapter are given in LRFD format, but most of the procedures are readily adaptable to ASD. See Chap. 5 for further discussion of design methods and terminology. For additional considerations in seismic applications, see AISC, "Seismic Provisions for Structural Steel Buildings," 2005, and Chap. 8.

3.1 GENERAL CONSIDERATIONS FOR CONNECTION DESIGN

3.1.1 To Connect, to Join, to Make Whole—the Job of the Connection Design Engineer

To connect or to join is to bring together so as to make continuous or form a unit. In steel structures, this bringing together is usually accomplished through the use of fasteners (primarily bolts) and welds, along with secondary plates, angles, or other steel pieces. When designing structural steel connections, the goal is to unite the parts in such a way that the basic assumptions made during the analysis are supported by the as-built conditions. Of course, the assumptions made during analysis can rarely be precisely replicated. Pinned supports are almost never truly pinned, and fixed supports are almost never truly fixed; but by diligently sizing connection elements to accommodate these differences, the connection design engineer can design connections that will closely approximate the assumed behavior, or at least accommodate the differences that inevitably exist.

3.1.2 Justifying Connection Design

A proposed connection design method can be justified in three ways: through precedence, testing, or analysis.

Precedence simply means that there is sufficient historical record of adequate performance of a connection configuration or an assumption to justify its use. Many valid arguments can be made against accepting a connection design method based on precedence. Because of the conservatism built into design loads and load factors and the fact that loads can often redistribute, connections in service may rarely see their full design loads. Therefore, a history of satisfactory service may not correlate directly to a safe design. However, some assumptions implicit in AISC, "Manual of Steel Construction," 2005 (referred to herein as the **AISC Manual**), are based largely on precedence. For instance, the bolts at the supported member of a double-angle connection are typically not designed to resist any eccentricity, though logically an eccentricity could exist. An argument can be made that the flexing of the angles relieves the eccentricity, and therefore the bolts do not have to be designed to resist this rotation. However, the support must now take this neglected moment. The argument can then be made that the eccentricity is small, and the supporting member probably has some excess capacity. All of these are qualitative arguments with little analytical basis. The only real justification that can be found to support this assumption is decades of satisfactory performance. Precedence should not be overlooked as a valid justification for engineering practices, but it must be used with caution and must be evaluated whenever paradigm shifts occur in design philosophies, especially when these shifts involve load determination or resistance factors.

Connection designs or design assumptions can also be justified by testing. This approach has been used to develop a handful of essentially prescribed connections, the standard single plate shear connection being the most notable. For many, this approach may be considered the "gold standard" for justifying a connection design, but it requires a great deal of financial investment, sometimes with relatively little return, since results are often valid only for a range of strictly defined parameters. Greater benefits from testing are more often achieved when an analytical model can be found to predict the results of testing. This analytical model can then be applied to a wider range of conditions. Often testing is performed to determine the effects of a single limit state. These data are then used to develop a model for use with more complex conditions.

The final and most common way to justify a connection design is through analysis. Precedence, testing, and engineering theory and judgment are coalesced to produce a rationale to justify the connection to be used. This is the art of connection design. Simple tests are extrapolated to more complex configurations. Load paths are analyzed and optimized. Assumptions are scrutinized to ensure their validity. In some cases these procedures are clearly codified. In many others they are not. The tools are essentially the same as those used in main-member design: statics to satisfy equilibrium, mechanics of materials to confirm strength and determine load paths, and statistical analysis to determine reliability. When combined with sound engineering judgment, these tools allow the connection design engineer to provide safe and economical connections for structural steel.

3.1.3 Choosing Load Distributions—Reconciling As-Built with As-Modeled

As previously stated, the as-built condition seldom re-creates the assumed as-modeled condition accurately. This fact sets up a paradox for the connection design engineer, who is charged with bringing the analytical model into existence. The connections must be configured in an attempt to re-create the assumed behavior, while at the same time recognizing that practical limitations prevent an exact re-creation.

As an example, consider a 30-ft-long W16 × 30 beam that supports a uniform load of 1.8 kips/ft. During the design of the beam, the beam ends are assumed to be pinned at the supports. For ease of erection, the connection design engineer chooses to use extended plates from the webs of the column to support the beam. This arrangement places a line of bolts 9 in from the center of the column, and a moment equal to (9 in)(27 kips) = 243 in·kips between the bolts and the center of the support. See Fig. 3.1. The connection design engineer is now faced with the decision as to where to take the

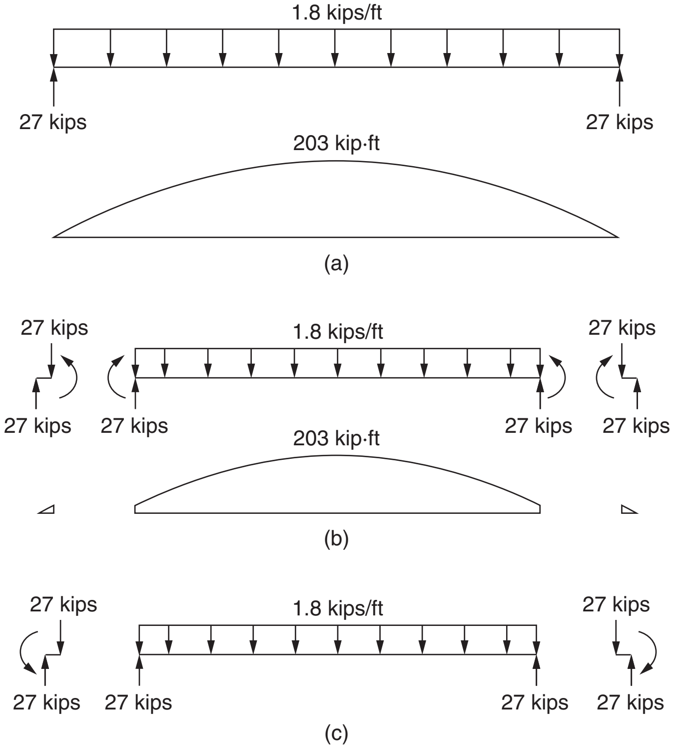


FIGURE 3.1 Free-body diagrams of simply supported beam. (*a*) Distribution assumed when designing the main member. (*b*) Distribution assuming the connection to the column is pinned and the bolts resist the eccentricity. (*c*) Distribution assuming the bolted connection is pinned and the connection to the column resists the eccentricity.

eccentricity. Since the objective is to maintain the original assumptions made during the analysis, the connection design engineer should choose to resist the eccentricity at the bolt line. This approach adds no additional load to the column, and lacking further information, is the safest approach. An alternative approach is to assume that the connection is pinned 9 in from the center of the support and that the eccentricity is taken at the column. This approach will add moment to the column, but the column may be able to accept it. Some analysis and design programs allow the eccentricity to be taken into account in the initial design.

Note that both approaches are only assumptions about the way the structure will behave. If the connection is designed so that the bolts can take all the eccentricity, this does not mean that they will actually see all the eccentricity. Since any practical beam-to-column connection will result in some rotational stiffness at the column, the column will undoubtedly experience some additional moment. Likewise, if the connection is designed assuming no rotational stiffness at the bolts, this does not mean that it will not resist some moment in practice. The loads will distribute based on the relative stiffnesses. Therefore, the actual load distribution will be somewhere between the two assumptions. However, if the connection is designed based on the first assumption, that the bolts will resist all the moment, then the resulting connection will be capable of delivering all of the intended load to the column regardless of the column's ability to support the additional moment. This is an example of the lower-bound theorem, which states that the applied external forces in equilibrium with the internal force field are less than, or at most equal to, the applied external force that would cause failure, provided that all the limit states are satisfied and sufficient ductility exists to allow redistribution of the forces.

3.1.4 Limit States for Connection Design

Limit states for connection elements are arrived at in a similar fashion as those for main-member design. Limit states that could result in sudden, fracture-type failures are required to have greater safety factors, or greater reliabilities, than limit states associated with yielding. Bolt and weld failures are treated as fracture-type failures, and are therefore required to be designed at the higher reliability level. Plates, angles, and other connection elements are designed to reliabilities based on the individual modes of failure in the same way that main members are designed. Generally, connections are not required to be designed to a higher reliability than the members they connect.

3.1.5 Ductility of "Pinned" Connections

In theory, a pinned connection will have no rotational stiffness. In reality, simple shear connections, which have been modeled as pinned during the structural analysis, will have varying degrees of rotational stiffness. The key, then, is to allow sufficient rotation to develop the simple end-beam rotations without fracturing the connection. This is accomplished through various means.

For double-angle, single-angle, end-plate, and tee shear connections, flexing of the connecting element accommodates the simple beam-end rotation. For seated connections, the top or side stability angle should be sized such that the simple beam-end rotation can be accommodated. For single-plate shear connections with either one or two vertical rows of bolts, bolt plowing at the plate can accommodate the simple beam-end rotation. For other types of single-plate shear connections, simple beam-end rotation is accommodated by flexing of the plate.

For tees cut from wide flange sections, double angles, and end plates, the thickness of the connection material at the support can be related to the minimum bolt diameter required to develop the simple beam-end rotation by the equation presented by Thornton (1996, 1995a)

$$d_b = 0.892t\sqrt{\frac{F_y s}{F_t b}\left(\frac{b^2}{L^2}+2\right)} \tag{3.1}$$

where t = thickness of end plate, tee flange, or angle leg, in
F_y = yield stress of endplate, tee, or angle, ksi
F_t = tensile strength of bolt, ksi
s = bolt spacing, in
b = flexible width of connection element, in
L = depth of connection element, in

Assuming A325 bolts (F_t = 90 ksi) and s = 3 in, the equation reduces to the relationship in the AISC Manual:

$$d_b = 0.163t\sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2}+2\right)} \tag{3.2}$$

When connections are welded to the support, the 70-ksi weld size, w, must be such that

$$w = 0.0158\frac{F_y t_f^2}{b}\left(\frac{b^2}{L^2}+2\right) \tag{3.3}$$

All of the above minimums are calculated assuming an end rotation of 0.03 rad, which exceeds the beam-end rotation of most beams when a plastic hinge forms at the center.

To prevent fracture at the weld of a single-plate shear connection, the plate is designed to yield before the weld fractures. From Astaneh (1989), interaction curves relating the shear and the moment caused by the shear can be conservatively approximated by an ellipse:

For the weld
$$\left(\frac{V}{V_w}\right)^2 + \left(\frac{M}{M_w}\right)^2 \le 1 \tag{3.4}$$

For the plate
$$\left(\frac{V}{V_y}\right)^2 + \left(\frac{M}{M_y}\right)^2 \le 1 \tag{3.5}$$

where V = applied shear
M = moment caused by shear = V_e
V_w = shear capacity of weld $= 2\frac{1}{\sqrt{2}} wL(0.6F_{\text{EXX}})$
M_w = moment capacity of weld $= 2\frac{1}{\sqrt{2}} w\frac{L^2}{4}(0.9F_{\text{EXX}})$
V_y = shear capacity of plate = $tL(0.6F_y)$
M_y = moment capacity of plate $= t\frac{L^2}{4}(0.6F_y)$
F_{EXX} = electrode strength classification, ksi

Since the plate must yield before the weld fractures,

$$\left(\frac{V}{V_y}\right)^2 + \left(\frac{M}{M_y}\right)^2 \le \left(\frac{V}{V_w}\right)^2 + \left(\frac{M}{M_w}\right)^2 \tag{3.6}$$

Solving for w, the weld size, in terms of t, the plate thickness, yields

$$w \ge \frac{tF_y\sqrt{1.39 + 9.89(e/L)^2}}{F_{\text{EXX}}\sqrt{2.78 + 16(e/L)^2}} \tag{3.7}$$

Substituting $F_y = 50$ ksi and $F_{\text{EXX}} = 70$ ksi, the above inequality yields

$w \ge 0.562t$ as e/L approaches infinity
$w \ge 0.505t$ as e/L approaches zero

Therefore, the required weld size is $0.562t$. For A36 steel, a slightly smaller weld size can be achieved, but due to the considerable overstrength typical of A36 steel, it is advisable to size the weld based on the Grade 50 value.

To prevent bolt fracture at a single-plate shear connection, the plate must deform sufficiently to redistribute unanticipated moments prior to bolt fracture. This can be achieved through two different actions. First, the bolts can plow through the material. In order for such plowing to occur, the thickness of the material joined cannot exceed one-half the diameter of the bolt, and sufficient edge distance must be present to prevent the bolt from tearing through the edge of the material. An edge distance of twice the bolt diameter is usually assumed.

The second action requires that the plate yield in bending prior to the bolts failing in shear. As an example, consider a two-row, two-column connection with 1-in-diameter A490-X bolts in single shear and F_y = 50-ksi steel. The assumed bolt strength here is 1.25 times the nominal strength = $1.25F_nA_b = 1.25[75(0.5)^2\pi] = 73.6$ kips/bolt. The bolt strength is assumed to be 25% higher than the specified nominal strength, because a 20% reduction in bolt strength is assumed in the AISC Specification to account for nonuniform loading in end-loaded connections. Since this is not an end-loaded connection, this reduction is not taken. The moment capacity of the connection is found to be 615 ft-kips. The maximum thickness for a 6-in-deep plate ($L = 6$ in) to facilitate plate yielding before bolt shearing is

$$t_{p\max} = \frac{6M}{F_y L^2} = \frac{6(615)}{50(6)^2} = 2.0 \text{ in} \tag{3.8}$$

3.1.6 Workpoints and Transfer Forces

Main-member design is usually performed with the members represented and analyzed as one-dimensional elements. Members are usually arranged so that the axial forces act concentrically at a point, thereby eliminating the need to consider additional moments in the member design. In practice, however, connecting multiple members to a single point can be difficult, if not impossible. Also, the need to support other elements of the structure, such as a floor slab or cladding, may force the members to move from their assumed concentric positions into an eccentric configuration. One common condition occurs when beams of different depths are required to transfer an axial load across a joint. Typically these beams are assumed to share a common mid-depth elevation during analysis, but in reality they will be positioned to a common top of steel elevation. This situation will result in moments being transferred to the main members, regardless of the approach used to design the connections.

Transfer forces are forces that are transmitted across joints in a structure. Such forces can occur in horizontal and vertical bracing systems, trusses, and even in beams that are not connected directly to braces. Both lateral and gravity loads can induce transfer forces. When lateral loads are delivered from a diaphragm system, such as a floor slab, into a skeletal system, such as a vertical bracing system, the beams in the unbraced bays, which transfer load into the braced bays, are sometimes referred to as drag beams or collector beams, denoting the fact that these beams collect or drag forces from one system and deliver it to another.

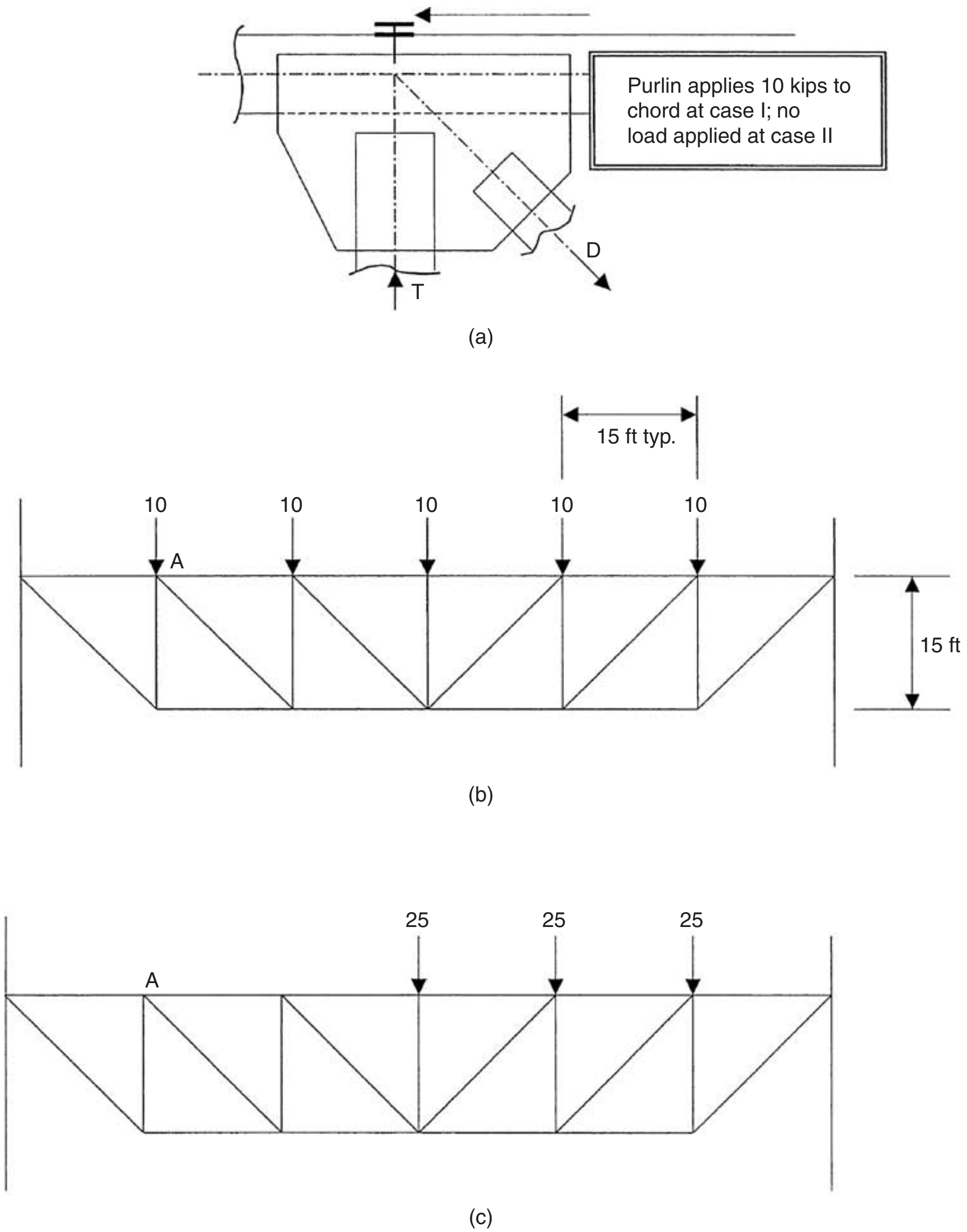
Transfer Force Example 1. It is often thought that the maximum transfer force can be determined from the maximum member forces in a system. This is not always correct, as can be shown using the relatively simple case of a roof truss subjected to uniform snow and snow drift loads, Fig. 3.2. In Case I (uniform snow load), the vertical transfer force (from the gusset to the chord at point A) is obviously 10.0 kips, but in Case II (snow drift load) the vertical transfer force is 0 kips, even though the member forces are larger (the maximums) for the second load case. Forces at point A are summarized below. This analysis becomes much more complex for larger structures subjected to both lateral and gravity loads.

Case	Force in vertical, kips	Force in diagonal, kips	Vertical transfer force, kips
I	25 (C)	21.2 (T)	10
II	25 (C)	35.4 (T)	0

C = compression
T = tension

Since the path that lateral loads take through the structure is often complex and involves numerous systems, both skeletal and diaphragm, and encompassing multiple load cases, determination of the required transfer forces at each joint can be cumbersome. A common mistake is to confuse transfer forces with member forces within a vertical bracing system. The transfer forces must be transferred through the beam-to-column connection from one bay to the next, while the member forces remain within a single bay.

Transfer Force Example 2. A typical bracing connection is shown in Fig. 3.3*a*. For the bracing along line 2 in Fig. 3.3*b*, the only transfer forces that may exist are those at the edge of the structure.



Purlin applies 10 kips to chord at case I; no load applied at case II

FIGURE 3.2 Transfer forces in a roof truss. (*a*) Connection configuration at point A. (*b*) Case I—uniform snow load. (*c*) Case II—snow drift load.

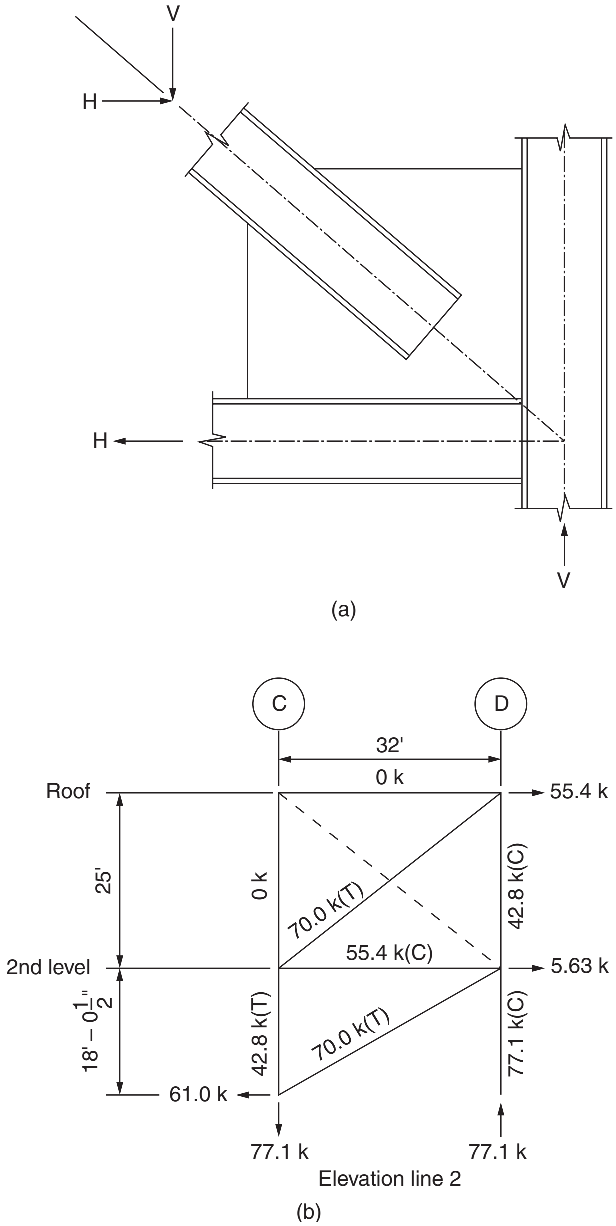
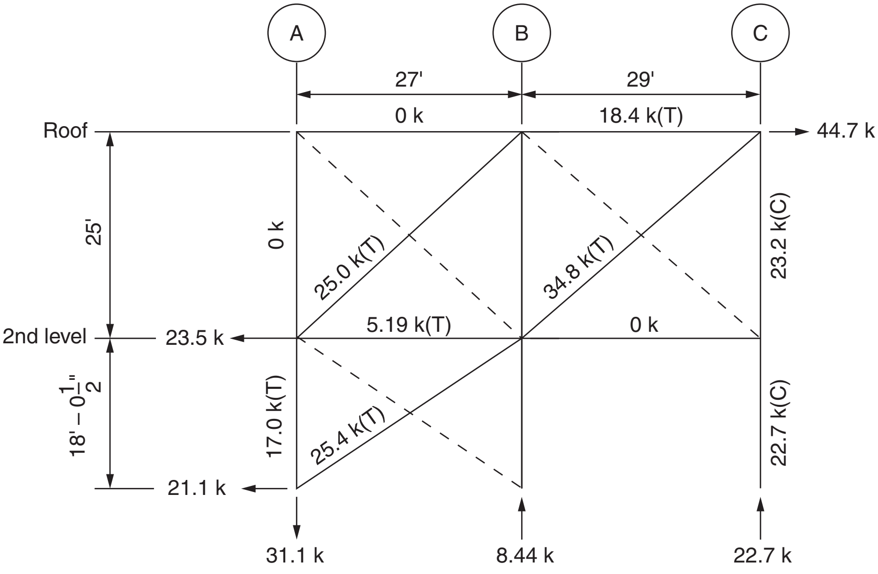
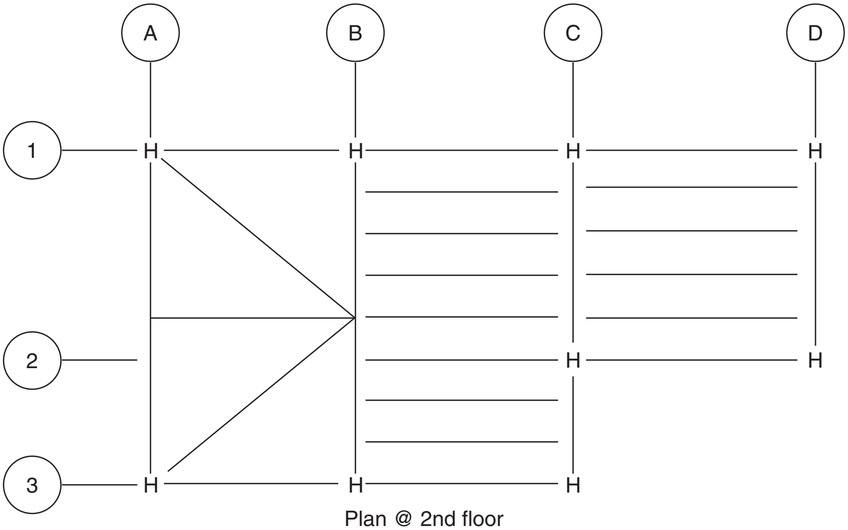


FIGURE 3.3 Analysis of a concentrically braced structure. (*a*) Assumed configuration of the bracing connection. (*b*) Elevation along line 2. (*c*) Elevation along line 3. (*d*) Plan at second floor.



Elevation line 3

(c)



Plan @ 2nd floor

(d)

FIGURE 3.3 (*Continued*)

These forces, however, may be transferred through the diaphragm and directly to the horizontal members, so that no transfer forces must be designed for. The 55.4-kip force at the second level is a member force and is not transferred through the beam-to-column connection. Designing the beam-to-column connection for an axial force of 55.4 kips would be extremely conservative, unnecessary, and costly.

For the bracing along line 3 in Fig. 3.3*c*, significant transfer forces occur at line B at both the roof and the second floor. In both cases the transfer force is equal to the horizontal component of the brace. At the roof this horizontal component, and therefore the transfer force, is equal to 18.4 kips, which is also the member force since the floor beam between lines A and B is a zero-force member. However, at the second floor, the transfer force through the column is 26.4 kips. If the connection to the column at line B were designed for the member force of the beams between A and B and B and C instead of the transfer force, the axial load that would be designed for would be zero or at most 5.19 kips, resulting in an unconservative design.

3.2 DESIGN OF FASTENERS AND WELDS

3.2.1 Limitations on Use of Fasteners and Welds

Structural steel fabricators prefer job specifications to state that "shop connections shall be made with bolts or welds" rather than restricting the type of connection that can be used. This allows the fabricator to make the best use of available equipment and to offer a more competitive price.

High-strength bolts may be used in either slip-critical or bearing-type connections. Bearing-type connections have higher allowable loads and should be used where permitted. Also, bearing-type connections may be either fully tensioned or, in most cases, snug-tight. Snug-tight bolts are generally more economical to install and should be allowed, except where loosening or fatigue due to vibration or load fluctuations are design considerations.

Carbon-steel (common) bolts should not be used in connections subject to fatigue.

The AISC Specification imposes special requirements on use of welded splices and similar connections in heavy sections. This includes ASTM A6 hot-rolled shapes with a flange thickness exceeding 2 in and built-up cross sections with plates over 2 in thick, subject to tensile stresses due to tension or flexure, and spliced using complete-joint-penetration groove welds that fuse through the thickness. Charpy V-notch tests are required, as well as special fabrication and inspection procedures. Where feasible, bolted connections are preferred to welded connections for such sections.

3.2.2 Bolts in Combination with Welds

Because of the significant differences in the load-deformation behavior of bolts and welds, it is difficult to properly design connections that employ both to share the load. For this reason the AISC Specification puts severe limitations on the design of connections employing both welds and bolts to resist loads on a common faying surface.

In new work, only longitudinally loaded welds can be considered to share loads with bolts in standard or short slotted holes loaded perpendicular to the axis of the slot. Transversely loaded welds do not have sufficient ductility to allow the bolts to "take up" before the weld fractures. In cases where bolts and welds act together to resist a common load, the capacity of the bolts is reduced by 50%.

These restrictions are sometimes interpreted, incorrectly, to mean that connections in general should not employ both welds and bolts. This is not the intent. For instance, direct flange-welded moment-connected beams can utilize bolted web connections without penalty. The reason is that the flanges are assumed to resist only the moment, while the web is assumed to resist only the shear. The weld access hole separates the web and the flanges sufficiently to allow this assumed behavior in practice.

In welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are assumed to carry the loads present at the time of alteration. The welding only needs to be adequate to carry the additional load. Of course this assumes

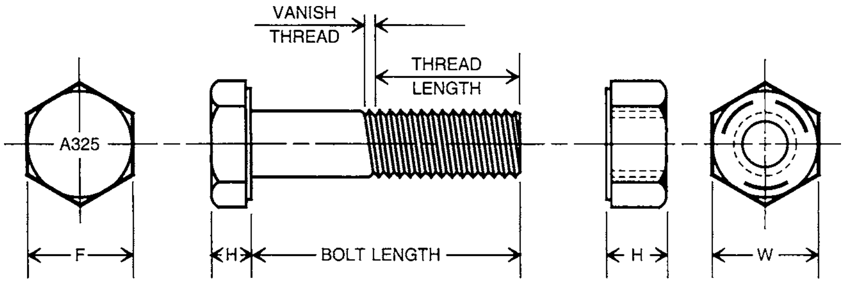


FIGURE 3.4 High-strength structural steel bolt and nut.

that there is no possibility of load reversal, which could overstress the weld before the bolts undergo sufficient deformation to participate.

3.2.3 General Considerations for Fasteners and Washers

In steel fabrication, commonly used fasteners include bolts, welded studs, and pins. Types of bolts that may be used in structural steel connections include high-strength bolts (ASTM A325 and ASTM A490) and common (carbon steel) bolts (ASTM A307). See Figs. 3.4 and 3.5. Since common bolts cannot be installed fully tensioned, their use is limited primarily to shear connections that do not experience fatigue. High-strength bolts are suitable for all structural steel connections. However, since A490 bolts cannot be galvanized, they should not be specified for use with galvanized work.

Washer requirements for connections with high-strength bolts, as given by the RCSC Specification ("Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections, AISC, Chicago, 2004), are as follows:

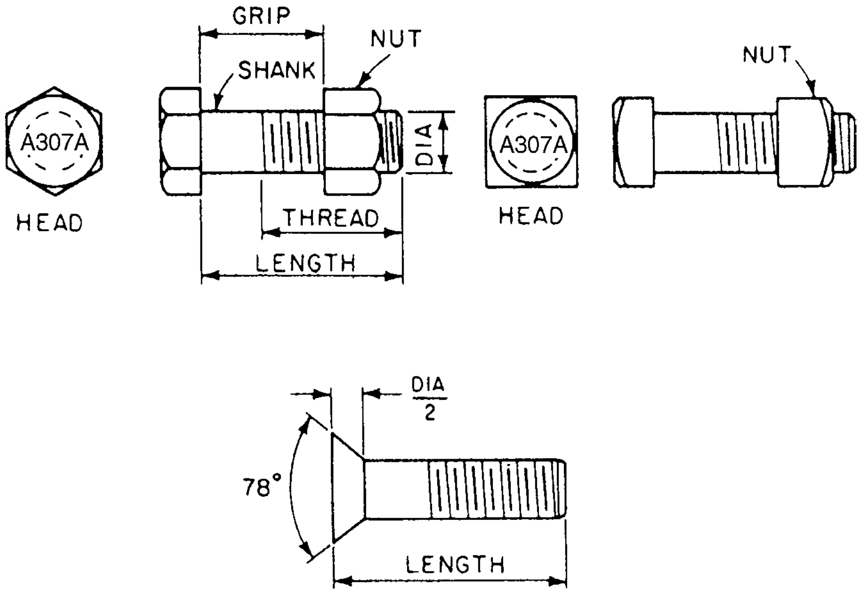


FIGURE 3.5 Unfinished (machine) or common bolts.

1. A hardened beveled washer should be used to compensate for the lack of parallelism where the outer face of the bolted parts has a greater slope than 1:20 with respect to a plane normal to the bolt axis.
2. For A325 and A490 bolts for slip-critical connections and connections subject to direct tension, hardened washers are required as specified in items 3 through 7 below. For bolts permitted to be tightened only snug-type, if a slotted hole occurs in an outer ply, a flat hardened washer or common plate washer shall be installed over the slot. For other connections with A325 and A490 bolts, hardened washers are not generally required.
3. When the calibrated-wrench method is used for tightening the bolts, hardened washers shall be used under the element turned by the wrench.
4. For A490 bolts tensioned to the specified tension, hardened washers shall be used under the head and nut in steel with a specified yield point less than 40 ksi.
5. A hardened washer conforming to ASTM F436 shall be used for A325 or A490 bolts 1 in or less in diameter tightened in an oversized or short slotted hole in an outer ply.
6. Hardened washers conforming to F436 but at least $^5/_{16}$ in thick shall be used, instead of washers of standard thickness, under both the head and nut of A490 bolts more than 1 in in diameter tightened in oversized or short slotted holes in an outer ply. This requirement is not met by multiple washers even though the combined thickness equals or exceeds $^5/_{16}$ in.
7. A plate washer or continuous bar of structural-grade steel, but not necessarily hardened, at least $^5/_{16}$ in thick and with standard holes, shall be used for an A325 or A490 bolt 1 in or less in diameter when it is tightened in a long slotted hole in an outer ply. The washer or bar shall be large enough to cover the slot completely after installation of the tightened bolt. For an A490 bolt more than 1 in in diameter in a long slotted hole in an outer ply, a single hardened washer (not multiple washers) conforming to F436, but at least $^5/_{16}$ in thick, shall be used instead of a washer or bar of structural-grade steel.

The requirements for washers specified in items 4 and 5 above are satisfied by other types of fasteners meeting the requirements of A325 or A490 and having a geometry that provides a bearing circle on the head or nut with a diameter at least equal to that of hardened F436 washers. Such fasteners include “twist-off” bolts with a splined end that extends beyond the threaded portion of the bolt. During installation, this end is gripped by a special wrench chuck and is sheared off when the specified bolt tension is achieved.

The RCSC Specification also permits direct tension-indicating devices, such as washers incorporating small, formed arches designed to deform in a controlled manner when subjected to the tightening force. The specification provides guidance on use of such devices to assure proper installation.

Carbon-steel bolts (also referred to as machine, common, or ordinary bolts) can prove economical in a number of applications. “Secondary connections may be made with unfinished bolts conforming to the Specification for Carbon Steel Bolts and Studs, ASTM A307” is an often-used job specification. When this specification is used, secondary connections should be carefully defined to preclude selection by ironworkers of the wrong type of bolt for a connection. A307 bolts generally have no identification marks on their square, hexagonal, or countersunk heads, as high-strength bolts do. Use of high-strength bolts where A307 bolts can provide the required strength merely adds to the cost of a structure. High-strength bolts cost at least 10% more than A307 bolts. A disadvantage of A307 bolts is the possibility that the nuts may loosen, but this may be eliminated by use of lock washers. Alternatively, lock nuts can be used or threads can be jammed, but either is more expensive than lock washers. Also, if A307 bolts in a connection carry calculated stress and have grips exceeding five diameters, the number of these fasteners used in the connection must be increased 1% for each additional $^1/_{16}$ in in the grip.

Fastener diameters for building construction should be $^1/_2$ in or more, and diameters of $^3/_4$, $^7/_8$, and 1 in are preferred. In general, a connection with a few large-diameter fasteners costs less than one of the same capacity with many small-diameter fasteners. The fewer the fasteners, the fewer the number of holes that must be formed and the less is the installation work required. Larger-diameter fasteners are generally favored in connections, because the available strength (load capacity) of a fastener varies with the square of the fastener diameter.

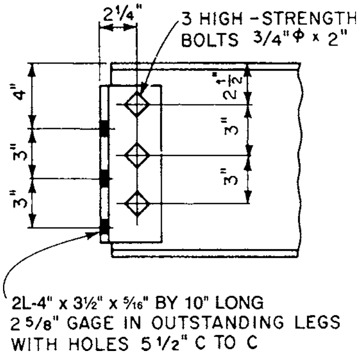


FIGURE 3.6 Staggered holes provide clearance for high-strength bolts.

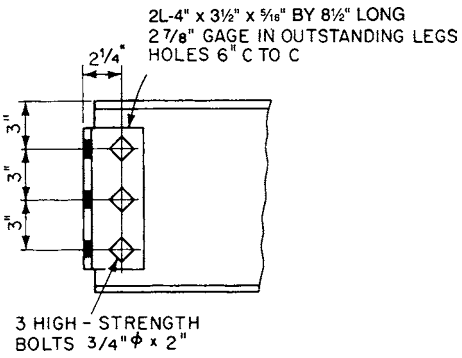


FIGURE 3.7 Increasing the gage in framing angles provides clearance for high-strength bolts.

Standard fastener holes for bolts are $^{1}/_{16}$ in larger than the nominal fastener diameter. In computing net area of a tension member, the diameter of the hole should be taken $^{1}/_{16}$ in larger than the hole diameter, to account for deformation that can occur around the hole during punching and drilling. The AISC Specification requires that the holes be punched or drilled, or cut thermally, with a surface roughness not exceeding 1000 μin. The method used varies with the available equipment and the thickness of the material. Punching is the most economical method in many cases. Holes for thick material may be either drilled from the solid or subpunched and reamed. The die for all subpunched holes and the drill for all subdrilled holes should be at least $^{1}/_{16}$ in smaller than the nominal fastener diameter.

Clearance for fasteners must be ample to provide for tightening high-strength bolts. Detailers who prepare shop drawings for fabricators generally are aware of the necessity of this and can, with careful detailing, secure necessary space. In tight situations, the solution may be staggering of holes (Fig. 3.6), variations from standard gages (Fig. 3.7), use of knife connections, or use of a combination of shop welds and field bolts. Minimum clearances for tightening high-strength bolts are indicated in Fig. 3.8 and Table 3.1.

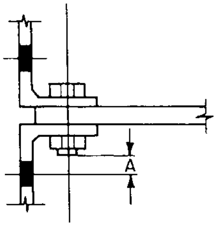


FIGURE 3.8 The usual minimum clearances *A* for high-strength bolts are given in Table 3.1.

Fastener spacing includes consideration of pitch and gage. **Pitch** is the distance (in) along the line of principal load between centers of adjacent fasteners. It may be measured along one or more lines of fasteners. For example, suppose bolts are staggered along two parallel lines. The pitch may

TABLE 3.1 Clearances for High-Strength Bolts

			Min. clearance *A** for twist-off bolts, in	
Bolt diameter, in	Nut height, in	Usual min. clearance *A**, in	Small tool	Large tool
$^{5}/_{8}$	$^{5}/_{8}$	1	$1^{5}/_{8}$	—
$^{3}/_{4}$	$^{3}/_{4}$	$1^{1}/_{4}$	$1^{5}/_{8}$	$1^{7}/_{8}$
$^{7}/_{8}$	$^{7}/_{8}$	$1^{3}/_{8}$	$1^{5}/_{8}$	$1^{7}/_{8}$
1	1	$1^{7}/_{16}$	—	$1^{7}/_{8}$
$1^{1}/_{8}$	$1^{1}/_{8}$	$1^{9}/_{16}$	—	—
$1^{1}/_{4}$	$1^{1}/_{4}$	$1^{11}/_{16}$	—	—

*See Fig. 3.8.

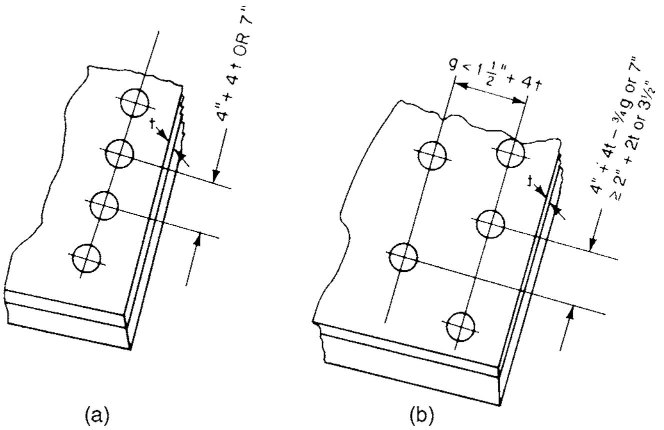


FIGURE 3.9 Maximum pitch of bolts for sealing. (*a*) Single line of bolts. (*b*) Double line of bolts.

be given as the distance between successive bolts in each line separately, or it may be given as the distance, measured parallel to the fastener lines, between a bolt in one line and the nearest bolt in the other line. **Gage** is the distance (in) between adjacent lines of fasteners along which pitch is measured, or the distance (in) from the back of an angle or other shape to the first line of fasteners.

The minimum distance between centers of fasteners should be at least three times the fastener diameter. The AISC Specification, however, permits it to be 2⅔ times the fastener diameter.

Limitations also are set on maximum spacing of fasteners, for several reasons. In built-up members, stitch fasteners, with restricted spacings, are used between components to ensure uniform action. Also, in compression members, such fasteners are required to prevent local buckling. Bolted joints in unpainted weathering steel require special limitations on pitch: 14 times the thickness of the thinnest part, not to exceed 7 in (AISC Specification). AASHTO sealing limits for pitch are shown in Fig. 3.9.

Minimum edge distance of fasteners, based on the AISC Specification, are summarized in Table 3.2. The AISC Specification includes the following provisions: The distance from the center of a standard

TABLE 3.2 Minimum Edge Distances[a] from Center of Standard Hole[b] to Edge of Connected Part for Fastener Holes in Steel Buildings

Bolt diameter, in	At sheared edges, in	At rolled edges of plates, shapes, or bars or gas-cut edges,[c] in
½	⅞	¾
⅝	1⅛	⅞
¾	1¼	1
⅞	1½[d]	1⅛
1	1¾[d]	1¼
1⅛	2	1½
1¼	2¼	1⅝
Over 1¼	1¾ × diam.	1¼ × diam.

[a] Lesser edge distances may be used provided equations from AISC Specification Sec. J3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see AISC Specification Table J3.5.

[c] All edge distances in this column may be reduced ⅛ in when the hole is at a point where stress does not exceed 25% of the maximum strength in the element.

[d] These may be 1¼ in at ends of beam connection angles and shear end plates.

hole to an edge of a connected part should not be less than the applicable value from Table 3.2 unless smaller distances are justified by analysis (see footnote *a* of Table 3.2). Note that Table 3.2 gives edge distances from the center of a standard hole to the edge. This is denoted L_e in the following.

Edge-distance strength limitations for bolted connections are given in the AISC Specification in terms of clear distance L_c, which is related to L_e by

$$L_e = L_c + \frac{d_h}{2} \tag{3.9}$$

where d_h is the hole diameter. For standard holes, oversized holes, short slotted holes regardless of the direction of the loading, and long slotted holes with the load perpendicular to the slot, the nominal strengths are as follows:

When hole deformation is a design consideration:

$$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u \tag{3.10}$$

When hole deformation is not a design consideration:

$$R_n = 1.5 L_c t F_u \leq 3.0 d t F_u \tag{3.11}$$

For long slotted holes with the load parallel to the slot, the nominal strength is

$$R_n = 1.0 L_c t F_u \leq 2.0 d t F_u \tag{3.12}$$

where F_u = minimum specified tensile strength
d = bolt diameter
t = thickness of critical connected part

Maximum edge distances are set for sealing and stitch purposes. The AISC Specification limits the distance from center of fastener to nearest edge of parts in contact to 12 times the thickness of the connected part, with a maximum of 6 in. For unpainted weathering steel, the maximum is 5 in or eight times the thickness of the thinnest outside plate.

3.2.4 Requirements for Fillers

A filler is a plate inserted in a splice between a gusset or splice plate and load-carrying members to fill a gap between them, such as results from differences in section depth. Requirements for fillers included in the AISC Specification for structural steel for buildings are as follows.

In welded construction, a filler $^1/_4$ in or more thick should extend beyond the edge of the splice plate and be welded to the part on which it is fitted (Fig. 3.10). The welds should be able to transmit the splice-plate stress, applied at the surface of the filler, as an eccentric load. The welds that join the splice plate to the filler should be able to transmit the splice-plate stress and should have sufficient length to prevent overstress of the filler along the toe of the welds. A filler less than $^1/_4$ in thick should have edges flush with the splice-plate edges. The size of the welds should equal the sum of the filler thickness and the weld size necessary to resist the splice-plate stress.

In bearing connections with bolts carrying computed stress passing through fillers thicker than $^1/_4$ in, the fillers should extend beyond the splice plate (Fig. 3.11). The filler extension should be secured by sufficient bolts to distribute the load on the member uniformly over the combined cross section of the member and filler. Alternatively, an equivalent number of bolts should be included in the connection. Fillers $^1/_4$ to $^3/_4$ in thick need not be extended if the allowable shear stress in the bolts is reduced by the factor $0.4(t - 0.25)$, where t is the total thickness of the fillers but not more than $^3/_4$ in.

Fillers in slip-critical connections need not be developed by the addition of extra bolts when prevention of slip is required for strength rather than serviceability.

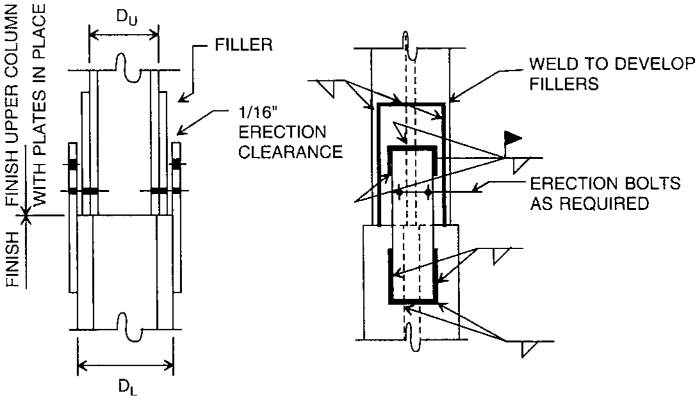


FIGURE 3.10 Typical welded splice of columns when depth D_U of the upper column is nominally 2 in less than depth D_L of the lower column.

3.2.5 Bolt Installation: Snug-Tight versus Fully Tensioned

High-strength bolts (A325 and A490) can be installed either snug-tight or fully tensioned. Common bolts (A307) can only be installed snug-tight. Snug-tight installation is achieved when all plies are in contact. It can be attained by a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench. Fully tensioned installation is achieved when the bolt is stressed in tension to 70% of its tensile strength. The RCSC Specification requires fully tensioned installation for the following conditions:

1. Joints in which fastener pretensioning is required in the specification or code that invokes the Specification
2. Joints that are subjected to significant load reversal
3. Joints that are subjected to fatigue load with no reversal of the loading direction
4. Joints with ASTM A325 or F1852 bolts that are subject to tensile fatigue
5. Joints with ASTM A490 bolts that are subject to tension or combined shear and tension, with or without fatigue

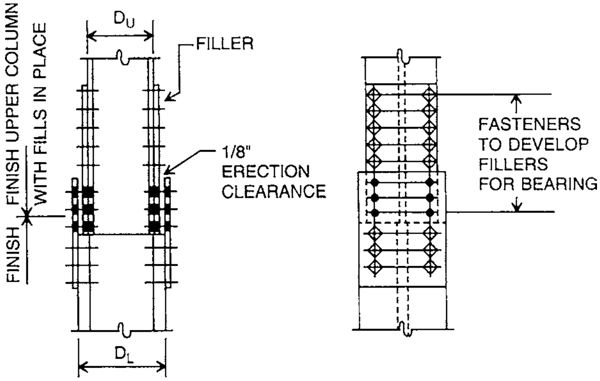


FIGURE 3.11 Typical bolted splice of columns when depth D_U of the upper column is nominally 2 in less than depth D_L of the lower column.

The AISC Specification defines the conditions for item 1 as

1. Column splices in all tier structures 125 ft or more in height
2. Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent, in structures over 125 ft in height
3. In all structures carrying cranes of over 5-ton capacity, roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports
4. Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress

Fully tensioned bolts can be installed using four different methods: calibrated wrench, turn-of-nut, as twist-off-type tension-control bolts, or with direct-tension indicators. In all installation methods, the plies are first brought together as in a snug-tight condition, before tensioning begins. Bolts should be tensioned starting with the most rigid element and moving to the most flexible element, to minimize relaxation in the previously tensioned bolts. The calibrated-wrench method is a torque-controlled method, in which the wrench is calibrated to stop torquing after the required tension is achieved in the bolt. An ASTM F436 washer must be used under the turned element, and the unturned element must be prevented from turning. The wrench should be set to cut off at 5% above the required tension. Because the torque-controlled methods of installation rely on so many variables for proper performance, it is imperative that the wrench be calibrated at least daily, and also when changes occur in the bolting setup such as changes in bolt diameter, hose length, or number of wrenches run off the same air supply. It is also important that fasteners be kept protected from dirt and moisture to ensure that the proper tension is achieved.

In the turn-of-nut method, the specified tension is achieved by turning the nut a specified rotation (Table 3.3), while the unturned element is prevented from turning.

Twist-off-type tension-control bolts consist of a splined end that extends beyond the threaded portion of the bolt. The splined end is held in place by the wrench during installation, so that the nut turns relative to the bolt. When the specified tension is achieved, the splined end is severed and rotation stops. An ASTM F436 washer must be provided under the nut. Like the calibrated-wrench method, the twist-off-type tension-control bolts behave as a torque-controlled installation method. However, since the torque is controlled within the fastener, the variability of the wrench and power supply are eliminated. Nevertheless, it is still important that fasteners be kept protected from dirt and moisture to ensure the proper tension is achieved. If the splined end is severed during the first

TABLE 3.3 Required Nut Rotation for Turn-of-Nut Installation[a,b]

	Disposition of outer face of bolted parts		
Bolt length[c]	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20[d]	Both faces sloped not more than 1:20 from normal to bolt axis[d]
Not more than $4d_b$	$^1/_3$ turn	$^1/_2$ turn	$^2/_3$ turn
More than $4d_b$ but not more than $8d_b$	$^1/_2$ turn	$^2/_3$ turn	$^5/_6$ turn
More than $8d_b$ but not more than $12d_b$	$^2/_3$ turn	$^5/_6$ turn	1 turn

[a] Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For required nut rotations of $^1/_2$ turn and less, the tolerance is ±30°. For required nut rotations of $^2/_3$ turn and more, the tolerance is ±45°.

[b] Applicable only to joints in which all material within the grip is steel.

[c] In terms of bolt diameter, d_b. When bolt length exceeds $12d_b$, the required nut rotation must be determined by testing in a suitable tension calibrator that simulates conditions of solidly fitting steel.

[d] Beveled washer not used.

step of installation, when the plies are being brought into contact, the fastener must be removed and replaced.

Direct-tension indicators are hardened washer-shaped discs with arched protrusions that flatten when the specified tension is achieved. The protrusions must bear against the bolt head or nut or against a hardened flat washer. If the protrusions flatten to the job-inspection gap while the connection is being brought into the snug-tight condition, the direct-tension indicator must be removed and replaced.

3.2.6 Connection Resistance: Bearing versus Slip-Critical Connections

It is common for bolts in structural steel connections to be referred to as being either bearing bolts or slip-critical bolts. This is a misnomer, since the same high-strength bolts can be used for both bearing and slip-critical connections, though common bolts (A307) are restricted to use in bearing connections. In bearing connections, movement within the joint is prevented through contact between the shank of the bolt and the material. In slip-critical connections, movement of the joint is resisted through the friction between the faying surfaces caused by the tension in the bolt. Therefore, though either snug-tight or fully tensioned bolts may be installed in bearing-type connections, only fully tensioned bolts may be installed in slip-critical connections.

By definition, slip-critical connections are required where slip cannot be tolerated, which would seem to be a definitive statement, but in reality there is a range of intolerance to slip. This range can be divided into two distinct levels, strength and serviceability.

Designing a slip-critical connection for serviceability can be viewed in two ways. It can be viewed as designing for a lower safety factor against slip than when designing as a strength limit state, or, more accurately, can be viewed as designing for an equal safety factor with a lower period of nonexceedance. This is analogous to what is done with sway due to wind loads in tall buildings. In such buildings, the limit states that govern strength and stability of the structure are designed to perform satisfactorily for a 50- or 100-year storm, while the sway is limited based on an 8- to 10-year storm. Because of this approach, all limit states for bearing-type connections must also be checked when designing against slip as a serviceability limit state. This includes bearing at the bolt, bolt tear-out, and development of fills. Though currently required by the AISC Specification, it is the opinion of the authors that these limit states, associated with bearing-type connections, need not be checked when designing against slip as a strength limit state.

Slip-criticality should be considered as a strength limit state where slip in the connection could be large enough to alter the usual analysis assumption that the undeformed structure can be used to calculate the internal forces. Examples might include braced frames where oversized holes are used, which could potentially result in large P-delta effects, or long-span roof trusses with oversized holes, where slip could result in excessively large loads due to ponding.

Slip-criticality should be considered as a serviceability limit state where slip in the connection would not violate the analysis assumptions of the structure. Since only a negligible amount of slip can occur at bolts installed in standard holes, these connections should be designed for serviceability, when slip must be resisted. Examples might include structures that contain sensitive communication or testing equipment, where slip is undesirable but would not result in structural failure. Slip should also be viewed as a serviceability limit state when slip-critical connections are used for joints subjected to fatigue load with reversal of the loading direction.

Slip-critical connections are required for very few situations in building design. The RCSC Specification requires the use of slip-critical connections for the following conditions:

1. Joints that are subject to fatigue load with reversal of the loading direction
2. Joints that utilize oversized holes
3. Joints that utilize slotted holes, except those with applied load approximately normal (within 80° to 100°) to the direction of the long dimension of the slot
4. Joints in which slip at the faying surface would be detrimental to the performance of the structure

Items 1, 2, and 3 are quantitative. Item 4 is qualitative and requires judgment. The previous two paragraphs are provided to aid that judgment. Slip in most structures that are not covered by items 1, 2, and 3 is rarely a concern. Specifying slip-critical connections where bearing connections would suffice leads to uneconomical designs, usually with no accompanying increase in the overall safety of the structure.

It should be noted that wind and seismic loads do not produce fatigue loads that would require the use of slip-critical connections. The AISC Specification states that "Fatigue need not be considered for seismic effects or for the effects of wind loading on normal building lateral force-resisting systems and building enclosure components." This is because most such load changes occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. On the other hand, crane runways and supporting structures for machinery and equipment are often subjected to fatigue loading conditions.

3.2.7 Threads-Included and Threads-Excluded Conditions

Bolts in bearing can be designed either assuming that the shear plane passes through the threads (threads included or N type) or that the shear plane does not pass through the threads (threads excluded or X type). It is commonly perceived that special detailing and field installation are required to assure a threads-excluded condition. Though this is true when connecting thin material, it is not true when heavier members are connected. Since bolts are manufactured with a constant thread length that does not vary with the overall length of the bolt, it is easy to calculate the minimum thickness of ply required to achieve a threads-excluded condition. This is shown in Table 3.4. Since it is relatively simple to provide inspection that ensures the use of washers and the 1/4-in bolt "stick-thru" beyond the nut, X-type bolts can be safely assumed for all but the thinnest beam webs.

3.2.8 Surface Class

Since a slip-critical connection relies on friction between the plies to resist movement at the joint, the coefficient of friction between the plies is important in determining slip resistance. The RCSC Specification defines three different surface classes for slip-critical connections, A, B, and C. Class A is the simplest surface class to achieve. It is defined as clean mill-scale steel, so no extensive blasting is required. Class B is defined as a blast-cleaned steel and requires blast cleaning of the members. Since the blast-cleaned Class B surface is often exposed to the elements prior to erection, there is sometimes concern that corrosion may reduce the slip resistance of these connections. However, test results (Yura 1981) have shown that the Class B surface can be maintained for up to 1 year under normal exposure conditions. Both Class A and Class B surfaces can also be achieved through the use of suitable coatings applied to blast-cleaned steel. Class C is used for hot-dip galvanized steel. In order

TABLE 3.4 Minimum Ply Thickness for Threads-Excluded Condition

Bolt diameter, in	Minimum plate thickness, in	Minimum plate thickness with 5/32-in washer, in	Minimum plate thickness with 5/32-in washer and 1/4-in stick-through, in
3/4	0.641	0.485	0.235
7/8	0.641	0.485	0.235
1	0.766	0.610	0.360
1 1/8	0.891	0.735	0.485
1 1/4	0.781	0.625	0.375

TABLE 3.5 Traditional Allowable Loads on Threaded Welded Studs*

Stud size, in	Tension, kips	Single shear, kips
$^5/_8$	6.9	4.1
$^3/_4$	10.0	6.0
$^7/_8$	13.9	8.3
1	18.2	10.9

*ASTM 108, Grade 1015, 1018, or 1020.

to achieve the Class C surface, the faying surfaces must be roughened by hand wire brushing. Power wire brushing should not be used, since it tends to polish rather than roughen the faying surfaces, thereby decreasing the slip resistance. Without the final step of hand wire brushing, the slip resistance is less than half that of a properly prepared Class C surface. Test results (Kulak et al., 1987) and field experience have shown that hot-dip galvanized slip-critical connections may experience creep over time. Thus it is advisable to use only standard holes for hot-dipped galvanized steel.

3.2.9 Specifying Bolts

When working with competent fabricators and detailers, engineers should avoid specifying bolts too tightly. The connection design engineer should be given the latitude to choose bolt grades, diameters, hole types, and connection types, within the limits allowed by AISC and RCSC, to achieve maximum efficiency and economy in both the shop and the field. Slip-critical connections should only be required where specified by AISC and RCSC, unless unusual circumstances exist where slip cannot be tolerated. Slip-critical connections should not be specified to obtain an added factor of safety, since in many cases additional bolts will not translate into additional reliability, but will result in added cost.

To prevent mistakes in the field, it is good practice to avoid choosing bolts with the same diameter but different grades. Therefore, if both A490 and A325 bolts are to be furnished, they should be of different diameters, such as 1-in-diameter A490 bolts and $^7/_8$-in-diameter A325 bolts. Since the oversized hole diameter of a bolt is often the same as the standard hole diameter of the next larger bolt, some engineers also prefer to skip a size to prevent field mistakes. Therefore, they would furnish $^3/_4$-in- and 1-in-diameter bolts, or $^7/_8$-in- and $1^1/_8$-in-diameter bolts, instead of $^7/_8$-in- and 1-in-diameter bolts.

3.2.10 Welded Studs

Studs, fasteners with one end welded to a steel member, frequently are used for connecting material. Shear connectors in composite construction are a common application. Welded studs also are used as anchors to attach wood, masonry, or concrete to steel. Threaded studs can also be used for steel-to-steel connections to cut costs. For example, fastening rail clips to crane girders with studs eliminates drilling of the top flange of the girders and may permit a reduction in flange size.

Types of studs and welding guns vary with manufacturers. Table 3.5 lists traditional allowable loads (safety factor included) for several sizes of threaded studs. Check manufacturer data for studs to be used. Chemical composition and physical properties may differ from those assumed for this table.

In designs with threaded studs, clearance must be provided for stud welds. Usual sizes of these welds are indicated in Fig. 3.12 and Table 3.6. The dimension C given is the minimum required to prevent burn-through in stud welding. Other design considerations may require greater thicknesses.

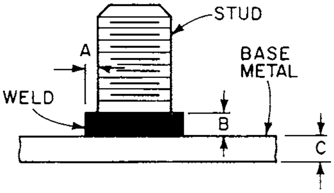


FIGURE 3.12 Welded stud.

TABLE 3.6 Minimum Weld and Base-Metal Dimensions for Threaded Welded Studs*

Stud size, in	Dimension *A*, in	Dimensions *B* and *C*, in
5/8	1/8	1/4
3/4	3/16	5/16
7/8	3/16	3/8
1	1/4	7/16

*Dimensions *A*, *B*, and *C* are shown in Fig. 3.12.

3.2.11 Pins

A pinned connection is used to permit rotation of the end of a connected member. Some aspects of the design of a pinned connection are the same as those of a bolted bearing connection. The pin (Fig. 3.13) serves the same purpose as the shank of a bolt, but since only one pin is present in a connection, forces acting on a pin are generally much greater than those on a bolt. Shear on a pin can be resisted by selecting a large enough pin diameter and an appropriate grade of steel. Bearing on thin webs or plates can be brought within required values by addition of reinforcing plates. Because a pin is relatively long, bending, ignored in bolts, must be investigated in choosing a pin diameter. Arrangements of plates on the pin affect bending stresses. Hence plates should be placed symmetrically and positioned to minimize stresses.

Finishing of the pin and its effect on bearing should be considered. Unless the pin is machined, the roundness tolerance may not permit full bearing, and a close fit of the pin may not be possible. The requirements of the pin should be taken into account before a fit is specified.

Pins may be made of any of the structural steels permitted by the AISC Specification, as well as ASTM A108 Grades 1016 through 1030, and A668 Classes C, D, F, and G. Design requirements for pins are given in Sec. D3 of the AISC Specification.

When reinforcing plates are needed on connected material, the plates should be arranged to reduce eccentricity on the pin to a minimum. One plate on each side should be as wide as the outstanding flanges will permit. At least one full-width plate on each segment should extend to the far

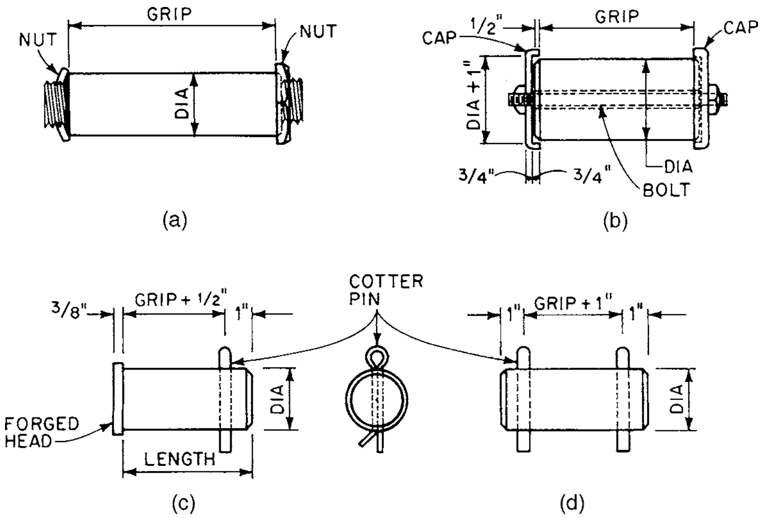


FIGURE 3.13 Pins. (*a*) With recessed nuts. (*b*) With caps and through bolt. (*c*) With forged head and cotter pin. (*d*) With cotter at each end (used in horizontal position).

end of the stay plate. Other reinforcing plates should extend at least 6 in beyond the near edge. All plates should be connected with fasteners or welds arranged to transmit the bearing pressure uniformly over the full section.

In buildings, pin hole diameters should not exceed pin diameters by more than $^1/_{32}$ in. The length of pin should be sufficient to secure full bearing on the turned body of the pin of all connected parts. Pins should be secured in position and connected material restrained against lateral movement on the pins. In building work, a pin may be secured with cotter pins (Fig. 3.13*c* and *d*). The most economical method is to drill a hole in each end for cotter pins. However, this method can be used only for horizontal pins. When a round pin must be turned down to obtain the required fit, a head can be formed to hold the pin at one end. The other end can be held by a cotter pin or threaded for a nut. This headed pin can be used in vertical installations with the head at the upper end.

3.2.12 General Considerations for Welds

Welded connections are used because of simplicity of design, fewer parts, less material, and decrease in shop handling and fabrication operations. Frequently, a combination of shop welding and field bolting is advantageous. With connection angles shop welded to a beam, field connections can be made with high-strength bolts without the clearance problems that may arise in an all-bolted connection.

Weldable structural steels permissible in buildings are listed in AISC Specification A3. Matching electrodes are given in American Welding Society AWS D1.1 (Table 3.1).

Welded connections have a rigidity that can be advantageous if properly accounted for in design. Welded trusses, for example, deflect less than bolted trusses, because the end of a welded member at a joint cannot rotate relative to the other members there. If the end of a beam is welded to a column, the rotation there is practically the same for column and beam.

A disadvantage of welding, however, is that shrinkage of large welds must be considered. This is particularly important in large structures, where there will be an accumulative effect.

Properly made, a weld is stronger than the base metal. Improperly made, even a good-looking weld may be worthless. Properly made, a weld has the required penetration and is not brittle.

Prequalified joints, welding procedures, and procedures for qualifying welders are covered by AWS D1.1, "Structural Welding Code—Steel." Common types of welds with structural steels, intended for welding when made in accordance with AWS specifications, can be specified by note or by symbol with assurance that a good connection will be obtained.

In making a welded design, designers should specify only the amount and size of weld actually required. Generally, a $^5/_{16}$-in weld is considered the maximum size for a single pass. A $^3/_8$-in weld, while only $^1/_{16}$ -in larger, requires three passes and engenders a great increase in cost.

The cost of fit-up for welding can range from about one-third to several times the cost of welding. In designing welded connections, therefore, designers should consider the work necessary for the fabricator and the erector in fitting members together so they can be welded.

3.2.13 Types of Welds

The main types of welds used for structural steel are fillet, groove, plug, and slot. The most commonly used weld is the fillet. For light loads, it is the most economical, because little preparation of material is required. For heavy loads, groove welds are the most efficient, because the full strength of the base metal can be obtained easily. Use of plug and slot welds generally is limited to special conditions where fillet or groove welds are not practical.

More than one type of weld may be used in a connection. If so, the available strength of the connection is the sum of the available strengths of each type of weld used, separately computed with respect to the axis of the group.

Tack welds may be used for assembly or shipping. They are not assigned any stress-carrying capacity in the final structure. In some cases, these welds must be removed after final assembly or erection.

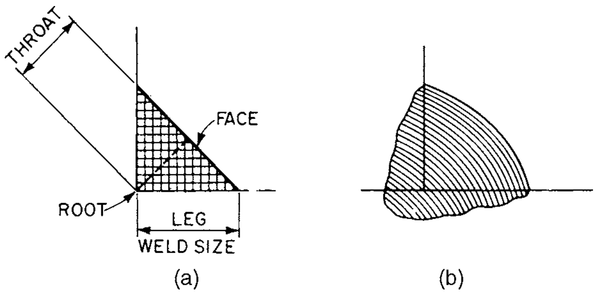


FIGURE 3.14 Fillet weld. (*a*) Theoretical cross section. (*b*) Actual cross section.

Fillet welds have the general shape of an isosceles right triangle (Fig. 3.14). The size of the weld is given by the length of leg. The strength is determined by the throat thickness, the shortest distance from the root (intersection of legs) to the face of the weld. If the two legs are unequal, the nominal size of the weld is given by the shorter of the legs. If welds are concave, the throat is diminished accordingly, and so is the strength.

Fillet welds are used to join two surfaces approximately at right angles to each other. The joints may be lap (Fig 3.15) or tee or corner (Fig 3.16). Fillet welds also may be used with groove welds to reinforce corner joints. In a skewed tee joint, the included angle of weld deposit may vary up to 30° from the perpendicular, and one corner of the edge to be connected may be raised, up to $^3/_{16}$ in. If the separation is greater than $^1/_{16}$ in, the weld leg must be increased by the amount of the root opening. A further discussion of this is given in Art. 3.2.20.

Groove welds are made in a groove between the edges of two parts to be joined. These welds generally are used to connect two plates lying in the same plane (butt joint), but they also may be used for tee and corner joints.

Standard types of groove welds are named in accordance with the shape given the edges to be welded: square, single V, double V, single bevel, double bevel, single U, double U, single J, and double J (Fig. 3.17). Edges may be shaped by thermal cutting, arc-air gouging, or edge planing. Material up to $^3/_8$ in thick, however, may be groove welded with square-cut edges, depending on the welding process used.

Groove welds should extend the full width of the parts joined. Intermittent groove welds, and butt joints not fully welded throughout the cross section, are prohibited.

Groove welds also are classified as complete-penetration and partial-penetration welds.

In a **complete-joint-penetration weld,** the weld material and the base metal are fused throughout the depth of the joint. This type of weld is made by welding from both sides of the joint or from one side to a backing bar. When the joint is made by welding from both sides, the root of the first-pass weld is chipped or gouged to sound metal before the weld on the opposite side, or back pass, is made. The throat dimension of a complete-joint-penetration groove weld, for stress computations, is the full thickness of the thinner part joined, exclusive of weld reinforcement.

Partial-joint-penetration welds should be used when forces to be transferred are less than those requiring a complete-joint-penetration weld. The edges may not be shaped over the full joint

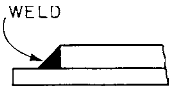


FIGURE 3.15 Welded lap joint.

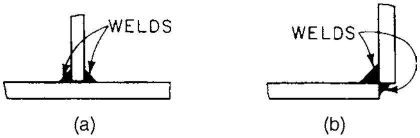


FIGURE 3.16 Welded joints. (*a*) Tee joint. (*b*) Corner joint.

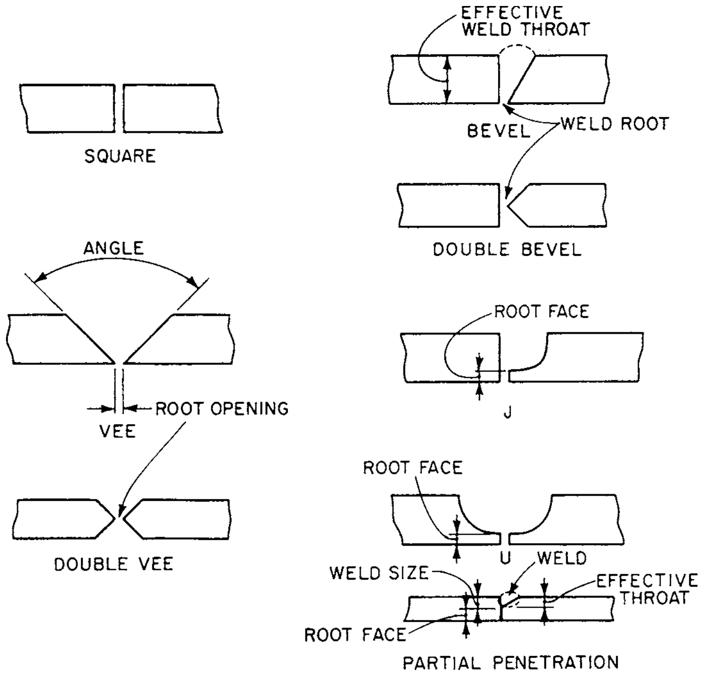


FIGURE 3.17 Groove welds.

thickness, and the depth of the weld may be less than the joint thickness (Fig. 3.18). However, even if the edges are fully shaped, groove welds made from one side without a backing bar or made from both sides without back gouging are considered partial-joint-penetration welds. They are often used for splices in building columns carrying axial loads only.

Plug welds and **slot welds** are used to transmit shear in lap joints and to prevent buckling of lapped parts. In buildings, they also may be used to join components of built-up members. (Plug or slot welds, however, are not permitted on A514 steel.) The welds are made, with lapped parts in contact, by depositing weld metal in circular or slotted holes in one part. The openings may be partly or completely filled, depending on their depth. Load capacity of a plug or slot completely welded equals the product of hole area and allowable stress. Unless appearance is a main consideration, a fillet weld in holes or slots is preferable.

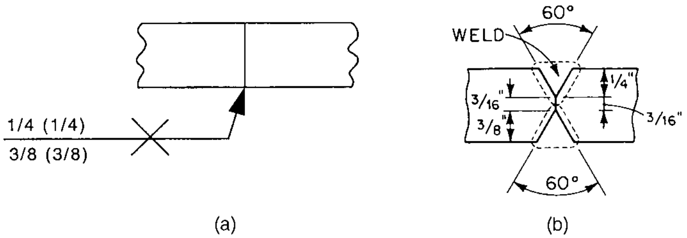


FIGURE 3.18 Penetration information given on the welding symbol in (*a*) for the weld shown in (*b*). Penetration must be at least $^5/_8$ in.

3.2.14 Economy in Weld Type Selection

In selecting a weld, designers should consider not only the type of joint but also the type of weld that will require a minimum amount of metal. This will yield a saving in both material and time.

While the strength of a fillet weld varies with size, the volume of metal varies with the square of the size. For example, a $^1/_2$-in fillet weld contains four times as much metal per inch of length as a $^1/_4$-in weld but is only twice as strong. In general, a smaller but longer fillet weld costs less than a larger but shorter weld of the same capacity.

Furthermore, small welds can be deposited in a single pass. Large welds require multiple passes. They take longer, absorb more weld metal, and cost more. As a guide in selecting welds, Table 3.7 lists the number of passes required for some frequently used types of welds. The values in this table are only approximate. The actual number of passes can vary depending on the welding process used.

Double-V and double-bevel groove welds contain about half as much weld metal as single-V and single-bevel groove welds, respectively (deducting effects of root spacing). Cost of edge preparation and added labor of gouging for the back pass, however, should be considered. Also, for thin material, for which a single weld pass may be sufficient, it is uneconomical to use smaller electrodes to weld from two sides. Furthermore, poor accessibility or less favorable welding position (Art. 3.2.17) may make an unsymmetrical groove weld more economical, because it can be welded from only one side.

When bevel or V grooves can be flame-cut, they cost less than J and U grooves, which require planning or arc-air gouging.

3.2.15 Weld Size and Length Limitations

For a given size of fillet weld, the cooling rate is faster and the restraint is greater with thick plates than with thin plates. To prevent cracking due to resulting internal stresses, the AISC Specification (Sec. J2.2) sets minimum sizes for fillet welds depending on plate thickness, Table 3.8.

To prevent overstressing of base material at a fillet weld, the maximum weld size is limited by the strength of the adjacent base metal.

TABLE 3.7 Number of Passes for Welds

		Single-bevel groove welds (back-up weld not included)		Single-bevel groove welds (back-up weld not included)		
Weld size,* in	Fillet welds	30° bevel	45° bevel	30° open	60° open	90° open
$^3/_{16}$	1					
$^1/_4$	1	1	1	2	3	3
$^5/_{16}$	1					
$^3/_8$	3	2	2	3	4	6
$^7/_{16}$	4					
$^1/_2$	4	2	2	4	5	7
$^5/_8$	6	3	3	4	6	8
$^3/_4$	8	4	5	4	7	9
$^7/_8$		5	8	5	10	10
1		5	11	5	13	22
$1^1/_8$		7	11	9	15	27
$1^1/_4$		8	11	12	16	32
$1^3/_8$		9	15	13	21	36
$1^1/_2$		9	18	13	25	40
$1^3/_4$		11	21			

*Plate thickness for groove welds.

TABLE 3.8 Minimum Plate Thickness for Fillet Welds

Size of fillet weld,[a] in	Maximum plate thickness,[b] in	Minimum plate thickness[b] for fillet welds on each side of plate, in Steel: $F_y = 36$ ksi	Steel: $F_y = 50$ ksi
1/8[c]	1/4		
3/16	1/2	0.32	0.29
1/4	3/4	0.43	0.38
5/16	Over 3/4	0.53	0.48

[a] Weld size need not exceed the thickness of the thinner part joined, but the AISC requires that care be taken to provide sufficient preheat to ensure weld soundness.
[b] Plate thickness is the thickness of the thinner part joined.
[c] Minimum weld size for structures subjected to dynamic loads is 3/16 in.

A limitation is also placed on the maximum size of fillet welds along edges. One reason is that edges of rolled shapes are rounded, and weld thickness consequently is less than the nominal thickness of the part. Another reason is that if weld size and plate thickness are nearly equal, the plate comer may melt into the weld, reducing the length of weld leg and the throat. Hence, the AISC Specification (Sec. J2.2b) requires the following: "Along edges of material less than 1/4 in thick, maximum size of fillet weld may equal material thickness. But along edges of material 1/4 in or more thick, the maximum size should be 1/16 in less than the material thickness." Weld size may exceed this, however, if drawings definitely show that the weld is to be built out to obtain full throat thickness.

AWS D1.1 requires that the minimum effective length of a fillet weld be at least four times the nominal size, or else the weld must be considered not to exceed 25% of the effective length. Subject to the preceding requirements, intermittent fillet welds maybe used in buildings to transfer calculated stress across a joint or faying surfaces when the required strength is less than that developed by a continuous fillet weld of the smallest permitted size. Intermittent fillet welds also may be used to join components of built-up members in buildings. Intermittent welds are advantageous with light members where excessive welding can result in straightening costs greater than the cost of welding. Intermittent welds often are sufficient and less costly than continuous welds. An exception is girder web-to-flange fillet welds, where automatic welding equipment makes continuous welds preferable.

Weld lengths specified on drawings are effective weld lengths. They include distances needed for start and stop of welding. No reduction in effective length need be made in design calculations for the start and stop weld craters. To avoid the adverse effects of starting or stopping a fillet weld at a corner, welds extending to corners should be returned continuously around the corners in the same plane for a distance of at least twice the weld size. This applies to side and top fillet welds connecting brackets, beam seats, and similar connections, on the plane about which bending moments are computed. End returns should be indicated on design and detail drawings.

Filet welds deposited on opposite sides of a common plane of contact between two parts must be interrupted at a corner common to both welds. An exception to this requirement must be made when seal-welding parts prior to hot-dipped galvanizing.

If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld should at least equal the perpendicular distance between the welds.

In material 5/8 in or less thick, the thickness of plug or slot welds should be the same as the material thickness. In material greater than 5/8 in thick, the weld thickness should be at least half the material thickness but not less than 5/8 in.

The diameter of the hole for a plug weld should be at least equal to the depth of the hole plus 5/16 in, but the diameter should not exceed 2 1/4 times the thickness of the weld. Thus, the hole diameter in $^3/_4$-in plate could be a minimum of $^3/_4 + ^5/_{16} = 1^1/_{16}$ in. The depth of metal would be at least $^5/_8$ in, because half the material thickness is only $^3/_8$ in.

Plug welds may not be spaced closer center-to-center than four times the hole diameter.

The length of the slot for a slot weld should not exceed 10 times the part thickness. The width of the slot should be at least equal to the depth of the hole plus $^5/_{16}$ in, but the width should not exceed $2^1/_4$ times the weld thickness.

Thus, the width of the slot in $^3/_4$-in plate should be a minimum of $^3/_4 + ^5/_{16} = 1^1/_{16}$ in. The weld metal depth would be at least $^5/_8$ in, because half the material thickness is only $^3/_8$ in. The slot could be up to $10 \times ^5/_8 = 6^1/_4$ in long.

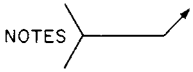
Slot welds may be spaced no closer than four times their width in a direction transverse to the slot length. In the longitudinal direction, center-to-center spacing should be at least twice the slot length.

3.2.16 Welding Symbols

Standard welding symbols should be used on drawings to designate welds and provide pertinent information concerning them. The basic parts of a weld symbol are a horizontal line and an arrow:

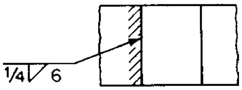
Extending from either end of the line, the arrow should point to the joint in the same manner as the electrode would be held to do the welding.

Welding symbols should clearly convey the intent of the designer. For this purpose, sections or enlarged details may have to be drawn to show the symbols, or notes may be added. Notes may be given as part of welding symbols or separately. When the notes are part of a symbol, they should be placed inside a tail at the opposite end of the line from the arrow:

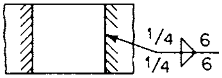


Type and length of weld are indicated above or below the line. If below the line, the symbol applies to a weld on the arrow side of the joint, the side to which the arrow points. If above the line, the symbol indicates that the other side, the side opposite the one to which the arrow points (not the far side of the assembly), is to be welded.

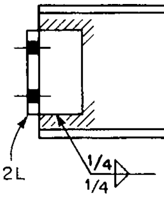
A fillet weld is represented by a right triangle extending above or below the line to indicate the side on which the weld is to be made. The vertical leg of the triangle is always on the left.



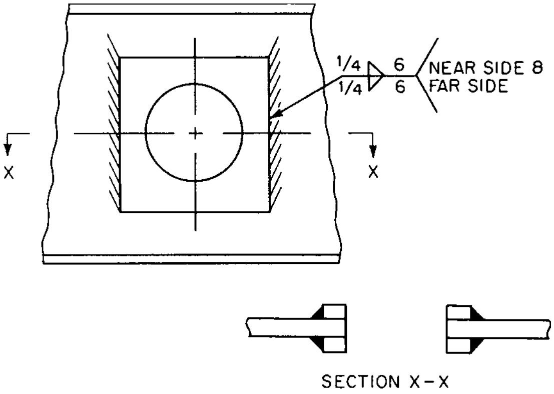
The preceding symbol indicates that a $^1/_4$-in fillet weld 6 in long is to be made on the arrow side of the assembly. The following symbol requires a $^1/_4$-in fillet weld 6 in long on both sides.



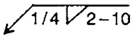
If a weld is required on the far side of an assembly, it may be assumed necessary from symmetry, shown in sections or details, or explained by a note in the tail of the welding symbol. For connection angles at the end of a beam, far-side welds generally are assumed:



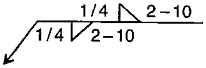
Length of weld is not shown on the symbol in this case, because the connection requires a continuous weld the full length of each angle on both sides of the angle. Care must be taken not to omit length unless a continuous full-length weld is wanted. "Continuous" should be written on the weld symbol to indicate length when such a weld is required. In general, a tail note is advisable to specify welds on the far side, even when the welds are the same size.



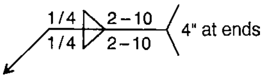
For many members, a stitch or intermittent weld is sufficient. It may be shown as



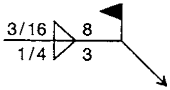
This symbol calls for ¼-in fillet welds on the arrow side. Each weld is to be 2 in long. Spacing of welds is to be 10 in center to center. If the welds are to be staggered on the arrow and other sides, they can be shown as



Usually, intermittent welds are started and finished with a weld at least twice as long as the length of the stitch welds. This information is given in a tail note:

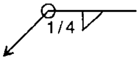


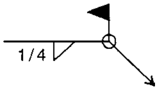
When the welding is to be done in the field rather than in the shop, a triangular flag should be placed at the intersection of arrow and line:



This is important in ensuring that the weld will be made as required. Often, a tail note is advisable for specifying field welds.

A continuous weld all around a joint is indicated by a small circle around the intersection of line and arrow:

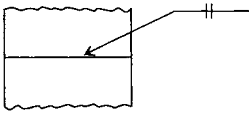




Such a symbol would be used, for example, to specify a weld joining a pipe column to a base plate. The all-around symbol, however, should not be used as a substitute for computation of actual weld length required. Note that the type of weld is indicated below the line in the all-around symbol, regardless of shape or extent of joint.

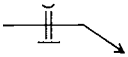
The preceding devices for providing information with fillet welds also apply to groove welds. In addition, groove-weld symbols also must designate material preparation required. This often is best shown on a cross section of the joint.

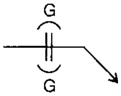
A square-groove weld (made in thin material) without root opening is indicated by



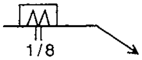
Length is not shown on the welding symbol for groove welds because these welds almost always extend the full length of the joint.

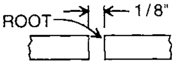
A short curved line below a square-groove symbol indicates weld contour. A short straight line in that position represents a flush weld surface. If the weld is not to be ground, however, that part of the symbol is usually omitted. When grinding is required, it must be indicated in the symbol.



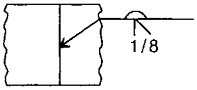


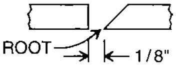
The root-opening size for a groove weld is written in within the symbol indicating the type of weld. For example, a $^1/_8$-in root opening for a square-groove weld is specified by





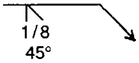
And a $^1/_8$-in root opening for a bevel weld, not to be ground, is indicated by



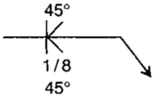


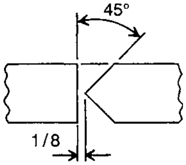
In this and other types of unsymmetrical welds, the arrow not only designates the arrow side of the joint but also points to the side to be shaped for the groove weld. When the arrow has this significance, the intention often is emphasized by an extra break in the arrow.

The angle at which the material is to be beveled should be indicated with the root opening:

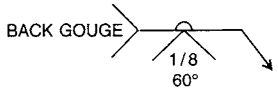


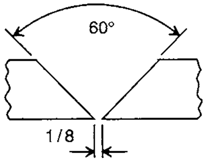
A double-bevel weld is specified by



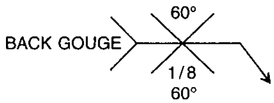


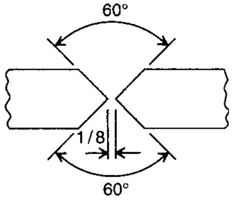
A single-V weld is represented by





A double-V weld is indicated by





Summary. In preparing a weld symbol, insert size, weld-type symbol, length of weld, and spacing, in that order from left to right. The perpendicular leg of the symbol for fillet, bevel, J, and flare-bevel welds should be on the left of the symbol. Bear in mind also that arrow-side and other-side welds are the same size, unless otherwise noted. When billing of detail material discloses the identity of the far side with the near side, the welding shown for the near side also will be duplicated on the far side. Symbols apply between abrupt changes in direction of welding unless governed by the all-around symbol or dimensioning shown.

Where groove preparation is not symmetrical and complete, additional information should be given on the symbol. Also, it may be necessary to give weld-penetration information, as in Fig. 3.18. For the weld shown, penetration from either side must be a minimum of $^3/_{16}$ in. The second side should be back-gouged before the weld there is made.

Welds also may be a combination of different groove and fillet welds. While symbols can be developed for these, designers will save time by supplying a sketch or enlarged cross section. It is important to convey the required information accurately and completely to the workers who will do the job. Actually, it is common practice for designers to indicate what is required of the weld and for fabricators and erectors to submit proposed procedures.

3.2.17 Welding Positions

The position of the electrode relative to the joint when a weld is being made affects welding economy and quality. The basic welding positions are as follows:

Flat, with the face of the weld nearly horizontal. The electrode is nearly vertical, and welding is performed from above the joint.

Horizontal, with the axis of the weld horizontal. For groove welds, the face of the weld is nearly vertical. For fillet welds, the face of the weld usually is about 45° relative to horizontal and vertical surfaces.

Vertical, with the axis of the weld nearly vertical. (Welds are made upward.)

Overhead, with the face of the weld nearly horizontal. The electrode is nearly vertical, and welding is performed from below the joint.

Where possible, welds should be made in the flat position. Weld metal can be deposited faster and more easily, and generally the best and most economical welds are obtained. In a shop, the work usually is positioned to allow flat or horizontal welding. With care in design, the expense of this positioning can be kept to a minimum. In the field, vertical and overhead welding sometimes may be necessary. The best assurance of good welds in these positions is use of proper electrodes by experienced welders.

AWS D1.1 requires that only the flat position be used for submerged-arc welding, except for certain sizes of fillet welds. Single-pass fillet welds may be made in the flat or the horizontal position in sizes up to $^5/_{16}$ in with a single electrode and up to $^1/_2$ in with multiple electrodes. Other positions are prohibited.

When groove-welded joints can be welded in the flat position, submerged-arc and gas-metal arc processes usually are more economical than the manual shielded metal arc process.

Designers and detailers should detail connections to ensure that welders have ample space for positioning and manipulating electrodes and for observing the operation with a protective hood in place. Electrodes may be up to 18 in long and $^3/_8$ in in diameter.

In addition, adequate space must be provided for deposition of the required size of the fillet weld. For example, to provide an adequate landing *c*, in, for the fillet weld of size *D*, in, in Fig. 3.19, *c* should be at least $D + {}^5/_{16}$. In building column splices, however, $c = D + {}^3/_{16}$ often is used for welding splice plates to fillers.

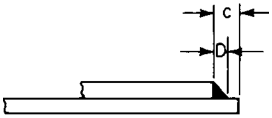


FIGURE 3.19 Minimum landing for a fillet weld.

3.2.18 Welding Procedures

Welds should be qualified and should be made only by welders, welding operators, and tackers qualified as required in AWS D1.1 for buildings. Welding should not be permitted under any of the following conditions:

When the ambient temperature is below 0°F

When surfaces are wet or exposed to rain, snow, or high wind

When welders are exposed to inclement conditions

Surfaces and edges to be welded should be free from fins, tears, cracks, and other defects. Also, surfaces at and near welds should be free from loose scale, slag, rust, grease, moisture, and other material that may prevent proper welding. AWS specifications, however, permit mill scale that withstands vigorous wire brushing, a light film of drying oil, or antispatter compound to remain. However, the specifications require all mill scale to be removed from surfaces on which flange-to-web welds of cyclically loaded girders are to be made.

Parts to be fillet-welded should be in close contact. The gap between parts should not exceed $^3/_{16}$ in. If the gap is more than $^1/_{16}$ in, the fillet weld size should be increased by the amount of separation. The separation between faying surfaces for plug and slot welds and for butt joints landing on a backing should not exceed $^1/_{16}$ in. Parts to be joined at butt joints should be carefully aligned. Where the parts are effectively restrained against bending due to eccentricity in alignment, an offset not exceeding 10% of the thickness of the thinner part joined, but in no case more than $^1/_8$ in, is permitted as a departure from theoretical alignment. When correcting misalignment in such cases, the parts should not be drawn in to a greater slope than $^1/_2$ in in 12 in.

For permissible welding positions, see Art. 3.2.17. Work should be positioned for flat welding whenever practicable.

In general, welding procedures and sequences should avoid needless distortion and should minimize shrinkage stresses. As welding progresses, welds should be deposited so as to balance the applied heat. Welding of a member should progress from points where parts are relatively fixed in position toward points where parts have greater relative freedom of movement. Where it is impossible to avoid high residual stresses in the closing welds of a rigid assembly, these welds should be made in compression elements. Joints expected to have significant shrinkage should be welded before joints expected to have lesser shrinkage, and restraint should be kept to a minimum. If severe external restraint against shrinkage is present, welding should be carried continuously to completion or to a point that will ensure freedom from cracking before the joint is allowed to cool below the minimum specified preheat and interpass temperatures.

In shop fabrication of cover-plated beams and built-up members, each component requiring splices should be spliced before it is welded to other parts of the member. Up to three subsections may be spliced to form a long girder or girder section.

With too rapid cooling, cracks may form in a weld. Possible causes are shrinkage of weld and heat-affected zone, austenite–martensite transformation, and entrapped hydrogen. Preheating the base metal can eliminate the first two causes. Preheating reduces the temperature gradient between the weld and the adjacent base metal, thus decreasing the cooling rate and resulting stresses. Also, if hydrogen is present, preheating allows more time for this gas to escape. Use of low-hydrogen electrodes, with suitable moisture control, is also advantageous in controlling hydrogen content.

High cooling rates occur at arc strikes that do not deposit weld metal. Hence, strikes outside the area of permanent welds should be avoided. Cracks or blemishes resulting from arc strikes should be ground to a smooth contour and checked for soundness.

To avoid cracks and for other reasons, AWS specifications require that under certain conditions, before a weld is made, the base metal must be preheated. Table 3.9 lists typical preheat and interpass temperatures. The table recognizes that as plate thickness, carbon content, or alloy content increases, higher preheats are necessary to lower cooling rates and to avoid microcracks or brittle heat-affected zones.

Preheating should bring the surface of the base metal to the specified preheat temperature within a distance equal to the thickness of the part being welded, but to not less than 3 in of the point of welding. This temperature should be maintained as a minimum interpass temperature while welding progresses.

Preheat and interpass temperatures should be sufficient to prevent crack formation. Temperatures above the minimums in Table 3.9 may be required for highly restrained welds.

To prevent cracking, peening sometimes is used on intermediate weld layers for control of shrinkage stresses in thick welds. Peening should be done with a round-nose tool and light blows from a power hammer after the weld has cooled to a temperature that feels warm to the hand. The root or surface layer of the weld or the base metal at the edges of the weld should not be peened. Care should be taken to prevent scaling or flaking of weld and base metal from overpeening.

TABLE 3.9 Requirements of AWS D1.1 for Minimum Preheat and Interpass Temperatures, °F, for Welds in Buildings for Some Commonly Used Structural Steels[a]

Thickness at thickest part at point of welding, in	Shielded metal-arc with other than low-hydrogen electrodes	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas-metal arc, or flux-cored arc	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas-metal arc, or flux-cored arc	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, with carbon or alloy steel wire neutral flux, gas-metal arc, or flux-cored arc with electrodes or electrode-flux combination capable of depositing weld metal with a maximum diffusible hydrogen content of 8 Ml/100 g when tested in accordance with AWS A4.3
	ASTM A36, A53 Grade B, A501, A529	ASTM A36; A53 Grade B; A242; A441; A501; A529; A572 Grades 42, 50, and 55; A588; A992	ASTM A572 Grades 60 and 65	ASTM A913[c] Grades 50, 60, and 65
To $\frac{3}{4}$	32[b]	32[b]	50	32[b]
Over $\frac{3}{4}$ to $1\frac{1}{2}$	150	50	150	32[b]
$1\frac{1}{2}$ to $2\frac{1}{2}$	225	150	225	32[b]
Over $2\frac{1}{2}$	300	225	300	32[b]

[a] In joints involving different base metals, preheat as specified for higher-strength base metal.

[b] When the base-metal temperature is below 32°F, the base metal shall be preheated to at least 70°F and the minimum interpass temperature shall be maintained during welding.

[c] The heat input limitations of AWS D1.1, paragraph 5-7, shall not apply to A913.

When required by plans and specifications, welded assemblies should be stress-relieved by heat treating, but this is rarely required in building construction. (See AWS D1.1 for temperatures and holding times required.) Finish machining should be done after stress relieving.

Tack and other temporary welds are subject to the same quality requirements as final welds. For tack welds, however, preheat is not mandatory for single-pass welds that are remelted and incorporated into continuous submerged-arc welds. Also, defects such as undercut, unfilled craters, and porosity need not be removed before final submerged-arc welding. Welds not incorporated into final welds should be removed after they have served their purpose, and the surface should be made flush with the original surface.

Before a weld is made over previously deposited weld metal, all slag should be removed, and the weld and adjacent material should be brushed clean.

Groove welds should be terminated at the ends of a joint in a manner that will ensure sound welds. Where possible, this should be done with the aid of weld tabs or runoff plates. AWS D1.1 does not require removal of weld tabs for statically loaded structures but does require it for dynamically loaded structures. The 2005 AISC Seismic Provisions also require their removal in zones of high seismicity. The ends of the welds then should be made smooth and flush with the edges of the abutting parts.

After welds have been completed, slag should be removed from them. The metal should not be painted until all welded joints have been completed, inspected, and accepted. Before paint is applied, spatter, rust, loose scale, oil, and dirt should be removed.

AWS D1.1 presents details of acceptable techniques for welding in buildings. These techniques include handling of electrodes and fluxes and maximum welding currents.

3.2.19 Weld Quality

A basic requirement of all welds is thorough fusion of weld and base metal and of successive layers of weld metal. In addition, welds should not be handicapped by craters, undercutting, overlap, porosity, or cracks. (AWS D1.1 gives acceptable tolerances for these defects.) If craters, excessive concavity, or undersized welds occur in the effective length of a weld, they should be cleaned and filled to the full cross section of the weld. Generally, all undercutting (removal of base metal at the toe of a weld) should be repaired by depositing weld metal to restore the original surface. Overlap (a rolling over of the weld surface with lack of fusion at an edge), which may cause stress concentrations, and excessive convexity, should be reduced by grinding away excess material (see Figs. 3.20

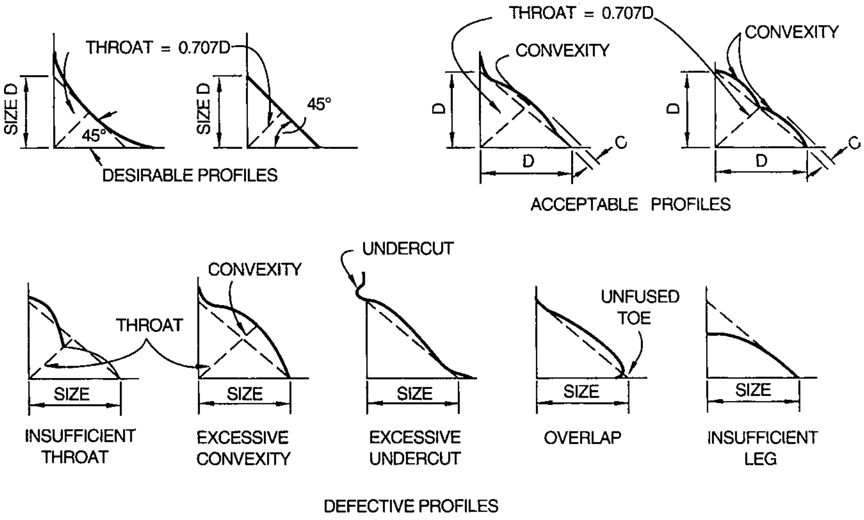


FIGURE 3.20 Profiles of fillet welds.

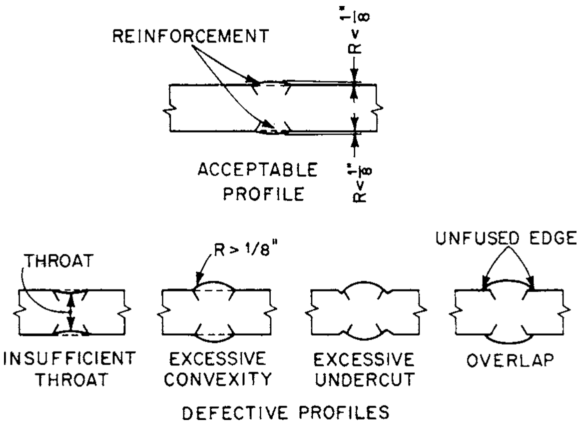


FIGURE 3.21 Profiles of groove welds.

and 3.21). If excessive porosity, excessive slag inclusions, or incomplete fusion occur, the defective portions should be removed and rewelded. If cracks are present, their extent should be determined by acid etching, magnetic-particle inspection, or other equally positive means. Not only the cracks but also sound metal 2 in beyond their ends should be removed and replaced with the weld metal. Use of a small electrode for this purpose reduces the chances of further defects due to shrinkage. An electrode not more than $^5/_{32}$ in in diameter is desirable for depositing weld metal to compensate for size deficiencies.

AWS D1.1 limits convexity (C dimension in Fig. 3.20) to the values in Table 3.10.

Weld-quality requirements should depend on the job the welds are to do. Excessive requirements are uneconomical. Size, length, and penetration are always important for a stress-carrying weld and should completely meet design requirements. Undercutting, on the other hand, should not be permitted in main connections, such as those in trusses and bracing, but small amounts may be permitted in less important connections, such as those in platform framing for an industrial building. Type of electrode, similarly, is important for stress-carrying welds but not so critical for many miscellaneous welds. Again, poor appearance of a weld is objectionable if it indicates a bad weld or if the weld will be exposed where esthetics is a design consideration, but for many types of structures, such as factories, warehouses, and incinerators, the appearance of a good weld is not critical. A sound weld is important, but a weld entirely free of porosity or small slag inclusions should be required only when the type of loading actually requires this perfection.

Welds may be inspected by one or more methods: visual inspection; nondestructive tests, such as ultrasonic, x-ray, dye penetration, and magnetic particles; and cutting of samples from finished welds. Designers should specify which welds are to be examined, extent of the examination, and methods to be used.

TABLE 3.10 AWS D1.1 Limits on Convexity of Fillet Welds

Measured leg size or width of surface bead, in	Maximum convexity, in
$^5/_{16}$ or less	$^1/_{16}$
Over $^5/_{16}$ but less than 1	$^1/_8$
1 or more	$^3/_{16}$

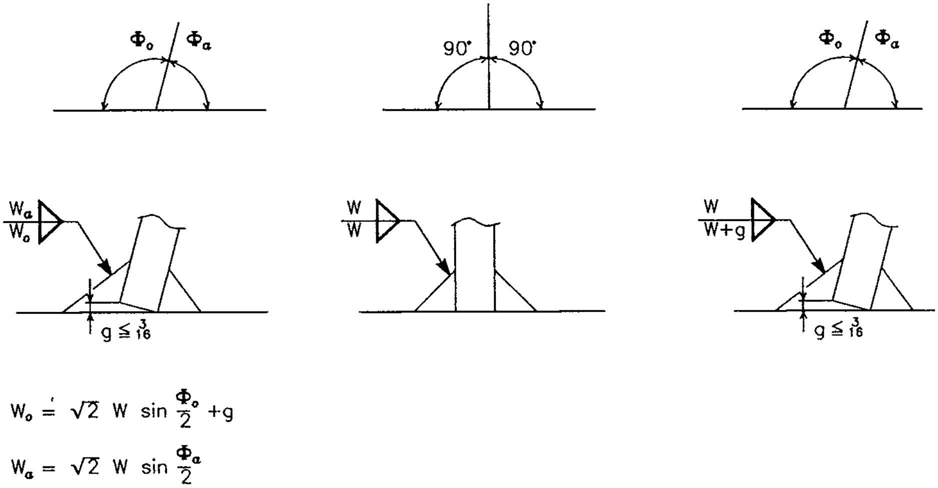
3.2.20 Strength of Skewed Fillet Welds

It is often beneficial to utilize skewed single-plate or end-plate shear connections to connect members that run nonorthogonal to their supports. In such cases, the welds attaching the connection material to the support must be designed to accommodate this skew. There are two ways to do this. AWS D1.1 provides a method to calculate the effective throat for skewed tee joints with varying dihedral angles, which is based on providing equal strength in the obtuse and acute welds. This is shown in Fig. 3.22*a*. The AISC method is simpler, and simply increases the weld size on the obtuse side by the amount of the gap, as shown in Fig. 3.22*c*.

Both methods can be shown to provide a strength equal to or greater than the required orthogonal weld size of W. The main difference with regard to strength is that the AWS method maintains equal strength in both fillets, whereas the AISC method increases the strength on the acute side by maintaining a constant fillet size, $W_a = W$, while the increased size on the obtuse side, $W_o = W + g$, actually loses strength because of the gap g. Nevertheless, it can be shown that the sum of the strengths of these two fillet welds, $W_a = W$ and $W_o = W + g$, is always greater than that of the $2W$ of the required orthogonal fillets. The gap g is limited to a maximum value of $^3/_{16}$ in for both methods.

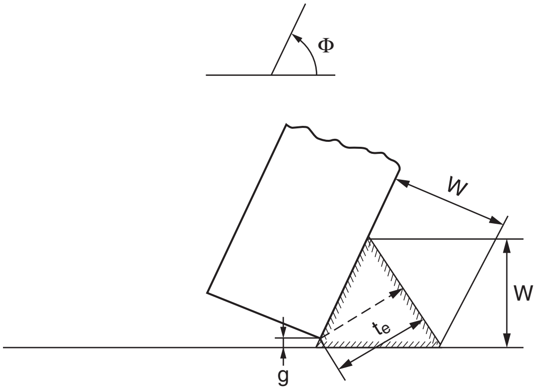
The effects of the skew on the effective throat of a fillet weld can be very significant, as shown in Fig. 3.23. Figure 3.23 also shows how fillet legs W_o and W_a are measured in the skewed configuration. On the acute side of the connection the effective throat for a given fillet weld size increases gradually as the connection intersection angle, ϕ, changes from 90° to 60°. From 60° to 30°, the weld changes from a fillet weld to a partial-penetration groove weld and the effective throat t_e decreases due to the allowance z for the unwelded portion at the root. See Fig. 3.24. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less $^1/_8$ in for 60° to 45° and less $^1/_4$ in for 45° to 30°. Joints less than 30° are not prequalified and generally should not be used.

Note in Fig. 3.23 how the skewed fillet welds are to be measured (dimension W) in accordance with AWS and AISC. The contact leg length is **not** the weld size.



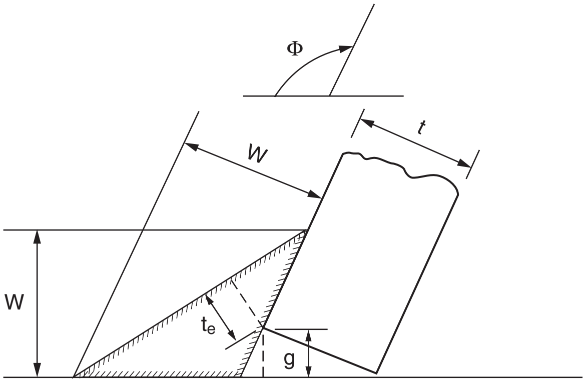
(a) (b) (c)

FIGURE 3.22 Skewed fillet weld sizes required to match strength of required orthogonal fillets. (*a*) AWS method. (*b*) Required orthogonal weld. (*c*) AISC method. (*Source*: *A. R. Tamboli,* Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)



Skewed fillet welds

(a)



Skewed fillet welds

(b)

FIGURE 3.23 Geometry of skewed fillet welds. (*a*) Acute side, 60° ≤ $\Phi \leq 90°$: AWS, $W = t_e\,[2\sin(\Phi/2)] + g$; AISC, $W = W + g$. (*b*) Obtuse side, $90° \leq \Phi \leq 135°$: AWS, $W = t_e\,[2\sin(\Phi/2)] + g$; AISC, $W = W$.

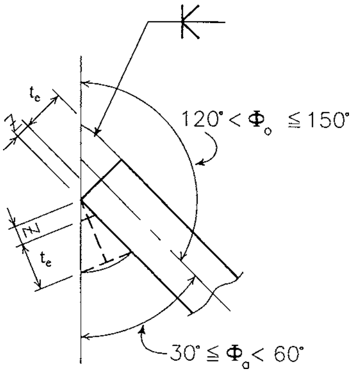


FIGURE 3.24 Acute angles less than 60° and obtuse angles greater than 120°. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

3.2.21 Obliquely Loaded Concentric Fillet Weld Groups

The strength of a fillet weld depends on the direction of loading. Welds that are loaded in their longitudinal direction have a nominal strength of $0.6F_{\mathrm{EXX}}$ times the effective weld area, while welds loaded transverse to their longitudinal axis have a strength 1.5 times greater. The nominal strength per unit area of welds loaded between these extremes can be found as

$$F_w = 0.6F_{\mathrm{EXX}}(1.0 + 0.50\sin^{1.5}\theta) \tag{3.13}$$

where F_{EXX} is the electrode classification number, ksi.

Equation (3.13) is easily applied to a single line weld, or a group of parallel line welds, but when applied to weld groups containing welds loaded at differing angles, such as depicted in Fig. 3.25, its application becomes much more complex. In such cases, deformation compatibility must also be satisfied. Since transversely loaded welds are considerably less ductile than longitudinally loaded welds, the transversely loaded welds will fracture before the longitudinally loaded welds reach their full capacity. This can be seen by examining the load–deformation plots in Fig. 3.26, where θ = 0 indicates longitudinal loading and θ = 90° indicates transverse loading. A weld loaded transverse to its longitudinal direction will fracture at a deformation equal to approximately 0.56 times the weld size. At this same deformation, the longitudinally loaded weld has reached only about 83% of its maximum strength.

To account for this effect, the components of the nominal strength of the weld, R_{nx} and R_{ny}, are calculated in the AISC Specification as

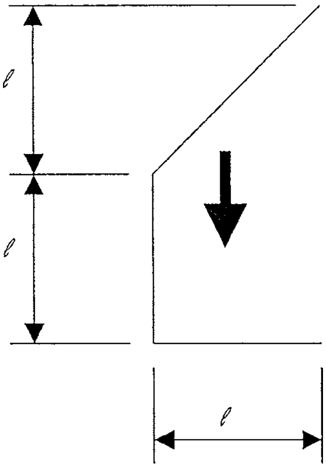


FIGURE 3.25 Obliquely loaded weld group.

$$R_{nx} = \sum F_{wix} A_{wi} \tag{3.14}$$

$$R_{ny} = \sum F_{wiy} A_{wi} \tag{3.15}$$

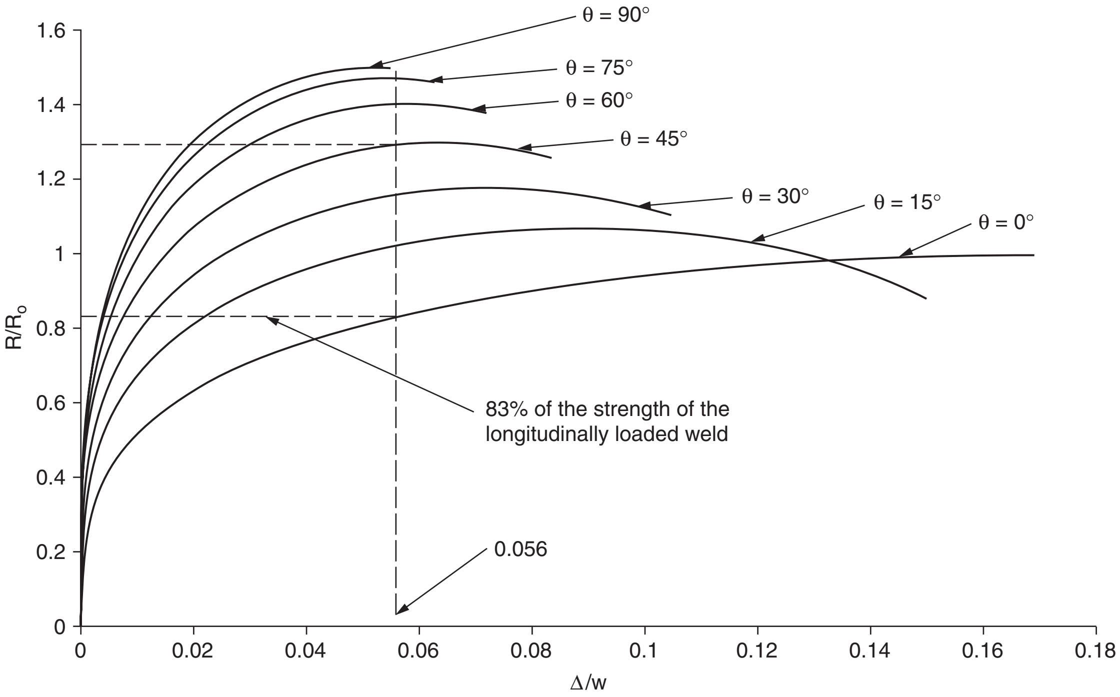


FIGURE 3.26 Load–deformation plots for graphical solution of strength of an obliquely loaded fillet weld group. R/R_o is ratio of nominal strength ratio at any Δ/w. Δ/w is ratio of deformation to weld size. θ = 0 is longitudinal weld. θ = 90 is transverse weld.

where A_{wi} = effective area of weld throat of any ith weld element, in^2

$F_{wi} = 0.6F_{\text{EXX}}\,(1 + 0.50 \sin^{1.5} \theta)f(p)$ (3.16)

$f(p) = [p(1.9 - 09p)]^{0.3}$ (3.17)

F_{wi} = nominal stress in any ith weld element, ksi

F_{wix} = x component of stress F_{wi}

F_{wiy} = y component of stress F_{wi}

$p = \Delta_i/\Delta_m$, ratio of element i deformation to its deformation at maximum stress

$\Delta_m = 0.209(\theta + 2)^{-0.32}w$, deformation of weld element at maximum stress, in (mm)

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation r_i, in

$$= \frac{r_i \Delta_u}{r_{\text{crit}}}$$

$\Delta_u = 1.087(\theta + 6)^{-0.65}w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element farthest from instantaneous center of rotation, in

w = leg size of the fillet weld, in

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u/r_i ratio, in

The calculations can be made graphically using the load–deformation curves in Fig. 3.26. For example, to find the strength of the concentrically loaded weld group shown in Fig. 3.25, first the least ductile weld (greatest θ) is determined. In this case it is the transversely loaded weld. By drawing a vertical line from the point of fracture, the strength increase or decrease for the remaining elements can be determined. In this case the strength of the weld group is found to be

$$\phi R_w = (D)(1.392)[1.5(1) + 1.29(1.41) + 0.83(1)] = 578D$$

where D is the weld size in $^1/_{16}$ths and 1.392 is the design strength of a weld with E70 electrodes, kip/in per $^1/_{16}$th. The latter is determined from

$$\phi R_n = \phi F_w A_w = 0.75 \times 0.60 \times 70 \times (^1/_{16}) \times 0.707 = 1.392 \text{ kips/in per } ^1/_{16}\text{th}$$

For fillet weld groups loaded concentrically, consisting of welds in the transverse and longitudinal directions, then the combined nominal strength of the weld group is permitted to be taken as the greater of the following:

$$R_n = R_{\text{wl}} + R_{\text{wt}} \tag{3.18}$$

$$R_n = 0.85R_{\text{wl}} + 1.5R_{\text{wt}} \tag{3.19}$$

where R_{wl} = total nominal strength of longitudinally loaded fillet welds

R_{wt} = total nominal strength of transversely loaded fillet welds

R_{wl} and R_{wt} are determined from basic provision for nominal strength,

$$R_n = F_w A_w \tag{3.20}$$

where $F_w = 0.60\ F_{\text{EXX}}$ is the nominal strength of the weld metal, ksi, and A_w, in^2, is the effective area of the weld.

3.3 GENERAL CONNECTION DESIGN PROCEDURE

Determine the external (applied) factored loads, also called required strengths, and their lines of action. Make a preliminary layout, preferably to scale. The connection should be as compact as possible to conserve material and to minimize interference with utilities, equipment, and access. Decide on where bolts and welds will be used and select bolt type and size. Decide on a load path through the connection. For a statically determinate connection, there is only one, but for indeterminate connections there are many possibilities. Use judgment, experience, and published information to arrive at the best load path. Now provide sufficient strength, stiffness, and ductility, using the limit states identified for each part of the load path, to give the connection sufficient design strength, that is, to make the connection adequate to carry the given loads. Complete the preliminary layout, check specification-required spacings, and finally check to ensure that the connection can be fabricated and erected. The examples of this chapter will demonstrate this procedure.

3.3.1 Economic Considerations

For any given connection situation, it is usually possible to arrive at more than one satisfactory solution. Where there is a possibility of using bolts or welds, let the economics of fabrication and erection play a role in the choice. Fabricators and erectors in different parts of the country have their preferred ways of working, and as long as the principles of connection design are followed to achieve a safe connection, local preferences should be accepted. Some additional considerations which will result in more economical connections (Thornton, 1995b) are as follows.

1. For shear connections, design for the specified factored loads and allow the use of single-plate and single-angle shear connections. Do not specify full-depth connections or rely on the AISC uniform load tables.

2. For moment connections, design for the specified factored moments and shears. Also, provide a "breakdown" of the total moment, that is, give the gravity moment and lateral moment due to wind or seismic loads separately. This is needed to do a proper check for column web doubler plates. If stiffeners are required, allow the use of fillet welds in place of complete joint-penetration welds. To avoid the use of stiffeners, consider redesigning with a heavier column to eliminate them.

3. For bracing connections, in addition to providing the brace force, also provide the beam shear and axial transfer force. As discussed in Art. 3.1.6, the transfer force is the axial force that must be transferred to the opposite side of the column. The transfer force is not necessarily the beam axial force that is obtained from a computer analysis of the structure. A misunderstanding of transfer forces can lead to both uneconomic and unsafe connections.

3.3.2 Types of Connections

There are three basic forces to which connections are subjected: axial force, shear force, and moment. Many connections are subject to two or more of these simultaneously. Connections are usually classified according to the major load type to be carried, such as shear connections, which carry primarily shear, moment connections, which carry primarily moment, and axial force connections, such as splices, bracing and truss connections, hangers, etc., which carry primarily axial force.

3.3.3 Strength Limit States

Many of the limit states that govern main-member design also must be considered in the design of connection elements.

Tension. Either tension yielding or fracture can govern the strength of a connecting element subjected to tension. The design strength for yielding in the gross section is

$$\phi R_n = \phi F_y A_g \tag{3.21}$$

and the design strength for fracture in the net section is

$$\phi R_n = \phi F_u A_n \tag{3.22}$$

where $\phi = 0.90$ for yielding or 0.75 for fracture
F_y = specified minimum yield stress of connecting element, ksi
F_u = specified minimum tensile strength of connecting element, ksi
A_g = gross area of the connecting element, in^2
A_n = net area of the connecting element, in^2

In some cases the entire gross or net areas of a connecting element cannot be considered effective. This is the case for a brace attaching to a large gusset, where the effective gross area is based on the Whitmore section. Also, for connecting elements, such as angles, where only one leg of the angle is connected, a shear lag factor must be included into the calculation of an effective net area.

Shear. Either shear yielding or fracture can govern the strength of a connecting element subjected to shear. The design strength for shear yielding in the gross section is

$$\phi R_n = \phi 0.6 F_y A_g \tag{3.23}$$

and the design strength for fracture in the net section is

$$\phi R_n = \phi 0.6 F_u A_n \tag{3.24}$$

where $\phi = 0.90$ for yielding or 0.75 for fracture and other terms are as given above. Due to the resistance provided by the flange, net shear fracture will govern the capacity of flanged members only when both flanges are coped.

Bending. Either tension yielding or fracture in the tension zone can govern the strength of a connecting element subjected to bending (flexure). The design strength for yielding (plastic moment) in the gross section is

$$\phi R_n = \phi F_y Z_g \tag{3.25}$$

and the design strength for fracture in the net section can be taken as

$$\phi R_n = \phi F_u Z_n \tag{3.26}$$

where $\phi = 0.90$ for yielding or 0.75 for fracture
Z_g = gross plastic section modulus of the connecting element
Z_n = net plastic section modulus of the connecting element

For a plate with equal edge distance top and bottom and constant bolt spacing, the net plastic section modulus can be calculated as

$$Z_n = Z_g\left(1 - \frac{d_h}{b}\right) \tag{3.27}$$

where d_h = hole diameter
b = bolt spacing

This is an exact result for connections with an even number of rows and a slightly conservative estimate for those with an odd number of rows.

Though contrary to historic procedures, recent tests indicate that it is acceptable to use the net plastic section modulus instead of the more traditional elastic section modulus. This more accurately represents the ultimate capacity of the element in bending.

Compression. The capacity of an element in bending can also be governed by buckling of the element in the compression zone. See Art. 3.3.5.

3.3.4 Localized Limit States

Connections are often subjected to, or subject main members to, localized stresses that are usually not considered in typical main-member design.

Bearing at Bolt Holes. As loads are transferred from one element to another through the bolts, large localized compression stresses can occur where the shank of the bolt bears on the connected material. The design strength at these locations, when deformation at the bolt hole at service load is a design consideration, is

$$\phi R_n = \phi 2.4 d_b t F_u \tag{3.28a}$$

where $\phi = 0.75$
d_b = bolt diameter
t = thickness of material

Alternatively, if deformation at the bolt hole under service loads is not a design consideration, the bearing strength can be determined as

$$\phi R_n = \phi 3.0 d_b t F_u \tag{3.29a}$$

In some cases, such as a single-plate shear connection, deformation at the bolt holes is desired for ductility and to relieve eccentricities.

Bolt Tear-out. An additional bearing check introduced in recent specifications actually represents a shear fracture failure mode in which the bolt tears out through the material. Although this tear-out requirement is included as a bearing check, it is not a bearing limit state but rather a shear fracture limit state. It modifies Eqs. (3.28a) and (3.29a) to the following:

$$\phi R_n = \phi 1.2 L_c t F_u \le \phi 2.4 d_b t F_u \tag{3.28b}$$

$$\phi R_n = \phi 1.5 L_c t F_u \le \phi 3.0 d_b t F_u \tag{3.29b}$$

This requirement complicates the checking of bolt shear, bearing, and bolt tear-out, by tying one to the other. At each bolt there are five possible limit states: bolt shear, bearing on the main material at the bolt, bearing on the connection material at the bolt, bolt tear-out through the main material, and bolt tear-out through the connection material. For each bolt in the bolt group, the minimum of these five limit states must be determined in order to calculate the available strength of the connection.

As an example, consider a tension strut connected to a support by a plate as shown in Fig. 3.27. The W8 × 15 tension strut is A570 Grade 50 steel with $F_u = 65$ ksi and the $^3/_8$-in-thick plate is A36 steel with $F_u = 58$ ksi. Calculations are as follows.

Design shear strength of $^7/_8$-in-diameter A490X bolt:

$$\phi R_n = \phi F_n A_b = 0.75 \times 75 \times 0.601 = 33.8 \text{ kips/bolt}$$

Design bearing strength of the strut web [Eq. (3.28a)]:

$$\phi R_n = 0.75(2.4)(0.875)(0.245)(65) = 25.1 \text{ kips/bolt}$$

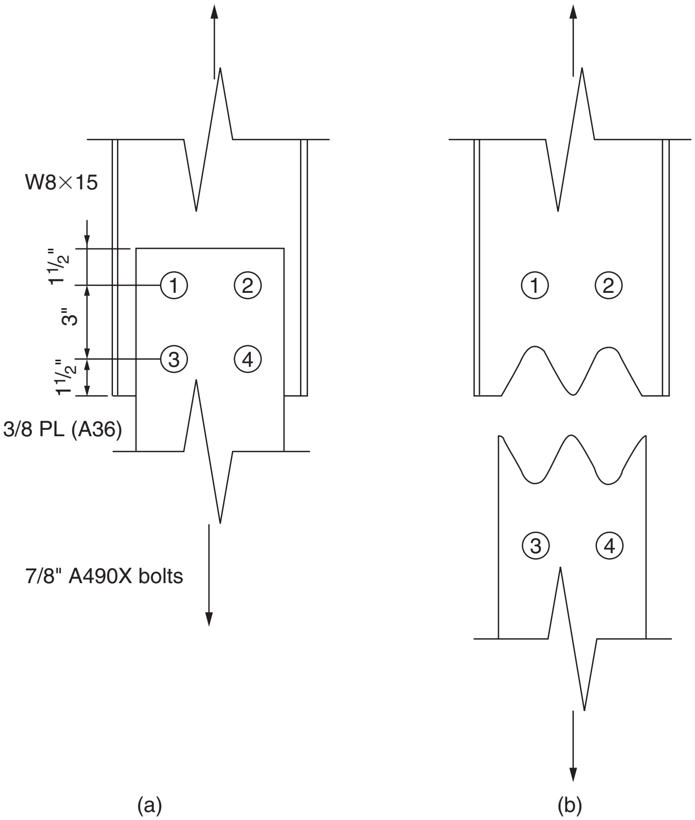


FIGURE 3.27 Illustration of bolt bearing/tear-out limit state. (*a*) As connected. (*b*) Showing tear-out.

Design bearing strength of the plate [Eq. (3.28a)]:

$$\phi R_n = 0.75(2.4)(0.875)(0.375)(58) = 34.3 \text{ kips/bolt}$$

Design strength for bolt tearing out through the edge of the strut web:

$$\phi R_n = \phi 1.2 L_c t F_b = (0.75)(1.2)(1.5 - {}^{15}\!/_{32})(0.245)(65) = 14.7 \text{ kips/bolt}$$

Design strength for bolt tearing out between bolt holes in the strut web:

$$(0.75)(1.2)(3 - {}^{15}\!/_{16})(0.245)(65) = 29.5 \text{ kips/bolt}$$

Design strength for bolt tearing out through the edge of the plate:

$$(0.75)(1.2)(1.5 - {}^{15}\!/_{32})(0.375)(58) = 20.2 \text{ kips/bolt}$$

Design strength for bolt tearing out between bolt holes in the plate:

$$(0.75)(1.2)(3 - {}^{15}\!/_{16})(0.375)(58) = 40.3 \text{ kips/bolt}$$

The design strength for bolts 1 and 2 is 20.2 kips, based on bolt tear-out through the edge of the plate, and the design strength for bolts 3 and 4 is 14.7 kips, based on bolt tear-out through the edge of the strut web. Therefore, the design strength of the connection is (2)14.7 + (2)20.2 = 69.8 kips.

It is possible to avoid the complications presented above involving bolt tear-out by increasing the edge distances and spacings such that bearing controls. To do this requires that

$$1.2F_uL_ct \geq 2.4F_ud_bt \tag{3.30}$$

Thus,

$$L_c \geq 2d_b \tag{3.31}$$

This equation was used to develop Table 3.11, which shows minimum edge distances to center of hole and minimum spacings center to center of holes to preclude tear-out. Note that the center edge distance L_e for all bolt diameters and the spacing s required for 1-in- and $1^1/_8$-in-diameter bolts are larger than the customary values. The examples in this chapter will generally use customary values and illustrate the interrelation of bolt shear, bolt bearing, and bolt tear-out.

Block Shear. The limit state of block shear is a fracture limit state in which a block of steel is torn from a member along the perimeter of the bolt holes or welds in a connection. Block shear can be distinguished from other fracture limit states by the fact that ultimate strength mobilizes tension on one plane and shear on a perpendicular plane. It is assumed that yielding will occur along one plane, while fracture occurs along the other.

Tests have shown that fracture always occurs first on the tension plane, with yielding on the shear plane. Because fracture strength can sometimes be less than yield strength on the shear plane, the design strength for the limit state of block shear is given as

$$\phi R_n = \phi[U_{\mathrm{bs}}F_uA_{\mathrm{nt}} + \min(0.6F_yA_{gv}, 0.6F_uA_{nv})] \tag{3.32}$$

where $\phi = 0.75$

A_{nt} = net area subjected to tension, in^2

A_{gv} = gross area subjected to shear, in^2

A_{nv} = net area subjected to shear, in^2

U_{bs} = shear lag factor for block shear

= 1.0 where the tension stress is uniform

= 0.5 where the tension stress is nonuniform

The factor U_{bs} is intended to account for nonuniform stress distributions, which can occur at the tension plane due to relatively large eccentricities. This is of particular concern at connections with larger eccentricities and multiple vertical rows of bolts, for which U_{bs} is taken as 0.5. Although the

TABLE 3.11 Minimum Edge Distance to Center of Hole and Minimum Spacing Center to Center of Holes to Preclude Tear-out Failure for Standard Holes

Bolt diameter, d, in	Clear edge distance L_c, in	Hole diameter d_h, in	Center edge distance[a] L_e, in	Center-to-center spacing s, in
$^3/_4$	$1^1/_2$	$^{13}/_{16}$	2	3[b]
$^7/_8$	$1^3/_4$	$^{15}/_{16}$	$2^1/_4$	3[b]
1	2	$1^1/_{16}$	$2^5/_8$	$3^1/_2$[c]
$1^1/_8$	$2^1/_4$	$1^3/_{16}$	$2^7/_8$	$3^1/_2$[c]

[a] Rounded up to nearest $^1/_8$ in.
[b] Rounded up to customary pitch of 3 in.
[c] Rounded up to nearest $^1/_2$ in.

AISC Specification does not distinguish between large and small eccentricities, it is the authors' opinion based on test results (Yura et al., 1982) that U_{bs} can be taken as 1.0 when the ratio of the eccentricity on the bolt or weld group to the length of the connection, e/L, is less than or equal to $^1/_3$.

When forces act obliquely to cause block shear, the resistance can be calculated using an elliptical interaction equation, analogous to the von Mises yield criterion:

$$\left(\frac{V}{\phi R_{\text{bsv}}}\right)^2 + \left(\frac{H}{\phi R_{\text{bsh}}}\right)^2 \leq 1 \tag{3.33}$$

where V = vertical component of tensile force, kips
H = horizontal component of tensile force, kips
R_{bsv} = nominal resistance to block shear in the vertical direction, kips
R_{bsh} = nominal resistance to block shear in the horizontal direction, kips

Block shear should also be checked at welded connections using $\phi = 0.75$ for both the fracture and yielding planes.

Local Web Yielding. Often a force is transmitted from one member to another in such a way that a large localized stress occurs in the web. This can occur in bearing connections, such as seats, in moment connections, and also at the interface between connectors and members, such as the beam-to-gusset interface in bracing connections. The effective length of web is determined assuming a $2^1/_2$:1 stress gradient through the flange of the supporting member. Both tension and compression forces can cause local web yielding. The design strength for the limit state of web local yielding is calculated as follows:

When the concentrated force is applied at a distance greater than the depth of the member,

$$\phi R_n = \phi(5k + N)F_{\text{yw}}t_w \tag{3.34}$$

When the concentrated force is applied at a distance less than or equal to the depth of the member,

$$\phi R_n = \phi(2.5k + N)F_{\text{yw}}t_w \tag{3.35}$$

where $\phi = 1.0$
F_{yw} = specified minimum yield stress of the web, ksi
N = length of bearing (not less than k for end-beam reactions), in
k = distance from outer face of flange to web toe of fillet, in
t_w = web thickness, in

Local Web Crippling. Like local web yielding, local web crippling can occur in bearing connections, moment connections, and at connector-to-beam interfaces. However, local web crippling is primarily a problem at bearing connections or at beam-to-connector interfaces and rarely governs the capacity of moment connections. With F_y less than or equal to 50 ksi, only moment connections to W12 × 50 or W10 × 33 columns are governed by local web crippling. The design strength for the limit state of web local crippling is calculated as follows:

When the concentrated force is applied at a distance greater than or equal to half the depth of the member,

$$\phi R_n = \phi 0.80 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{\text{yw}}t_f}{t_w}} \tag{3.36}$$

When the concentrated force is applied at a distance less than half the depth of the member,

For $N/d \leq 0.2$,
$$\phi R_n = \phi 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{3.37}$$

For $N/d \leq 0.2$,
$$\phi R_n = \phi 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{3.38}$$

where $\phi = 0.75$
E = modulus of elasticity, 29,000 ksi
F_{yw} = specified minimum yield stress of the web, ksi
N = length of bearing, in
d = overall depth of member, in
t_w = web thickness, in
t_f = flange thickness, in

In the design of vertical bracing connections, it is advisable to use Eq. (3.36), due to the additional restraint provided by the connection of the beam web to the column.

Local Web Compression Buckling. Unlike local web yielding and local web crippling, the checks given by AISC for local web compression buckling apply only to moment connections where compression forces exist at the same location at opposite flanges of a column and where N/d is small (< 1). The design strength for the limit state of web local buckling is calculated as

$$\phi R_n = \frac{\phi 24 t_w^3 \sqrt{EF_{yw}}}{h} \tag{3.39}$$

where $\phi = 0.90$
h = clear distance between the flanges less the fillet

When the compressive forces are applied near the end of the member, the calculated resistance to web compression buckling should be reduced by 50%. For conditions with N/d ratios greater than 1, the web should be designed as a compression member.

Local Flange Bending. When a tension load is applied to a member through a plate welded across the flange, flange bending will occur. This can occur at the tension flange of a moment connection and also at tension hangers. The design strength for the limit state of flange local bending can be calculated as

$$\phi R_n = \phi 6.25 t_f^2 F_{yf} \tag{3.40}$$

where $\phi = 0.90$
F_{yf} = specified minimum yield stress of the flange, ksi
t_f = flange thickness

Axial Yield Line or Plate Plastification. Axial yield line or plate plastification can be thought of as an "oil can effect," where an axial load is applied normal to the weak axis of an element, such as a beam web or tube wall. Although not treated specifically in the AISC Specification, the strength of such an element can be determined by performing a yield line analysis as illustrated in Fig. 3.28. Such an analysis assumes a plastic moment collapse mechanism (along the dashed lines in Fig. 3.28)

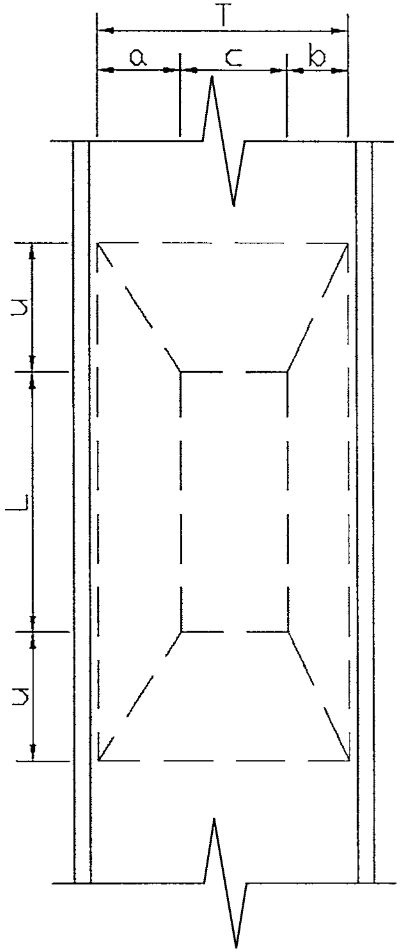


FIGURE 3.28 Yield-line analysis of web subjected to transverse axial load.

and then uses virtual work to determine the ultimate load. The value produced is an upper bound. Therefore, assuming an incorrect collapse mechanism will produce unconservative results. The literature contains numerous examples of yield line analyses for various conditions, so the correct mechanism is usually easily determined. Typical cases are discussed in the following. P_u represents the nominal strength for a load applied normal to the plane of the web and M_p is the plastic moment ($M_p = 0.25t_w^2F_y$). Multiply the nominal strength by $\phi = 0.90$ to obtain the design strength.

Web Simply Supported Subjected to an Axial Load. In general, the web should be assumed as simply supported when checking the webs of wide flange members. If the distance from the top of the connection to the end of the member is less than u, then dividing the calculated strength by 2 will yield a conservative result.

$$u = \sqrt{\frac{2Tab}{a+b}} \tag{3.41}$$

$$P_u = 2M_p\left\{\frac{[2\sqrt{(2Tab)/(a+b)} + L/2](a+b)}{ab}\right\} \tag{3.42}$$

If the connection is centered on the web, $a = b$, then the equation becomes

$$u = \sqrt{\frac{T(T-c)}{2}} \tag{3.43}$$

$$P_u = 8M_p\left[\sqrt{\frac{2T}{T-c}} + \frac{L}{2(T-c)}\right] \tag{3.44}$$

Conservatively, the c dimension can be assumed to be zero, further simplifying the equation to

$$u = \frac{T\sqrt{2}}{2} \tag{3.45}$$

$$P_u = 8M_p\left(\sqrt{2} + \frac{L}{2T}\right) \tag{3.46}$$

Web Fixed Supported Subjected to an Axial Load. The web should be assumed to be fixed supported when checking the walls of square and rectangular tube members. If the distance from the top of the connection to the end of the member is less than u, then dividing the calculated capacity by 2 will yield a conservative result.

$$u = \sqrt{\frac{Tab}{a+b}} \tag{3.47}$$

$$P_u = 2M_p\left[\frac{a+b}{ab}\left(4\sqrt{\frac{Tab}{a+b}}+L\right)\right] \tag{3.48}$$

For a hollow structural section, substitution of $B = T$, $N = L$, $a = b$, $t_1 = T - a - b$, and $0.25t_w^2F_y = M_p$ results in

$$P_u = \frac{t_w^2F_y}{[1-(t_1/B)]}\left(4\sqrt{1-\frac{t_1}{B}}+\frac{2N}{B}\right) \tag{3.49}$$

This is the equation given in the AISC "Hollow Structural Sections Connection Manual."

Axial Load Effect. When members are subjected to a concentrically applied axial load as well as the transverse load, the strength derived by the yield line equations should be reduced by a factor, Q, defined as

$$Q = 1 - 0.3\left(\frac{f}{F_y}\right) - 0.3\left(\frac{f}{F_y}\right)^2 \tag{3.50}$$

where f is the magnitude of the compressive stress.

3.3.5 Stability

In addition to satisfying the limit states of yielding and fracture, connections must also be checked for limit states of stability, the resistance of the connecting elements to buckling. Buckling can occur in plates loaded in pure compression, plates in bending, or plates subjected to both compression and bending forces.

Plates in Uniform Compression. Oftentimes plates are checked to resist buckling in a manner similar to columns. As with columns, the end restraint is important in determining the plate's resistance to buckling. This is reflected in the effective length factor K. Several conditions can exist, as illustrated in Fig. 3.29.

A gusset connected to only one supporting member and supporting a single brace is free to translate out of plane. The welded connection of the gusset to the support, and the welded or bolted connection of the brace to the gusset, are both assumed fixed, since considerable rotational stiffness is present relative to the stiffness of the plate. Therefore, $K = 1.2$ is assumed.

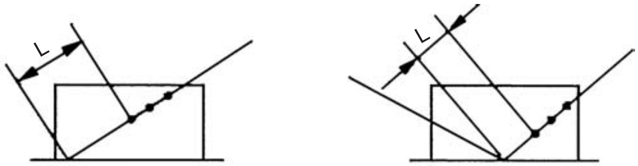
The behavior of a gusset connected to only one supporting member and supporting two or more braces depends on the loads in the braces. If all the braces act in compression simultaneously, then the gusset should be considered free to translate out of plane, and $K = 1.2$ is assumed. However, if one of the braces acts in tension, this brace will tend to resist movement out of plane, and $K = 0.65$ can be assumed. Of course, some engineering judgment must be used in determining if the tension force is sufficient to resist the out-of-plane movement. If the tension force counteracts the compression force, the most common case, the tension will be sufficient.

It is typically assumed that a gusset attached to a beam and a column, as is the usual case for vertical bracing, is retrained against both rotation and translation. For this case the recommended K value is 0.65. However, test data support the use of the theoretical K value of 0.5, so this has become the accepted approach.

At the intersection of X bracing, the end of the plate connected to the compression brace is assumed fixed. However, the connection to the tension brace is assumed pinned, because the tension brace often has little torsional stiffness. Since out-of-plane translation can occur, $K = 2.0$ is assumed. If the tension brace is deemed to have sufficient torsional stiffness, a less conservative $K = 1.2$ can be used.

Often a connection framing to another element can provide resistance to out-of-plane movement. This is the case with a flange-plate moment connection where the shear connection to the web of the member prevents the translation of flange plate. This allows a $K = 0.65$ to be assumed.

The last condition illustrated, that of a simple strut, is essentially the same condition as the single brace, and therefore utilizes $K = 1.2$.

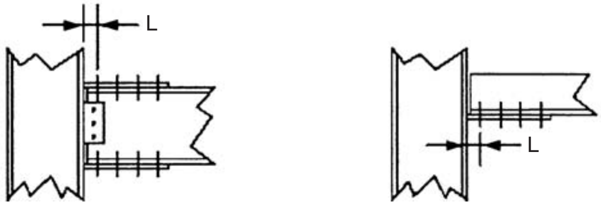


Single brace to gusset
along beam span
K = 1.2

Double brace to gusset
along beam span
K = 0.65 with one brace in tension
and one brace in compression
K = 1.2 with both braces in compression

Brace at beam-column
connection
K = 0.5

X-bracing intersection
(brace with kicker sim.)
K = 2.0



Flange plate moment
connection
K = 0.65

Strut connection
K = 1.2

FIGURE 3.29 *K* factors and buckling lengths for plates in compression.

In many cases, the full width of the plate cannot be considered effective to resist buckling. Therefore, the Whitmore section illustrated in Fig. 3.30 is taken as the effective width of the plate. The Whitmore section is assumed to extend along the length of the connection at a 30° angle to either side.

Plates in Bending. The strength of plates in bending can also be governed by buckling. Again, the boundary conditions are important in determining behavior. Typically, plates in bending are assumed to be free along one unloaded edge and simply supported along the other. The loaded edges are both usually assumed to be simply supported. This is the approach used in the AISC Manual. The critical buckling strength in bending is defined (Muir and Thornton, 2004) as

$$\phi F_{cr} = 0.9 F_y Q \tag{3.51}$$

where $Q = 1$ for $\lambda \leq 0.7$
$= 1.34 - 0.486\lambda$ for $0.7 < \lambda \leq 1.41$
$= 1.30/\lambda^2$ for $\lambda > 1.41$

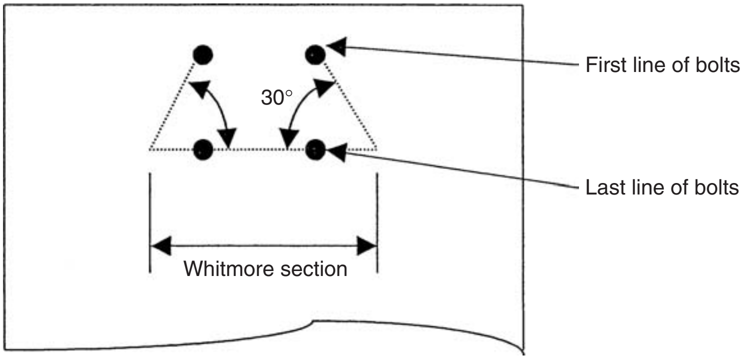


FIGURE 3.30 Whitmore section for distribution of stresses.

and

$$\lambda = \frac{d\sqrt{F_y}}{t_w\sqrt{47{,}500 + 112{,}000(d/2c)^2}} \tag{3.52}$$

where t_w is web thickness (in) and dimensions d and c (in) are defined in Fig. 3.31. This procedure can also be used to check buckling for a beam web in bending with flanges coped both top and bottom.

The above analysis is based on plate buckling coefficient k, which is equal to

$$k = 0.608\left[0.7 + 1.64\left(\frac{bm}{a}\right)^2\right] \tag{3.53}$$

where $a = c =$ length of plate parallel to the compressive force (c)
$b = d/2 =$ width of plate perpendicular to the compressive force
$m =$ number of half sine waves in buckled plate at minimum compressive stress ($m = 1$ for a plate simply supported along both loaded edges and one unloaded edge and free along the other unloaded edge)

The assumption that the supported unloaded edge is pinned is conservative. If the supported unloaded edge is assumed fixed, then

$$k = 0.83 - 0.93\nu + 1.34\left(\frac{a}{mb\pi}\right)^2 + 0.10\left(\frac{mb\pi}{a}\right)^2 \tag{3.54}$$

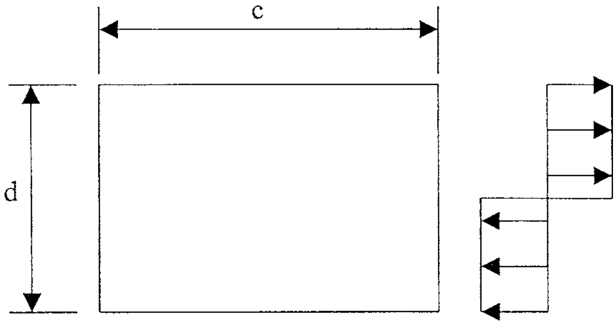


FIGURE 3.31 Bending may cause plate buckling.

Taking the derivative with respect to m and setting the result equal to zero gives

$$m = 0.609\left(\frac{a}{b}\right) \tag{3.55}$$

If m was a continuous function, Eq. (3.55) could be used to calculate k and then used to find the buckling capacity of the plate. However, since m must be an integer, Eq. (3.55) provides only an approximation. However, for any relationship of plate width to depth, the approximate value of m can be calculated by Eq. (3.55), rounded to the nearest integer, and used to determine k. Then the next highest integer can be chosen for m and k recalculated. The lower k value from these two calculations is the correct value from which to calculate the buckling capacity.

Using the more general form of the equation, λ can be calculated for the condition of fixed supported unloaded edges as

$$\lambda = \frac{1}{167}\sqrt{\frac{F_y}{k}}\,\frac{d}{2t_w} \tag{3.56}$$

This is a less conservative alternative to Eq. (3.52).

3.4 SHEAR AND AXIAL BEAM END CONNECTIONS

3.4.1 Example of Shear End-Plate Connection

An end-plate connection is to be used with a W16 × 31 A992 beam with the top flange coped as shown in Fig. 3.32. Check to see if the design strength is adequate for a factored shear force of 80 kips.

Bolt design strength for each pair of $^7/_8$-in-diameter bolts:

Bolt shear: $\phi R_n = \phi F_n A_b = 0.75 \times 48 \times 0.601 \times 2 = 21.6 \times 2 = 43.3$ kips
Bearing on W30 × 99: $\phi R_n = \phi 2.4 d_b t F_u = 0.75 \times 2.4 \times 0.875 \times 0.52 \times 65 \times 2 = 106$ kips
Bearing on end plate: $\phi R_n = 0.75 \times 2.4 \times 0.875 \times 0.375 \times 58 \times 2 = 68.5$ kips

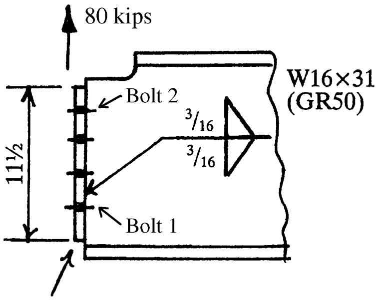


Plate: $^3/_8$×8×11½ in (A36)

Bolts: $^7/_8$ A325N in horizontal short slots at 5½ -in gage

Note: Cope in beam is 1$^3/_4$ in deep × 5$^3/_4$ in long

FIGURE 3.32 Shear end-plate connection. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

Tear-out at extreme bolt no. 1:

- At end plate $\phi R_n = \phi 1.2 L_c t F_u$ $\phi R_b = 0.75 \times 1.2 \times 2.06 \times 0.375 \times 58 \times 2 = 80.7$ kips

Tear-out at extreme bolt no. 2:

- At W30 × 99, $\phi R_n = 0.75 \times 1.2 \times 2.06 \times 0.52 \times 65 \times 2 = 125$ kips
- At end plate, $\phi R_n = 0.75 \times 1.2 \times 0.781 \times 0.375 \times 58 \times 2 = 30.6$ kips

Tear-out at intermediate bolts:

- At W30 × 99, $\phi R_n = 0.75 \times 1.2 \times 2.06 \times 0.52 \times 65 \times 2 = 125$ kips
- At end plate, $\phi R_n = 0.75 \times 1.2 \times 2.06 \times 0.375 \times 58 \times 2 = 80.7$ kips

Design strength with four rows of bolts ($n = 4$):

$$\begin{aligned}\phi R_n &= \phi R_n \text{ ext bolt 1 min} + \phi R_n \text{ ext bolt 2 min} + \phi R_n \text{ int bolt min} \times (n - 2)\\ &= 43.3 + 30.6 + 43.3 \times (4 - 2) = 161 \text{ kips} \geq 80 \text{ kips} \qquad \text{OK}\end{aligned}$$

Design strength of welds (assume $F_{\text{EXX}} = 70$ ksi):

$$\phi R_n = \phi F_w A_w = 0.75 \times 0.60 \times 70 \times {}^1\!/_{16} \times 0.707 = 1.392 \text{ kips/in} \qquad \text{for } {}^1\!/_{16}\text{-in fillet}$$

$$\phi R_n = 1.392 \times 3 \times 11.125 \times 2 = 92.916 \text{ kips} \qquad \text{for the two } {}^3\!/_{16}\text{-in fillets}$$

Since $t_w = 0.275$ in is less than the AISC base metal minimum of 0.286 in,

$$\phi R_n = 92.916 \times \frac{0.275}{0.286} = 89.4 \text{ kips} \geq P = 80 \text{ kips} \qquad \text{OK}$$

The minimum thickness noted assures that the base metal has sufficient strength to transmit the load from the weld, that is, the design shear rupture strength of the base metal matches the design shear rupture strength of the welds. It is calculated from

$$t_{\text{min}} = \frac{3.09D}{F_u} = 2 \times 3.09 \times \frac{3}{65} = 0.286 \text{ in}$$

where D is the weld size in ${}^1\!/_{16}$ths and the multiple of 2 accounts for the pair of welds.

Design strength for shear in end plate and beam web:

Gross shear on end plate:

$$\phi R_n = \phi \times 0.60 A_g F_y = 0.90 \times 0.60 \times 11.5 \times 0.75 \times 36 = 168 \text{ kips} \geq 80 \text{ kips} \qquad \text{OK}$$

Net shear on end plate:

$$\begin{aligned}\phi R_n &= \phi \times 0.60 A_{nv} F_u = 0.75 \times 0.60 \times [11.5 - (4 \times 1)] \times 0.375 \times 2 \times 58\\ &= 147 \text{ kips} \geq 80 \text{ kips} \qquad \text{OK}\end{aligned}$$

Gross shear on beam web:

$$\phi R_n = 0.90 \times 0.60 \times 15.9 \times 0.275 \times 50 = 118 \text{ kips} \geq 80 \text{ kips} \qquad \text{OK}$$

Shear on remaining W16 × 31 web:

$$\phi R_n = 0.90 \times 0.60 \times 14.1 \times 0.275 \times 50 = 105 \text{ kips} \geq 80 \text{ kips} \qquad \text{OK}$$

Design strength for bending in net section of single cope:

Follow procedure in AISC Manual

Area of remaining tee = 6.2 in^2

Center of gravity of remaining tee = 4.52 in above bottom of beam

Moment of inertia of remaining tee = 133 in^4

Section modulus at top of tee = 13.8 in^3

Section modulus at bottom of tee = 29.4 in^3

$M_u = R_u e = 80 \times (5.75 + 0.375) = 490$ in·kips

$\phi M_n = \phi F_u S_{\text{net}} = 0.75 \times 65 \times 13.8 = 673$ in·kips ≥ 490 in·kips OK

Design strength for web buckling in net section of single cope:

Follow procedure in AISC Manual

c = cope length = 5.75 in; d = beam depth = 15.9 in; h_o = net depth = 14.1 in

$c/d = 0.362 \leq 1$; therefore, adjustment factor is $f = 2(c/d) = 0.723$

$c/h_o = 0.408 \leq 1$; therefore, plate buckling coefficient is $k = 2.2(h_o/c)^{1.65} = 9.66$

$$\phi F_{\text{cr}} = 23{,}590\left(\frac{t_w}{h_o}\right)^2 fk$$

$$\phi F_{\text{cr}} = 23{,}590\left(\frac{0.275}{14.1}\right)^2 \times 0.723 \times 9.66 = 64.8 \text{ ksi} < 0.9F_y = 45 \text{ ksi}$$

$\phi M_n = \phi F_{\text{cr}} S_{\text{net}} = 45 \times 13.8 = 621$ in·kips ≥ 490 in·kips OK

3.4.2 Example of Double-Angle Connections under Shear and Axial Load

A double-angle connection is to be used for a simply supported W18 × 50 A992 beam with the top flange coped as shown in Fig. 3.33. Check to see if the design strength is adequate for factored forces of 33 kips shear and 39 kips axial.

Design strength under shear load of 33 kips with ¾-in-diameter bolts:

Bolt shear: $\phi R_n = \phi F_n A_b = 0.75 \times 48 \times 0.442 \times 2 = 15.9 \times 2 = 31.8$ kips

Bearing on W18 × 50: $\phi R_n = \phi 2.4 d_b t F_u = 0.75 \times 2.4 \times 0.75 \times 0.355 \times 65 = 31.2$ kips/bolt

Bearing on ⅝-in-thick angles: $\phi R_n = 0.75 \times 2.4 \times 0.75 \times (2 \times 0.625) \times 58 = 97.9$ kips/bolt pair

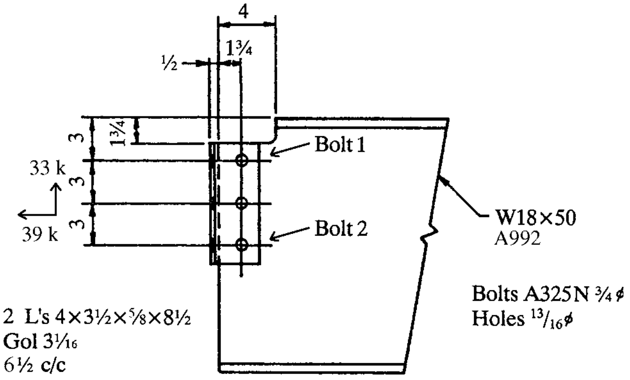


FIGURE 3.33 Double-angle framed connection. (*Source*: *A. R. Tamboli,* Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

Tear-out at extreme bolt no. 1:

- At W18 × 50, $\phi R_n = 0.75 \times 1.2 \times (1.75 - {}^{13}/_{32}) \times 0.355 \times 62 = 27.9$ kips/bolt
- At angles, $\phi R_n = \phi 1.2 L_c t F_u \phi R_b = 0.75 \times 1.2 \times (3 - {}^{13}/_{16}) \times (2 \times 0.625) \times 58 = 143$ kips/bolt pair

Tear-out at extreme bolt no. 2:

- At W18 × 50, $\phi R_n = 0.75 \times 1.2 \times (3 - {}^{13}/_{16}) \times 0.355 \times 65 = 45.4$ kips/bolt
- At angles, $\phi R_n = 0.75 \times 1.2 \times (1.25 - {}^{13}/_{32}) \times (2 \times 0.625) \times 58 = 55.1$ kips/bolt pair

Tear-out at intermediate bolts:

- At W18 × 50, $\phi R_n = 0.75 \times 1.2 \times (3 - {}^{13}/_{16}) \times 0.355 \times 65 = 45.4$ kips/bolt
- At angles, $\phi R_n = 0.75 \times 1.2 \times (3 - {}^{13}/_{16}) \times (2 \times 0.625) \times 58 = 143$ kips/bolt pair

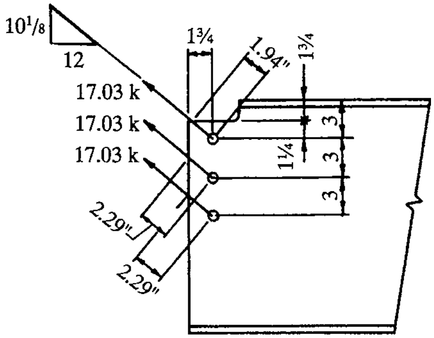


FIGURE 3.34 Edge distances along line of action. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Design strength with three rows of bolts ($n = 3$):

$$\begin{aligned}\phi R_n &= \phi R_n \text{ ext bolt 1 min} + \phi R_n \text{ ext bolt 2 min} + \phi R_n \text{ int bolt min} \times (\text{n} - 2)\\ &= 27.9 + 31.2 + 31.2 \times (3 - 2) = 90.3 \text{ kips} \geq 33 \text{ kips} \qquad \text{OK}\end{aligned}$$

Design strength under combined forces:

Resultant force at connection $= \sqrt{V^2 + T^2} = 51.1$ kips.

Edge distances along line of resultant force are shown in Fig. 3.34.

Critical bearing (including tear-out) at top or bottom bolt per Eq. (3.28b):

$$\phi R_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d_b t F_u$$

$$\phi R_n = 0.75 \times 1.2 \times (1.94 - {}^{13}/_{32}) \times 0.355 \times 65 \leq 0.75 \times 2.4 \times 0.75 \times 0.355 \times 65$$

$$31.8 < 31.2 = 31.2 \text{ kips/bolt}$$

Bearing at interior bolt per Eq. (3.28b):

$$\phi R_n = 0.75 \times 1.2 \times (2.29 - {}^{13}/_{32}) \times 0.355 \times 65 \leq 0.75 \times 2.4 \times 0.75 \times 0.355 \times 65$$

$$39.1 \leq 31.2 = 31.2 \text{ kips/bolt}$$

Design strength with three bolts ($n = 3$):

$$\begin{aligned}\phi R_n &= 2 \times \phi R_n \text{ top or btm bolt min} + \phi R_n \text{ int bolt min} \times (n - 2)\\ &= 2 \times 31.2 + 31.2 \times (3 - 2) = 93.6 \text{ kips} \geq 51.1 \text{ kips} \qquad \text{OK}\end{aligned}$$

Design strength for shear in angles and beam web:

Gross section shear on angles:

$$\phi R_n = \phi \times 0.60 A_g F_y = 0.90 \times 0.60 \times 8.5 \times (2 \times 0.625) \times 36 = 207 \text{ kips} \geq 33 \text{ kips} \qquad \text{OK}$$

Net section shear on angles:

$$\begin{aligned}\phi R_n &= \phi \times 0.60 A_{nv} F_u = 0.75 \times 0.60 \times [8.5 - (3 \times 0.875)] \times (2 \times 0.625) \times 58\\ &= 192 \text{ kips} \geq 33 \text{ kips} \qquad \text{OK}\end{aligned}$$

Gross shear on full beam web:

$$\phi R_n = 0.90 \times 0.60 \times 18 \times 0.355 \times 50 = 173 \text{ kips} \geq 33 \text{ kips} \qquad \text{OK}$$

Gross shear on remaining beam web:

$$\phi R_v = 0.9 \times 0.60 \times 16.3 \times 0.355 \times 50 = 156 \text{ kips} \geq 33 \text{ kips} \qquad \text{OK}$$

Design strength for block shear on beam web [Eq. (3.32)]:

$$\phi R_n = \phi[U_{\text{bs}} F_u A_{\text{nt}} + \min(0.6 F_y A_{gv},\ 0.6 F_u A_{nv})]$$

Check block shear for vertical force:

$$\begin{aligned}
A_{\text{gt}} &= 0.355 \times 1.75 = 0.621\ \text{in}^2 \\
A_{\text{nt}} &= 0.355 \times (1.75 - 0.5 \times 0.875) = 0.466\ \text{in}^2 \\
A_{gv} &= 0.355 \times [1.25 + (3-1) \times 3] = 2.57\ \text{in}^2 \\
A_{nv} &= 0.355 \times [1.25 + (3-1) \times 3 - (3-0.5) \times 0.875] = 1.8\ \text{in}^2 \\
\phi R_n &= 0.75[1 \times 65 \times 0.466 + \min(0.6 \times 50 \times 2.57,\ 0.6 \times 65 \times 1.8)] \\
&= 0.75[30.3 + \min(77.1,\ 70.2)] = 80.5,\ 75.4 \\
&= 75.4\ \text{kips}
\end{aligned}$$

Check block shear for horizontal force:

$$\begin{aligned}
A_{gv} &= 0.355 \times 1.75 = 0.621\ \text{in}^2 \\
A_{nv} &= 0.355 \times (1.75 - 0.5 \times 0.875) = 0.466\ \text{in}^2 \\
A_{\text{gt}} &= 0.355 \times [1.25 + (3-1) \times 3] = 2.57\ \text{in}^2 \\
A_{\text{nt}} &= 0.355 \times [1.25 + (3-1) \times 3 - (3-0.5) \times 0.875] = 1.8\ \text{in}^2 \\
\phi R_n &= 0.75[1 \times 65 \times 1.8 + \min(0.6 \times 50 \times 0.621,\ 0.6 \times 65 \times 0.466)] \\
&= 0.75[117.0 + \min(18.6,\ 18.2)] = 101.7,\ 101.4 \\
&= 101\ \text{kips}
\end{aligned}$$

Check block shear for combined force:

$$\left(\frac{V}{\phi R_n}\right)^2 + \left(\frac{T}{\phi R_n}\right)^2 = \left(\frac{33}{75.4}\right)^2 + \left(\frac{39}{101}\right)^2 = 0.340 \le 1 \qquad \text{OK}$$

Design strength for bending in net section of single cope:

Follow procedure in AISC Manual.

Area of remaining tee = 9.85 in^2

Center of gravity of remaining tee = 4.88 in above bottom of beam

Moment of inertia of remaining tee = 274 in^4

Section modulus at top of tee = 24.1 in^3

Section modulus at bottom of tee = 56.1 in^3

$M_u = R_u e = 33 \times (4.00 + 0.50) = 148$ in·kips

$\phi M_n = \phi F_u S_{\text{net}} = 0.75 \times 65 \times 24.1 = 1175$ in·kips ≥ 148 in·kips OK

Design strength for web buckling in net section of single cope:

Follow procedure in AISC Manual.

c = cope length = 4.00 in; d = beam depth = 18.0 in; h_o = net depth = 16.25 in

$c/d = 0.222 \le 1$; therefore, adjustment factor is $f = 2(c/d) = 0.444$

$c/h_o = 0.246 \le 1$; therefore, plate buckling coefficient is $f = 2.2(h_o/c)^{1.65} = 22.2$

$$\phi F_{\text{cr}} = 23{,}590\left(\tfrac{t_w}{h_o}\right)^2 fk$$

$$\phi F_{\text{cr}} = 23{,}590\left(\tfrac{0.355}{16.25}\right)^2 \times 0.444 \times 22.22 = 111\ \text{ksi} < 0.9F_y = 45\ \text{ksi}$$

$\phi M_n = \phi F_{\text{cr}} S_{\text{net}} = 45 \times 13.8 = 621$ in·kips ≥ 490 in·kips OK

Design strength for web buckling with combined force (39 kips applied axially):

$$\phi R_n = \left(\phi F_{\text{cr}} - \frac{V_e}{S}\right)A = \left(45 - \frac{33 \times 4.50}{24.1}\right)9.85 = 382 \text{ kips} < 39 \text{ kips} \qquad \text{OK}$$

Design tensile strength of bolt for shear/tension interaction:

Design strength is $\phi F'_{\text{nt}}$, where $F'_{\text{nt}} = 1.3F_{\text{nt}} - (F_{\text{nt}}/\phi F_{nv})f_v \le F_{\text{nt}}$

$$\phi F'_{\text{nt}} = 0.75\left\{1.3 \times 90 - \frac{90}{36}\left[\frac{33}{6(0.75)^2(\pi/4)}\right]\right\} = 64.5 \le 0.75 \times 90 = 67.5 \text{ ksi}$$

$$\phi F'_{\text{nt}} = 64.5 \text{ ksi}$$

Design tensile strength of $^3/_4$-in-diameter A325N bolt reduced for shear is

$$\phi r_n' = \phi F'_{\text{nt}} A_b = 64.5 \times 0.442 = 28.5 \text{ kips/bolt}$$

For 6 bolts = 28.5 × 6 = 171 kips ≥ 39 kips OK

Design strength for prying action of angles:

Follow procedure in AISC Manual. See Art. 3.5 for background.

a = distance from bolt centerline to edge of angle = 1.93 in

b = distance from bolt centerline to centerline of angle leg = 2.76 in

d_b = bolt diameter = 0.75 in

d_h = bolt diameter = 0.875 in

p = tributary length per bolt = 2.83 in

d' = width of hole along length of angle = 0.875 in

$a' = a + d_b/2 \le 1.25 \times \text{b} + d_b/2 = 1.93 + 0.75/2 \le 1.25 \times 2.76 + 0.75/2$

$= 2.30 \le 2.98$ Use $a' = 2.30$ in

$b' = b - d_b/2 = 2.76 - 0.75/2 = 2.38$ in

$\rho = b'/a' = 2.38/2.30 = 1.03$

$\delta = 1 - d'/p = 1 - 0.875/2.83 = 0.691$

t_c = angle thickness required to develop strength of bolt with no prying action, in

$$t_c = \sqrt{\frac{4.44\phi R_n b'}{pF_t}} \qquad t_c = \sqrt{\frac{4.44 \times 28.5 \times 2.38}{2.83 \times 58}} = 1.35 \text{ in}$$

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \qquad \alpha' = \frac{1}{0.691(1+1.03)}\left[\left(\frac{1.35}{0.625}\right)^2 - 1\right] = 2.61$$

$$\alpha' > 1; \text{ therefore } T_d = \phi r_n'\left(\frac{t}{t_c}\right)^2(1+\delta) = 28.5\left(\frac{0.625}{1.35}\right)^2(1+0.691) = 10.3 \text{ kips/bolt}$$

For 6 bolts = 10.3 × 6 = 61.8 kips ≥ 39 kips OK

3.4.3 Example: Extended Single-Plate Shear Connections

Single-plate shear connections as shown in Fig. 3.35 can be very economical connections. In-fill beams can be drilled on the fabricator's drill line with no further handling, since the beams will

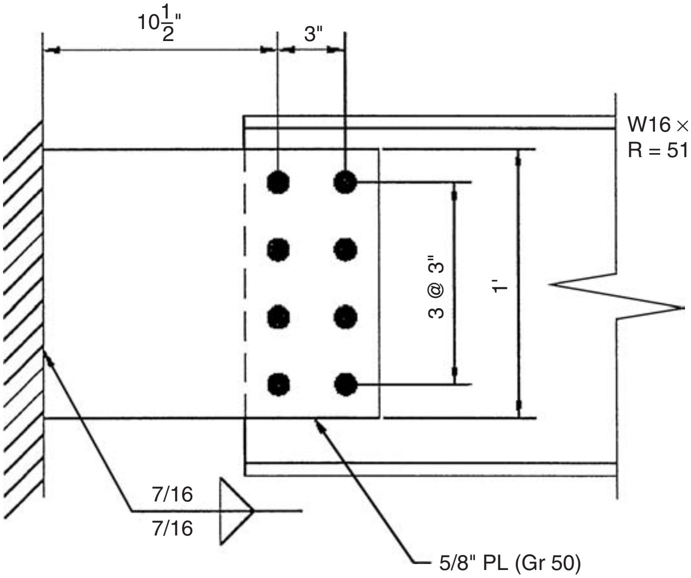


FIGURE 3.35 Extended single-plate shear connection.

not require the coping needed for more traditional beam-to-beam connections. Beam-to-column web connections are also made easier. Since the beam can be connected beyond the column flanges, erection is greatly eased. Unlike double-angle, end-plate, and sometimes single-angle connections, there will be no bolts at the support common to more than one member, so safety is also improved.

A new procedure for the design of extended single-plate shear connections was introduced in the 2005 AISC Manual. This procedure, illustrated in the following example, addresses many of the past concerns for these connections, including plate buckling, ductility at both the welds and the bolts, and support rotation.

Consider a $^5/_8$-in-thick single shear plate of Grade 50 steel, which connects to the web of a W16 × 45 A992 beam (Fig. 3.35). Bolts are 1-in-diameter A325X. The factored shear force is 51 kips. To check the eccentric bolt group for bolt shear, bearing, and tear-out, refer to Table 7-9 of the AISC Manual. With $s = 3$ in (bolt spacing), $n = 4$ (number of bolts in a vertical row), and $e_x = 12$ in (eccentricity, support to c.g. bolts), the coefficient for eccentrically loaded bolt group is $C = 2.06$.

Design strength for bolt shear:

$$\phi R_n = C\phi F_n A_b = 2.06 \times (0.75 \times 60 \times 0.785) = 2.06 \times 35.3 = 72.8 \text{ kips} > 51 \text{ kips} \qquad \text{OK}$$

Design strength for bolt bearing and tear-out: Since the direction of the force on each bolt is not known, it is conservative to determine the capacity of a single bolt tearing through the smallest edge distance and prorate the connection capacity accordingly.

$$\phi R_n = 0.75 \times 1.2 \times \left(1.5 - \frac{1.0625}{2}\right) \times 0.345 \times 65 \leq 0.75 \times 2.4 \times 1 \times 0.345 \times 65$$
$$= 19.6 < 40.4 \text{ kips/bolt}$$

This results in a conservative connection capacity of

$$\phi R_n = 2.06 \times 19.6 = 40.4 \text{ kips} < 51 \text{ kips} \qquad \text{No good}$$

Alternatively, the direction of the force on each bolt can be found using the instantaneous center method. The bolt tear-out can then be found along the direction of the force. Though this procedure (Muir and Thornton, 2004) is too complex to show here, it results in an effective bolt value of 24.8 kips. Thus,

$$\phi R_n = 2.06 \times 24.8 = 51.0 \text{ kips} \geq 51 \text{ kips} \qquad \text{OK}$$

Determine maximum plate thickness to ensure connection ductility at bolts: The maximum nominal strength of a 1-in-diameter A325X bolt (with the 1.25 factor discussed in Art. 3.1.5) is

$$R_n = 1.25 \times 60 \left(\frac{1}{2}\right)^2 \pi = 58.9 \text{ kips/bolt}$$

For the "moment only" condition, $C' = 26.0$ from AISC Table 7-9. This results in a moment M of

$$M = 58.9 \times 26.0 = 1530 \text{ in}\cdot\text{kips}$$

The maximum plate thickness of

$$t_{p\max} = \frac{6M}{F_y L^2} = \frac{6(1530)}{50(12)^2} = 1.28 \text{ in}$$

A plate up to $1^1/_4$-in-thick is allowed for ductility. Thus, proceed with checks for $^5/_8$-in-thick plate.

Design strength for interaction of shear and bending in plate:

- Shear stress on gross section of plate $= f_v = \frac{51.0}{0.625 \times 12} = 6.80$ ksi
- Plastic section modulus of gross section of plate $= Z_{\text{gross}} = \frac{0.625 \times 12^2}{4} = 22.5 \text{ in}^3$
- Bending stress on gross section of plate $= f_b = \frac{51.0 \times 10.5}{22.5} = 23.8$ ksi
- Shear and bending interaction on gross section of plate $= \left(\frac{6.80}{0.9 \times 0.6 \times 50}\right)^2 + \left(\frac{23.8}{0.9 \times 50}\right)^2 = 0.343 \leq 1$
- Shear stress on net section in plate $= f_v = \frac{51.0}{0.625 \times (12 - 4 \times 1.125)} = 10.9$ ksi
- Plastic section modulus of net section of plate $= Z_{\text{net}} = 22.5\left(1 - \frac{1.125}{3}\right) = 14.1 \text{ in}^3$
- Bending stress on net section of plate $= f_b = \frac{51.0 \times 10.5}{14.1} = 38.0$ ksi
- Shear and bending interaction on net section of plate $= \left(\frac{10.9}{0.75 \times 0.6 \times 65}\right)^2 + \left(\frac{38.0}{0.75 \times 65}\right)^2 = 0.746 \leq 1$

Plate buckling:

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}500 + 112{,}000(L/2e)^2}} = \frac{12\sqrt{50}}{0.625\sqrt{47{,}500 + 112{,}000[12/2(10.5)]^2}} = 0.468$$

Since $\lambda = 0.472 \leq 0.7$, buckling does not govern.

Size weld: For ductility, the weld size must be equal to $^5/_8\, t_p = 0.625(0.625) = 0.391$ in. Therefore, use $^7/_{16}$-in fillet welds. Checks show that the welds meet design strength requirements for shear and bending.

3.5 AXIAL CONNECTIONS

Axial connections carry primarily tension or compression axial force, but may also involve shear and moment as well.

3.5.1 Hanger Connections

The most interesting connection of the genre is the type that involves prying action, sometimes of both the connection fitting and the supporting member. Figure 3.36 shows a typical example. The calculations to determine the capacity of this connection are as follows. The connection can be broken into three main parts: the angles, the W16 × 57 piece, and the W18 × 50 supporting member. The angles are A36 and the W sections are A992 steel. Bolts are A325N. The three main parts are joined by two additional parts, the bolts of the angles to the piece W16 and the bolts from the piece W16 to the W18. The load path in this connection is unique. The load *P* passes from the angles through the bolts into the piece W16, thence through bolts again into the supporting W18. The latter bolt group is arranged to straddle the brace line of action. These bolts thus see only direct tension and shear, and no additional tension due to moment. Statics is sufficient to establish this. Consider now the determination of the design strength of this connection.

Angles. The limit states for the angles are gross tension, net tension, block shear rupture, and bearing. The load can be compression as well as tension in this example. Compression will affect the angle design, but tension will control the above limit states.

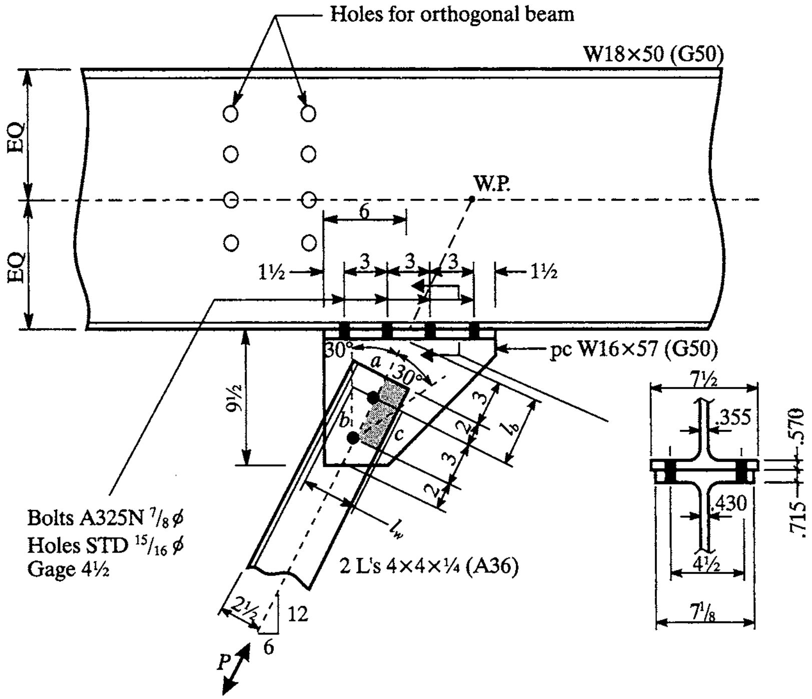


FIGURE 3.36 Typical bolted hanger connection. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Gross Tension. The gross area A_{gt} is $1.94 \times 2 = 3.88$ in^2. The gross tension design strength is

$$\phi R_{\mathrm{gt}} = \phi F_y A_{\mathrm{gt}} = 0.90 \times 36 \times 3.88 = 126 \text{ kips}$$

Net Tension. The net tension area is $A_{\mathrm{nt}} = 3.88 - 0.25 \times 1.0 \times 2 = 3.38$ in^2. The effective net tension area A_e is less than the net area because of shear lag, because only one of the two angle legs is connected. From the AISC Specification, Sec. D3.3,

$$U = \max(0.60,\ 1 - 1.09/3) = 0.637$$
$$A_e = U \times A_{\mathrm{nt}} = 0.637 \times 3.38 = 2.15 \text{ in}^2$$

The net tension design strength is

$$\phi R_{\mathrm{nt}} = \phi F_t A_e = 0.75 \times 58 \times 2.15 = 93.5 \text{ kips}$$

Block Shear Rupture. This failure mode involves the tearing out of the shaded block at the end of the angles in Fig. 3.36. Ultimate strength is characterized by yielding on the longitudinal line through the bolts (line *ab*) and a simultaneous fracture on the perpendicular line from the bolts longitudinal line to the angle toe (line *bc*). Because yielding on the longitudinal section may sometimes exceed fracture on this section, AISC Specification J4.3 limits the strength to the lesser of the two. Thus, the block shear limit state is

$$\phi R_{\mathrm{bs}} = 0.75[U_{\mathrm{bs}} F_u A_{\mathrm{nt}} + \min(0.6 F_y A_{gv},\ 0.6 F_u A_{nv})] \tag{3.57}$$

For line *ab*, the gross shear area is $A_{gv} = 5 \times 0.25 \times 2 = 2.5$ in^2 and the net shear area is $A_{nv} = 2.5 - (1.5 \times 0.25 \times 1.0)2 = 1.75$ in^2. For line *bc*, the gross tension area is $A_{gt} = 1.5 \times 0.25 \times 2 = 0.75$ in^2 and the net tension area is $A_{\mathrm{nt}} = 0.75 - 0.5 \times 1.0 \times 0.25 \times 2 = 0.5$ in^2. The term U_{bs} accounts for the fact that for highly eccentric connections, the tension force distribution on section *bc* will not be uniform. In this case, U_{bs} is taken as 0.5. In the present case, the force distribution is essentially uniform because the angle gage line and the angle gravity axis are close to each other. Thus $U_{\mathrm{bs}} = 1.0$, and the block shear strength is

$$\phi R_{\mathrm{bs}} = 0.75[1.0 \times 58 \times 0.5 + \min(0.6 \times 36 \times 2.5,\ 0.6 \times 58 \times 1.75)] = 62.2 \text{ kips}$$

Shear/Bearing/Tear-out on Bolts and Parts. As pointed out in Art. 3.3.4, bearing, tear-out, and bolt shear are tied together for each bolt. Therefore, it is not possible to check bolt shear for the bolt group as a whole, and bearing/tear-out for each part separately, and then take the minimum of these limit states as the controlling limit state. The procedure is as follows for each bolt.

ANGLES TO PIECE W16. For the upper bolt, the design strengths for the various limit states are as follows:

Bolt shear: $\phi R_v = 0.75 \times 48 \times (\pi/4) \times 0.875^2 \times 2 = 43.3$ kips

Bearing on angles: $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 2 \times 0.25 \times 58 = 45.7$ kips

Bearing on W16 × 57: $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 0.43 \times 65 = 44.0$ kips

Tear-out on angles: $\phi R_{\mathrm{to}} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 2 \times 0.25 \times 58 = 40.0$ kips

Tear-out on W16 × 57: $\phi R_{\mathrm{to}} = 0.75 \times 1.2(3 - 0.9375) \times 0.430 \times 65 = 51.9$ kips

The design strength for shear/bearing/tear-out of the upper bolt is the lowest value, 40.0 kips.

For the lower bolt, the design strengths for the various limit states are as follows:

Bolt shear: $\phi R_v = 43.4$ kips

Bearing on the angles: $\phi R_p = 45.7$ kips

Bearing on the W16 × 57: $\phi R_p = 44.0$ kips

Tear-out on the angles: $\phi R_{\text{to}} = 0.75 \times 1.2(3 - 0.9375) \times 2 \times 0.25 \times 58 = 53.8$ kips

Tear-out on the W16 × 57: $\phi R_{\text{to}} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 0.430 \times 65 = 38.5$ kips

The shear/bearing/tear-out strength of the lower bolt is thus 38.5 kips, and the capacity of the connection in these limit states is $\phi R_{vp} = 40.0 \times 1 + 38.5 \times 1 = 78.5$ kips.

BOLTS FOR ANGLES TO PIECE W16. The limit state for the bolts is shear. The shear capacity of one A325N bolt is

$$\phi r_v = 0.75 \times 48 \times \frac{\pi}{4} \times 0.875^2 = 21.6 \text{ kips}$$

In this case, the bolts are in double shear and the double-shear value per bolt is 21.6 × 2 = 43.3 kips/bolt. Note that because of bearing limitations, shown in the preceding calculations, this value cannot be achieved. The bolt shear strength is limited by the bearing strength of the parts. Thus the design strength for the bolts in shear is limited to the bearing strength, so

$$\phi R_v = \phi R_p = 78.5 \text{ kips}$$

PIECE W16 × 57. The limit states for this part of the connection are Whitmore section yield and buckling, bearing, and prying action in conjunction with the W16 flange–to–W18 flange bolts. Because there is only one line of bolts, block shear is not a limit state. Bearing has already been considered with the angle checks.

Whitmore Section. The Whitmore section is denoted by l_w on Fig. 3.36. It is formed by 30° lines from the bolt farthest away from the end of the brace to the intersection of these lines with a line through and perpendicular to the bolt nearest the end of the brace. Whitmore (1952) determined that this 30° spread gave an accurate estimate of the stress in gusset plates at the end of the brace. The length of the Whitmore section $l_w = 3(\tan 30°)2 = 3.46$ in.

The design strength for the limit state of yielding on the Whitmore section in the W16 × 50 with a web thickness of 0.430 in is

$$\phi R_{\text{wy}} = \phi F_y A_g = 0.90 \times 50 \times 3.46 \times 0.430 = 67.0 \text{ kips}$$

Tests (Gross, 1990) have shown that the Whitmore section can be used as a conservative estimate for buckling of a gusset, such as the web of the W16 × 57. If the load P is compression, the gusset can buckle laterally in a sidesway mode and, for this mode, the K factor is 1.2. The buckling length is $l_b = 5$ in in Fig. 3.36. The radius of gyration of the rectangular section is $r = t/\sqrt{12}$. Thus the slenderness ratio is

$$\frac{Kl}{r} = \frac{1.2 \times 5 \times \sqrt{12}}{0.430} = 48.3$$

Since $(Kl/r) > 25$, AISC Specification Sec. J4.4 on strength of elements in compression does not apply. Instead, the column buckling equations of Sec. E3 apply. Thus,

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 29{,}000}{(48.3)^2} = 123 \text{ ksi}$$

$$\begin{aligned} F_{\text{cr}} &= (0.658^{F_y/F_e})F_y \\ &= [0.658^{(50/123)}]50 = 42.2 \text{ ksi} \end{aligned}$$

$$\phi F_{\text{cr}} = 0.9 \times 42.2 = 38.0 \text{ ksi}$$

$$\phi R_{\text{wb}} = 38.0 \times 3.46 \times 0.430 = 56.5 \text{ kips}$$

Bearing. Bearing has been considered with the angles, above.

Prying Action. Prying action refers explicitly to the extra tensile force in bolts that connect flexible plates or flanges subjected to loads normal to the flanges. For this reason, prying action involves not only the bolts but the flange thickness, bolt pitch, and gage, and, in general, the geometry of the entire connection.

The AISC LRFD Manual presents a method for calculating the effects of prying. This method was originally developed by Struik (Struik and deBack, 1969; Kulak et al., 1987). The form used in the AISC LRFD Manual was developed by Thornton (1985), for ease of calculation and to provide optimum results, that is, maximum capacity for a given connection (analysis) and minimum required thickness for a given load (design). Thornton (1992, 1997) has shown that this method gives a very conservative estimate of ultimate load and that very close estimates of ultimate load can be obtained by using the flange ultimate strength F_u in place of yield strength F_y in the prying action formulas. More recently, Swanson (2002) has confirmed Thornton's (1992, 1997) results with modern materials. For this reason, the AISC Manual now uses F_u in place of F_y in the prying action formulas. Note that the resistance factor, ϕ, used with the F_u is 0.90, because the flange failure mode is yielding with strain hardening rather than fracture.

From the foregoing calculations, the design strength of this connection is limited by buckling in the Whitmore section to 56.5 kips. Take this as the required strength and proceed with prying calculations. The vertical component of 56.5 kips is 50.5 kips and the horizontal component is 25.3 kips. Thus, the shear per bolt is $V = 25.3/8 = 3.16$ kips and the tension per bolt is $T = 50.5/8 = 6.31$ kips. Since $3.16 < \phi r_v = 21.6$ kips, the bolts are OK for shear. Note that the bolts also need to be checked for bearing, as was done for the angles. In this case, bearing is seen to be OK by inspection.

Bolt Design Tensile Strength. Bolt design tensile strength must be checked by the interaction equation for bolts in bearing-type connections:

$$F'_{\text{nt}} = 1.3F_{\text{nt}} - \frac{F_{\text{nt}}}{\phi F_{nv}} f_v \le F_{\text{nt}}$$

where F_{nt} = bolt nominal tensile strength = 90 ksi for A325N

F_{nv} = bolt nominal shear strength = 48 ksi for A325N

$\phi = 0.75$

f_v = required shear strength per bolt = $\frac{3.16}{0.6013} = 5.26$ ksi

$F'_{\text{nt}} = 1.3 \times 90 - \frac{90}{0.75 \times 48} \times 5.26 = 104 \text{ ksi} \le 90 \text{ ksi}$ Use $F'_{\text{nt}} = 90$ ksi

Design tensile strength per bolt is

$$\phi r_t' = 0.75 \times 90 \times 0.6013 = 40.6 \text{ kips}$$

Since this is greater than the required strength per bolt $T = 6.31$ kips, the bolts are OK.

Prying of the W16 Piece. Prying of the W16 piece is checked following the procedure in the AISC Manual (see Art. 3.4 for terms).

$$b = \frac{4.5 - 0.430}{2} = 2.035 \text{ in}$$

$$a = \frac{7.125 - 4.5}{2} = 1.3125 \text{ in}$$

Check that $a < 1.25b = 1.25 \times 2.035 = 2.544$. Since $a = 1.3125 < 2.544$, use $a = 1.3125$ in. If $a > 1.25b$, $a = 1.25b$ should be used.

$$b' = 2.035 - \frac{0.875}{2} = 1.598 \text{ in}$$

$$a' = 1.3125 + \frac{0.875}{2} = 1.75 \text{ in}$$

$$\rho = \frac{b'}{a'} = 0.91$$

$$p = 3 \text{ in}$$

$$\delta = 1 - \frac{b'}{p} = 1 - \frac{0.9375}{3} = 0.6875$$

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$$

$$t_c = \sqrt{\frac{4.44(\phi r_t')b'}{pF_u}} = \sqrt{\frac{4.44 \times 40.6 \times 1.598}{3 \times 65}} = 1.215 \text{ in}$$

$$\alpha' = \frac{1}{0.6875 \times 1.91}\left[\left(\frac{1.215}{0.715}\right)^2 - 1\right] = 1.44$$

Since $\alpha' > 1$, use $\alpha' = 1$ in subsequent calculations. $\alpha' = 1.44$ means that the bending of the W16 × 57 flange will be the controlling limit state. The bolts will not be critical, that is, the bolts will not limit the prying strength. The design tensile strength T_d per bolt including the flange strength is

$$T_d = \phi r_t'\left(\frac{t}{t_c}\right)^2(1+\delta) = 40.6 \times \left(\frac{0.715}{1.215}\right)^2 \times 1.6875 = 23.7 \text{ kips} > 5.96 \text{ kips} \qquad \text{OK}$$

In addition to the prying check on the piece W16 × 57, a check should also be made on the flange of the W18 × 50 beam. See Art. 3.7.3. Thus,

$$\bar{b} = \frac{4.5 - 0.355}{2} = 2.073 \text{ in}$$

$$\bar{a} = \frac{7.5 - 4.5}{2} = 1.50 \text{ in}$$

$$n = 4$$

$$p = 3 \text{ in}$$

$$p_{\text{eff}} = \frac{3(4-1) + (\pi \times 2.073) + (2 \times 1.50)}{4} = 4.63 \text{ in}$$

Following the procedure in the AISC Manual,

$$b = \bar{b} = 2.073 \text{ in}$$

$$a = 1.3125 \text{ in}$$

Note that the prying lever arm is controlled by the narrower of the two flanges.

$$b' = 2.073 - \frac{0.875}{2} = 1.636 \text{ in}$$

$$a' = 1.3125 + \frac{0.875}{2} = 1.75 \text{ in}$$

$$\rho = 0.93$$

$$p = p_{\text{eff}} = 4.63 \text{ in}$$

$$\delta = 1 - \frac{0.9375}{4.63} = 0.798$$

$$t_c = \sqrt{\frac{4.44 \times 40.6 \times 1.636}{4.63 \times 65}} = 0.990 \text{ in}$$

$$\alpha' = \frac{1}{0.798 \times 1.93}\left[\left(\frac{0.990}{0.570}\right)^2 - 1\right] = 1.31$$

Use $\alpha' = 1$.

The design strength of the bolt in tension is

$$T_d = 40.6 \times \left(\frac{0.570}{0.990}\right)^2 \times 1.798 = 24.2^k > 5.96^k \qquad \text{OK}$$

Web Yielding on the W18 × 50 Beam. Since $5k = 5 \times 1.25 = 6.25 > p = 3$ in, the web tributary to each bolt at the k distance exceeds the bolt spacing and thus $N = 9$.

$$\phi R_{\text{wy}} = 1.0 \times (9 + 5 \times 1.25) \times 50 \times 0.355 = 271 \text{ kips} > 50.5 \text{ kips} \qquad \text{OK}$$

Web Crippling on the W18 × 50 Beam. Web crippling occurs when the load is compression, thus $N = 12$, the length of the W16 piece.

$$\phi R_{wcp} = 0.75 \times 0.80 \times 0.355^2 \left[1 + 3\left(\frac{12}{18.0}\right)\left(\frac{0.355}{0.570}\right)^{1.5}\right]\sqrt{\frac{29{,}000 \times 50 \times 0.570}{0.355}}$$
$$= 229 \text{ kips} > 50.5 \text{ kips} \qquad \text{OK}$$

This completes the design calculations for this connection. A load path has been provided through every element of the connection. For this type of connection, the beam designer should make sure that the bottom flange is stabilized if P can be compressive. A transverse beam framing nearby as shown in Fig. 3.36 by the W18 × 50 web hole pattern, or a bottom flange stay (kicker) will provide stability.

3.5.2 Column Base Plates

The geometry of a column base plate is shown in Fig. 3.37. The area of the base plate is $A_1 = B \times N$. The area of the pier which is concentric with A_1 is A_2. If the pier is not concentric with the base plate, only that portion which is concentric can be used for A_2. The design strength of the concrete in bearing is

$$\phi_c F_p = 0.6 \times 0.85 f'_c \sqrt{\frac{A_2}{A_1}} \tag{3.58}$$

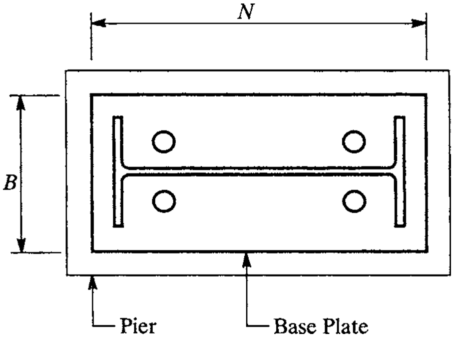


FIGURE 3.37 Column base plate. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

where f_c' is the concrete compressive strength in ksi and

$$1 \le \sqrt{\frac{A_2}{A_1}} \le 2 \tag{3.59}$$

The required bearing strength is

$$f_p = \frac{P}{A_1} \tag{3.60}$$

where P is the factored column load in kips. In terms of these variables, the required base plate thickness is

$$t_p = l\sqrt{\frac{2f_p}{\phi F_y}} \tag{3.61}$$

where $l = \max(m, n, \lambda n')$

ϕF_y = base plate design strength = $0.90F_y$

$m = \frac{N-0.95d}{2}$

$n = \frac{B-0.8b_f}{2}$

$n' = \frac{\sqrt{db_f}}{4}$

$\lambda = \frac{2\sqrt{x}}{1+\sqrt{1-x}} \le 1$

$x = \frac{4db_f}{(d+b_f)^2}\frac{f_p}{\phi_c F_p}$

d = depth of column

b_f = flange width of column

For simplicity, λ can always be conservatively taken as unity. The formulation given here was developed by Thornton (1990a, 1990b), based on previous work by Murray (1983), Fling (1970), and Stockwell (1975). It is the method given in the 2005 AISC Manual.

Example of Base Plate Calculation. The column of Fig. 3.37 is a W24 × 84 carrying 600 kips. The concrete has $f_c' = 4.0$ ksi. Try a base plate of A36 steel, 4 in bigger than the column in both directions. Since $d = 24.125$ in and $b_f = 9$ in, $N = 24.125 + 4 = 28.125$ in, $B = 9 + 4 = 13$ in. Try a plate 28 × 13 in. Assume that 2 in of grout will be used, so the minimum pier size is 32 × 17 in. Thus $A_1 = 28 \times 13 = 364$ in^2, $A_2 = 32 \times 17 = 544$ in^2, $\sqrt{A_2/A_1} = 1.22 < 2$. OK.

$$\phi_c F_p = 0.6 \times 0.85 \times 4 \times 1.22 = 2.49 \text{ ksi}$$

$$f_p = \frac{600}{364} = 1.65 \text{ ksi} < 2.49 \text{ ksi} \qquad \text{OK}$$

$$m = \frac{28 - 0.95 \times 24.125}{2} = 2.54 \text{ in}$$

$$n = \frac{13 - 0.8 \times 9}{2} = 2.90 \text{ in}$$

$$n' = \frac{\sqrt{24.125 \times 9}}{4} = 3.68 \text{ in}$$

$$x = \frac{4 \times 24.125 \times 9.0}{(24.125 + 9.0)^2}\frac{1.65}{2.49} = 0.52$$

$$\lambda = \frac{2\sqrt{0.52}}{1+\sqrt{1-0.52}} = 0.85$$

$$l = \max(2.54,\ 2.90,\ 0.85 \times 3.68) = 3.13 \text{ in}$$

$$t_p = 3.13\sqrt{\frac{2 \times 1.65}{0.9 \times 36}} = 0.99 \text{ in}$$

Use a plate 1 × 13 × 28 in of A36 steel. If the conservative assumption of $\lambda = 1$ were used, $t_p = 1.17$ in, which indicates a $1^1/_4$-in-thick base plate.

Erection Considerations. In addition to designing a base plate for the column compression load, loads on base plates and anchor rods during erection should be considered. The latest OSHA requirements postulate a 300-lb load 18 in off the column flange in the strong axis direction, and the same load 18 in off the flange tips in the weak axis direction. Note that these loads are applied sequentially. A common design load for erection, which is much more stringent than the OSHA load, is a 1-kip working load, applied at the top of the column in any horizontal direction. If the column is, say, 40 ft high, this 1-kip force at a lever arm of 40 ft will cause a significant couple at the base plate and anchor bolts. The base plate, anchor bolts, and column-to-base plate weld should be checked for this construction load condition. A paper by Murray (1983) gives some yield line methods that can be used for doing this. Figure 3.37 shows four anchor rods. This is an OSHA erection requirement for all columns except minor posts.

3.5.3 Splices of Columns and Truss Chords

Section J1.4 of the AISC Specification says that finished-to-bear compression splices in columns need be designed only to hold the parts "securely in place." For this reason, the AISC provides a series of "standard" column splices in the AISC Manual. These splices are nominal in the sense that they are designed for no particular loads. Section J1.4 of the AISC Specification also requires that splices in trusses be designed for at least 50% of the design load (required compression strength), or for the moment and shear resulting from a transverse load equal to 2% of the required compressive strength of the member, whichever is less severe. The difference between columns and "other compression members," such as compression chords of trusses, is that for columns, splices are usually near lateral support points, such as floors, whereas trusses can have splices at the mid-panel point, where there is no lateral support. Either the 50% requirement or the 2% requirement can be used to address this situation.

Column Splice Moment Capacity. Figure 3.38 shows a standard AISC column splice for a W14 × 99 to a W14 × 109. If the column load remains compression, the strong-axis column shear can be carried by friction. The coefficient of static friction of steel to steel is of the order of 0.5 to 0.7, so quite high shears can be carried by friction. Suppose the compression load on this column is 700 kips. How much major-axis bending moment can this splice carry? Even though these splices are nominal, they can carry quite significant bending moment. The flange area of the W14 × 99 is $A_f = 0.780 \times 14.6 = 11.4 \text{ in}^2$. Thus, the compression load per flange is 700 × 11.4/29.1 = 274 kips. For a bending moment to cause a tension in the column flange, this load of 274 kips must first be unloaded. Assuming that the flange force acts at the flange centroid, the moment in the column can be represented as

$$M = T(d - t_f) = T(14.2 - 0.780) = 13.4T$$

If $T = 274$ kips, one flange will be unloaded, and $M = 13.4 \times 274 = 3670$ in·kips = 306 kip·ft. The design strength in bending for this column (assuming sufficient lateral support) is $\phi M_p = 649$ kip·ft. Thus, because of the compression load, the nominal AISC splice, while still seeing no load, can carry almost 50% of the column's bending capacity.

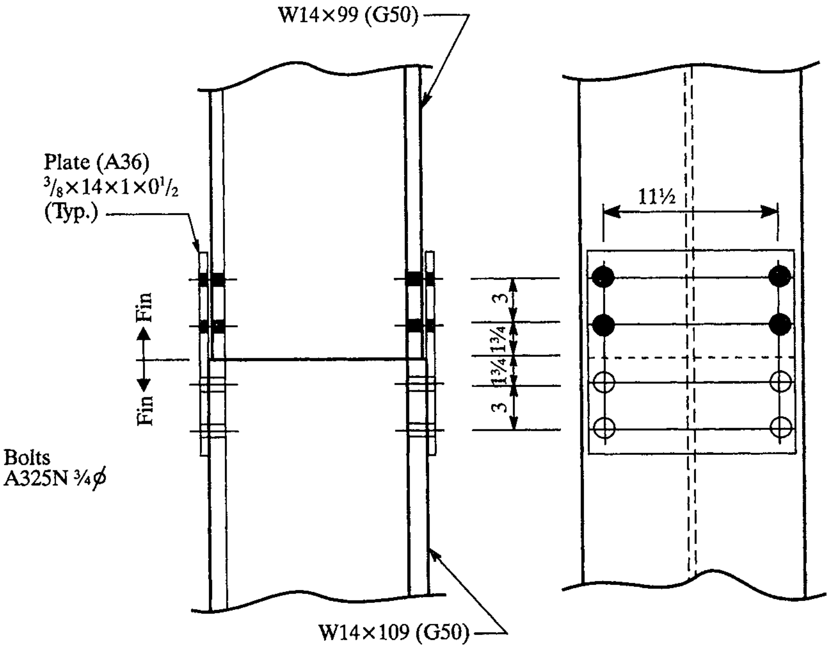


FIGURE 3.38 AISC standard column splice. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

The splice plates and bolts allow additional moment to be carried. It can be shown that the controlling limit state for the splice material is bolt shear. For one bolt, A325N, ¾ in in diameter, $\phi r_v =$ 15.9 kips. Thus, for four bolts, $\phi R_v = 15.9 \times 4 = 63.6$ kips. The splice forces are assumed to act at the faying surface of the deeper member. Thus, the moment capacity of the splice plates and bolts is $M_s =$ $63.6 \times 14.3 = 911$ in·kips = 75.9 kip·ft. The total moment capacity of this splice with zero compression is thus 75.9 kip·ft and with 700 kips compression, it is 306 + 75.9 = 382 kip·ft. The role of compression in providing moment capability is often overlooked in column splice design.

Erection Stability of Columns. As discussed earlier for base plates, the stability of columns during erection must also be a consideration for splice design. The usual nominal erection load for columns is a 1-kip horizontal force at the column top in any direction. In LRFD format, the 1-kip working load is converted to a factored load by multiplying by a load factor of 1.5. This load of 1 × 1.5 = 1.5 kips will result in connections that will be similar to those obtained in allowable strength design (ASD) with a working load of 1 kip. It has been established that for major-axis bending, the splice is good for 75.9 kip·ft. This means that the 1.5-kip load can be applied at the top of a column 75.9/1.5 = 50.6 ft tall. Most columns will be shorter than 50.6 ft, but if not, a more robust splice should be considered.

Minor-axis stability must also be considered. If the 1.5-kip erection load is applied in the minor- or weak-axis direction, the forces at the splice will be as shown in Fig. 3.39. The upper shaft will tend to pivot about point *O*. Taking moments about point *O*, with *d* and *g* as shown,

$$PL = T\left(\frac{d}{2} + \frac{g}{2}\right) + T\left(\frac{d}{2} - \frac{g}{2}\right) = Td \tag{3.62}$$

Thus the erection load *P* that can be carried by the splice is

$$P = \frac{Td}{L} \tag{3.63}$$

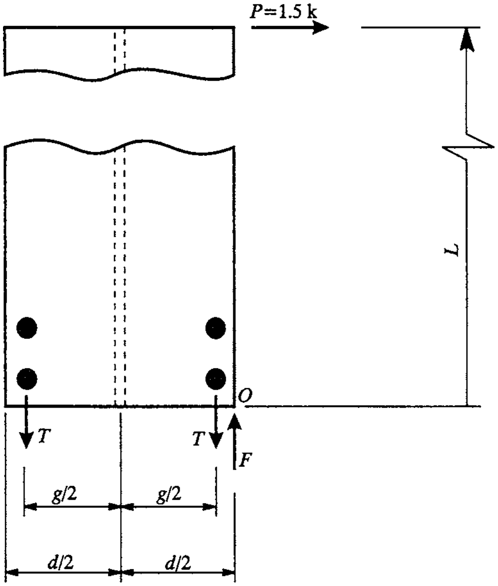


FIGURE 3.39 Weak-axis stability forces for column splice. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

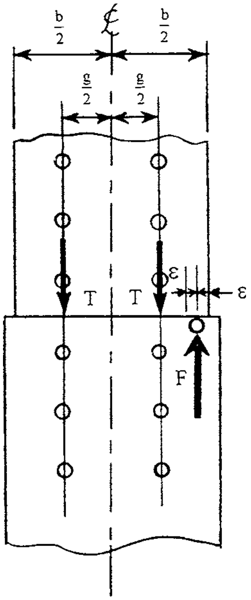


FIGURE 3.40 Force distribution for minor axis bending. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Note that this erection load capacity (design strength) is independent of the gage g. This is why the AISC splices carry the note, "Gages shown can be modified if necessary to accommodate fittings elsewhere on the column." The standard column gages are 5.5 and 7.5 in for beams framing to column flanges. Errors can be avoided by making all column gages the same. The gages used for the column splice can also be 5.5 or 7.5 in without affecting erection stability.

If the upper column of Fig. 3.38 is 40 ft long and T is the shear strength of four (two per splice plate) bolts, then

$$P = \frac{4 \times 15.9 \times 14.565}{40 \times 12} = 1.93 \text{ kips}$$

Since 1.93 kips > 1.5 kips, this splice is satisfactory for a 40-ft-long column. If it were not, larger or stronger bolts could be used.

Column Splices for Biaxial Bending. The simplest method for designing column splices for biaxial bending is to establish a flange force (required strength) that is statically equivalent to the applied moments and then to design the bolts, welds, plates, and fillers (if required) for this force.

If M_x is the major-axis applied moment and d is the depth of the deeper of the two columns, the flange force (required strength) is $F_{fx} = M_x/d$. For minor-axis bending, the force distribution is similar to that shown in Fig. 3.39 for erection stability. The force F in the case of factored design loads can be quite large and will need to be distributed over some finite bearing area as shown in Fig. 3.40. In Fig. 3.40, the bearing area is $2\varepsilon t$, where t is the thickness of the thinner flange, ε is the position of the force F from the toe of the flange of the smaller column, and T is the force per gage

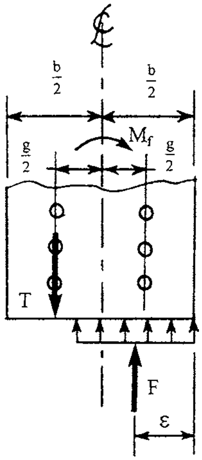


FIGURE 3.41 Splice force distribution when bolts on bearing side are ineffective. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

line of bolts. The quantities T and F are for each of the two flanges. If M_y is the weak-axis applied moment, $M_f = M_y/2$ is the weak-axis applied moment per flange. Taking moments about O gives a moment per flange of

$$M_f = T\left(\frac{b}{2} - \frac{g}{2} - \varepsilon\right) + T\left(\frac{b}{2} + \frac{g}{2} - \varepsilon\right) = T(b - 2\varepsilon) \tag{3.64}$$

The bearing area is determined by requiring that the bearing stress reach its design strength at the load F. Thus, $0.75(1.8F_y)(2\varepsilon)t = F$, and since from vertical equilibrium $F = 2T$,

$$T = 0.75(1.8F_y)t\varepsilon \tag{3.65}$$

Substituting in Eq. (3.64) to find $M_f = 0.75(1.8F_y)t\varepsilon(b - 2\varepsilon)$ and solving for ε gives

$$\varepsilon = \frac{1}{4}b - \frac{1}{2}\sqrt{\left(\frac{b}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_y}\right)} = \frac{1}{4}b\left(1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}}\right) \tag{3.66}$$

where $M_{py} = F_yZ_y = {}^1\!/\!_2F_ytb^2$. This expression for ε is valid while

$$M_f \le \frac{27}{40}\left(\frac{F_ytb^2}{4}\right) = \frac{3}{8}\phi M_{py} \tag{3.67}$$

When $M_f > {}^3\!/\!_8\,\phi M_{py}$, the tension T on the bolts on the bearing side vanishes and Fig. 3.41 applies. In that case, $F = T = 0.75(1.8F_y)t(2\varepsilon)$, and

$$\begin{gathered} M_f = T\left(\frac{b+g}{2} - \varepsilon\right) \\ \varepsilon = \frac{1}{4}(b+g) - \frac{1}{2}\sqrt{\left(\frac{b+g}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_yt}\right)} = \frac{1}{4}b\gamma\left[1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}\left(\frac{1}{\gamma}\right)^2}\,\right] \end{gathered} \tag{3.68}$$

where $\gamma = 1 + g/b$. This expression for ε is valid while

$$M_f \le \frac{27}{40}\frac{F_yt(b+g)^2}{4} = \frac{3}{8}\gamma^2\phi M_{py} \tag{3.69}$$

but T need never exceed M_f/g. The flange force in every case is $F_{fy} = 2T$.

Example for Design of Column Splice for Bending. Design a bolted splice for a W14 × 99 upper shaft to a W14 × 193 lower shaft. Design the splice for 15% of the axial capacity of the smaller member plus 20% of the smaller member's bending capacity about either the major or minor axis, whichever produces the greater flange force F_f. The columns are ASTM A572-50, the splice plates are ASTM A36, and the bolts are ASTM A490X, 1-in diameter. The holes are standard $1^1\!/\!_{16}$-in diameter and the gage is $7^1\!/\!_2$ in.

The completed splice is shown in Fig. 3.42. The flange force due to tension is

$$F_{f_t} = 0.15 \times \phi\frac{F_y}{2}A_g = \frac{0.15 \times 0.9 \times 50 \times 29.1}{2} = 98.2 \text{ kips}$$

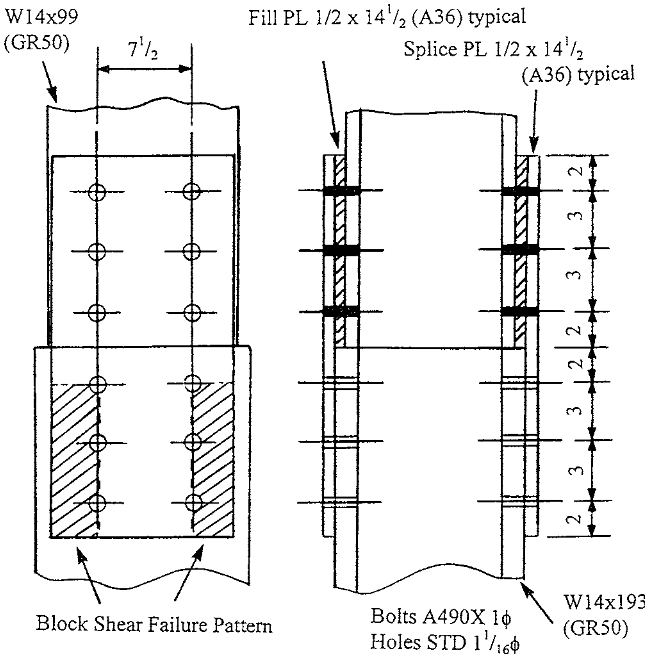


FIGURE 3.42 Bolted column splice for biaxial bending. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

The flange force due to major-axis bending is

$$F_{f_x} = \frac{0.20\phi M_{px}}{d} = \frac{0.20 \times 649 \times 12}{14.2} = 110 \text{ kips}$$

The flange force due to minor-axis bending is calculated as follows:

$$M_f = \frac{0.20\phi M_{py}}{2} = \frac{0.20 \times 0.9 \times 50 \times 836}{2} = 376 \text{ in}\cdot\text{kips}$$

Check that

$$M_f = 376 \le \frac{3}{8} \times 0.9 \times 50 \times 83.6 = 1410 \text{ in}\cdot\text{kips} \qquad \text{OK}$$

Calculate

$$\varepsilon = \frac{1}{4} \times 14.6\left(1 - \sqrt{1 - \frac{376}{1410}}\right) = 0.523 \text{ in}$$

$$T = 0.75(1.8 \times 50) \times 0.780 \times 0.523 = 27.5 \text{ kips} \qquad \text{and} \qquad F_{f_y} = 2 \times 27.5 = 55.0 \text{ kips}$$

The flange force for design of the splice is thus

$$F_f = F_{f_t} + \max(F_{f_x},\ F_{f_y}) = 98.2 + \max(110,\ 55.0) = 208 \text{ kips}$$

Suppose that $M_f > \frac{3}{8}\,\phi M_{py}$. Let $M_f = 1500$ in·kips, $\gamma = 1 + 7.5/14.6 = 1.51$, and check $M_f = 1500$ in·kips $< \frac{3}{8}\gamma^2\phi M_{py} = \frac{3}{8}(1.51)^2 \times 3760 = 3200$ in·kips. Proceeding,

$$\varepsilon = \frac{1}{4} \times 14.6 \times 1.51 \left(1 - \sqrt{1 - \frac{1500}{3200}}\right) = 1.49 \text{ in}$$

$$T = 0.75 \times (1.8 \times 50) \times 0.780 \times 2 \times 1.49 = 157 \text{ kips}$$

$$F_{fy} = 2T = 314 \text{ kips}$$

This is still less than the maximum possible value of $F_{fy} = (1500/7.5) \times 2 = 400$ kips.

Returning to the splice design example, the splice will be designed for a factored load of 208 kips. Since the columns are of different depths, fill plates will be needed. The theoretical fill thickness is $(15\frac{1}{2} - 14\frac{1}{8})/2 = \frac{11}{16}$ in, but for ease of erection AISC suggests subtracting either $\frac{1}{8}$ in or $\frac{3}{16}$ in, whichever results in $\frac{1}{8}$-in multiples of fill thickness. Thus, use actual fills $\frac{11}{16} - \frac{3}{16} = \frac{1}{2}$ in thick. Since this splice is a bearing splice, the fills either must be developed, or the shear strength of the bolts must be reduced. It is usually more economical to do the latter in accordance with AISC Specification Sec. J6 when the total filler thickness is not more than $\frac{3}{4}$ in. With the reduced bolt shear design strength, $\phi r_v = 44.2[1 - 0.4(0.5 - 0.25)] = 39.8$ kips. The number of bolts required is $208/39.8 = 5.23$ or 6 bolts. By contrast, if the fillers were developed, the number of bolts required would be $208[1 + 0.5/(0.5 + 0.780)]/44.2 = 6.54$ or 8 bolts. By reducing the bolt shear strength instead of developing the fills, $[(8 - 6)/8]100 = 25\%$ fewer bolts are required for this splice.

Next, the splice plates are designed. These plates will be approximately as wide as the narrower column flange. Since the W14 × 99 has a flange width of $14\frac{5}{8}$ in, use a plate $14\frac{1}{2}$ in wide. The following limit states are checked.

Gross Area. The required plate thickness based on gross area is $t_p = 208/(0.9 \times 36 \times 14.5) = 0.44$ in. Try a $\frac{1}{2}$ in plate.

Net Area. The net area is $A_n = (14.5 - 2 \times 1.125) \times 0.5 = 6.125 \text{ in}^2$, but this cannot exceed 0.85 of the gross area or $0.85 \times 14.5 \times 0.5 = 6.16 \text{ in}^2$. Since $6.16 \text{ in}^2 > 6.125 \text{ in}^2$, the effective net area is $A_e = A_n = 6.125 \text{ in}^2$. The design strength in net tension is

$$\phi R_n = 0.75 \times 58 \times 6.125 = 266 \text{ kips} > 208 \text{ kips} \qquad \text{OK}$$

Block Shear Rupture. Since $b - g < g$, failure will occur as shown in Fig. 3.42 on the outer parts of the splice plate.

$$A_{gv} = 8 \times 0.5 \times 2 = 8.0 \text{ in}^2$$

$$A_{\text{gt}} = (14.5 - 7.5) \times 0.5 = 3.5 \text{ in}^2$$

$$A_{nv} = 8.0 - 2.5 \times 1.125 \times 0.5 \times 2 = 5.1875 \text{ in}^2$$

$$A_{\text{nt}} = 3.5 - 1 \times 1.125 \times 0.5 = 2.9375 \text{ in}^2$$

$$F_u A_{\text{nt}} = 58 \times 2.9375 = 170 \text{ kips}$$

$$0.6 F_y A_{gv} = 0.6 \times 36 \times 8.0 = 173 \text{ kips}$$

$$0.6 F_u A_{nv} = 0.6 \times 58 \times 5.1875 = 181 \text{ kips}$$

$$U_{\text{bs}} = 1.0 \qquad \text{(uniform tension)}$$

$$\phi R_{\text{bs}} = 0.75[1.0 \times 170 + \min(173,\ 181)] = 257 \text{ kips} > 208 \text{ kips} \qquad \text{OK}$$

Bearing/Tear-out. Although we initially determined that six bolts are required, the following bearing/tear-out check may require an adjustment in this number.

Bolt shear: $\phi R_v = 39.8$ kips (reduced value calculated above)

Bearing on splice plate: $\phi R_p = 0.75 \times 2.4 \times 1.0 \times 0.5 \times 58 = 52.2$ kips

Bearing on W14 × 99 flange: $\phi R_p = 0.75 \times 2.4 \times 1.0 \times 0.780 \times 65 = 91.3$ kips

Tear-out on splice plate: $\phi R_{\text{to}} = 0.75 \times 1.2 \times (2 - 0.5 \times 1.0625) \times 0.5 \times 58 = 38.3$ kips

Tear-out on W14 × 99 flange: $\phi R_{\text{to}} = 0.75 \times 1.2(2 - 0.5 \times 1.0625)0.780 \times 65 = 67.1$ kips

(Two more tear-out limit states are related to the spacing of the bolts, but these are obviously not critical.) Since the bolt shear value of 39.8 kips is greater than the splice plate tear-out, the shear bearing/tear-out limit state for the six bolts is

$$\phi R_{vpt} = 4 \times 39.8 + 2 \times 38.3 = 236 \text{ kips} > 208 \text{ kips} \qquad \text{OK}$$

Whitmore Section. Calculate distance l_w and design strength for limit state of yielding as follows:

$$l_w = (6 \tan 30)2 + 7.5 = 14.43 \text{ in.}$$

$$\phi R_n = 0.9 \times 36 \times 14.43 \times 0.5 = 234 \text{ kips} > 208 \text{ kips}$$

Note that if $l_w > 14.5$ in, 14.5 in would be used in the calculation of design strength.

In addition to the checks for the bolts and splice plates, the column sections should also be checked for bearing and block shear rupture. These are not necessary in this case because $t_f = 0.780 > t_p = 0.50$, the edge distances for the column are the same as for the plates, and the column material is stronger than the plate material.

Splices in Truss Chords. Splices in truss chords must be designed for 50% of the chord load as an axial force, or 2% of the chord load as a transverse force, as discussed in Art. 3.5.3, even if the load is compression and the members are finished to bear. As discussed earlier, these splices may be positioned in the center of a truss panel, and therefore must provide some degree of continuity to resist bending. For the tension chord, the splice must be designed to carry the full tensile load.

Example of Design of Splice in Truss Chord. Design the tension chord splice shown in Fig. 3.43 for a factored load of 800 kips. The bolts are A325X, $^7/_8$ in in diameter, $\phi r_v = 27.1$ kips. The load at this location is controlled by the smaller W14 × 90 shape, so the loads should be apportioned to flanges and the web based on this member. Thus, the flange load P_f and the web load P_w are

$$P_f = \frac{0.710 \times 14.520}{26.5} \times 800 = 311 \text{ kips}$$

$$p_w = 800 - 2 \times 311 = 178 \text{ kips}$$

The load path is such that the flange load P_f passes from the W14 × 90 through the bolts into the flange plates and into the W14 × 120 flanges through a second set of bolts. The web load path is similar.

FLANGE SPLICE DESIGN

Member Limit States. **Bolt pattern,** although not a member limit state, must be established to check the chords. The minimum number of bolts required in double shear is $\frac{311}{2} \times 27.1 = 5.74$. Try six bolts in two rows of three as shown in Fig. 3.43. This may need to be adjusted because of bearing/tear-out.

Chord net section is checked to see if the holes in the W14 × 90 reduce its capacity below 800 kips. Assume that there will be two web holes in alignment with the flange holes.

$$A_{\text{net}} = 26.5 - 4 \times 1 \times 0.710 - 2 \times 1 \times 0.440 = 22.8 \text{ in}^2$$

$$\phi R_{\text{net}} = 22.8 \times 0.75 \times 65 = 1111 \text{ kips} > 800 \text{ kips} \qquad \text{OK}$$

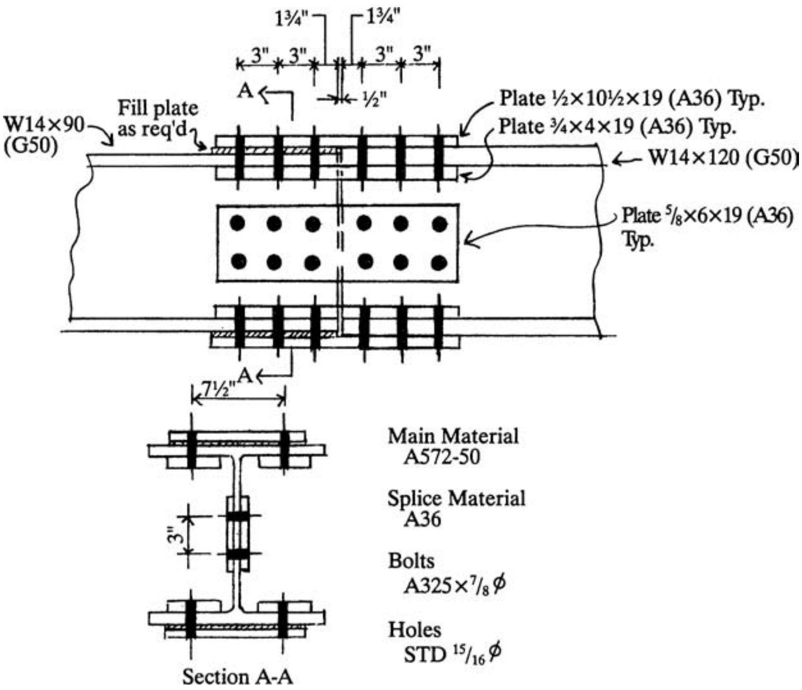


FIGURE 3.43 Truss chord tension splice. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

Bearing/tear-out will be checked after the splice plates are designed.
Block shear rupture is checked as follows:

$$A_{nv} = (7.75 - 2.5 \times 1.0)0.710 \times 2 = 7.46 \text{ in}^2$$

$$A_{\text{nt}} = \left[\frac{(14.5 - 7.5)}{2} - 0.5 \times 1\right]0.710 \times 2 = 4.27 \text{ in}^2$$

$$A_{gv} = 7.75 \times 0.710 \times 2 = 11.0 \text{ in}^2$$

$$A_{\text{gt}} = 3.50 \times 0.710 \times 2 = 4.98 \text{ in}^2$$

$$F_u A_{\text{nt}} = 65 \times 4.27 = 278 \text{ kips}$$

$$0.6F_y A_{gv} = 0.6 \times 50 \times 11.00 = 330 \text{ kips}$$

$$0.6F_u A_{nv} = 0.6 \times 65 \times 7.46 = 291 \text{ kips}$$

$$U_{\text{bs}} = 1.0$$

$$\phi R_{\text{bs}} = 0.75[278 + \min(330, 291)] = 427 \text{ kips} > 311 \text{ kips} \qquad \text{OK}$$

Flange Connection. Since the bolts are assumed to be in double shear, the load path is such that one-half of the flange load goes into the outer plate, and one-half goes into the inner plates.

Outer Plate: Gross and Net Area. Since the bolt gage is 7.5 in, try a plate 10.5 in wide. The gross area in tension required is

$$A_{\text{gt}} = \frac{311/2}{0.90 \times 36} = 4.80 \text{ in}^2$$

and the thickness required is 4.80/10.5 = 0.46 in. Try a plate 0.50 × 10.5 in

$$A_{gt} = 0.5 \times 10.5 = 5.25 \text{ in}^2$$
$$A_{nt} = (10.5 - 2 \times 1) \times 0.5 = 4.25 \text{ in}^2$$
$$0.85A_{gt} = 0.85 \times 5.25 = 4.46 \text{ in}^2$$

Since $0.85A_{gt} > A_{nt}$, use $A_{nt} = 4.25 \text{ in}^2$ as the effective net tension area:

$$\phi R_{nt} = 0.75 \times 58 \times 4.25 = 185 \text{ kips} > 311/2 = 156 \text{ kips} \qquad \text{OK}$$

Use a plate 0.50 × 10.5 in for the outer flange splice plate for the following limit state checks.

Outer Plate: Block Shear Rupture

$$A_{gv} = 7.5 \times 0.5 \times 2 = 7.5 \text{ in}^2$$
$$A_{gt} = 1.5 \times 0.5 \times 2 = 1.5 \text{ in}^2$$
$$A_{nv} = (7.5 - 2.5 \times 1)0.5 \times 2 = 5.0 \text{ in}^2$$
$$A_{nt} = (1.5 - 0.5 \times 1)0.5 \times 2 = 1.0 \text{ in}^2$$
$$F_uA_{nt} = 58 \times 1.0 = 58.0 \text{ kips}$$
$$0.6F_yA_{gy} = 0.6 \times 36 \times 7.5 = 162 \text{ kips}$$
$$0.6F_uA_{nv} = 0.6 \times 58 \times 5.0 = 174 \text{ kips}$$
$$\phi R_{bs} = 0.75[58 + \min(162, 174)] = 165 \text{ kips} > 156 \text{ kips} \qquad \text{OK}$$

Outer Plate: Bearing

$$\phi R_p = 0.75 \times 2.4 \times 58 \times 0.50 \times 0.875 \times 6 = 274 \text{ kips} > 156 \text{ kips} \qquad \text{OK}$$

Note that bearing/tear-out still needs to be checked.

Inner Plate: Gross and Net Area. The load to each plate is 156/2 = 78 kips. The gross area in tension required is

$$A_{gt} = \frac{78}{0.9 \times 36} = 2.41 \text{ in}^2$$

If the plate is 4 in wide, the required thickness will be 2.41/4 = 0.60 in. Try a plate 0.75 × 4 in (A36):

$$A_{gt} = 0.75 \times 4 = 3 \text{ in}^2$$
$$A_{nt} = (4 - 1.0)0.75 = 2.25 \text{ in}^2$$
$$0.85A_{gt} = 0.85 \times 3 = 2.55 \text{ in}^2$$
$$\phi R_{nt} = 0.75 \times 58 \times 2.25 = 97.9 \text{ kips} > 78 \text{ kips} \qquad \text{OK}$$

Inner Plate: Block Shear Rupture. Since there is only one line of bolts, this limit state is not possible. The plate limit state would be tension rupture on the net section instead. Note that bearing/tear-out still needs to be checked.

Bearing/Tear-out of Flange Plates and W14 × 90. Now that the bolts, the outer plate, and the inner plates have been tentatively selected, bearing/tear-out can be checked for the connection as a whole:

Bolt shear: $\phi r_v = 54.2$ kips (double shear): $\phi r_v = 27.1$ kips (single shear)

Bearing on W14 × 99 flange: $\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.710 \times 65 = 72.7$ kips

Bearing on outer plate: $\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.50 \times 58 = 45.7$ kips
Bearing on inner plate: $\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.75 \times 58 = 68.5$ kips
Tear-out on W14 × 99 flange: $L_c = 1.75 - 0.50 \times 0.9375 = 1.281$ in

$$\phi r_{\text{to}} = 0.75 \times 1.2 \times 1.281 \times 0.710 \times 65 = 53.2 \text{ kips}$$

Tear-out on outer plate: $L_c = 1.5 - 0.5 \times 0.9375 = 1.031$ in

$$\phi r_{\text{to}} = 0.75 \times 1.2 \times 1.031 \times 0.50 \times 58 = 26.9 \text{ kips}$$

Tear-out on inner plate: $L_c = 1.031$ in

$$\phi r_{\text{to}} = 0.75 \times 1.2 \times 1.031 \times 0.75 \times 58 = 40.4 \text{ kips}$$

Tear-out between bolts will not control in this case, since $3 - 0.9375 = 2.0625 > 1.281$ or 1.031 in. From the above, the shear/bearing/tear-out strength of the flange connection is

$$\phi R_{vpt} = 2 \times 54.2 + 2 \times 26.9 + 2 \times 27.1 + 2 \times 54.2 = 325 \text{ kips} > 311 \text{ kips} \qquad \text{OK}$$

In the expression for ϕR_{vpt}, the first term is for the two bolts in the center, which are controlled by shear; the second and third terms are for the outer two bolts controlled by outer-plate edge distance and bolt shear; and the fourth term is for the two inner bolts, again controlled by bolt shear.

This completes the calculation for the flange portion of the splice. The bolts, outer plate, and inner plates, as chosen above, are OK.

WEB SPLICE DESIGN

Member Limit States. The calculations for the web connection involve the same limit states as the flange connection, except for tension rupture of the chord net section, which involves flanges and web.

Bolt pattern must be established to check the web. The minimum number of bolts in double shear required is (178/2)/27.1 = 3.28. Try four bolts.

Bearing/tear-out will be checked after the web splice plates are designed.

Block shear rupture will be checked as follows. As a first trial, assume the bolts have a 3-in pitch longitudinally.

$$A_{nv} = (4.75 - 1.5 \times 1) \times 0.440 \times 2 = 2.86 \text{ in}^2$$
$$A_{\text{nt}} = (3 - 1 \times 1) \times 0.440 = 0.88 \text{ in}^2$$
$$A_{gv} = 4.75 \times 0.440 \times 2 = 4.18 \text{ in}^2$$
$$A_{\text{gt}} = 3 \times 0.440 = 1.32 \text{ in}^2$$
$$F_u A_{\text{nt}} = 65 \times 0.88 = 57.2 \text{ kips}$$
$$0.6F_u A_{nv} = 0.60 \times 65 \times 2.86 = 112 \text{ kips}$$
$$0.6F_y A_{gy} = 0.60 \times 50 \times 4.18 = 125 \text{ kips}$$
$$U_{\text{bs}} = 1.0$$
$$\phi R_{\text{bs}} = 0.75[57.2 + \min(115, 125)] = 127 \text{ kips} < 178 \text{ kips} \qquad \text{No good}$$

Since the block shear limit state fails, the bolts can be spaced out to increase the capacity. Increase the bolt pitch, from the 3 in assumed above to 6 in, and repeat the calculations.

$$A_{nv} = (7.75 - 1.5 \times 1) \times 0.440 \times 2 = 5.50 \text{ in}^2$$
$$A_{\text{nt}} = 0.88 \text{ in}^2$$

$$
\begin{aligned}
A_{gv} &= 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2 \\
A_{\text{gt}} &= 1.32 \text{ in}^2 \\
U_{\text{bs}} &= 1.0 \\
F_u A_{\text{nt}} &= 65 \times 0.88 = 57.2 \text{ kips} \\
0.6 F_u A_{nv} &= 0.60 \times 65 \times 5.50 = 214 \text{ kips} \\
0.6 F_y A_{gv} &= 0.60 \times 50 \times 6.82 = 205 \text{ kips} \\
\phi R_{\text{bs}} &= 0.75[57.2 + \min(214, 205)] = 197 \text{ kips} > 178 \text{ kips} \qquad \text{OK}
\end{aligned}
$$

Figure 3.43 shows the web bolt pattern with four bolts in the web at 6-in pitch. Other checks must be made as follows.

Web Plates. Try two A36 plates, one on each side of web, 0.50 × 6 in.

Gross area:

$$\phi R_{\text{gt}} = 0.9 \times 36 \times 0.5 \times 6 \times 2 = 194 \text{ kips} > 178 \text{ kips} \qquad \text{OK}$$

Net area:

$$
\begin{aligned}
A_{\text{nt}} &= (6 - 2 \times 1) \times 0.50 \times 2 = 4.0 \text{ in}^2 \\
0.85 A_{\text{gt}} &= 0.85 \times 0.50 \times 6 \times 2 = 5.1 \text{ in}^2 \\
\phi R_{\text{nt}} &= 0.75 \times 58 \times 4.0 = 174 \text{ kips} < 178 \text{ kips} \qquad \text{No good}
\end{aligned}
$$

Increase web plates to $^5/_8$ in thick. Net area will be OK by inspection. Block shear rupture: Check as shown in previous calculations; not critical here.

Bearing/Tear-out of Web Plates and W14 × 90. The bolt pattern and plates have been tentatively selected, so this combined limit state can be checked.

Bolt shear: $\phi r_v = 54.2$ kips double shear; $\phi r_v = 27.1$ kips single shear

Bearing on W14 × 99 web: $\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.440 \times 65 = 45.0$ kips

Bearing on splice plates: $\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.625 \times 2 \times 58 = 114$ kips

Tear-out on W14 × 99 web: $L_c = 1.75 - 0.5 \times 0.9375 = 1.281$ in

$$\phi r_{\text{to}} = 0.75 \times 1.2 \times 1.281 \times 0.440 \times 65 = 33.0 \text{ kips}$$

Tear-out on splice plates: $\phi r_{\text{to}} = 0.75 \times 1.2 \times 1.281 \times 0.625 \times 2 \times 58 = 83.6$ kips

Tear-out between bolts: Will not control in this case.

From the above, the shear/bearing/tear-out strength of the web splice is

$$\phi R_{vpt} = 2 \times 33.0 + 2 \times 45.0 = 156 \text{ kips} < 178 \text{ kips} \qquad \text{No good}$$

Add two bolts in the web so that the 6-in pitch becomes 3-in pitch. The shear/bearing/tear-out capacities per bolt given above do not change. Tear-out between bolts is still not critical. Thus

$$\phi R_{vpt} = 2 \times 33.0 + 4 \times 45.0 = 246 \text{ kips} > 178 \text{ kips} \qquad \text{OK}$$

Note that, in this case, none of the bolts was able to achieve its double shear value.

Additional Checks because of Change in Web Bolt Pattern

Block Shear Rupture

$$A_{nv} = (7.75 - 2.5 \times 1.0)\ (0.440 \times 2) = 4.62 \text{ in}^2$$

$$A_{gv} = 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2$$

$$A_{\text{nt}} = (3 - 1 \times 1.0) \times 0.440 = 0.88 \text{ in}^2$$

$$F_u A_{\text{nt}} = 65 \times 0.88 = 57.2 \text{ kips}$$

$$0.6F_u A_{nv} = 0.6 \times 65 \times 4.62 = 180 \text{ kips}$$

$$0.6F_y A_{gv} = 0.6 \times 50 \times 6.82 = 205 \text{ kips}$$

$$U_{\text{bs}} = 1.0$$

$$\phi R_{\text{bs}} = 0.75[57.2 + \min(180, 205)] = 178 \text{ kips} \le 178 \text{ kips} \qquad \text{OK}$$

Buckling. If this were a nonbearing compression splice, the splice plates would be checked for buckling. The following shows the method, although it is obviously not required for a tension splice.

The plates at the flange splice line are unsupported for a 4.0-in length between bolts. Check for a load of 311/2 = 156 kips/plate. The slenderness ratio is

$$\frac{Kl}{r} = 0.65 \times 4.0 \times \sqrt{12} / 0.5 = 18.0$$

Since this value is less than 25, AISC Specification Sec. J4.4 allows the plate to be checked for yielding rather than buckling. This limit state has been checked in the preceding calculations, and for the thicker web plates as well.

3.6 MOMENT CONNECTIONS

The most commonly used moment connection is the field-welded moment connection as shown in Fig. 3.44. This connection is in common use in all regions of the United States, where the Seismic Design Category (SDC) is A, B, or C, and the response modification factor R is 3 or less (AISC, "Seismic Provisions for Structural Steel Buildings").

3.6.1 Example of Three-Way Moment Connection

The moment connection of Fig. 3.44*a* is termed a **three-way moment connection** because, in addition to the strong-axis bending, the column is subjected to minor-axis bending from both sides. Additional views are shown in Figs. 3.44*b* and *c*. If the strong-axis connection requires stiffeners opposite the beam flanges, there will be an interaction between the flange forces of the strong- and weak-axis beams. If the primary function of these moment connections is to resist lateral maximum load from wind or seismic sources, the interaction can generally be ignored because the maximum lateral loads will act in only one direction at any one time. If the moment connections are used primarily to carry gravity loads, such as would be the case when stiff floors with small deflections and high natural frequencies are desired, there will be interaction between the weak- and strong-beam flange forces. The calculations here assume gravity moments in both directions, although much of the procedure is also applicable to wind or low-to-moderate seismic conditions.

The load path through this connection that is usually assumed is that the moment is carried entirely by the flanges, and the shear entirely by the web. This load path has been verified by testing (Huang et al., 1973) and will be the approach used here. Proceeding to the connection design, the strong-axis beam (beam no. 1) will be designed first.

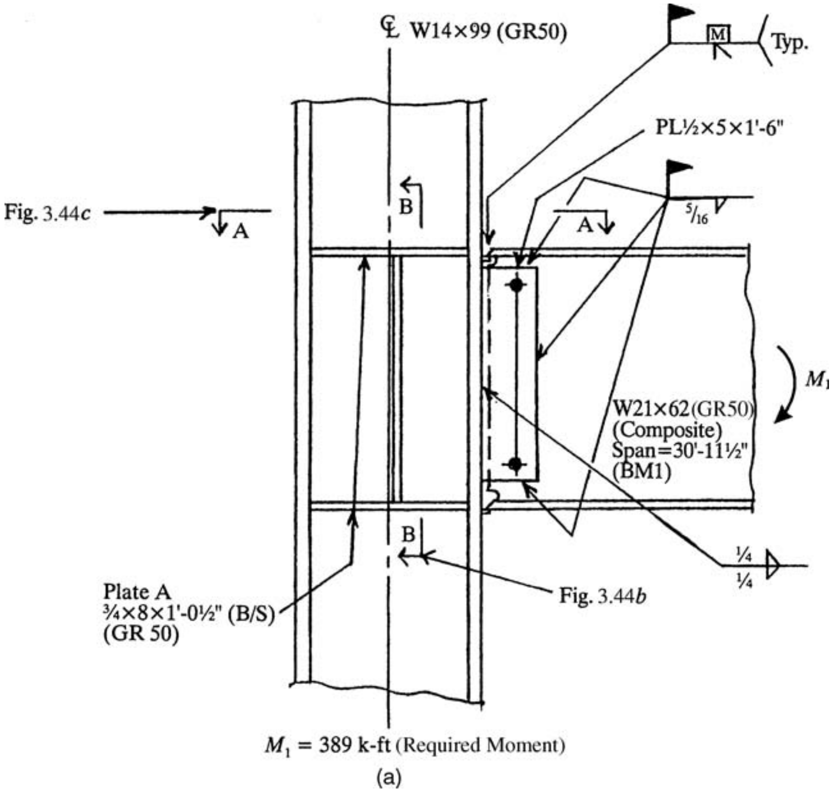


FIGURE 3.44 (*a*) Field-welded moment connection. (*b*) Section B–B of (*a*). (*c*) Section A–A of (*a*). (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Beam No. 1: W21 × 62 (A36) Composite. The flange connection is a complete joint penetration (CJP) weld that develops the strength of the flange, so no weld design is required. The column must be checked for stiffeners and doublers.

Stiffener Requirements. The flange force F_f is

$$F_f = \frac{\phi M_p}{d - t_f} = \frac{389 \times 12}{21.0 - 0.615} = 229 \text{ kips}$$

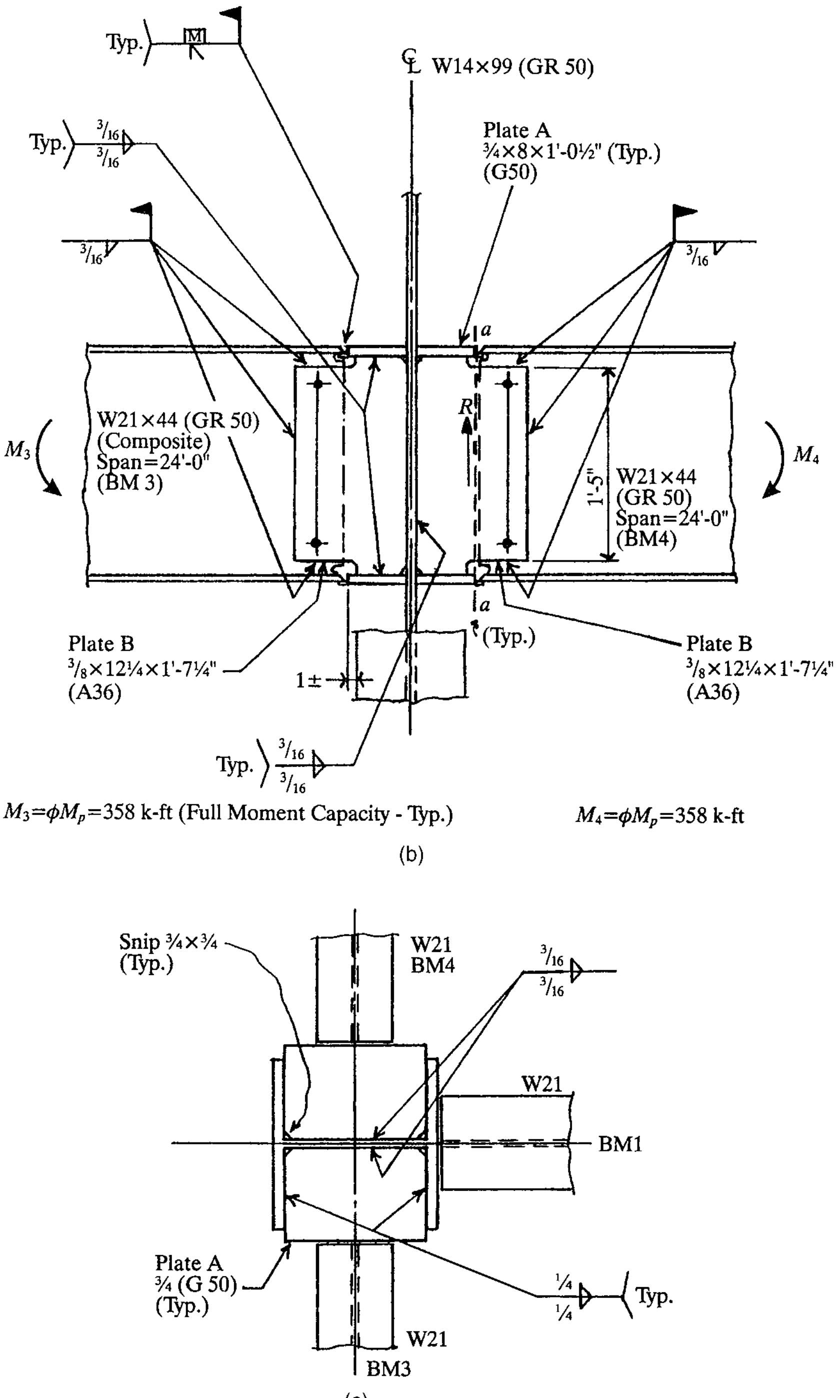
Using the column load tables of the AISC Manual, consider the following limit states:

Web yielding: $P_{\mathrm{wy}} = P_{wo} + t_b P_{wi} = 167 + 0.615 \times 24.3 = 182$ kips $<$ 229 kips, thus stiffeners are required at both flanges

Web buckling: $P_{\mathrm{wb}} = 260$ kips $>$ 229 kips—no stiffener required at compression flange

Flange bending: $P_{\mathrm{fb}} = 171$ kips $<$ 229 kips—stiffener required at tension flange

From the preceding three checks (limit states), a stiffener is required at both flanges. For the tension flange, the total stiffener force is 229 – 171 = 58 kips, and for the compression flange, the stiffener force



(b)

(c)

FIGURE 3.44 (*Continued*)

is 229 –182 = 47 kips. But the loads may reverse, so use the larger of 58 and 47 as the stiffener force for both flanges. Then, the force in *each* stiffener is 58/2 = 29 kips, both top and bottom.

Stiffener Size Determination. The minimum stiffener width w_s is

$$\frac{b_{\text{fb}}}{3}-\frac{t_{wc}}{2}=\frac{8.24}{3}-\frac{0.485}{2}=2.5 \text{ in}$$

Use a stiffener 6.5 in wide to match the column.

The minimum stiffener thickness t_s is

$$\frac{t_{\text{fb}}}{2}=\frac{0.615}{2}=0.31 \text{ in}$$

Use a stiffener at least $^3/_8$ in thick.

The minimum stiffener length is

$$l_s=\frac{d_c}{2}-t_{fc}=\frac{14.2}{2}-0.78=6.3 \text{ in}$$

This is the minimum length for a half-depth stiffener, which is not possible here because of the weak-axis connections. Therefore, use a full-depth stiffener, 12.5 in long.

A final stiffener size check is a plate buckling check, which requires that

$$t_s \geq \frac{W_s}{15}=\frac{6.5}{15}=0.433 \text{ in}$$

Therefore, assume a minimum stiffener thickness of $^1/_2$ in. The final stiffener size for the strong-axis beam is $^1/_2 \times 6.5 \times 12.5$ in. The contact area of this stiffener against the inside of the column flange is 6.5 − 1.25 = 5.25 in due to the snip to clear the column web-to-flange fillet. The stiffener design strength is thus

$$0.90 \times 36 \times 5.25 \times 0.5 = 85.1 \text{ kips} > 29 \text{ kips} \qquad \text{OK}$$

Welds of Stiffeners to Column Flange and Web. Putting aside for the moment that the weak-axis moment connections still need to be considered and will affect both the strong-axis connection stiffeners and welds, the welds for the $^1/_2 \times 6.5 \times 12.5$ in strong-axis stiffener are designed as follows. For the weld to the inside of the flange, the force to be developed by the weld to the connected portion is 29 kips. Thus, the 5.25-in width in contact, which is the connected portion, is designed for 29 kips. The fillet weld design strength is $\phi F_w A_w = 0.75 \times 0.60 \times 70 \times (^1/_{16}) = 1.392$ kips per $^1/_{16}$th of weld size. Thus, for the weld to the flange,

$$D_f=\frac{29}{2\times 5.25\times 1.392\times 1.5}=1.32$$

where D_f = number of $^1/_{16}$ths in the fillet weld size. In this case, an AISC minimum fillet weld is indicated. The factor 1.5 in the denominator above comes from the AISC Specification, Sec. J2.4, for transversely loaded fillets. The weld to the web has a length 12.5 − 1.25 − 1.25 = 10.0 in, and is designed to transfer the unbalanced force in the stiffener to the web. The unbalanced force in the stiffener is 29 kips in this case. Since the weld at the web and the weld at the flange do not share load in this case, both the longitudinally and transversely loaded welds can develop their full strength. Thus,

$$D_w=\frac{29}{2\times 10.0\times 1.392}=1.04$$

where D_w = number of $^1/_{16}$ths in the fillet weld size. An AISC minimum fillet is indicated.

Doublers. The beam flange force (required strength) delivered to the column is $F_f = 229$ kips. The design shear strength of the column is

$$\phi V_v = \phi \times 0.60 \times F_y d_c t_w = 0.90 \times 0.60 \times 50 \times 14.2 \times 0.485 = 186 \text{ kips} < 229 \text{ kips}$$

so a doubler appears to be required. However, if the moment that is causing doublers is $M_1 =$ 389 ft·kip, then from Fig. 3.45, the column story shear is $V_s = \phi M_p/H$, where H is the story height. If $H = 13$ ft, then $V_s = 389/13 = 30$ kips and the shear delivered to the column web is $F_f - V_s =$ $229 - 30 = 199$ kips. Since 199 kips > 185 kips, a doubler (or doublers) is still indicated. However, if some panel zone deformation is acceptable, AISC Specification Sec. J10.6, Eqs. J10.11 and J10.12, contain an extra term that increases the panel zone strength. The term is calculated for this example as

$$\frac{3b_{f_c}t_{f_c}^{\ 2}}{d_b d_c t_{wc}} = \frac{3 \times 14.6 \times 0.780^2}{21.0 \times 14.2 \times 0.485} = 0.184$$

For the usual case where the column load (required strength) is less than $0.75P_y$, as it is here, $(0.75P_y = 0.75 \times A_c F_{yc} = 0.75 \times 29.1 \times 50 = 1091 \text{ kips})$, ϕV_v is multiplied by 1 plus the above factor. Hence, $\phi V_v = 186 \times 1.184 = 220$ kips. Since 220 kips > 199 kips, no doubler is required. In a high-rise building where the moment connections are used for drift control, the extra term can still be used, but an analysis that includes inelastic joint shear deformation should be considered.

Placement of Doubler Plates. If a doubler plate (or plates) is required in this example, the most inexpensive arrangement is to place the doubler plate against the column web between the stiffeners (the panel zone) and to attach the weak-axis shear connection plates, plates B (Fig. 3.44*b*), to the face of the doubler. This is permissible provided that the doubler is capable of carrying the entire weak-axis shear load $R = 163$ kips on one vertical cross section of the doubler plate. To see this, consider Fig. 3.46. The portion of the shear force induced in the doubler plate by the moment-connection flange force F_f is H. For the doubler to be in equilibrium under the forces H, vertical shear forces $V = Hd/w$ must exist. The welds of the doubler at its four edges develop the shear strength of the doubler. Let the shear force R from the weak axis connection be applied to the face of the doubler at or near its horizontal center as shown in Fig. 3.46. If it is required that all of the shear R can be carried by one vertical section *a–a* of Fig. 3.46, that is, $0.90 \times 0.60 \times F_y t_d d \geq R$, where t_d is the doubler thickness and F_y is the yield stress of the doubler (and the column), then the free-body diagram of Fig. 3.46 is possible. In this figure, all of the shear force R is delivered to the side of the doubler where it is opposite in direction to the shear delivered by the moment connection, thereby avoiding overstressing the other side where the two shears would add. Since the doubler and its welds are capable

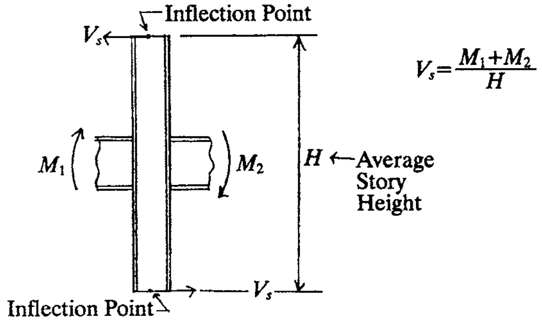


FIGURE 3.45 Relationship between column story shear and moments that induce it. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

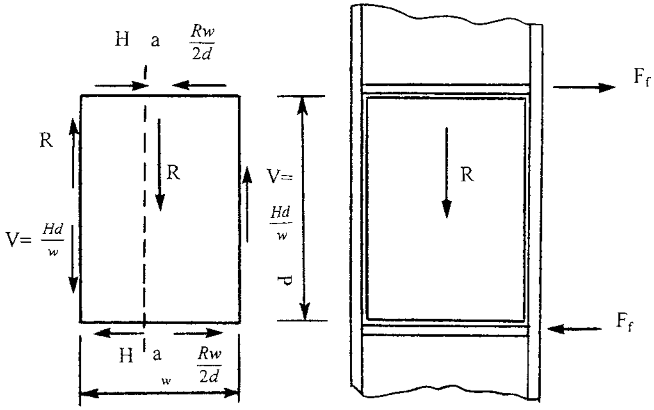


FIGURE 3.46 Equilibrium of doubler plate with weak-axis shear load. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999*, *with permission*.)

of carrying V or R alone, they are capable of carrying their difference. The same argument applies to the top and bottom edges of the doubler. Also, the same argument holds if the moment and/or weak-axis shear reverse(s).

Associated Shear Connections: Beam 1. The specified shear for the web connection is R = 163 kips, which is the shear capacity of the W21 × 62 (A36) beam. The connection is a shear plate with two erection holes for erection bolts. The shear plate is shop welded to the column flange and field welded to the beam web. The limit states are shear yielding on the plate gross section, weld strength, and beam web strength.

Plate Gross Shear. Try a plate $\frac{1}{2} \times 5 \times 18$ in.

$$\phi R_{gv} = 0.5 \times 18 \times 0.9 \times 0.6 \times 36 = 175 \text{ kips} > 163 \text{ kips} \qquad \text{OK}$$

Plate net shear need not be checked here, because it is not a valid limit state.

Weld to Column Flange. This weld sees shear only. Thus, weld size in $\frac{1}{16}$ths is

$$D = \frac{163}{2 \times 18 \times 1.392} = 3.25$$

Use a $\frac{1}{4}$-in fillet weld.

Weld to Beam Web. This weld sees the shear plus a small couple. Using AISC Manual Table 8-9, $l = 18$, $kl = 4.25$, $k = 0.24$, $x = 0.04$, $xl = 0.72$, $al = 4.28$, $a = 0.24$, $c = 2.70$, and

$$D = \frac{163}{0.75 \times 2.70} = 4.47$$

Use a $\frac{5}{16}$-in fillet weld.

Beam Web. To support a $\frac{5}{16}$-in fillet weld on both sides of a plate, AISC Manual Table 10.2 shows that a 0.476-in web is required. For a $\frac{5}{16}$-in fillet on one side, a 0.238-in web is required. Since the W21 × 62 web is 0.400 in thick, it is OK.

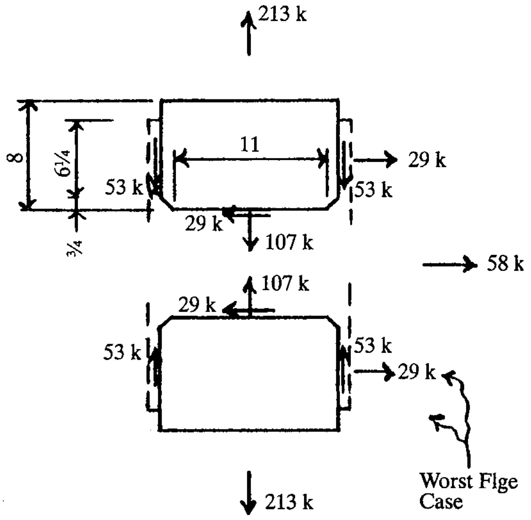


FIGURE 3.47 Distribution of forces on plates A. (*Source: A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

Beams Nos. 3 and 4: W21 × 44 (Grade 50) Composite. The flange connection is a full-penetration weld, so no weld design is required. Section *A–A* of Fig. 3.44*a*, depicted in Fig. 3.44*c*, shows the arrangement in plan. The connection plates A are made ¼ in thicker than the W21 × 44 beam flange to accommodate under- and overrolling and other minor misfits. Also, the plates are extended beyond the toes of the column flanges by ¾ to 1 in to improve ductility. The plates A should also be welded to the column web, even if not required to carry load, to provide improved ductility. A good discussion of this is provided in the AISC Manual.

The flange force for the W21 × 44 is based on the full moment capacity as required in this example, so $\phi M_p = 358$ ft·kip and $F_f = (358 \times 12)/(20.7 - 0.45) = 213$ kips. Figure 3.47 shows the distribution of forces on the plates A, including the forces from the strong-axis connection. The weak-axis force of 213 kips is distributed one-fourth to each flange and one-half to the web. This is done to cover the situation where the beams may not be reacting against each other. In this case, all of the 213 kips must be passed to the flanges. To see this, imagine that beam 4 is removed and the plate A for beam 4 remains as a back-up stiffener. One-half of the 213 kips from beam 3 passes into the beam 3 near-side column flanges, while the other half is passed through the column web to the back-up stiffener, and thence into the far-side flanges, so that all of the load is passed to the flanges. This is the load path usually assumed, although others are possible.

Merging of Stiffeners from Strong- and Weak-Axis Beams. The strong-axis beam, beam no. 1, requires stiffeners ½ × 6.5 × 12.5 in. The weak-axis beams no. 3 and no. 4 require plates A, ¾ × 8 × 12.5 in. These plates occupy the same space because the beams are all of the same depth. Therefore, the larger of the two plates is used, as shown in Fig. 3.44*a*. Since the stiffeners are merged, the welds that were earlier determined for the strong-axis beam must be revisited.

Weld to Web. From Fig. 3.47, orthogonal forces are 29 and 107 kips. Following the alternative provisions in AISC Specification Sec. J2.4, the angle between the weld and the resultant load is $\theta = \tan^{-1}(107/29) = 74.8°$ and the strength factor $\mu = 1 + 0.5\sin^{1.5}\theta° = 1.47$. Thus, the weld size in $^1/_{16}$ths is

$$D_w = \frac{\sqrt{29^2 + 107^2}}{2 \times 10.0 \times 1.392 \times 1.47} = 2.61$$

Use $^3/_{16}$-in fillet weld or AISC minimum.

Weld to Flanges. From Fig. 3.47, the worst-case combined flange loads are 53 kips shear and 29 kips axial. The length of weld is 5.25 in. Proceeding as above, $\theta = \tan^{-1}(29/53) = 28.7°$ and $\mu = 1.17$. However, to maintain deformation compatibility, the weld at the flanges can only develop 99% of its capacity before the less ductile weld at the web fractures (see Art. 3.2.21). Therefore μ is taken equal to 1.16. Note that in more extreme cases, the more ductile welds (those with small θ) will be able to develop as little as 85% of their strength before the less ductile welds (those with large θ) fracture. Thus, the weld size in $^1/_{16}$ths is

$$D_f = \frac{\sqrt{29^2 + 53^2}}{2 \times 5.25 \times 1.392 \times 1.16} = 3.13$$

Use a $^1/_4$-in fillet weld, which is also the AISC minimum.

Strength of Stiffeners (Plate A). The weak-axis beams are Grade 50 steel and are butt welded to plates A. Therefore, plates A should also be Grade 50 steel. Previous calculations involving this plate assumed it was A36, but changing to Grade 50 will not change the final results in this case because the stiffener contact force is limited by the beam no. 1 delivered force, rather than by the stiffener strength.

Design strength for gross section shear (to resist shear at flange):

$$\phi R_{gv} = \phi 0.60 A_g F_y = 0.90 \times 0.60 \times 0.75 \times 5.75 \times 50 = 116 \text{ kips} > 53 \text{ kips} \qquad \text{OK}$$

Design strength for gross section tension (to resist tension at flange):

$$\phi R_{\text{gt}} = \phi A_g F_y = 0.90 \times 0.75 \times 5.75 \times 50 = 194 \text{ kips} > 29 \text{ kips} \qquad \text{OK}$$

Design strength for gross section shear (to resist shear at web):

$$\phi R_{gv} = \phi 0.60 A_g F_y = 0.90 \times 0.60 \times 0.75 \times 10 \times 50 = 203 \text{ kips} > 29 \text{ kips} \qquad \text{OK}$$

Design strength for gross section tension (to resist tension at web):

$$\phi R_{\text{gt}} = \phi A_g F_y = 0.90 \times 0.75 \times 10 \times 50 = 338 \text{ kips} > 107 \text{ kips} \qquad \text{OK}$$

Associated Shear Connections: Beams 3 and 4. The factored shear force for these beams is $R = 107$ kips.

Weld to Beam Web. As with the strong-axis beam web connection, this is a field-welded connection with bolts used for erection only. The design load (required strength) is $R = 107$ kips. The beam web shear R is essentially constant in the area of the connection and is assumed to act at the edge of plate A (section *a–a* of Fig. 3.44*b*). This being the case, there will be a small eccentricity on the C-shaped field weld. Following AISC Manual Table 8-9, $l = 17$, $kl = 4$, $k = 0.24$, $x = 0.04$, $xl = 0.68$, $al = 4.25 - 0.68 = 3.57$, and, by interpolation, $c = 2.80$. Thus, the required weld size in $^1/_{16}$ths is

$$D = \frac{107}{0.75 \times 2.80 \times 17} = 2.99$$

Use a $^3/_{16}$-in fillet weld.

Plate B (Shear Plate) Gross Shear. Try a $^3/_8$-in plate of A36 steel:

$$\phi R_v = 0.9 \times 0.6 \times 36 \times 0.375 \times 17 = 124 \text{ kips} > 107 \text{ kips} \qquad \text{OK}$$

Weld of Plate B to Column Web. This 17.75-in-long weld carries all of the beam shear. The required weld size in $^1/_{16}$ths is thus

$$D = \frac{107}{2 \times 17.75 \times 1.392} = 2.17$$

Use $^3/_{16}$-in fillet weld.

Because this weld occurs on both sides of the column web, the column web should have sufficient shear strength per unit length ($\phi R_v = 0.75 \times 0.60 \times F_u t_w$) to transmit the force from the welds through its thickness. Thus, the thickness should satisfy the relationship $0.75 \times 0.60 \times 65 t_w \geq 1.392 \times D \times 2$ or $t_w > 0.207$. Since the column web thickness is 0.485 in, the web can support the $^3/_{16}$-in fillets. The same result can be achieved using AISC Manual Table 10.2.

Weld of Plate B to Plate A. There is a shear flow $q = VQ/I$ acting on this interface, where the shear force $V = R = 107$ kips, and Q is the statical moment of plate A with respect to the neutral axis of the I section formed by plates A as flanges and plate B as web. Thus,

$$I = \frac{1}{12} \times 0.375 \times 19.25^3 + 0.75 \times 12.5 \times \left(\frac{19.25 + 0.75}{2} \right)^2 \times 2 = 2100 \text{ in}^4$$

$$Q = 0.75 \times 12.5 \times 10 = 93.8 \text{ in}^3$$

$$q = \frac{107 \times 93.8}{2100} = 4.78 \text{ kips/in}$$

Thus,

$$D = \frac{4.78}{2 \times 1.392} = 1.72$$

Since plate B is $^3/_8$ in thick, use the AISC minimum fillet weld size, $^3/_{16}$ in.

The total shear flow force acting on plate A is 4.78 × 6.25 = 29.9 kips. This force does not affect the welds of stiffener A to the column. Rather, stiffener A can be considered an extension of the beam flange, and the shear flow force is taken as part of the flange force. Since the beam flange is full-penetration welded to the stiffener A, no further analysis is required.

A Further Consideration. It sometimes happens in the design of this type of connection that the beam is much stronger in bending than the column. In the example just completed, this is not the case. For the strong-axis W21 × 62 beam, design $M = 389$ ft·kip, while for the column, $\phi M_p = 647$ ft·kip. If the ϕM_p of the column were less than half the design M of the beam, then the connection should be designed for $2(\phi M_p)$ of the column because this is the maximum moment that can be developed between the beam and column, that is, that the system can deliver. Similar conclusions can be arrived at for other arrangements.

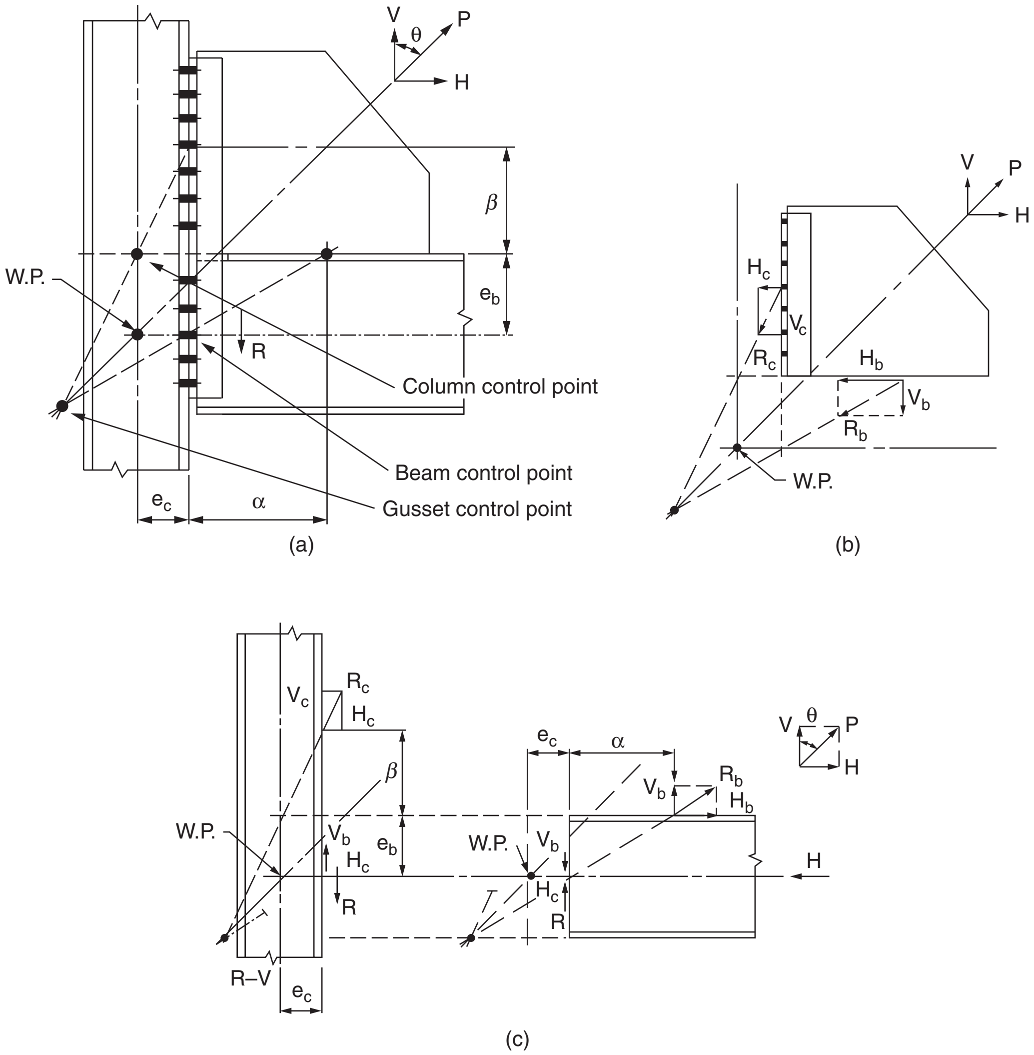
3.7 VERTICAL BRACE DESIGN BY UNIFORM FORCE METHOD

The vertical bracing system in a structure acts as a vertical truss, providing stability for the structure and resisting lateral loads resulting from wind and seismic forces. When the bracing system is concentric, lateral loads will cause only axial loads in the members. On a global level, a concentrically braced frame is a determinate system and the force distribution is easily determined. On a local level, however, the force distribution, or load path, through the connection is not as obvious, and assumptions must be made to establish a reasonable load path.

As discussed in Art. 3.1.3, an endless variety of possible load paths exist, and any design based on a load path that satisfies equilibrium, and for which none of the limit states is exceeded, can be considered as a lower bound to the design strength of the connection.

The uniform force method produces a load path that is consistent with the gusset plate boundaries and eliminates moments at the connection interfaces. See Thornton (1991, 1995b). For instance, if the gusset-to-column connection is to a column web, no horizontal force is directed perpendicular to the column web because, unless it is stiffened, the web will not be able to sustain this force. This is clearly shown in the physical test results of Gross (1990), where it was reported that bracing connections to column webs were unable to mobilize the column weak-axis stiffness because of web flexibility.

The uniform force method is strongly tied to the geometry of the structure, as can be seen in Fig. 3.48. The relationship of the angle of the brace, the depth of the beam, the depth of the column, and the choice of α or β determines the force distribution at all member interfaces. To calculate the forces at



(a) (b) (c)

FIGURE 3.48 The uniform force method. (*a*) Geometry including control points. (*b*) Forces acting on the gusset. (*c*) Forces acting on the beam and column.

the member interfaces, four control points must be established: the work point, the beam control point, the column control point, and the gusset control point.

The intersection of the centers of gravity of the beam, the column, and the brace at the work point ensures that a force distribution exists that will produce no moments in any of the members. The intersection of the brace line, the gusset-to-beam line, and the gusset-to-column line at the gusset control point ensure that the force distribution chosen is the one required to eliminate moments for the system.

The work point, the beam control point, and the column control point are established by the geometry of the given situation. The position of the fourth, the gusset control point, must be calculated. The uniform force method uses the following relationship to determine the location of the fourth control point. (Since the location of the gusset control point is not required in the determination of the force distribution, it is usually not calculated.)

3.7.1 Control Points

Refer to Fig. 3.48 for definition of dimensional terms and forces. From geometry, the angle of the force P from the vertical θ is related to the key dimensions by $\alpha - \beta \tan\theta = e_b \tan\theta - e_c$. With θ determined, the interface forces are then calculated using

$$V_c = \frac{\beta}{r}P \qquad H_c = \frac{e_c}{r}P \qquad V_b = \frac{e_b}{r}P \qquad H_b = \frac{\alpha}{r}P$$

where $r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$

Note that for every choice of α or β, there will be a different location for the gusset control point and a different force distribution at the interfaces. In this way, distribution of the forces can be manipulated to the extent that the geometry of the gusset plate will allow.

3.7.2 Conditions with High Beam Reactions: ΔV_b and Special Case 2

Problems can arise when the beam reaction is high and the additional vertical load V_b delivered by the bracing connection to the beam flange would overstress the member. Rather than reinforcing the beam web, it is usually more economical to reduce V_b. This is done by introducing a force ΔV_b, which acts opposite V_b. Obviously, this will change the force distribution assumed in the uniform force method, and moments in both the beam-to-gusset interface and locally within the column will have to be introduced to maintain equilibrium. AISC recommends calculating the moment introduced at the beam-to-gusset interface as $\Delta V_b(\alpha)$. A more direct and versatile method is to calculate the moment caused by the gusset-to-beam force about the beam control point. This results in the equation

$$M_b = \overline{\alpha}(\overline{V_b}) - e_b(H_b) \tag{3.70}$$

where $\overline{V_b} = V_b - \Delta V_b$

$\overline{\alpha}$ = actual distance from face of support to centroid of gusset-to-beam connection

AISC does not address the moment in the column at this section containing the column control point, though one will exist. However, it can be found by calculating the moment caused by the gusset-to-column force about the column control point, which results in

$$M_c = e_c(\overline{V_c}) - \overline{\beta}(H_c) \tag{3.71}$$

where $\overline{V_c} = V_c + \Delta V_b$

$\overline{\beta}$ = actual distance from face of beam flange to centroid of gusset-to-column connection

The moment M_c occurs in the column cross section that contains the column control point. If there is a connection at this cross section (see Art. 3.7.4), M_c needs to be considered in the design of this connection. Since the column is usually continuous at this cross section, this moment will have no effect on the column. If the column is in compression, the moment M_c will cause increased compressive stress at one column flange, and a tensile stress which decreases the column compressive stress at the other flange. This decreased column compressive stress will prevent any yielding from occurring on the column cross section as a whole and the moment M_c can therefore be neglected. If the column is in tension, the same argument applies. In Art. 3.7.4, M_c will have an effect because there is a connection at this point.

To complete the picture, the free moment on the gusset needs to be calculated. This can be done by multiplying the distance from the intersection of the brace force line and the gusset-to-column force line to the gusset-to-beam line by the gusset-to-beam force. Multiplying the distance from the intersection of the brace force line and the gusset-to-beam force line to the gusset-to-column line by the gusset-to-column force will yield the same results. When simplified, this leads to

$$M_g = -\overline{\beta}(H_c) + \overline{\alpha}(\overline{V_b}) = e_c(V) - e_b(H) \tag{3.72}$$

In this case M_b is equal to M_g, so no moment can exist at the gusset-to-column interface. Therefore, the calculated moment M_c must exist internal to the column.

When taken to the extreme, $\Delta V_b = V_b$, the result is Special Case 2 (Fig. 3.49), illustrated in the AISC Manual.

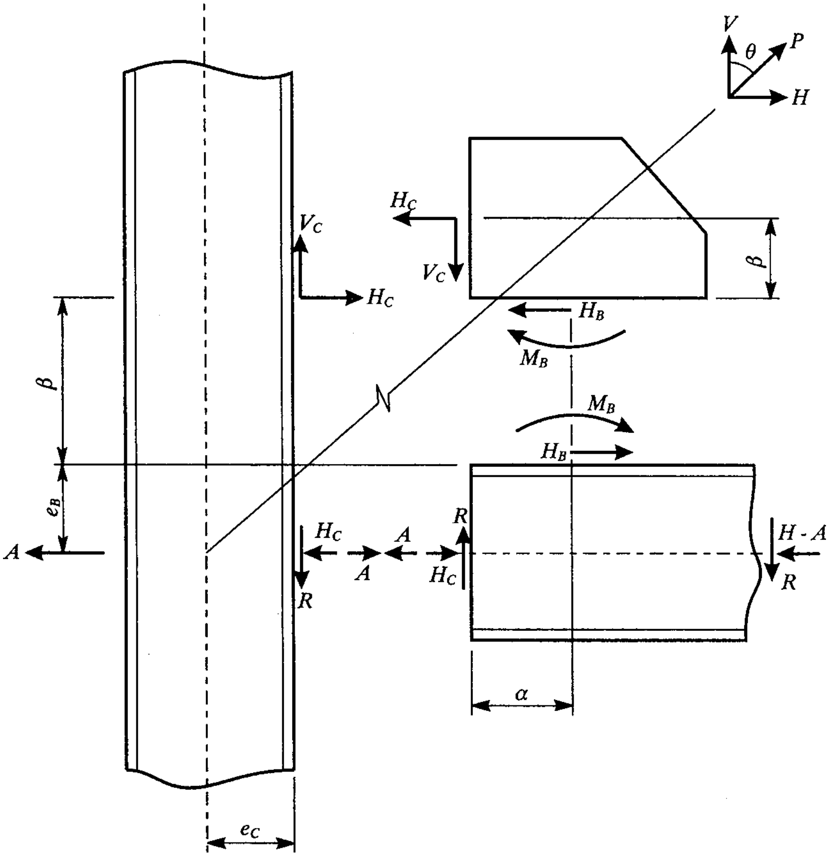


FIGURE 3.49 Force distribution for Special Case 2 of uniform force method. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

3.7.3 Satisfying Geometric Constraints $\bar{\beta}$ and $\bar{\alpha}$

Another problem arises when the geometry will not allow either α or β to be located as prescribed by the uniform force method. In such cases it is logical to assume that the more rigid connection takes all of the moment necessary to satisfy equilibrium. For instance, where the gusset plate is welded to the beam and bolted to the column, $\bar{\beta}$ should be assumed equal to β and $\bar{\alpha}$ should, if necessary, be unequal to α so that all the moment is distributed to the more rigid beam-to-gusset connection.

The AISC Manual introduces two new equations to handle these situations, $M_b = V_b(\bar{\alpha} - \alpha)$ and $M_c = H_c(\bar{\beta} - \beta)$. However, the equations for M_g, M_b, and M_c already presented will work equally well. If $\bar{\beta} \neq \beta$ then M_c will act at the gusset-to-column interface. If $\bar{\alpha} \neq \alpha$, then M_b will act at the gusset-to-column interface. One exception is when ΔV_b is introduced and $\bar{\beta} \neq \beta$. In this case, as shown earlier, ΔV_b will cause an internal moment in the column. The result from the M_c equation will then reflect the total moment both internal and external to the column. The moment that exists at the gusset-to-column interface can be calculated as

$$M_{c\text{–interface}} = M_g - M_b \tag{3.73}$$

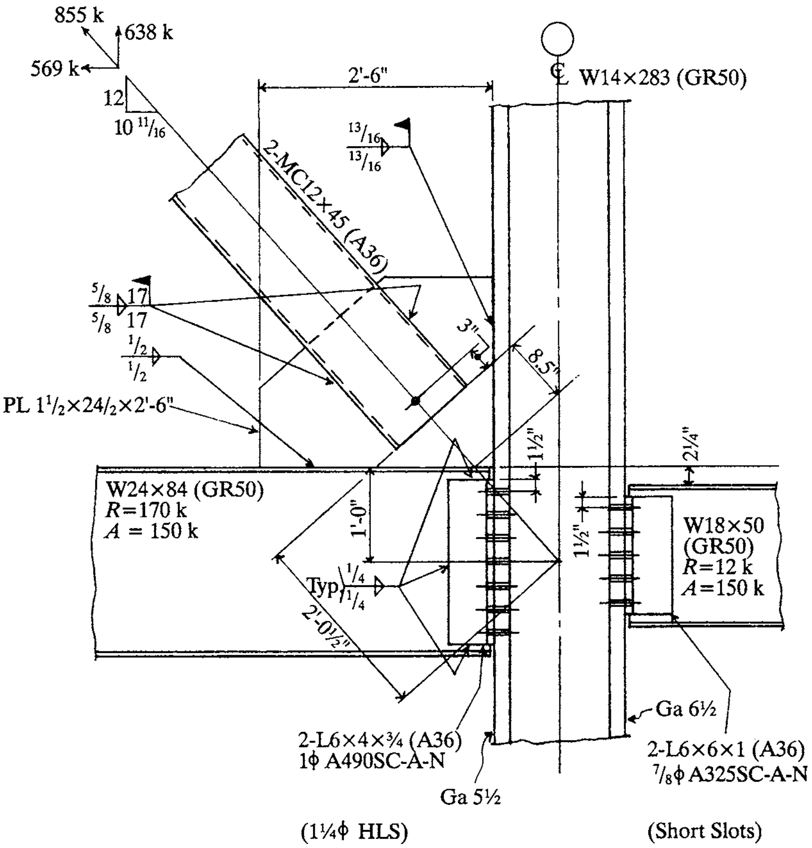


FIGURE 3.50 Example of bracing connection design. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Though it does not affect connection design, if desired, the internal moment in the column can be calculated as

$$M_{c\text{–internal}} = M_c - M_{c\text{–interface}} \tag{3.74}$$

Example of Bracing Connection. Consider the connection shown in Fig. 3.50. The member on the right of the joint is a “collector” that adds load to the bracing truss. The design in this example is for seismic loads. The brace consists of a pair of MC12 × 45 with toes 1.5 in apart. The trial gusset thickness is thus chosen to be 1.5 in. The completed design is shown in Fig. 3.50. In this case, because of the high specified beam shear of 170 kips, it is proposed to use a special case of the uniform force method that sets the vertical component of the load between the gusset and the beam, V_b, to zero. Figure 3.50 shows the resultant force distribution. This method is called Special Case 2 of the uniform force method and is discussed in detail elsewhere (AISC, 1992: AISC, 1994).

Brace-to-Gusset Connection

WELD. The brace is field welded to the gusset with fillet welds. Because of architectural constraints, the gusset size is to be kept to 30 in horizontally and 24.5 in vertically. From the geometry of the gusset and brace, about 17 in of fillet weld can be accommodated. The weld size in $^1/_{16}$ths is

$$D = \frac{855}{4 \times 17 \times 1.392} = 9.03$$

A $^5/_8$-in fillet weld is indicated, but the flange of the MC12 × 45 must be checked to see if an adequate load path exists. The average thickness of 0.700 in occurs at the center of the flange, which is 4.012 in wide. The thickness at the toe of the flange, because of the usual inside flange slope of 2/12 or $16^2/_3$%, is $0.700 - 2/12 \times 2.006 = 0.366$ in (see Fig. 3.51). The thickness at the toe of the fillet is $0.366 + 2/12 \times 0.625 = 0.470$ in. The design shear rupture strength of the MC12 × 45 flange at the toe of the fillet is

$$\phi R_t = 0.75 \times 0.60 \times 58 \times 0.470 \times 17 \times 4 = 834 \text{ kips}$$

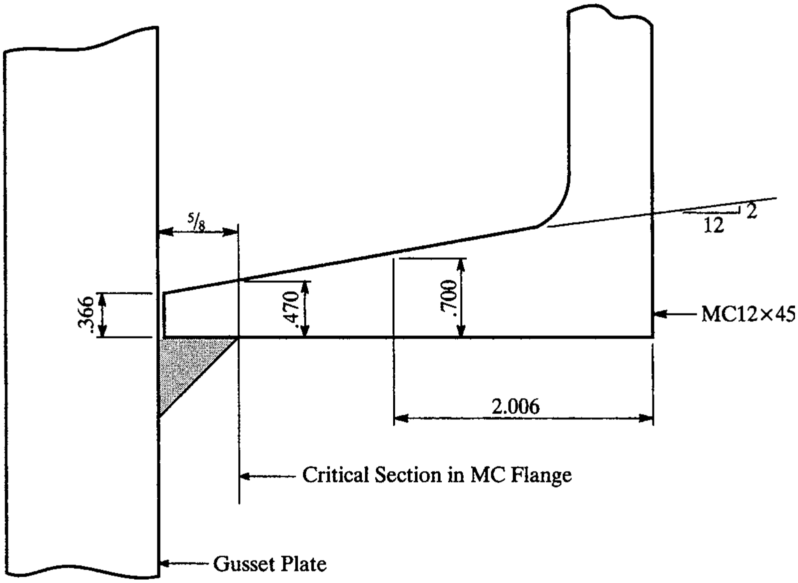


FIGURE 3.51 Critical section at toe of fillet weld. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

The design tensile rupture strength of the toe of the MC flange under the fillet is

$$\phi R_t = 0.75 \times 36\left(\frac{0.366 + 0.470}{2}\right)0.625 \times 4 = 28 \text{ kips}$$

Thus the total strength of the load path in the channel flange is 834 + 28 = 862 kips > 855 kips; OK.

GUSSET-TO-BRACE RUPTURE. Design strength for limit state of shear rupture of gusset is

$$\phi R_v = 0.75 \times 0.6 \times 58 \times 1.5 \times 17 \times 2 = 1331 \text{ kips}$$

Design strength for limit state of tension rupture of gusset is

$$\phi R_t = 0.75 \times 58 \times 1.5 \times 12 = 783 \text{ kips}$$

Design strength for limit state of block shear rupture of gusset is

$$\phi R_{\text{bs}} = 1331 + 0.75 \times 36 \times 1.5 \times 12 = 1817 \text{ kips} > 855 \text{ kips} \qquad \text{OK}$$

WHITMORE SECTION. The theoretical length of the Whitmore section is (17 tan 30°)2 + 12 = 31.6 in. The Whitmore section extends into the column by 5.40 in. The column web is stronger than the gusset since 1.29 × 50/36 = 1.79 > 1.5 in. The Whitmore also extends into the beam web by 6.80 in, but since 0.470 × 50/36 = 0.653 < 1.5 in, the beam web is not as strong as the gusset. The effective Whitmore section length is therefore taken as

$$l_{w\text{eff}} = (31.6 - 6.80) + 6.80 \times \frac{0.470}{1.5} \times \frac{50}{36} = 27.8 \text{ in}$$

The effective length is based on $F_y = 36$ ksi and the gusset thickness of 1.5 in.

Since the brace force can be tension or compression, compression will control. The slenderness ratio of the unsupported length of gusset is

$$\frac{Kl}{r} = \frac{0.5 \times 8.5\sqrt{12}}{1.5} = 9.8$$

From the AISC Specification, Sec. J4.4, the buckling strength is

$$\phi F_a = 0.9 \times 36 = 32.4 \text{ ksi}$$

and the buckling strength of the gusset is

$$\phi R_{\text{wb}} = 27.8 \times 1.5 \times 32.4 = 1350 > 855 \text{ kips} \qquad \text{OK}$$

This completes the brace-to-gusset part of the design. Before proceeding, the distribution of forces to the gusset edges must be determined. From Figs. 3.49 and 3.50,

$$e_b = \frac{24.10}{2} = 12.05 \text{ in} \qquad e_c = 8.37 \text{ in} \qquad \bar{\beta} = 12.25 \text{ in} \qquad \bar{\alpha} = 15.0 \text{ in}$$

$$\theta = \tan^{-1}\frac{10.6875}{12} = 41.6°$$

$$V_c = P\cos\theta = 855 \times 0.747 = 638 \text{ kips}$$

$$H_c = \frac{V_c e_c}{e_b + \beta} = \frac{638 \times 8.37}{12.05 + 12.25} = 220 \text{ kips}$$

$$H_b = P\sin\theta - H_c = 855 \times 0.665 - 220 = 349 \text{ kips}$$

$$M_b = H_b e_b = 349 \times 12.05 = 4205 \text{ in·kips}$$

Note that, in this Special Case 2, the calculations can be simplified as shown in the preceding. The same results can be obtained formally with the uniform force method by setting $\beta = \bar{\beta} = 12.25$ and proceeding as follows. With $\tan\theta = 0.8906$,

$$\alpha - 0.8906\beta = 12.05 \times 0.8906 - 8.37 = 2.362 \text{ in}$$

Setting $\beta = \bar{\beta} = 12.25$ in, $\alpha = 13.27$ in. Since $\bar{\alpha} = 15.0$ in, there will be a couple, M_b, on the gusset-to-beam edge. Continuing,

$$r = \sqrt{(13.27 + 8.37)^2 + (12.25 + 12.05)^2} = 32.54$$

$$\frac{P}{r} = 26.27$$

$$H_b = \frac{\alpha}{r} P = 349 \text{ kips}$$

$$H_c = \frac{e_c}{r} P = 220 \text{ kips}$$

$$V_b = \frac{e_b}{r} P = 317 \text{ kips}$$

$$V_c = \frac{\beta}{r} P = 322 \text{ kips}$$

$$M_b = | V_b(\alpha - \bar{\alpha}) | = 548 \text{ in} \cdot \text{kips}$$

This couple is clockwise on the gusset edge. Now, introducing Special Case 2, set $\Delta V_B = V_B = 317$ kips. This reduces the vertical force between the gusset and beam to zero, and increases the gusset-to-column shear, V_C, to 317 + 322 = 638 kips and creates a counterclockwise couple on the gusset-to-beam edge of $\Delta V_B \bar{\alpha} = 317 \times 15.0 = 4755$ in·kips. The total couple on the gusset-to-beam edge is thus $M_B = 4755 - 548 = 4207$ in·kips. It can be seen that these gusset interface forces are the same as those obtained from the simpler method.

Gusset-to-Column Connection. The loads are 638 kips shear and 220 kips axial.

GUSSET STRENGTH. Design strength for gross section shear is

$$\phi R_{gv} = \phi 0.60 A_g F_y = 0.90 \times 0.60 \times 1.5 \times 24.5 \times 36 = 714 \text{ kips} > 638 \text{ kips} \qquad \text{OK}$$

Design strength for gross section tension is

$$\phi R_{\text{gt}} = \phi A_g F_y = 0.90 \times 1.5 \times 24.5 \times 36 = 1190 \text{ kips} > 220 \text{ kips} \qquad \text{OK}$$

WELD OF GUSSET-TO-COLUMN FLANGE

$$P_u = \sqrt{638^2 + 220^2} = 675 \text{ kips}$$

The angle from the longitudinal weld axis is $\tan^{-1}$ (220/638) = 19°. From the AISC Manual, Table 8.5 for 15° with $k = a = 0.0$, $C = 3.84$. The weld size in $^1/_{16}$ths is

$$D = \frac{675}{0.75 \times 3.84 \times 24.5} 1.25 = 12.0$$

Use a $^3/_4$-in fillet. The ductility factor of 1.25 is used because the weld is assumed to be uniformly loaded, but research shows that the ratio of peak-to-average stresses is about 1.25 (Hewitt and Thornton, 2004).

Column Web

WEB YIELDING (UNDER NORMAL LOAD H_c)

$$\phi R_{\text{wy}} = 1.0 \times 50 \times 1.290(24.5 + 5 \times 2^3/_4) = 2470 \text{ kips} > 220 \text{ kips} \qquad \text{OK}$$

WEB CRIPPLING (UNDER NORMAL LOAD H_c)

$$\phi R_{wcp} = 0.75 \times 0.8 \times 1.29^2 \left[1 + 3\left(\frac{24.5}{16.7}\right)\left(\frac{1.29}{2.07}\right)^{1.5}\right]\sqrt{\frac{29{,}000 \times 50 \times 2.07}{1.29}}$$

$$= 4820 \text{ kips} > 220 \text{ kips} \qquad \text{OK}$$

WEB SHEAR. The horizontal force, H_c, is transferred to the column by the gusset-to-column connection and back into the beam by the beam-to-column connection. Thus, the column web sees H_c = 220 kips as a shear. The column shear capacity is

$$\phi R_v = 0.9 \times 0.6 \times 50 \times 1.29 \times 16.7 = 583 \text{ kips} > 200 \text{ kips} \qquad \text{OK}$$

Gusset-to-Beam Connection. Design for 349 kips shear and 4205 in·kips moment.

GUSSET STRENGTH. Design strength for the gross section shear is

$$f_v = 349/(1.5 \times 30) = 7.76 \text{ ksi} < 19.4 \text{ ksi} \qquad \text{OK}$$

Design strength for flexure is

$$f_b = (4205 \times 4)/(1.5 \times 30^2) = 12.5 \text{ ksi} < 32.4 \text{ ksi} \qquad \text{OK}$$

WELD OF GUSSET-TO-BEAM FLANGE

$$f_{\text{peak}} = \sqrt{7.76^2 + 12.5^2} \times \frac{1.5}{2} = 11.0 \text{ kips/in}$$

$$\theta_w = a\tan\left(\frac{12.5}{7.76}\right) = 58.2°$$

$$f_{\text{ave}} = \frac{1}{2}\left[\sqrt{7.76^2 + 12.5^2} + \sqrt{7.76^2 + 12.5^2}\right]\frac{1.5}{2} = 11.0 \text{ kips/in}$$

Since 11.0/11.0 = 1.0 < 1.25, the weld size based on the average force in the weld is $1.25 f_{\text{ave}}$. Therefore,

$$D = \frac{11.0 \times 1.25}{1.392[1 + 0.5\sin^{1.5}(58.2)]} = 7.10$$

Use a ½-in fillet weld.

Beam Web

WEB YIELD. Although there is no axial component, the couple M_B = 4205 in·kips is statically equivalent to equal and opposite vertical shears at a lever arm of one-half the gusset length or 15 in. The shear is thus

$$V_s = \frac{4205}{15} = 280 \text{ kips}$$

This shear is applied to the flange as a transverse load over 15 in of flange. It is convenient for analysis purposes to imagine this load doubled and applied over the contact length N = 30 in. The design web yielding strength is

$$\phi R_{\text{wy}} = 1.0 \times 50 \times 0.47(30 + 5 \times 1.27) = 854 \text{ kips} > 280 \times 2 = 560 \text{ kips} \qquad \text{OK}$$

WEB CRIPPLING

$$\phi R_{wcp} = 0.75 \times 0.8 \times 0.47^2 \left[1 + 3\left(\frac{30}{24.1}\right)\left(\frac{0.47}{0.77}\right)^{1.5}\right]\sqrt{\frac{29{,}000 \times 50 \times 0.77}{0.47}}$$

$$= 568 \text{ kips} > 560 \text{ kips} \qquad \text{OK}$$

WEB SHEAR

$$\phi P_v = 0.90 \times 0.60 \times 50 \times 0.47 \times 24.1 = 306 \text{ kips} > 280 \text{ kips} \qquad \text{OK}$$

The maximum shear due to the couple is centered on the gusset 15 in from the beam end. It does not reach the beam-to-column connection, where the beam shear is 170 kips. Because of the total vertical shear capacity of the beam and the gusset acting together, there is no need to check the beam web for a combined shear of V_s and R of 280 + 170 = 450 kips.

Beam-to-Column Connection. The shear load is 170 kips and the axial force is $H_c \pm A = 220 \pm 150$ kips. Since the W18 × 50 is a collector, it adds load to the bracing system. Thus, the axial load is 220 + 150 = 370 kips. However, the AISC book on connections (AISC, 1992) addresses this situation and states that because of frame action (distortion), which will always tend to reduce H_c, it is reasonable to use the larger of H_c and A as the axial force. Thus the axial load would be 220 kips in this case. It should be noted, however, that when the brace load is not due to primarily lateral loads, frame action might not occur.

BOLTS AND CLIPS. Though loads caused by wind and seismic forces are not considered cyclic (fatigue) loads and bolts in tension are not required to be designed as slip critical, the bolts are specified to be designed as A490 SC-A-N 1-in diameter to accommodate the use of oversize 1¼-in-diameter holes. Thus, for shear,

$$\phi_{rstr} = 0.85 \times 1.13 \times 0.33 \times 64 = 20.3 \text{ kips/bolt}$$

and for tension,

$$\phi_{rt} = 0.75 \times 113 \times 0.7854 = 66.6 \text{ kips/bolt}$$

The clips are angles 4 × 4 × ¾ with seven rows of bolts. For shear,

$$\phi R_v = 20.3 \times 14 = 284 \text{ kips} > 170 \text{ kips} \qquad \text{OK}$$

Bolt bearing and tear-out are not controlling limit states.

For bearing,

$$\phi R_{\text{brg}} = 0.75 \times 2.4 \times 1 \times 0.75 \times 58 = 78.3 \text{ kips/bolt} > 20.3 \text{ kips/bolt at the clips}$$
$$\phi R_{\text{brg}} = 0.75 \times 2.4 \times 1 \times 2.07 \times 65 = 242 \text{ kips/bolt} > 20.3 \text{ kips/bolt at the column}$$

At the column there is no possibility of the bolt tearing out through an edge, so the only limit state is tear-out between the bolts. $L_c = 3 - 1.06 = 1.94$ in. At the angles, since the clear-edge distance is smaller than the clear spacing distance, tear-out through the edge will govern. $L_c = 1.5 - 0.625 = 0.875$ in. For bolt tear-out,

$$\phi R_{\text{brg}} = 0.75 \times 1.2 \times 0.875 \times 0.75 \times 58 = 34.3 \text{ kips/bolt} > 20.3 \text{ kips/bolt at the clips}$$
$$\phi R_{\text{brg}} = 0.75 \times 1.2 \times 1.94 \times 2.07 \times 65 = 235 \text{ kips/bolt} > 20.3 \text{ kips/bolt at the column}$$

For tension, the bolts and clips are checked together for prying action.

Since all of the bolts are subjected to tension simultaneously, there is interaction between tension and shear. The reduced tensile capacity is

$$\phi r_t' = 1.13 \times 64\left(1 - \frac{170/4}{20.3}\right) = 29.1 \text{ kips/bolt}$$

Since 29.1 kips > 220/14 = 15.7 kips, the bolts are OK for tension. The bearing-type interaction expression should also be checked, but it will not control.

Prying action is now checked using the method and notation of the AISC Manual:

$$b = \frac{5.5 - 0.47}{2} - 0.375 = 2.14 \text{ in}$$

$$a = \frac{8 + 0.47 - 5.5}{2} = 1.49 \text{ in}$$

Check 1.25b = 1.25 × 2.14 = 2.68 in. Since 2.68 > 1.49, use a = 1.49 in.

$$b' = 2.14 - \frac{1.0}{2} = 1.64 \text{ in}$$

$$a' = 1.49 + \frac{1.0}{2} = 1.99$$

$$\rho = 0.824$$

$$\delta = 1 - \frac{1.25}{3} = 0.583$$

$$t_c = \sqrt{\frac{4.44 \times 29.1 \times 1.64}{3 \times 58}} = 1.10 \text{ in}$$

$$\alpha' = \frac{1}{0.583 \times 1.824}\left[\left(\frac{0.10}{0.75}\right)^2 - 1\right] = 1.08$$

The design strength per bolt, including prying, is

$$T_d = 29.1\left(\frac{0.75}{1.10}\right)^2 (1 + 0.583 \times 1.00)$$

$$= 21.4 \text{ kips} > 15.7 \text{ kips} \qquad \text{OK}$$

In addition to the prying check, the clips should also be checked for gross and net shear, but these will not control in this case.

WELD OF CLIPS TO BEAM WEB. The weld is a C-shaped weld with length l = 21 in, kl = 3.5 in, k = 3.5/21 = 0.167. From the AISC Manual, Table 8-9, $xl = 0.0220 \times 21 = 0.462$ in, so $al = 6 - 0.462 = 5.538$ in, and a = 5.538/21 = 0.264. Since $\tan^{-1}$ 220/170 = 52.3°, use the chart for 45°. By interpolation, $C = 2.56$. A $^1\!/_4$-in fillet weld has a capacity of $\phi R_w = 0.75 \times 2.56 \times 4 \times 2 \times 21 = 323$ kips. To support this weld, the web thickness required is $0.90 \times 0.60 \times 50 \times t_w > 1.392 \times 4 \times 2$. Thus, t_w required ≥ 0.41 in. Since the actual web thickness is 0.470 in, the weld is fully effective and has the calculated capacity. Thus, since 323 kips > $\sqrt{220^2 + 170^2}$ = 278 kips, the $^1\!/_4$-in fillet weld is OK.

BENDING OF THE COLUMN FLANGE. Because of the axial force, the column flange can bend just as the clip angles. A yield-line analysis derived from Mann and Morris (1979) can be used to determine an effective tributary length of column flange per bolt. The yield lines are shown in Fig. 3.52. First, determine the effective pitch, p_{eff}:

$$p_{\text{eff}} = \frac{(n-1)p + \pi\bar{b} + 2\bar{a}}{n}$$

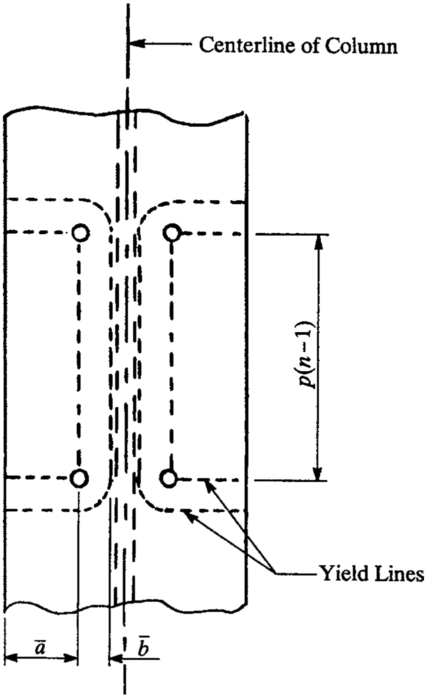


FIGURE 3.52 Yield lines for flange bending. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

where $\bar{b} = \frac{5.5-1.29}{2} = 2.11$ in

$\bar{a} = \frac{16.1-5.5}{2} = 5.31$ in

$p = 3$ in

$n = 7$

Thus,

$$p_{\text{eff}} = \frac{6\times 3+\pi\times 2.11+2\times 5.31}{7} = 5.03 \text{ in}$$

Using p_{eff} in place of p, and following the AISC procedure,

$$b = \bar{b} = 2.11 \text{ in}$$

$$b' = 2.11 - \frac{1.0}{2} = 1.61 \text{ in}$$

$$a = \min\left(\frac{4+4+0.47-5.5}{2}, 5.31, 1.25\times 2.11\right) = \min(1.48, 5.31, 2.63) = 1.49 \text{ in}$$

$$a' = 1.49+0.5 = 1.99 \text{ in}$$

$$\rho = \frac{b'}{a'} = 0.81$$

$$\delta = 1 - \frac{1.06}{5.03} = 0.79$$

Note that standard holes are used in the column flange.

$$t_c = \sqrt{\frac{4.44\times 29.1\times 1.61}{5.03\times 65}} = 0.798 \text{ in}$$

$$\alpha' = \frac{1}{0.79\times 1.81}\left[\left(\frac{0.798}{2.07}\right)^2 - 1\right] = -0.595$$

Since $\alpha' < 0$, use $\alpha' = 0$.

$$T_d = 29.1 \text{ kips/bolt} > 15.7 \text{ kips/bolt} \qquad \text{OK}$$

When $\alpha' < 1$, the bolts, not the flange, control the strength of the connection.

3.7.4 Vertical Brace Connections Using Extended Single Plate

It often makes sense when designing the lateral force resisting system for a building to design a moment frame along the strong axis of the columns and a braced frame along the weak one. This can lead to a situation in which stiffeners are required in the column webs due to the moment connections, and the weak-axis beam and bracing connections need to be kept clear of the column flanges for erection. In such cases the connections to the columns are often made using extended single-plate connections. The uniform force method is particularly well suited to analyzing this type of connection.

By moving e_c from the center of the column to the center of the bolt group in the single-plate connections, the eccentricity which would normally have to be considered in the bolt group can now be taken as a couple, H_c, producing a more efficient connection. See Figs. 3.53 and 3.54. The uniform force method equations can now be applied to the connection, with the forces normally associated with the column now associated with the single-plate connection. As long as the forces derived from the uniform force method are used without the introduction of ΔV_b or a change in α or β, no moments will need to be considered in any of the connection interfaces or within the plates. However, the single-plate connector must now be checked to transfer H_c as a horizontal shear. If α or β is varied,

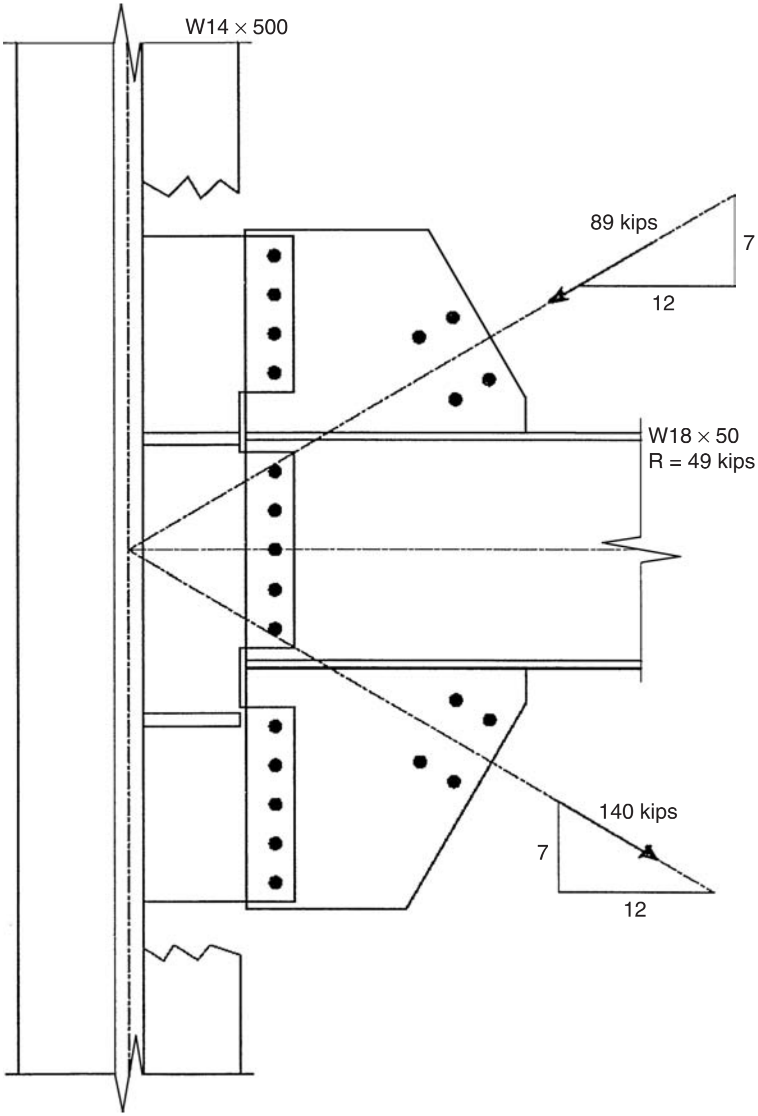


FIGURE 3.53 Vertical bracing using an extended single-plate connection.

a moment must be developed at the appropriate gusset interface. If ΔV_b is introduced, an internal moment will develop in the single-plate connector and all appropriate checks must be made.

Note that although the moment due to the brace force has been eliminated, a moment due to the beam reaction must still be considered. Since the beam reaction does not have a line of action that passes through the workpoint, it is impossible to eliminate all moments from the system, as is accomplished through the use of the uniform force method for the brace force. However, it is possible to move the moment from the beam-to-column connection to the beam-to-gusset connection. Since the beam-to-gusset connection is usually a welded and not a bolted connection, and is not limited by the depth of the

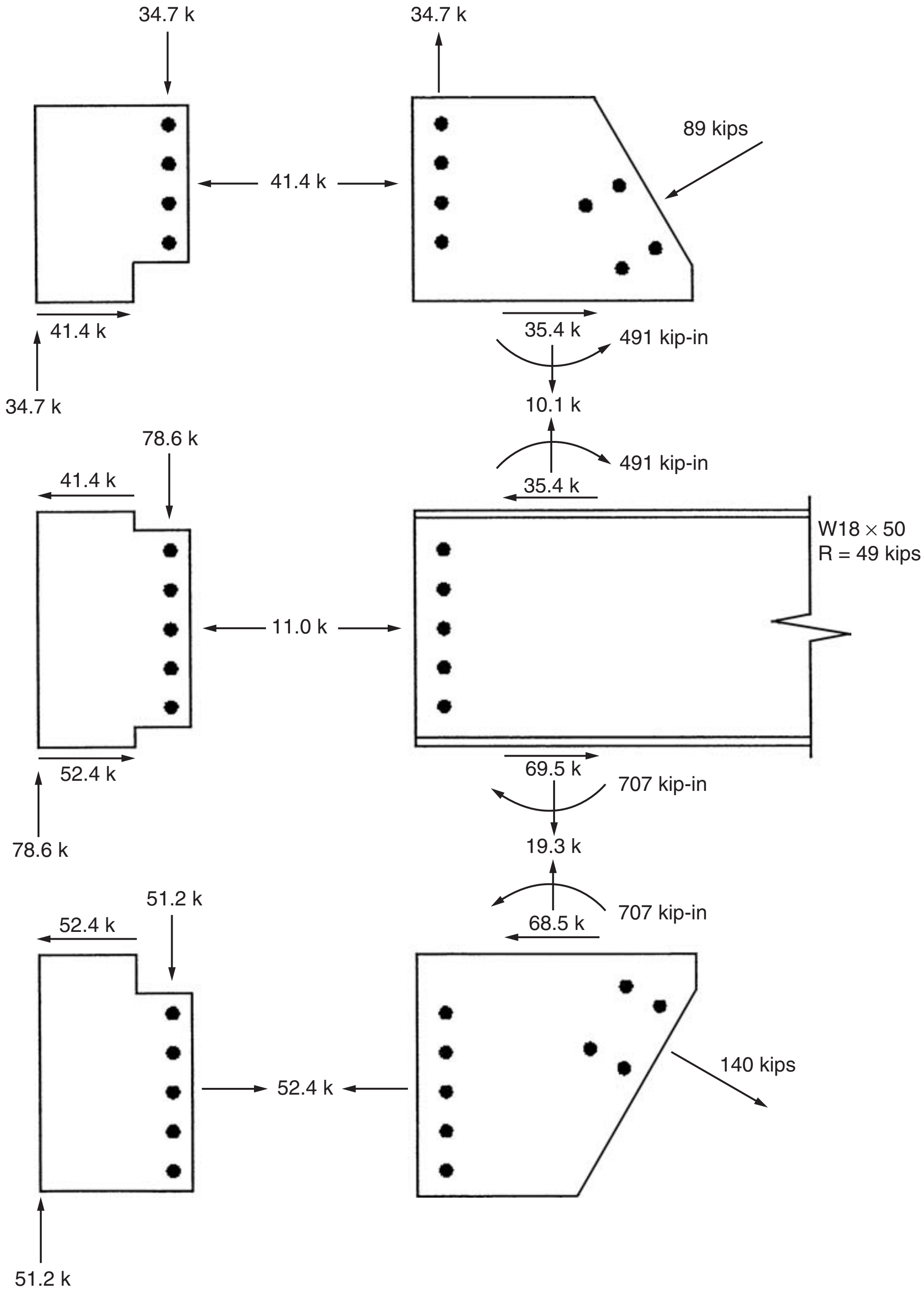


FIGURE 3.54 Free-body diagram of vertical bracing using extended single-plate connection.

beam as the beam-to-column connection is, it is advantageous to resist the moment here. To accomplish this, an H_c' is calculated in a manner similar to the H_c calculation in the uniform force method, as follows:

$$H_c' = \frac{e_c}{e_b + \overline{\beta}} R \tag{3.75}$$

The beam reaction is then distributed between the beam-to-column connection and the gusset-to-column connection so as to eliminate moments in the extended single-plate connection. This is done as follows:

$$V_c' = \frac{\overline{\beta}}{e_b + \overline{\beta}} R \tag{3.76}$$

$$V_b' = \frac{e_b}{e_b + \overline{\beta}} R \tag{3.77}$$

These forces can then be included in the equations used with the uniform force method so that

$$\overline{V_c} = \frac{\beta}{r} P + \Delta V_b + V_c' \tag{3.78}$$

$$H_c = \frac{e_c}{r} P + H_c' \tag{3.79}$$

$$V_b = \frac{e_b}{r} P - \Delta V_b + V_b' \tag{3.80}$$

The moments at the interfaces can be calculated as

$$M_b = \overline{\alpha}\left(\overline{V_b} - \frac{R}{2}\right) - e_b \overline{H_b} - e_c\left(\frac{R}{2}\right) \tag{3.81}$$

$$M_c = e_c \overline{V_c} - \overline{H_c}\overline{\beta} \tag{3.82}$$

$$M_g = -\overline{\beta}\overline{H_c} + \overline{\alpha}(V - \overline{V_c}) + e_c V - e_b H \tag{3.83}$$

Example of Extended Single-Plate Connection for Vertical Brace. The above formulation of the uniform force method will be applied to the design of the connection shown in Figs. 3.54 and 3.55.

Determine Interface Forces

$$H = \sin(\theta)P = \sin(59.74)89 = 76.9 \text{ kips}$$

$$V = \cos(\theta)P = \cos(59.74)89 = 44.8 \text{ kips}$$

$$\alpha = e_b \tan(\theta) - e_c + \beta \tan(\theta) = (9.00)\ \tan(59.74) - 10.8 + (9)\ \tan(59.74) = 20.1 \text{ in}$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(20.1 + 10.8)^2 + (9.00 + 9.00)^2} = 35.7 \text{ in}$$

$$V_c = P\left(\frac{\beta}{r}\right) = 89\left(\frac{9.00}{35.7}\right) = 22.4 \text{ kips}$$

$$H_c = P\left(\frac{e_c}{r}\right) = 89\left(\frac{10.8}{35.7}\right) = 26.8 \text{ kips}$$

$$V_b = P\left(\frac{e_b}{r}\right) = 89\left(\frac{9.00}{35.7}\right) = 22.4 \text{ kips}$$

$$H_b = P\left(\frac{\alpha}{r}\right) = 89\left(\frac{20.1}{35.7}\right) = 50.1 \text{ kips}$$

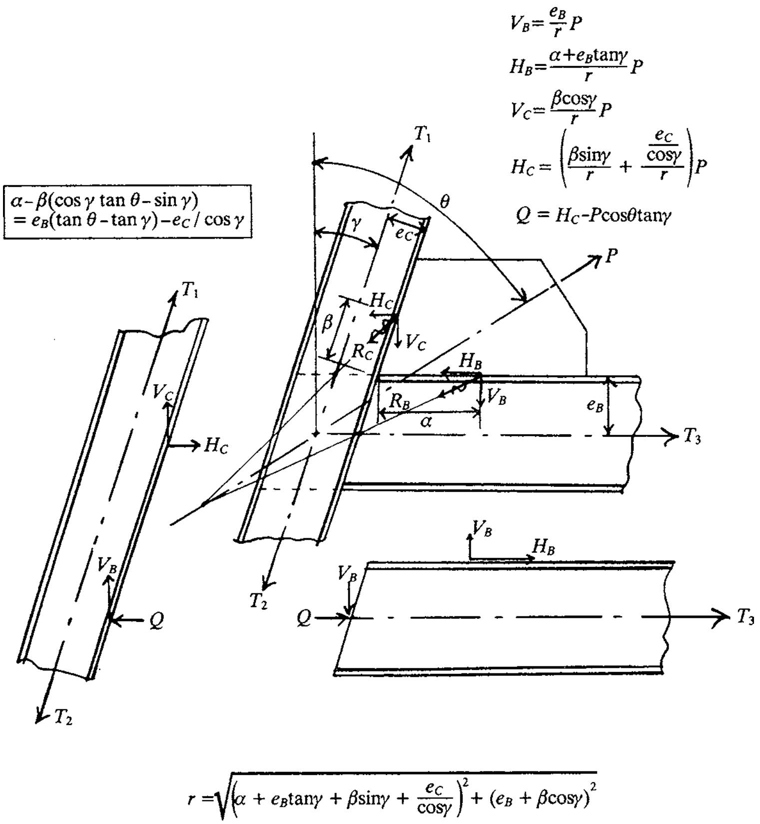


FIGURE 3.55 Generalized uniform force method. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

Distribute Half of Beam Reaction to Top Gusset

$$H_c' = \frac{(R/2)e_c}{e_b + \overline{\beta}} = \frac{24.5(10.8)}{9.00 + 9.00} = 14.6 \text{ kips}$$

$$V_c' = \frac{(R/2)\overline{\beta}}{e_b + \overline{\beta}} = \frac{24.5(9.00)}{9.00 + 9.00} = 12.3 \text{ kips}$$

$$V_b' = \frac{(R/2)e_b}{e_b + \overline{\beta}} = \frac{24.5(9.00)}{9.00 + 9.00} = 12.2 \text{ kips}$$

$$\overline{V_b} = V_b - \Delta V_b + V_b' = 22.4 - 0 + 12.2 = 34.6 \text{ kips}$$

$$\overline{V_c} = V_c + \Delta V_b + V_c' = 22.4 + 0 + 12.3 = 34.7 \text{ kips}$$

$$\overline{H_c} = H_c + H_c' = 26.8 + 14.6 = 41.4 \text{ kips}$$

$$M_b = \overline{\alpha}\left(\overline{V_b} - \frac{R}{2}\right) - e_b\overline{H_b} - e_c\left(\frac{R}{2}\right) = 9.00(34.6 - 24.5) - 9.00(35.4) - 10.8(24.5) = 491 \text{ in} \cdot \text{kips}$$

$$M_c = e_c\bar{V}_c - \bar{H}_c\bar{\beta} = 10.8(34.7) - 9.00(41.4) = 0 \text{ in} \cdot \text{kips}$$

$$M_g = -\bar{\beta}\bar{H}_c + \bar{\alpha}(V - \bar{V}_c) + e_c V - e_b H = -9.00(41.4) + 9.00(10.2) + 10.75(44.8) - 8.995(76.9)$$
$$= 491 \text{ in} \cdot \text{kips}$$

Determine Interface Forces for Bottom Bracket

$$H = \sin(\theta)P = \sin(59.74)140 = 121 \text{ kips}$$
$$V = \cos(\theta)P = \cos(59.74)140 = 70.5 \text{ kips}$$
$$\alpha = e_b\tan(\theta) - e_c + \beta\tan(\theta) = (9.00)\tan(59.74) - 10.8 + (10.5)\tan(59.74) = 22.7 \text{ in}$$
$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(22.7 + 10.8)^2 + (10.5 + 9.00)^2} = 38.7 \text{ in}$$
$$V_c = P\left(\frac{\beta}{r}\right) = 140\left(\frac{10.5}{38.7}\right) = 38.0 \text{ kips}$$
$$H_c = P\left(\frac{e_c}{r}\right) = 140\left(\frac{10.8}{38.7}\right) = 39.1 \text{ kips}$$
$$V_b = P\left(\frac{e_b}{r}\right) = 140\left(\frac{9.00}{38.7}\right) = 32.6 \text{ kips}$$
$$H_b = P\left(\frac{\alpha}{r}\right) = 140\left(\frac{22.7}{38.7}\right) = 82.1 \text{ kips}$$

Distribute Half of Beam Reaction to Top Gusset

$$H_c' = \frac{(R/2)e_c}{e_b + \bar{\beta}} = \frac{24.5(10.8)}{9.00 + 10.5} = 13.6 \text{ kips}$$
$$V_c' = \frac{(R/2)\bar{\beta}}{e_b + \bar{\beta}} = \frac{24.5(10.5)}{9.00 + 10.5} = 13.2 \text{ kips}$$
$$V_b' = \frac{(R/2)e_b}{e_b + \bar{\beta}} = \frac{24.5(9.00)}{9.00 + 10.5} = 11.3 \text{ kips}$$
$$\bar{V}_b = V_b - \Delta V_b + V_b' = 32.6 - 0 + 11.3 = 43.9 \text{ kips}$$
$$\bar{V}_c = V_c - \Delta V_b + V_c' = 38.0 + 0 + 13.2 = 51.2 \text{ kips}$$
$$\bar{H}_c = H_c + H_c' = 39.1 + 13.6 = 52.7 \text{ kips}$$
$$M_b = \bar{\alpha}\left(\bar{V}_b - \frac{R}{2}\right) - e_b\bar{H}_b - e_c\left(\frac{R}{2}\right) = 9.00(43.9 - 24.5) - 9.00(68.5) - 10.8(24.5) = 707 \text{ in} \cdot \text{kips}$$
$$M_c = e_c\bar{V}_c - \bar{H}_c\bar{\beta} = 10.8(51.2) - 10.5(52.7) = 0 \text{ in} \cdot \text{kips}$$
$$M_g = -\bar{\beta}\bar{H}_c + \bar{\alpha}(V - \bar{V}_c) + e_c V - e_b H = -10.5(52.7) + 9.00(19.3) + 10.8(70.5) - 8.995(121)$$
$$= 707 \text{ in} \cdot \text{kips}$$

Nonorthogonal Trusses. The uniform force method as originally formulated can be applied to trusses as well as to bracing connections. After all, a vertical bracing system is just a truss, as can be seen in Fig. 3.55, which shows various arrangements. However, bracing systems generally involve orthogonal members whereas trusses, especially roof trusses, often have a sloping top chord. To handle this situation, the uniform force method has been generalized as shown in Fig. 3.55 to include nonorthogonal members. As before, α and β locate the centroids of the gusset-edge connections

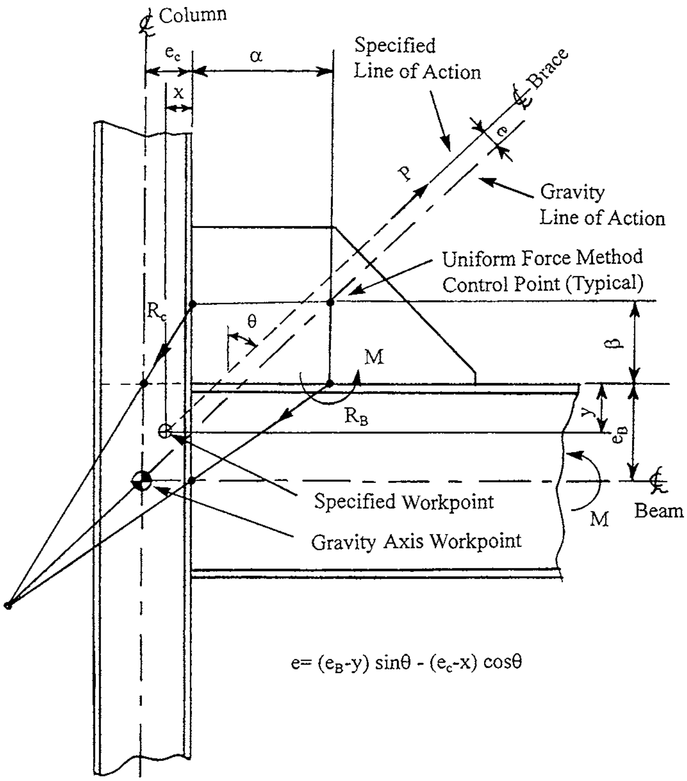


FIGURE 3.56 Nonconcentric uniform force method. (*Source*: *A. R. Tamboli*, Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission*.)

and must satisfy the constraint shown in the box in Fig. 3.55. This can always be arranged when designing a connection, but in checking a given connection designed by some other method, the constraint may not be satisfied. The result is gusset-edge couples that must be considered in the design. In Fig. 3.55, the angle, γ, is positive as shown. If the angle between the column and the beam is greater than 90°, γ is negative.

Connections with Nonconcentric Workpoints. The uniform force method can be easily generalized to this case as shown in Fig. 3.56, where x and y locate the specified nonconcentric workpoint (WP) from the intersection of the beam and column flanges. All of the forces on the connection interfaces are the same as for the concentric uniform force method, except that there is an extra moment on the gusset plate $M = Pe$, which can be applied to the stiffer gusset edge. It should be noted that this nonconcentric force distribution is consistent with the findings of Richard (1986), who found very little effect on the force distribution in the connection when the work point is moved from concentric to nonconcentric locations. It should also be noted that a nonconcentric work-point location induces a moment in the structure of $M = Pe$, and this may need to be considered in the design of the frame members. In the case of Fig. 3.56, since the moment $M = Pe$ is assumed to act on the gusset-to-beam interface, it also must be assumed to act on the beam outside of the connection, as shown. In the case of a connection to a column web, this will be the actual distribution (Gross, 1990), unless the connection to the column mobilizes the flanges by means of stiffeners.

An alternative analysis can be performed where the joint is considered rigid, such as a connection to a column flange. Here, the moment M is distributed to the beam and column in accordance with their stiffnesses. The brace is usually assumed to remain an axial force member and so is not included in the moment distribution. If η denotes the fraction of the moment that is distributed to the beam, then horizontal and vertical forces H' and V', respectively, acting at the gusset-to-beam, gusset-to-column, and beam-to-column connection centroids due to the distribution of M, are

$$H' = \frac{(1-\eta)M}{\bar{\beta}+e_b} \tag{3.84}$$

$$V' = \frac{M - H'\bar{\beta}}{\bar{\alpha}} \tag{3.85}$$

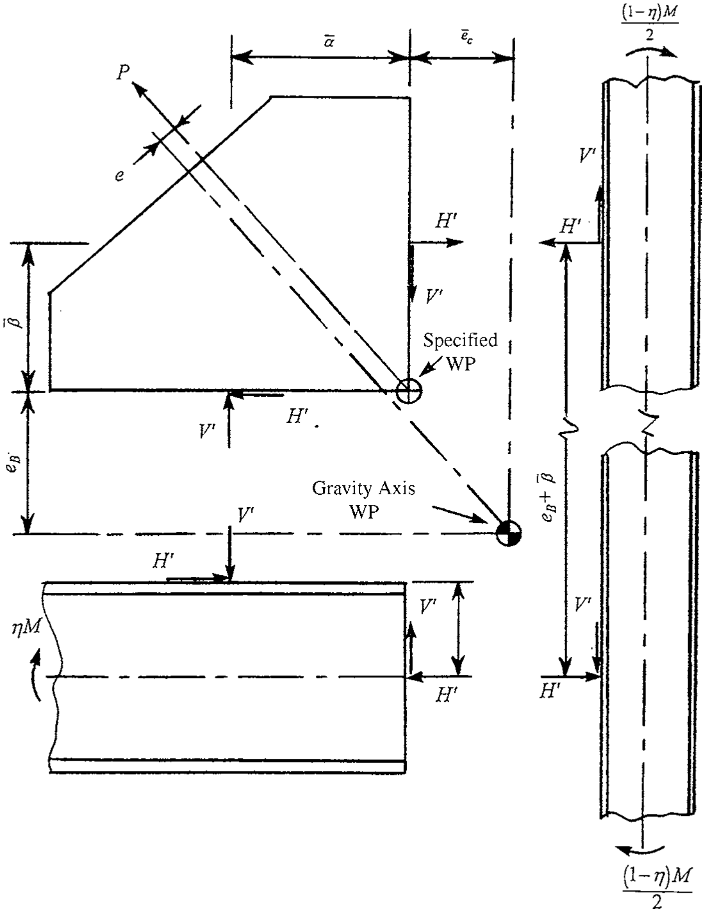


FIGURE 3.57 Vertical brace connection with nonconcentric workpoint. (*Source: A. R. Tamboli,* Handbook of Structural Steel Connection Design and Details, *McGraw-Hill, 1999, with permission.*)

These forces, shown in Fig. 3.57, are added algebraically to those of the concentric uniform force method acting at the three connection interfaces. Note that for connections to column webs, $\eta = 1$, $H' = 0$, and $V' = M/\overline{\alpha}$, unless the gusset-to-column web and beam-to-column web connections positively engage the column flanges. Figure 3.57 shows the specified WP at the corner of the gusset. This is a special case of the location shown in Fig. 3.56.

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CHAPTER 4

BUILDING CODES, LOADS, AND FIRE PROTECTION*

Nestor R. Iwankiw, Ph.D., P.E.
Senior Engineer
Hughes Associates, Inc.
Chicago, Illinois

Building designs generally are controlled by local or state building codes. In addition, designs must satisfy owner requirements and specifications. For buildings on sites not covered by building codes, or for conditions not included in building codes or owner specifications, designers must use their own judgment in selecting design criteria. This chapter has been prepared to aid in understanding current codes and to provide information that will be useful to the designer. It summarizes the requirements of model building codes, particularly regarding wind loads, seismic loads, and fire protection. (See also Chap. 8.)

4.1 BUILDING CODES

A **building code** is a legal ordinance enacted by public bodies, such as city councils, regional planning commissions, states, or federal agencies, establishing regulations governing building design and construction. Building codes are enacted to protect public health, safety, and welfare.

A building code presents minimum requirements to protect the public from harm. It does not necessarily indicate the most efficient or most economical practice.

Building codes specify design techniques in accordance with generally accepted theory. They present rules and procedures that represent generally accepted engineering practices and present knowledge for common conditions.

A building code is a consensus document that relies on information contained in other recognized codes or standard specifications, e.g., national model building codes promulgated by building officials associations and the standards of the American Institute of Steel Construction (AISC), the American Society for Testing and Materials (ASTM), the American Society of Civil Engineers (ASCE), and the American National Standards Institute (ANSI). Information in a building code generally addresses all aspects of building design and construction, e.g., fire protection, mechanical and electrical installations, plumbing installations, design loads and member strengths, types of construction and materials, and safeguards during construction. For its purposes, a building code adopts provisions of other codes

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*Revised and updated from "Building Design Criteria" by R. A. LaBoube, P.E., with contributions from Delbert F. Boring, P.E., Sec. 6 in the Third Edition.

or specifications either by direct reference or with modifications. The two current national model building codes are the International Building Code (IBC) and the National Fire Protection Association (NFPA) 5000.

4.2 APPROVAL OF SPECIAL CONSTRUCTION

Increasing use of specialized types of construction not covered by building codes has stimulated preparation of special-use permits or approvals. Model codes individually and collectively have established formal review procedures that enable manufacturers to attain approval of building products. These code-approval procedures entail a rigorous engineering review of all aspects of product design.

4.3 STANDARD SPECIFICATIONS

Standard specifications are consensus documents sponsored by professional or trade associations to protect public safety and promote responsible use of a product or method. Examples of such specifications are the AISC "Specification for Structural Steel Buildings," the American Iron and Steel Institute (AISI) "Specification for the Design of Cold-Formed Steel Structural Members," the Steel Joist Institute (SJI) "Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders," and the American Welding Society (AWS) "Structural Welding Code-Steel" (AWS D1.1).

Another important class of standard specifications defines acceptable standards of quality of building materials, standard methods of testing, and required workmanship in fabrication and erection. Many of these widely used specifications are developed by the ASTM. As need arises, ASTM specifications are revised to incorporate the latest technological advances. The complete ASTM designation for a specification includes the year in which the latest revision was approved. For example, A992/A992M-05 refers to specification A992, last revised in 2005. The "M" indicates that it includes alternative metric units.

In addition to standards for product design and building materials, there are standard specifications for minimum design loads, e.g., the ASCE "Minimum Design Loads for Buildings and Other Structures" (SEI/ASCE 7-02), and the Metal Building Manufacturers Association "Low-Rise Building Systems Manual."

It is always advisable to use the latest editions of standards, recommended practices, and building codes. Also, it is wise to consult the cited source references for the complete criteria and full context of the provisions.

4.4 BUILDING OCCUPANCY LOADS

Safe yet economical building designs necessitate application of reasonable and prudent design loads. Computation of design loads can require a complex analysis involving such considerations as building end use, location, and geometry.

4.4.1 Building Code-Specified Loads

Before initiating a design, engineers must become familiar with the load requirements of the local building code. All building codes specify minimum design loads. These include, when applicable, dead, live, wind, earthquake, and impact loads, as well as earth pressures.

Dead, floor live, and roof live loads are considered vertical loads and generally are specified as force per unit area, e.g., lb/ft^2 or kPa. These loads are often referred to as **gravity loads.** In some cases, concentrated dead or live loads also must be considered. Wind loads are assumed to act normal to building surfaces and are expressed as pressures, e.g., lb/ft^2 or kPa. Depending on the direction of the wind and the geometry of the structure, wind loads may exert either a positive or negative pressure on a building surface.

All building codes and project specifications require that a building have sufficient strength to resist imposed loads without exceeding the available strength of any element of the structure. Of equal importance to design strength is the design requirement that a building be functional as stipulated by serviceability considerations. Serviceability requirements are often given as allowable or permissible maximum static deflections, either vertical or horizontal, or both. They may also be in the form of dynamic response characteristics, such as natural frequency or acceleration.

4.4.2 Dead Loads

The dead load of a building includes weights of walls, permanent partitions, floors, roofs, framing, fixed service equipment, and all other permanent construction (Tables 4.1 and 4.2). The ASCE standard, "Minimum Design Loads for Buildings and Other Structures" (SEI/ASCE 7-02), gives detailed information regarding computation of dead loads for both normal and special considerations.

4.4.3 Floor Live Loads

Typical requirements for live loads on floors for different occupancies are summarized in Table 4.3. These minimum design loads may differ from requirements of local or state building codes or project specifications. The engineer of record for the building to be constructed is responsible for determining the appropriate load requirements.

Temporary or movable partitions should be considered a floor live load. For structures designed for live loads exceeding 80 lb/ft^2, however, the effect of partitions may be ignored, if permitted by the local building code.

Live Load Reduction. Because of the small probability that a member supporting a large floor area will be subjected to full live loading over the entire area, building codes permit a reduced live load based on the areas contributing loads to the member.

The ASCE standard, "Minimum Design Loads for Buildings and Other Structures" (SEI/ASCE 7-02), permits a reduced live load L (lb/ft^2) computed from the following for design of members with a value of $K_{LL}A_T$ of 400 ft^2 or more:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \tag{4.1}$$

where L_o = unreduced design live load (psf) supported by the member
K_{LL} = live load element factor (see Table 4.4)
A_T = tributary area (ft^2)

The tributary area A_T for one-way slabs must not exceed the area defined by the slab span multiplied by the width normal to the span, nor an area equal to 1.5 times the slab span squared. The reduced live load should not be less than $0.5L_o$ for members supporting one floor nor $0.4L_o$ for all other loading situations. If live loads exceed 100 lb/ft^2, and for garages for passenger cars only, design live loads may be reduced 20% for members supporting more than one floor. For members supporting garage floors, roofs, or areas used for public assembly, no reduction is permitted if the design live load is 100 lb/ft^2 or less.

4.4.4 Concentrated Loads

Some building codes require that members be designed to support a specified concentrated live load in addition to the uniform live load. The concentrated live load may be assumed to be uniformly distributed over an area of 2.5 ft^2 and located to produce the maximum load effects in the members. Table 4.3 lists some typical concentrated loads that may be specified in building codes.

TABLE 4.1 Minimum Design Dead Loads*

Component	Load, lb/ft^2	Component	Load, lb/ft^2
CEILINGS		COVERINGS, ROOF, AND WALL (*cont.*)	
Acoustical fiber board	1	Insulation, roof boards (per inch thickness):	
Gypsum board (per mm thickness)	0.55	Cellular glass	0.7
Mechanical duct allowance	4	Fibrous glass	1.1
Plaster on tile or concrete	5	Fiberboard	1.5
Plaster on wood lath	8	Perlite	0.8
Suspended steel channel system	2	Polystyrene foam	0.2
Suspended metal lath and cement plaster	15	Urethane foam with skin	0.5
Suspended metal lath and gypsum plaster	10	Plywood (per $^1/_8$-in thickness)	0.4
Wood furring suspension system	2.5	Rigid insulation, $^1/_2$-in	0.75
COVERINGS, ROOF, AND WALL		Skylight, metal frame, $^3/_8$-in wire glass	8
Asbestos-cement shingles	4	Slate, $^3/_{16}$-in	7
Asphalt shingles	2	Slate, $^1/_4$-in	10
Cement tile	16	Waterproofing membranes:	
Clay tile (for mortar add 10 lb/ft^2):		Bituminous, gravel-covered	5.5
Book tile, 2-in	12	Bituminous, smooth surface	1.5
Book tile, 3-in	20	Liquid applied	1
Ludowici	10	Single-ply, sheet	0.7
Roman	12	Wood sheathing (per inch thickness)	3
Spanish	19	Wood shingles	3
Composition:		FLOOR FILL	
Three-ply ready roofing	1	Cinder concrete, per inch	9
Four-ply felt and gravel	5.5	Lightweight concrete, per inch	8
Five-ply felt and gravel	6	Sand, per inch	8
Copper or tin	1	Stone concrete, per inch	12
Corrugated asbestos-cement roofing	4	FLOORS AND FLOOR FINISHES	
Deck, metal, 20 gage	2.5	Asphalt block (2-in), $^1/_2$-in mortar	30
Deck, metal, 18 gage	3	Cement finish (1-in) on stone-concrete fill	32
Decking, 2-in wood (Douglas fir)	5	Ceramic or quarry tile ($^3/_4$-in) on $^1/_2$-in mortar bed	16
Decking, 3-in wood (Douglas fir)	8	Ceramic or quarry tile ($^3/_4$-in) on 1-in mortar bed	23
Fiberboard, $^1/_2$-in	0.75	Concrete fill finish (per inch thickness)	12
Gypsum sheathing, $^1/_2$-in	2	Hardwood flooring, $^7/_7$-in	4

(*Continued*)

TABLE 4.1 Minimum Design Dead Loads* (*Continued*)

Component				Load, lb/ft^2
FLOORS AND FLOOR FINISHES (*cont.*)				
Linoleum or asphalt tile, $^{1}/_{4}$-in				1
Marble and mortar on stone-concrete fill				33
Slate (per mm thickness)				15
Solid flat tile on 1-in mortar base				23
Subflooring, $^{3}/_{4}$-in				3
Terrazzo ($1^{1}/_{2}$-in) directly on slab				19
Terrazzo (1-in) on stone-concrete fill				32
Terrazzo (1-in), 2-in stone concrete				32
Wood block (3-in) on mastic, no fill				10
Wood block (3-in) on $^{1}/_{2}$-in mortar base				16
FLOORS, WOOD-JOIST (NO PLASTER)				
DOUBLE WOOD FLOOR				
Joist sizes (in)	12-in spacing (lb/ft^2)	16-in spacing (lb/ft^2)	24-in spacing (lb/ft^2)	
2 × 6	6	5	5	
2 × 8	6	6	5	
2 × 10	7	6	6	
2 × 12	8	7	6	
FRAME PARTITIONS				
Movable steel partitions				4
Wood or steel studs, $^{1}/_{2}$-in gypsum board each side				8
Wood studs, 2 × 4, unplastered				4
Wood studs, 2 × 4, plastered one side				12
Wood studs, 2 × 4, plastered two sides				20
FRAME WALLS				
Exterior stud walls:				
2 × 4 @ 16-in, $^{5}/_{8}$-in gypsum, insulated, $^{3}/_{8}$-in siding				11
2 × 6 @ 16-in, $^{5}/_{8}$-in gypsum, insulated, $^{3}/_{8}$-in siding				12
Exterior stud walls with brick veneer				48
Windows, glass, frame and sash				8

Component						Load, lb/ft^2
FRAME WALLS (*cont.*)						
Clay brick wythes:						
4 in						39
8 in						79
12 in						115
16 in						155
Hollow concrete masonry unit wythes:						
Wythe thickness (in inches)		4	6	8	10	12
Density of unit (16.49 kN/m^3)						
No grout		22	24	31	37	43
48″ o.c.			29	38	47	55
40″ o.c.	grout		30	40	49	57
32″ o.c.	spacing		32	42	52	61
24″ o.c.			34	46	57	67
16″ o.c.			40	53	66	79
Full grout			55	75	95	115
Density of unit (125 lb/ft^3)						
No grout		26	28	36	44	50
48″ o.c.			33	44	54	62
40″ o.c.	grout		34	45	56	65
32″ o.c.	spacing		36	47	58	68
24″ o.c.			39	51	63	75
16″ o.c.			44	59	73	87
Full grout			59	81	102	123
Density of unit (21.21 kN/m^3)						
No grout		29	30	39	47	54
48″ o.c.			36	47	57	66
40″ o.c.	grout		37	48	59	69
32″ o.c.	spacing		38	50	62	72
24″ o.c.			41	54	67	78
16″ o.c.			46	61	76	90
Full grout			62	83	105	127
Solid concrete masonry unit wythes:						
Wythe thickness (in mm)		4	6	8	10	12
Density of unit (105 lb/ft^3)		32	51	69	87	105
Density of unit (125 lb/ft^3)		38	60	81	102	124
Density of unit (135 lb/ft^3)		41	64	87	110	133

*Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft^2 for each face plastered. Values given represent averages. In some cases, there is a considerable range of weight for the same construction.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.2 Minimum Densities for Design Loads from Materials

Material	Load, lb/ft^3
Aluminum	170
Bituminous products	
Asphaltum	81
Graphite	135
Paraffin	56
Petroleum, crude	55
Petroleum, refined	50
Petroleum, benzine	46
Petroleum, gasoline	42
Pitch	69
Tar	75
Brass	526
Bronze	552
Cast-stone masonry (cement, stone, sand)	144
Cement, portland, loose	90
Ceramic tile	150
Charcoal	12
Cinder fill	57
Cinders, dry, in bulk	45
Coal	
Anthracite, piled	52
Bituminous, piled	47
Lignite, piled	47
Peat, dry, piled	23
Concrete, plain	
Cinder	108
Expanded-slag aggregate	100
Haydite (burned-clay aggregate)	90
Slag	132
Stone (including gravel)	144
Vermiculite and perlite aggregate, non-load-bearing	25–50
Other light aggregate, load-bearing	70–105
Concrete, reinforced	
Cinder	111
Slag	138
Stone (including gravel)	150
Copper	556
Cork, compressed	14
Earth (not submerged)	
Clay, dry	63
Clay, damp	110
Clay and gravel, dry	100
Silt, moist, loose	78
Silt, moist, packed	96
Silt, flowing	108
Sand and gravel, dry, loose	100
Sand and gravel, dry, packed	110
Sand and gravel, wet	120
Earth (submerged)	
Clay	80
Soil	70
River mud	90
Sand or gravel	60
Sand or gravel and clay	65
Glass	160
Gravel, dry	104
Gypsum, loose	70
Gypsum, wallboard	50
Ice	57
Iron	
Cast	450
Wrought	48
Lead	710
Lime	
Hydrated, loose	32
Hydrated, compacted	45
Masonry, Ashlar stone	
Granite	165
Limestone, crystalline	165
Limestone, oolitic	135
Marble	173
Sandstone	144
Masonry, brick	
Hard (low absorption)	130
Medium (medium absorption)	115
Soft (high absorption)	100
Masonry, concrete*	
Lightweight units	105
Medium weight units	125
Normal weight units	135
Masonry, grout	140
Masonry, rubble stone	
Granite	153
Limestone, crystalline	147
Limestone, oolitic	138
Marble	156
Sandstone	137
Mortar, cement or lime	130
Particleboard	45
Plywood	36
Riprap (not submerged)	
Limestone	83
Sandstone	90
Sand	
Clean and dry	90
River, dry	106
Slag	
Bank	70
Bank screenings	108
Machine	96
Sand	52
Slate	172
Steel, cold-drawn	492
Stone, quarried, piled	
Basalt, granite, gneiss	96
Limestone, marble, quartz	95
Sandstone	82
Shale	92
Greenstone, hornblende	107

(*Continued*)

TABLE 4.2 Minimum Densities for Design Loads from Materials (*Continued*)

Material	Load, lb/ft^3	Material	Load, lb/ft^3
Terra cotta, architectural		Wood, seasoned (*cont.*)	
Voids filled	120	Fir, Douglas, coast region	34
Voids unfilled	72	Hem fir	28
Tin	459	Oak, commercial reds and whites	47
Water		Pine, southern yellow	37
Fresh	62	Redwood	28
Sea	64	Spruce, red, white, and Sitka	29
Wood, seasoned		Western hemlock	32
Ash, commercial white	41	Zinc, rolled sheet	449
Cypress, southern	34		

*Tabulated values apply to solid masonry and to the solid portion of hollow masonry.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

4.4.5 Pattern (Partial) Loading

Pattern or partial loading is an arrangement of live loads that produces maximum possible stresses at a point in a structure or member such as a continuous beam. The member carries full dead and live loads, but full live load may occur only in alternating spans or some combination of spans. In a high-rise building frame, maximum positive moments are produced by a checkerboard pattern of live load, i.e., by full live load on alternate spans horizontally and alternate bays vertically. Maximum negative moments at a joint occur, for most practical purposes, with full live loads only on the continuous spans directly adjacent to the joint. Thus, pattern loading may produce critical bending moments in such members and should be investigated.

4.5 ROOF LOADS

In northern areas, roof loads are determined by the expected maximum snow loads. However, in southern areas, where snow accumulation is not a problem, minimum roof live loads are specified to accommodate the weight of workers, equipment, and materials during maintenance and repair.

4.5.1 Roof Live Loads

SEI/ASCE 7-02 requires that structural members in flat, pitched, or curved roofs be designed for a live load L_r (lb/ft^2 of horizontal projection) computed from

$$L_r = 20R_1R_2 \geq 12 \tag{4.2}$$

where R_1 = reduction factor for size of tributary area
$= 1$ for $A_T \leq 200$
$= 1.2–0.001A_T$ for $200 < A_T < 600$
$= 0.6$ for $A_T \geq 600$
A_T = tributary area, or area contributing load to the structural member, ft^2 (Art. 4.4.3)
R_2 = reduction factor for slope of roof
$= 1$ for $F \leq 4$
$= 1.2–0.05F$ for $4 < F < 12$
$= 0.6$ for $F \geq 12$
F = rate of rise for a pitched roof, in/ft
= rise-to-span ratio multiplied by 32 for an arch or dome

TABLE 4.3 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or use	Uniform lb/ft^2 (kN/m^2)	Conc. lb (kN)
Apartments (see residential)		
Access floor systems		
Office use	50 (2.4)	2000 (8.9)
Computer use	100 (4.79)	2000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)	
Lobbies	100 (4.79)	
Movable seats	100 (4.79)	
Platforms (assembly)	100 (4.79)	
Stage floors	150 (7.18)	
Balconies (exterior)	100 (4.79)	
On one- and two-family residences only, and not exceeding 100 ft^2 (9.3 m^2)	60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwelling (see residential)		
Elevator machine room grating [on area of 4 in^2 (2580 mm^2)]		300 (1.33)
Finish light floor plate construction [on area of 1 in^2 (645 mm^2)]		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders		See Sec. 4.4
Garages (passenger vehicles only)	40 (1.92)	Note (1)
Trucks and buses	Note (2)	
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	100 (4.79) [Note (4)]	
Handrails, guardrails, and grab bars	See Sec. 4.4	
Hospitals		
Operating rooms, laboratories	60 (2.87)	1000 (4.45)
Private rooms	40 (1.92)	1000 (4.45)
Wards	40 (1.92)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Hotels (see residential)		
Libraries		
Reading rooms	60 (2.87)	1000 (4.45)
Stack rooms	150 (7.18) [Note (3)]	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Manufacturing		
Light	125 (6.00)	2000 (8.90)
Heavy	250 (11.97)	3000 (13.40)
Marquees and canopies	75 (3.59)	
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100 (4.79)	2000 (8.90)
Offices	50 (2.40)	2000 (8.90)
Corridors above first floor	80 (3.83)	2000 (8.90)

(*Continued*)

TABLE 4.3 Minimum Uniformly Distributed Live Loads, L_O, and Minimum Concentrated Live Loads (*Continued*)

Occupancy or use	Uniform lb/ft^2 (kN/m^2)	Conc. lb (kN)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79) [Note (4)]	
Roofs	See Secs. 4.3 and 4.9	
Schools		
Classrooms	40 (1.92)	1000 (4.45)
Corridors above first floor	80 (1.92)	1000 (4.45)
First-floor corridors	100 (4.79)	1000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (9.58)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97) [Note (5)]	8000 (35.60) [Note (6)]
Stadiums and arenas		
Bleachers	100 (4.79) [Note (4)]	
Fixed seats (fastened to floor)	60 (2.87) [Note (4)]	
Stairs and exit-ways	100 (4.79)	[Note (7)]
One- and two-family residences only	40 (1.92)	
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)	
Heavy	250 (11.97)	
Stores		
Retail		
First floor	100 (4.79)	1000 (4.45)
Upper floors	75 (3.59)	1000 (4.45)
Wholesale, all floors	125 (6.00)	1000 (4.45)
Vehicle barriers	See Sec. 4.4	
Walkways and elevated platforms (other than exit-ways)	60 (2.87)	
Yards and terraces, pedestrians	100 (4.79)	

Notes:

(1) Floors in garages or portions of building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4.3 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3000 lb (13.35 kN) acting on an area of 4.5 in by 4.5 in (114 mm by 114 mm, footprint of a jack); (2) for mechanical parking structures without slab or deck which are used for storing passenger car only, 2250 lb (10 kN) per wheel.

(2) Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

(3) The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:
  a. The nominal bookstack unit height shall not exceed 90 in (2290 mm).
  b. The nominal shelf depth shall not exceed 12 in (305 mm) for each face.
  c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 36 in (914 mm) wide.

(4) In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lb/linear ft of seat applied in a direction parallel to each row of seats and 10 lb/ linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

(5) Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

(6) The concentrated wheel load shall be applied on an area of 4.5 in by 4.5 in (114 mm by 114 mm, footprint of a jack).

(7) Minimum concentrated load on stair treads [on area of 4 in^2 (2580 mm^2)] is 300 lb (1.33 kN).

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.4 Live Load Element Factor, K_{LL}

Element	K_{LL}*
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified above, including: Edge beams with cantilever slabs Cantilever beams One-way slabs Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

*In lieu of the values above, K_{LL} is permitted to be calculated.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

This roof live load can be conveniently summarized as shown in Table 4.5.

4.5.2 Snow Loads

Determination of design snow loads for roofs is often based on the maximum ground snow load in a 50-year mean recurrence period (2% probability of being exceeded in any year). This load, or data for computing it from an extreme-value statistical analysis of weather records of snow on the ground, may be obtained from the local building code or the National Weather Service. Maps showing ground snow loads for various regions are presented in model building codes and standards, such as the ASCE standard, “Minimum Design Loads for Buildings and Other Structures” (SEI/ASCE 7-02). The map scales, however, may be too small for use for some regions, especially where the amount of local variation is extreme or high country is involved.

Some building codes and SEI/ASCE 7-02 specify an equation that takes into account the consequences of a structural failure in view of the end use of the building to be constructed and the wind exposure of a flat (low-slope) roof:

$$p_f = 0.7C_eC_tIp_g \tag{4.3}$$

where C_e = wind exposure factor (Table 4.6)
C_t = thermal effects factor (Table 4.7)
I = importance factor for end use (Table 4.8)
p_f = roof snow load, lb/ft^2
= Ip_g when $p_g \leq 20$ lb/ft^2
= $20I$ when $p_g > 20$ lb/ft^2
p_g = ground snow load for 50-year recurrence period, lb/ft^2

TABLE 4.5 Roof Live Loads

	Tributary loaded area (A_t) in ft^2 for any structural member		
Roof slope, F:12	$A_t \leq 200$	$200 < A_t < 600$	$A_t \geq 600$
$F \leq 4$	20	20 (1.2–0.001 A_t)	12
$4 < F < 12$	20 (1.2–0.05F)	20 (1.2–0.001 A_t) (1.2–0.05F) ≥ 12	12
$F \geq 12$	12	12	12

Source: From *Metal Buildings System Manual*, 2002, Metal Buildings Manufacturers Association, Cleveland, Ohio, with permission.

TABLE 4.6 Wind Exposure Factor C_e

	Exposure of roof*		
Terrain category	Fully exposed	Partially exposed	Sheltered
A (see Sec. 6.5.6)	N/A	1.1	1.3
B (see Sec. 6.5.6)	0.9	1.0	1.2
C (see Sec. 6.5.6)	0.9	1.0	1.1
D (see Sec. 6.5.6)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2-mi (3-km) radius of the site.	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

*Definitions:

Partially exposed: All roofs except as indicated below.

Fully exposed: Roofs exposed on all sides with no shelter† afforded by terrain, structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load (h_b), or other obstructions are not in this category.

Sheltered: Roofs located tight in among conifers that qualify as obstructions.

†Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used except for terrain Category "A." Note that these are heights above the roof. Heights used to establish the terrain category in Sec. 6.5.3 are heights above the ground.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.7 Thermal Factor C_t

Thermal condition*	C_t
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R/value) between the ventilated space and the heated space exceeds 25°F·h·ft^2/Btu (4.4 K·m^2/W)	1.1
Unheated structures and structures intentionally kept below freezing	1.2
Continuously heated greenhouses† with a roof having a thermal resistance (R-value) less than 2.0°F·h·ft^2/Btu (0.4 K·m^2/W)	0.85

*These conditions shall be representative of the anticipated conditions during winters for the life of the structure.

†Greenhouses with a constantly maintained interior temperature of 50°F (10°C) or more at any point 3 ft above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.8 Importance Factor for Snowloads, Eq. (4.3)

Category*	Importance factor I
I	0.8
II	1.0
III	1.1
IV	1.2

*See Table 4.10 for description of categories.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

In their provisions for roof design, codes and standards also allow for the effect of roof slopes, snow drifts, unbalanced snow loads, rain-on-snow surcharge, and ponding instability. The structural members should be investigated for the maximum possible load effects that might be induced.

4.6 WIND LOADS

Wind loads are randomly applied dynamic loads. The intensity of the wind pressure on the surface of a structure depends on wind velocity, air density, orientation of the structure, area of contact surface, and shape of the structure. Because of the complexity involved in defining both the dynamic wind load and the behavior of an indeterminate steel structure when subjected to wind loads, the design criteria adopted by building codes and standards have been based on the application of an equivalent static wind pressure. This simplified equivalent static design wind pressure p_s (lb/ft^2) is defined by SEI/ASCE 7-02 as

$$p_s = \lambda I_w p_{S30} \tag{4.4}$$

where p_s = simplified net design wind pressure for the main wind force-resisting system of low-rise, simple, regular, enclosed, diaphragm buildings (lb/ft^2)
λ = adjustment factor for building height and exposure
I_w = importance factor for wind loads
p_{S30} = net design wind pressure for Exposure B, at height of 30 ft, with $I_w = 1.0$

For low-rise buildings with conditions discussed below that qualify for application of this simplified method, SEI/ASCE 7-02 Method 1, the net wind design pressure, p_{net}, on the components and cladding can be similarly obtained from Eq. (4.4) by substitution of appropriate variables.

SEI/ASCE 7-02 specifies a minimum net wind pressure of 10 lb/ft^2 for the main wind-force resisting system as well as the components and cladding.

Velocity pressure is computed from

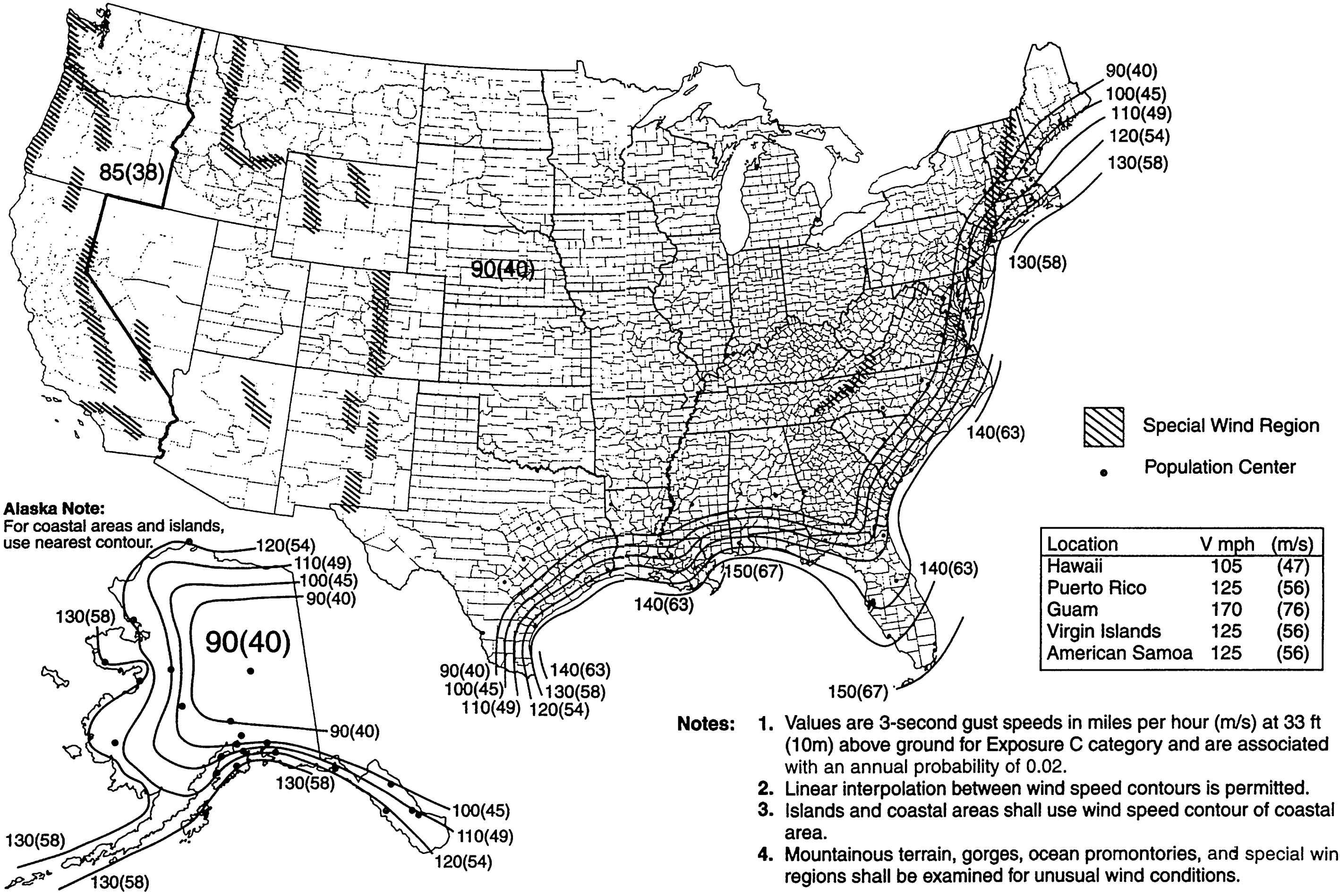
$$q_z = 0.00256 K_z K_{zt} K_d V^2 I_w \tag{4.5}$$

where K_z = velocity exposure coefficient evaluated at height z
K_{zt} = topographic factor
K_d = wind directionality factor
I_w = importance factor
V = basic wind speed (mph) corresponding to a 3-s gust speed at 33 ft above the ground in Exposure C

Velocity pressures due to wind to be used in building design vary with type of terrain, distance above ground level, importance of building, likelihood of hurricanes, and basic wind speed recorded near the building site. The wind pressures are assumed to act horizontally on the building area projected on a vertical plane normal to the wind direction.

Unusual wind conditions often occur over rough terrain and around ocean promontories. Basic wind speeds applicable to such regions should be selected with the aid of meteorologists and the application of extreme-value statistical analysis to anemometer readings taken at or near the site of the proposed building. Generally, however, minimum basic wind velocities are specified in local building codes and in national model building codes but should be used with discretion, because actual velocities at a specific site and on a specific building may be significantly larger. In the absence of code specifications and reliable data, basic wind speed at a height of 10 m above grade may be estimated from Fig. 4.1.

For design purposes, wind pressures should be determined in accordance with the degree to which terrain surrounding the proposed building exposes it to the wind. Exposures are defined in Table 4.6.



Alaska Note:
For coastal areas and islands, use nearest contour.

Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	125	(56)
Guam	170	(76)
Virgin Islands	125	(56)
American Samoa	125	(56)

Notes:

1. Values are 3-second gust speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category and are associated with an annual probability of 0.02.
2. Linear interpolation between wind speed contours is permitted.
3. Islands and coastal areas shall use wind speed contour of coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

FIGURE 4.1 Contours on map of the United States showing basic wind speeds for open terrain and grasslands with 50-year mean recurrence interval. (*Source:* Minimum Design Loads for Buildings and Other Structures, *SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.*).

SEI/ASCE 7-02 permits the use of either Method 1 or Method 2 to define the design wind loads. Method 1 is a simplified procedure and may be used for enclosed or partially enclosed buildings meeting the following conditions:

1. Wind loads are transmitted through floor and roof diaphragms to the vertical main wind force-resisting system in each direction with either a flat roof or a gable or hip roof with angle ≤ 45°.
2. Approximately symmetrical cross section of building with roof slopes less than 10°.
3. Low-rise building, i.e., mean roof height less than or equal to 60 ft and height does not exceed the least horizontal dimension.
4. Building having no unusual geometric irregularity in spatial form.
5. Building whose fundamental frequency is greater than 1 Hz.
6. Building structure having no expansion joints or separations.
7. Building is not subject to topographical effects.

The design procedure for Method 1 involves the following considerations:

1. Determine the basic design speed, V, from Fig. 4.1.
2. Select the wind importance factor, I_w, using Table 4.9.
3. Define the exposure category, i.e., A, B, C, or D, using Table 4.6.
4. Define the building enclosure classification, i.e., enclosed or partially enclosed.
5. Using Fig. 4.2, determine the design wind load for the main wind force-resisting system.
6. Using Fig. 4.3, determine the design wind load for the component and cladding elements.

Method 2 is a rigorous computation procedure that accounts for the external and internal pressure variation as well as gust effects. The following is the general equation for computing the design wind pressure p:

$$p = qGC_p - q_i(GC_{\text{pi}}) \tag{4.6}$$

where q and q_i = velocity pressure as given by SEI/ASCE 7-02
G = gust effect factor as given by SEI/ASCE 7-02
C_p = external pressure coefficient as given by SEI/ASCE 7-02
GC_{pi} = internal pressure coefficient as given by SEI/ASCE 7-02

Building codes and standards may present the gust factors and pressure coefficients in different formats. Coefficients from different codes and standards should not be mixed.

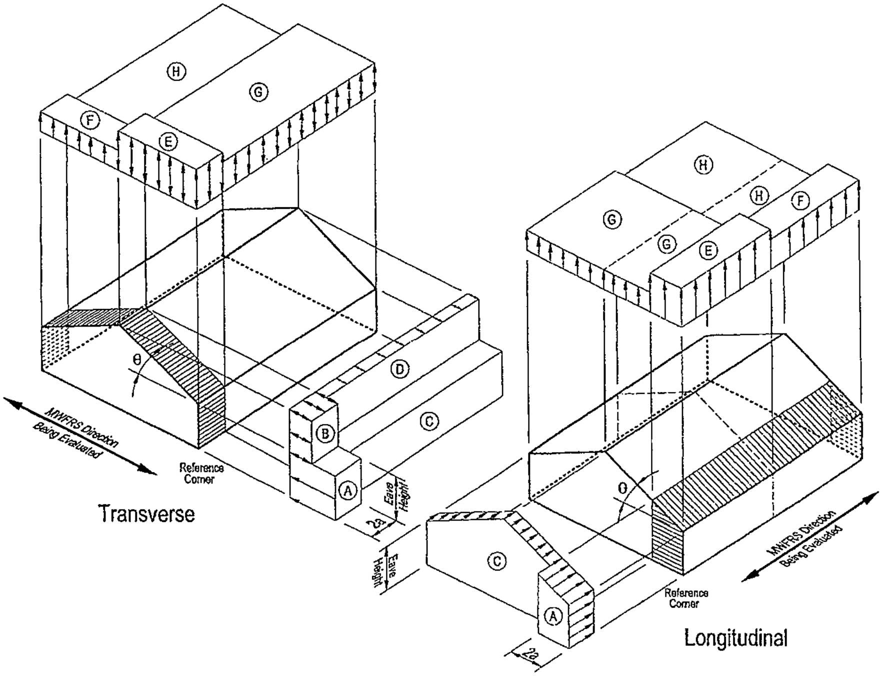
Designers must exercise judgment in selecting wind loads for a building with unusual shape, response-to-load characteristics, or site exposure where channeling of wind currents or buffeting in

TABLE 4.9 Importance Factor *I* for Wind Loads

Category	Non-hurricane-prone regions and hurricane-prone regions with V = 85–100 mi/h and Alaska	Hurricane-prone regions with V > 100 mi/h
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Note: The building and structure classification categories are listed in Table 4.9.
Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

Main Wind Force Resisting System – Method 1		**h ≤ 60 ft**
	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h=30 ft (9.1m), for I=1.0. Adjust to other exposures and heights with adjustment factor λ.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner.
3. For the design of the longitudinal MWFRS use $\theta = 0°$, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for $25° < \theta \leq 45°$. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_S = 0$ in zones B & D.
8. The zone pressures represent the following:

   Horizontal pressure zones – Sum of the windward and leeward net (sum of internal and external) pressures on vertical projection of:

   A - End zone of wall
   B - End zone of roof
   C - Interior zone of wall
   D - Interior zone of roof

   Vertical pressure zones – Net (sum of internal and external) pressures on horizontal projection of:

   E - End zone of windward roof
   F - End zone of leeward roof
   G - Interior zone of windward roof
   H - Interior zone of leeward roof
9. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
10. Notation:

    a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

    h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.

    θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 4.2 Simplified design wind load (Method I) for main wind force-resisting system. (*Source:* Minimum Design Loads for Buildings and Other Structures, *SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.*).

Main Wind Force Resisting System – Method 1 | **h ≤ 60 ft**

Design Wind Pressures

Enclosed Buildings

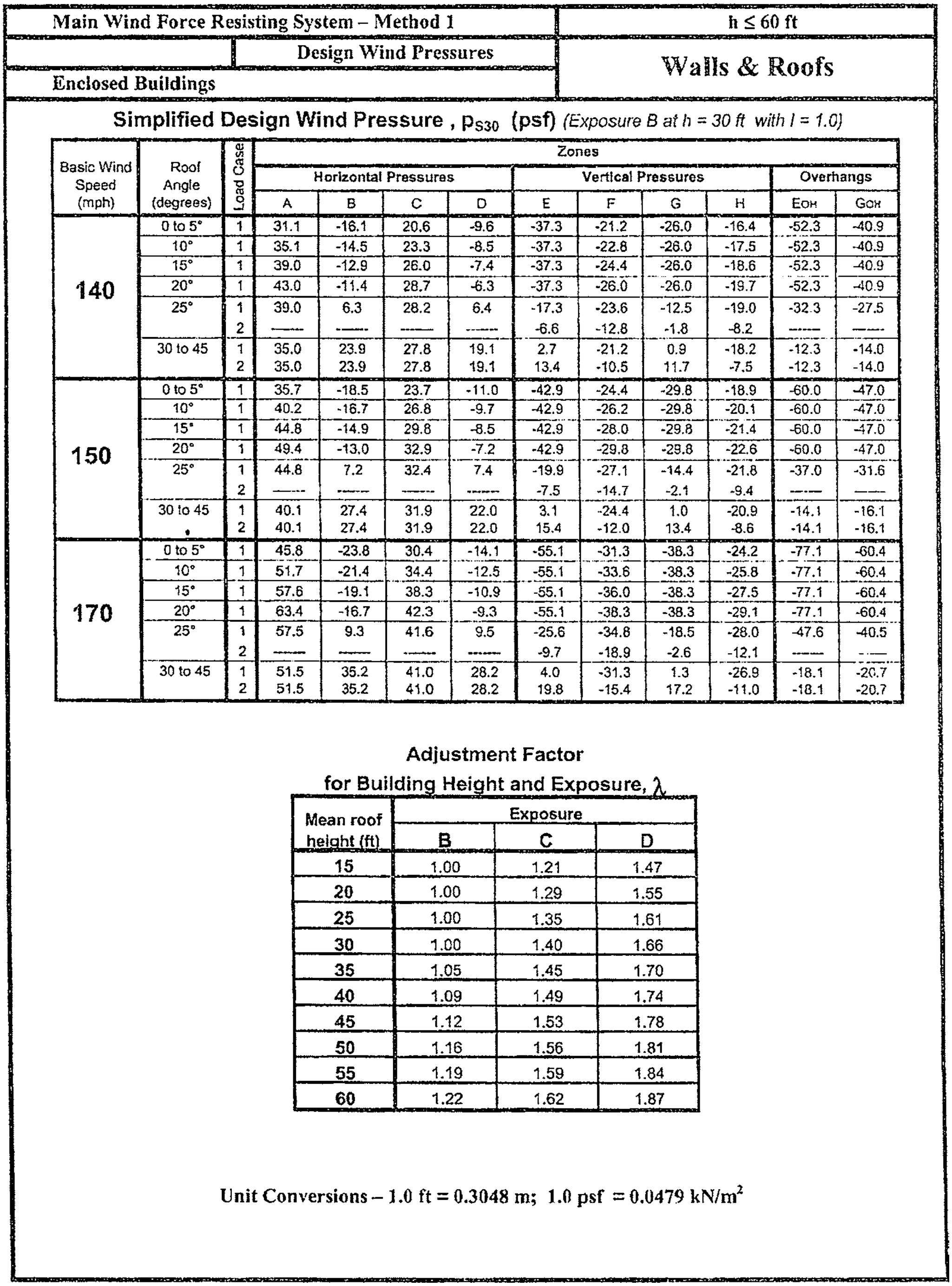
Walls & Roofs

Simplified Design Wind Pressure , p_{S30} (psf) *(Exposure B at h = 30 ft with I = 1.0)*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
85	0 to 5°	1	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1
	10°	1	12.9	-5.4	8.6	-3.1	-13.8	-8.4	-9.6	-6.5	-19.3	-15.1
	15°	1	14.4	-4.8	9.6	-2.7	-13.8	-9.0	-9.6	-6.9	-19.3	-15.1
	20°	1	15.9	-4.2	10.6	-2.3	-13.8	-9.6	-9.6	-7.3	-19.3	-15.1
	25°	1	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7.0	-11.9	-10.1
		2	——	——	——	——	-2.4	-4.7	-0.7	-3.0	——	——
	30 to 45	1	12.9	8.8	10.2	7.0	1.0	-7.8	0.3	-6.7	-4.5	-5.2
		2	12.9	8.8	10.2	7.0	5.0	-3.9	4.3	-2.8	-4.5	-5.2
90	0 to 5°	1	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9
	10°	1	14.5	-6.0	9.6	-3.5	-15.4	-9.4	-10.7	-7.2	-21.6	-16.9
	15°	1	16.1	-5.4	10.7	-3.0	-15.4	-10.1	-10.7	-7.7	-21.6	-16.9
	20°	1	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9
	25°	1	16.1	2.6	11.7	2.7	-7.2	-9.8	-5.2	-7.8	-13.3	-11.4
		2	——	——	——	——	-2.7	-5.3	-0.7	-3.4	——	——
	30 to 45	1	14.4	9.9	11.5	7.9	1.1	-8.8	0.4	-7.5	-5.1	-5.8
		2	14.4	9.9	11.5	7.9	5.6	-4.3	4.8	-3.1	-5.1	-5.8
100	0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9
	10°	1	17.9	-7.4	11.9	-4.3	-19.1	-11.6	-13.3	-8.9	-26.7	-20.9
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-12.4	-13.3	-9.5	-26.7	-20.9
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9
	25°	1	19.9	3.2	14.4	3.3	-8.8	-12.0	-6.4	-9.7	-16.5	-14.0
		2	——	——	——	——	-3.4	-6.6	-0.9	-4.2	——	——
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	-10.8	0.5	-9.3	-6.3	-7.2
		2	17.8	12.2	14.2	9.8	6.9	-5.3	5.9	-3.8	-6.3	-7.2
110	0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
	10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0
		2	——	——	——	——	-4.1	-7.9	-1.1	-5.1	——	——
	30 to 45	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7
120	0 to 5°	1	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
	10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
	15°	1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1
	20°	1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1
	25°	1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-23.7	-20.2
		2	——	——	——	——	-4.8	-9.4	-1.3	-6.0	——	——
	30 to 45	1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3
130	0 to 5°	1	26.8	-13.9	17.8	-8.2	-32.2	-18.3	-22.4	-14.2	-45.1	-35.3
	10°	1	30.2	-12.5	20.1	-7.3	-32.2	-19.7	-22.4	-15.1	-45.1	-35.3
	15°	1	33.7	-11.2	22.4	-6.4	-32.2	-21.0	-22.4	-16.1	-45.1	-35.3
	20°	1	37.1	-9.8	24.7	-5.4	-32.2	-22.4	-22.4	-17.0	-45.1	-35.3
	25°	1	33.6	5.4	24.3	5.5	-14.9	-20.4	-10.8	-16.4	-27.8	-23.7
		2	——	——	——	——	-5.7	-11.1	-1.5	-7.1	——	——
	30 to 45	1	30.1	20.6	24.0	16.5	2.3	-18.3	0.8	-15.7	-10.6	-12.1
		2	30.1	20.6	24.0	16.5	11.6	-9.0	10.0	-6.4	-10.6	-12.1

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m^2

FIGURE 4.2 (*Continued*)



Main Wind Force Resisting System – Method 1 | **h ≤ 60 ft**

Design Wind Pressures

Walls & Roofs

Enclosed Buildings

Simplified Design Wind Pressure , p_{S30} (psf) *(Exposure B at h = 30 ft with I = 1.0)*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
140	0 to 5°	1	31.1	-16.1	20.6	-9.6	-37.3	-21.2	-26.0	-16.4	-52.3	-40.9
	10°	1	35.1	-14.5	23.3	-8.5	-37.3	-22.8	-26.0	-17.5	-52.3	-40.9
	15°	1	39.0	-12.9	26.0	-7.4	-37.3	-24.4	-26.0	-18.6	-52.3	-40.9
	20°	1	43.0	-11.4	28.7	-6.3	-37.3	-26.0	-26.0	-19.7	-52.3	-40.9
	25°	1	39.0	6.3	28.2	6.4	-17.3	-23.6	-12.5	-19.0	-32.3	-27.5
		2	——	——	——	——	-6.6	-12.8	-1.8	-8.2	——	——
	30 to 45	1	35.0	23.9	27.8	19.1	2.7	-21.2	0.9	-18.2	-12.3	-14.0
		2	35.0	23.9	27.8	19.1	13.4	-10.5	11.7	-7.5	-12.3	-14.0
150	0 to 5°	1	35.7	-18.5	23.7	-11.0	-42.9	-24.4	-29.8	-18.9	-60.0	-47.0
	10°	1	40.2	-16.7	26.8	-9.7	-42.9	-26.2	-29.8	-20.1	-60.0	-47.0
	15°	1	44.8	-14.9	29.8	-8.5	-42.9	-28.0	-29.8	-21.4	-60.0	-47.0
	20°	1	49.4	-13.0	32.9	-7.2	-42.9	-29.8	-29.8	-22.6	-60.0	-47.0
	25°	1	44.8	7.2	32.4	7.4	-19.9	-27.1	-14.4	-21.8	-37.0	-31.6
		2	——	——	——	——	-7.5	-14.7	-2.1	-9.4	——	——
	30 to 45	1	40.1	27.4	31.9	22.0	3.1	-24.4	1.0	-20.9	-14.1	-16.1
		2	40.1	27.4	31.9	22.0	15.4	-12.0	13.4	-8.6	-14.1	-16.1
170	0 to 5°	1	45.8	-23.8	30.4	-14.1	-55.1	-31.3	-38.3	-24.2	-77.1	-60.4
	10°	1	51.7	-21.4	34.4	-12.5	-55.1	-33.6	-38.3	-25.8	-77.1	-60.4
	15°	1	57.6	-19.1	38.3	-10.9	-55.1	-36.0	-38.3	-27.5	-77.1	-60.4
	20°	1	63.4	-16.7	42.3	-9.3	-55.1	-38.3	-38.3	-29.1	-77.1	-60.4
	25°	1	57.5	9.3	41.6	9.5	-25.6	-34.8	-18.5	-28.0	-47.6	-40.5
		2	——	——	——	——	-9.7	-18.9	-2.6	-12.1	——	——
	30 to 45	1	51.5	35.2	41.0	28.2	4.0	-31.3	1.3	-26.9	-18.1	-20.7
		2	51.5	35.2	41.0	28.2	19.8	-15.4	17.2	-11.0	-18.1	-20.7

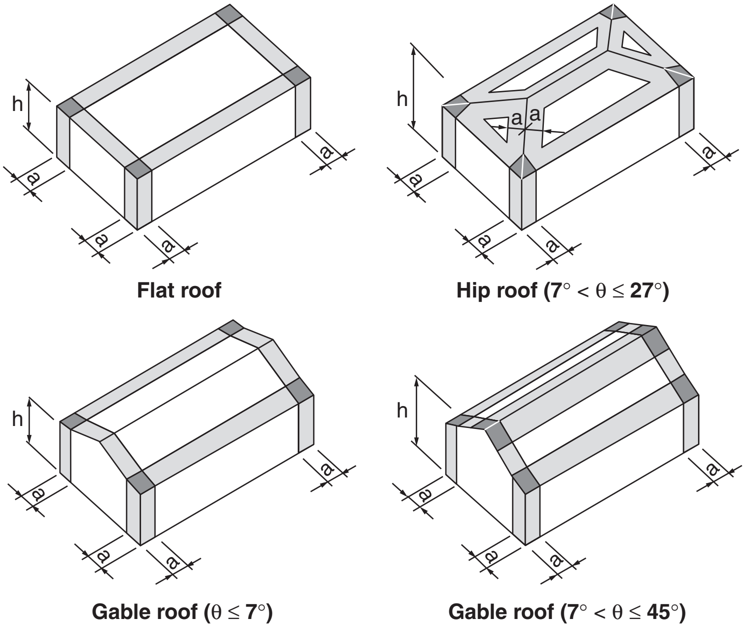
Adjustment Factor
for Building Height and Exposure, λ

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m^2

FIGURE 4.2 (*Continued*)

Components and Cladding–Method 1	h ≤ 60 ft
Design Wind Pressures Enclosed Buildings	Walls & Roofs



☐ Interior zones
Roofs–Zone 1/walls–Zone 4

▨ End zones
Roofs–Zone 2/walls–Zone 5

■ Corner zones
Roofs–Zone 3

Notes:

1. Pressures shown are applied normal to the surface, for exposure B, at h = 30 ft (9.1 m), for I = 1.0. Adjust to other exposures and heights with adjustment factor λ.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For hip roofs with θ ≤ 25°, Zone 3 shall be treated as Zone 2.
4. For effective wind areas between those given, value may be interpolated; otherwise use the value associated with the lower effective wind area.
5. Notation:

a: 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.
θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 4.3 Simplified design wind load (Method 1) for component and cladding elements. (*Source:* Minimum Design Loads for Buildings and Other Structures, *SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.*)

Components and Cladding – Method 1

h ≤ 60 ft

Design Wind Pressures

Walls & Roofs

Enclosed Buildings

Net Design Wind Pressure , p_{net30} (psf) *(Exposure B at h = 30 ft with I = 1.0)*

	Zone	Effective wind area (sf)	Basic Wind Speed V (mph)																	
			85		90		100		110		120		130		140		150		170	
Roof 0 to 7 degrees	1	10	5.3	-13.0	5.9	-14.6	7.3	-18.0	8.9	-21.8	10.5	-25.9	12.4	-30.4	14.3	-35.3	16.5	-40.5	21.1	-52.0
	1	20	5.0	-12.7	5.6	-14.2	6.9	-17.5	8.3	-21.2	9.9	-25.2	11.6	-29.6	13.4	-34.4	15.4	-39.4	19.8	-50.7
	1	50	4.5	-12.2	5.1	-13.7	6.3	-16.9	7.6	-20.5	9.0	-24.4	10.6	-28.6	12.3	-33.2	14.1	-38.1	18.1	-48.9
	1	100	4.2	-11.9	4.7	-13.3	5.8	-16.5	7.0	-19.9	8.3	-23.7	9.8	-27.8	11.4	-32.3	13.0	-37.0	16.7	-47.6
	2	10	5.3	-21.8	5.9	-24.4	7.3	-30.2	8.9	-36.5	10.5	-43.5	12.4	-51.0	14.3	-59.2	16.5	-67.9	21.1	-87.2
	2	20	5.0	-19.5	5.6	-21.8	6.9	-27.0	8.3	-32.6	9.9	-38.8	11.6	-45.6	13.4	-52.9	15.4	-60.7	19.8	-78.0
	2	50	4.5	-16.4	5.1	-18.4	6.3	-22.7	7.6	-27.5	9.0	-32.7	10.6	-38.4	12.3	-44.5	14.1	-51.1	18.1	-65.7
	2	100	4.2	-14.1	4.7	-15.8	5.8	-19.5	7.0	-23.6	8.3	-28.1	9.8	-33.0	11.4	-38.2	13.0	-43.9	16.7	-56.4
	3	10	5.3	-32.8	5.9	-36.8	7.3	-45.4	8.9	-55.0	10.5	-65.4	12.4	-76.8	14.3	-89.0	16.5	-102.2	21.1	-131.3
	3	20	5.0	-27.2	5.6	-30.5	6.9	-37.6	8.3	-45.5	9.9	-54.2	11.6	-63.6	13.4	-73.8	15.4	-84.7	19.8	-108.7
	3	50	4.5	-19.7	5.1	-22.1	6.3	-27.3	7.6	-33.1	9.0	-39.3	10.6	-46.2	12.3	-53.5	14.1	-61.5	18.1	-78.9
	3	100	4.2	-14.1	4.7	-15.8	5.8	-19.5	7.0	-23.6	8.3	-28.1	9.8	-33.0	11.4	-38.2	13.0	-43.9	16.7	-56.4
Roof > 7 to 27 degrees	1	10	7.5	-11.9	8.4	-13.3	10.4	-16.5	12.5	-19.9	14.9	-23.7	17.5	-27.8	20.3	-32.3	23.3	-37.0	30.0	-47.6
	1	20	6.8	-11.6	7.7	-13.0	9.4	-16.0	11.4	-19.4	13.6	-23.0	16.0	-27.0	18.5	-31.4	21.3	-36.0	27.3	-46.3
	1	50	6.0	-11.1	6.7	-12.5	8.2	-15.4	10.0	-18.6	11.9	-22.2	13.9	-26.0	16.1	-30.2	18.5	-34.6	23.8	-44.5
	1	100	5.3	-10.8	5.9	-12.1	7.3	-14.9	8.9	-18.1	10.5	-21.5	12.4	-25.2	14.3	-29.3	16.5	-33.6	21.1	-43.2
	2	10	7.5	-20.7	8.4	-23.2	10.4	-28.7	12.5	-34.7	14.9	-41.3	17.5	-48.4	20.3	-56.2	23.3	-64.5	30.0	-82.8
	2	20	6.8	-19.0	7.7	-21.4	9.4	-26.4	11.4	-31.9	13.6	-38.0	16.0	-44.6	18.5	-51.7	21.3	-59.3	27.3	-76.2
	2	50	6.0	-16.9	6.7	-18.9	8.2	-23.3	10.0	-28.2	11.9	-33.6	13.9	-39.4	16.1	-45.7	18.5	-52.5	23.8	-67.4
	2	100	5.3	-15.2	5.9	-17.0	7.3	-21.0	8.9	-25.5	10.5	-30.3	12.4	-35.6	14.3	-41.2	16.5	-47.3	21.1	-60.8
	3	10	7.5	-30.6	8.4	-34.3	10.4	-42.4	12.5	-51.3	14.9	-61.0	17.5	-71.6	20.3	-83.1	23.3	-95.4	30.0	-122.5
	3	20	6.8	-28.6	7.7	-32.1	9.4	-39.6	11.4	-47.9	13.6	-57.1	16.0	-67.0	18.5	-77.7	21.3	-89.2	27.3	-114.5
	3	50	6.0	-26.0	6.7	-29.1	8.2	-36.0	10.0	-43.5	11.9	-51.8	13.9	-60.8	16.1	-70.5	18.5	-81.0	23.8	-104.0
	3	100	5.3	-24.0	5.9	-26.9	7.3	-33.2	8.9	-40.2	10.5	-47.9	12.4	-56.2	14.3	-65.1	16.5	-74.8	21.1	-96.0
Roof > 27 to 45 degrees	1	10	11.9	-13.0	13.3	-14.6	16.5	-18.0	19.9	-21.8	23.7	-25.9	27.8	-30.4	32.3	-35.3	37.0	-40.5	47.6	-52.0
	1	20	11.6	-12.3	13.0	-13.8	16.0	-17.1	19.4	-20.7	23.0	-24.6	27.0	-28.9	31.4	-33.5	36.0	-38.4	46.3	-49.3
	1	50	11.1	-11.5	12.5	-12.8	15.4	-15.9	18.6	-19.2	22.2	-22.8	26.0	-26.8	30.2	-31.1	34.6	-35.7	44.5	-45.8
	1	100	10.8	-10.8	12.1	-12.1	14.9	-14.9	18.1	-18.1	21.5	-21.5	25.2	-25.2	29.3	-29.3	33.6	-33.6	43.2	-43.2
	2	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	19.9	-25.5	23.7	-30.3	27.8	-35.6	32.3	-41.2	37.0	-47.3	47.6	-60.8
	2	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	19.4	-24.3	23.0	-29.0	27.0	-34.0	31.4	-39.4	36.0	-45.3	46.3	-58.1
	2	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	18.6	-22.9	22.2	-27.2	26.0	-32.0	30.2	-37.1	34.6	-42.5	44.5	-54.6
	2	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	18.1	-21.8	21.5	-25.9	25.2	-30.4	29.3	-35.3	33.6	-40.5	43.2	-52.0
	3	10	11.9	-15.2	13.3	-17.0	16.5	-21.0	19.9	-25.5	23.7	-30.3	27.8	-35.6	32.3	-41.2	37.0	-47.3	47.6	-60.8
	3	20	11.6	-14.5	13.0	-16.3	16.0	-20.1	19.4	-24.3	23.0	-29.0	27.0	-34.0	31.4	-39.4	36.0	-45.3	46.3	-58.1
	3	50	11.1	-13.7	12.5	-15.3	15.4	-18.9	18.6	-22.9	22.2	-27.2	26.0	-32.0	30.2	-37.1	34.6	-42.5	44.5	-54.6
	3	100	10.8	-13.0	12.1	-14.6	14.9	-18.0	18.1	-21.8	21.5	-25.9	25.2	-30.4	29.3	-35.3	33.6	-40.5	43.2	-52.0
Wall	4	10	13.0	-14.1	14.6	-15.8	18.0	-19.5	21.8	-23.6	25.9	-28.1	30.4	-33.0	35.3	-38.2	40.5	-43.9	52.0	-56.4
	4	20	12.4	-13.5	13.9	-15.1	17.2	-18.7	20.8	-22.6	24.7	-26.9	29.0	-31.6	33.7	-36.7	38.7	-42.1	49.6	-54.1
	4	50	11.6	-12.7	13.0	-14.3	16.1	-17.6	19.5	-21.3	23.2	-25.4	27.2	-29.8	31.6	-34.6	36.2	-39.7	46.6	-51.0
	4	100	11.1	-12.2	12.4	-13.6	15.3	-16.8	18.5	-20.4	22.0	-24.2	25.9	-28.4	30.0	-33.0	34.4	-37.8	44.2	-48.6
	4	500	9.7	-10.8	10.9	-12.1	13.4	-14.9	16.2	-18.1	19.3	-21.5	22.7	-25.2	26.3	-29.3	30.2	-33.6	38.8	-43.2
	5	10	13.0	-17.4	14.6	-19.5	18.0	-24.1	21.8	-29.1	25.9	-34.7	30.4	-40.7	35.3	-47.2	40.5	-54.2	52.0	-69.6
	5	20	12.4	-16.2	13.9	-18.2	17.2	-22.5	20.8	-27.2	24.7	-32.4	29.0	-38.0	33.7	-44.0	38.7	-50.5	49.6	-64.9
	5	50	11.6	-14.7	13.0	-16.5	16.1	-20.3	19.5	-24.6	23.2	-29.3	27.2	-34.3	31.6	-39.8	36.2	-45.7	46.6	-58.7
	5	100	11.1	-13.5	12.4	-15.1	15.3	-18.7	18.5	-22.6	22.0	-26.9	25.9	-31.6	30.0	-36.7	34.4	-42.1	44.2	-54.1
	5	500	9.7	-10.8	10.9	-12.1	13.4	-14.9	16.2	-18.1	19.3	-21.5	22.7	-25.2	26.3	-29.3	30.2	-33.6	38.8	-43.2

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 sf = 0.0929 m^2; 1.0 psf = 0.0479 kN/m^2

FIGURE 4.3 (*Continued*)

Components and Cladding – Method 1

h ≤ 60 ft

Design Wind Pressures

Walls & Roofs

Enclosed Buildings

Roof Overhang Net Design Wind Pressure, p_{net30} (psf)

(Exposure B at h = 30 ft with I = 1.0)

	Zone	Effective Wind Area (sf)	Basic Wind Speed V (mph)							
			90	100	110	120	130	140	150	170
Roof 0 to 7 degrees	2	10	-21.0	-25.9	-31.4	-37.3	-43.8	-50.8	-58.3	-74.9
	2	20	-20.6	-25.5	-30.8	-36.7	-43.0	-49.9	-57.3	-73.6
	2	50	-20.1	-24.9	-30.1	-35.8	-42.0	-48.7	-55.9	-71.8
	2	100	-19.8	-24.4	-29.5	-35.1	-41.2	-47.8	-54.9	-70.5
	3	10	-34.6	-42.7	-51.6	-61.5	-72.1	-83.7	-96.0	-123.4
	3	20	-27.1	-33.5	-40.5	-48.3	-56.6	-65.7	-75.4	-96.8
	3	50	-17.3	-21.4	-25.9	-30.8	-36.1	-41.9	-48.1	-61.8
	3	100	-10.0	-12.2	-14.8	-17.6	-20.6	-23.9	-27.4	-35.2
Roof > 7 to 27 degrees	2	10	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	20	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	50	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	2	100	-27.2	-33.5	-40.6	-48.3	-56.7	-65.7	-75.5	-96.9
	3	10	-45.7	-56.4	-68.3	-81.2	-95.3	-110.6	-126.9	-163.0
	3	20	-41.2	-50.9	-61.6	-73.3	-86.0	-99.8	-114.5	-147.1
	3	50	-35.3	-43.6	-52.8	-62.8	-73.7	-85.5	-98.1	-126.1
	3	100	-30.9	-38.1	-46.1	-54.9	-64.4	-74.7	-85.8	-110.1
Roof > 27 to 45 degrees	2	10	-24.7	-30.5	-36.9	-43.9	-51.5	-59.8	-68.6	-88.1
	2	20	-24.0	-29.6	-35.8	-42.6	-50.0	-58.0	-66.5	-85.5
	2	50	-23.0	-28.4	-34.3	-40.8	-47.9	-55.6	-63.8	-82.0
	2	100	-22.2	-27.4	-33.2	-39.5	-46.4	-53.8	-61.7	-79.3
	3	10	-24.7	-30.5	-36.9	-43.9	-51.5	-59.8	-68.6	-88.1
	3	20	-24.0	-29.6	-35.8	-42.6	-50.0	-58.0	-66.5	-85.5
	3	50	-23.0	-28.4	-34.3	-40.8	-47.9	-55.6	-63.8	-82.0
	3	100	-22.2	-27.4	-33.2	-39.5	-46.4	-53.8	-61.7	-79.3

Adjustment Factor for Building Height and Exposure, λ

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Unit Conversions – 1.0 ft = 0.3048 m; 1.0 sf = 0.0929 m^2; 1.0 psf = 0.0479 kN/m^2

FIGURE 4.3 (*Continued*)

the wake of upwind obstructions should be considered in design. Wind-tunnel tests (SEI/ASCE 7-02, Method 3) on a model of the structure and its neighborhood may be helpful in supplying design data as an alternative to Methods 1 and 2.

4.7 SEISMIC LOADS

Earthquakes have occurred in many states. Figures 4.4 and 4.5 show current contour maps of the United States that reflect the severity of seismic ground motion, as indicated in the ASCE standard, "Minimum Design Loads for Buildings and Other Structures" (SEI/ASCE 7-02).

The engineering approach to seismic design differs from that for other load types. For live, wind, or snow loads, the intent of a structural design is to preclude structural damage. However, to achieve an economical seismic design, codes and standards permit local yielding of a structure during a major earthquake. **Local yielding** absorbs energy but results in permanent deformations of structures. Thus, seismic design incorporates not only application of anticipated seismic forces but also use of structural details that ensure adequate ductility to absorb the seismic forces without compromising the stability of structures. Requirements for this approach are included in the AISC standard, "Seismic Provisions for Structural Steel Buildings."

The forces transmitted by an earthquake to a structure result from vibratory excitation of the ground. The vibration has both vertical and horizontal components. However, it is customary for building design to neglect the vertical component because most structures have reserve strength in the vertical direction due to gravity-load design requirements.

Seismic requirements in building codes and standards attempt to translate the complicated dynamic phenomenon of earthquake force into a simplified equivalent lateral static force to be applied to structure of regular geometry for design purposes. For example, SEI/ASCE 7-02 stipulates that the total lateral force, or base shear, V (kips) acting in the direction of each of the principal axes of the main structural system should be computed from

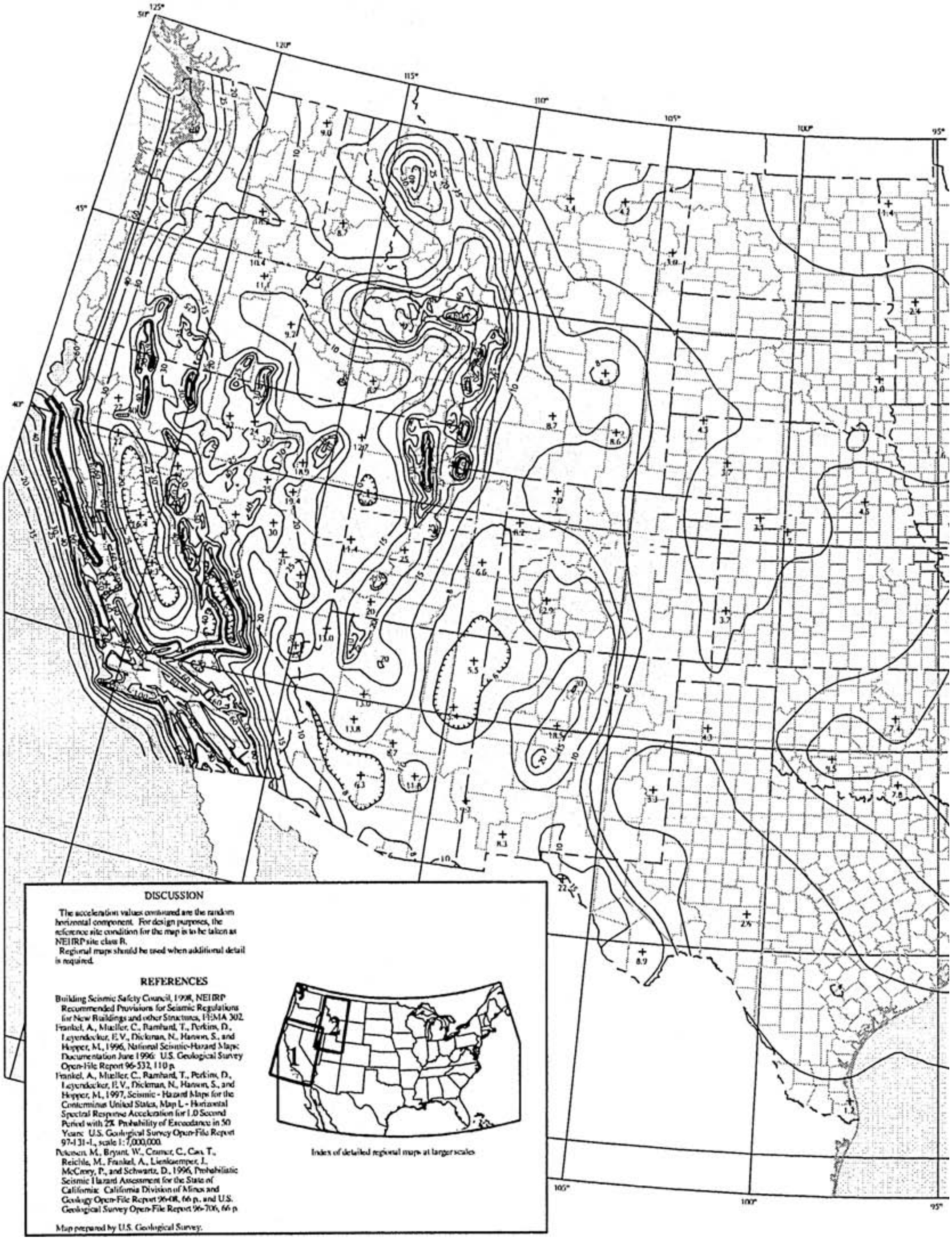
$$V = C_s W \tag{4.7}$$

where C_s = seismic response coefficient
W = total dead load and applicable portions of other loads, kips

Applicable portions of other loads are considered to be as follows:

1. In areas for storage, a minimum of 25% of the floor live load is applicable. The floor live load in parking garages and open parking structures need not be considered.
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 lb/ft^2 of floor area, whichever is greater, is applicable.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load exceeds 30 lb/ft^2, the design snow load should be included in W. Where the authority having jurisdiction approves, the amount of snow load included in W may be reduced to no less than 20% of the design snow load.

From the intended building occupancy classification (Table 4.10), the appropriate Seismic Use Group (I, II, or III) and its corresponding importance factor, I_s, is established by means of Tables 4.11 and 4.12, respectively. The site classification (A–F) must be ascertained in accordance with Table 4.13. Tables 4.14 and 4.15 can then provide the site coefficients, F_a and F_v, for the short and 1-s period maximum considered earthquake (MCE) spectral acceleration, respectively. The MCE maps given in Figs. 4.4 and 4.5 can be used to read the spectral response values of S_s and S_1 for the selected construction site. The seismic design category (A–F) and response modification factor R for the basic seismic force-resisting structural system must then be identified per SEI/ASCE 7-02, or as required by the applicable building code. The R-factor value is proportional to the amount of ductility,



DISCUSSION

The acceleration values contoured are the random horizontal component. For design purposes, the reference site condition for the map is to be taken as NEHRP site class B.

Regional maps should be used when additional detail is required.

REFERENCES

Building Seismic Safety Council, 1998, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, FEMA 302.

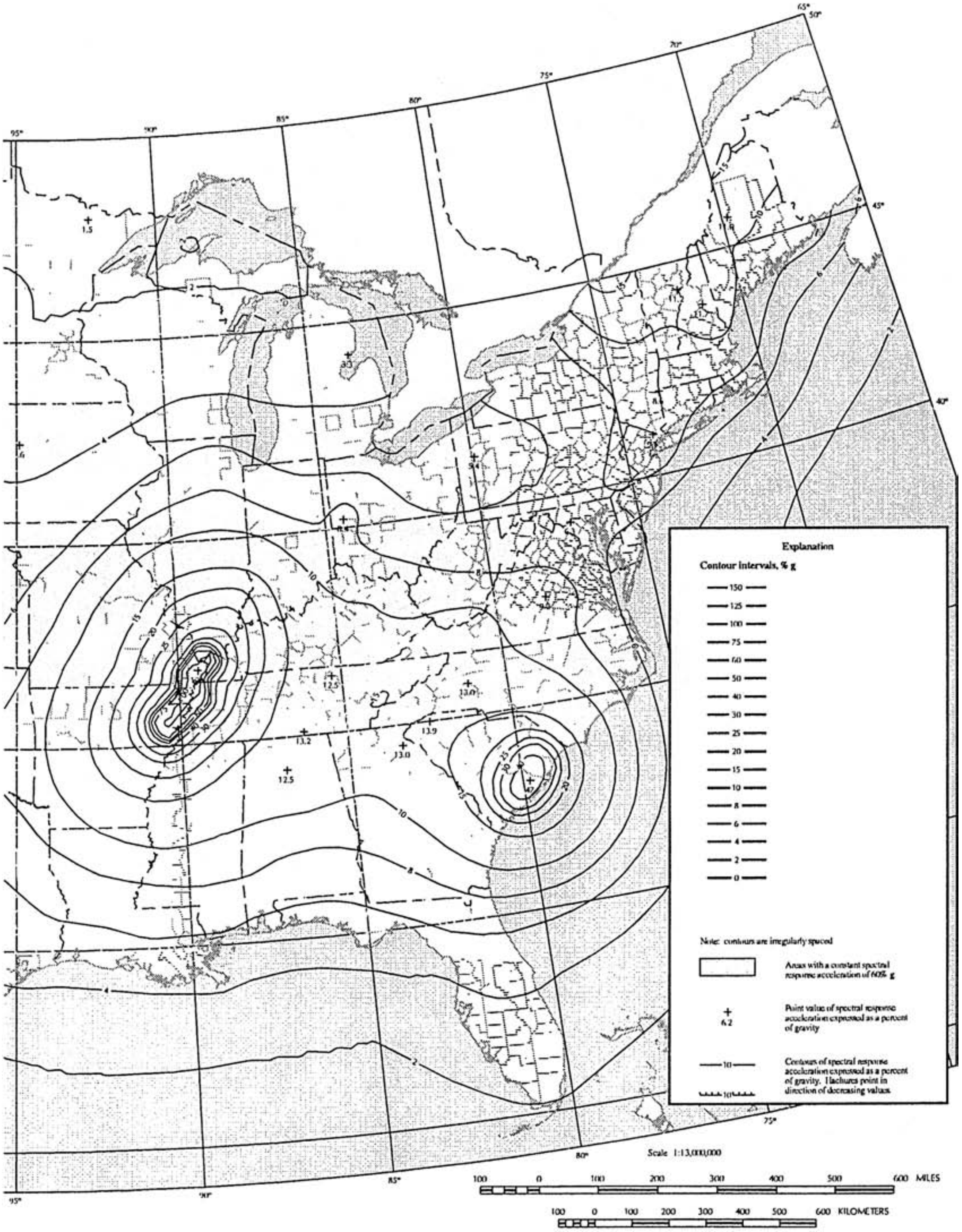
Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, National Seismic-Hazard Maps: Documentation June 1996: U.S. Geological Survey Open-File Report 96-532, 110 p.

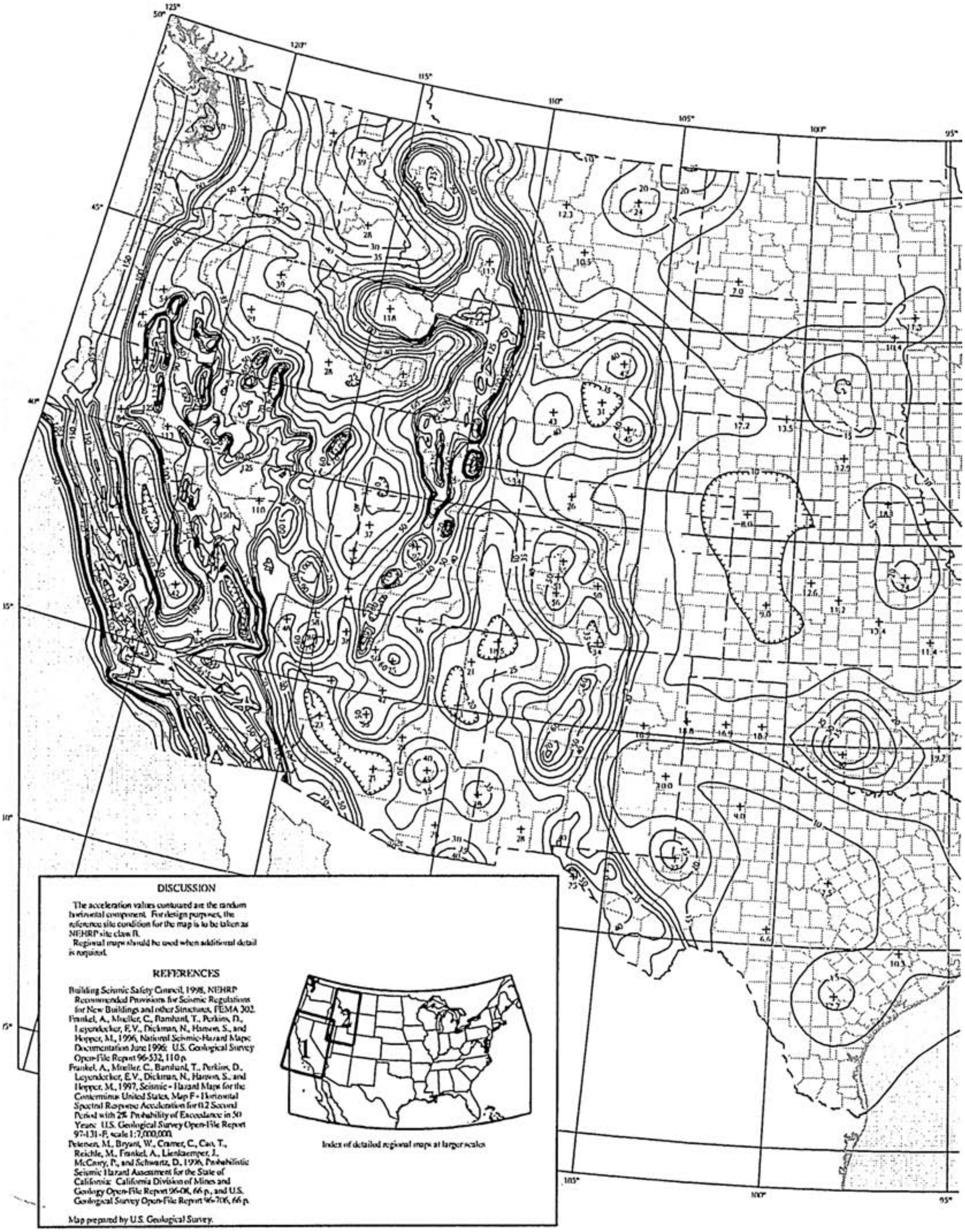
Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1997, Seismic - Hazard Maps for the Conterminous United States, Map L - Horizontal Spectral Response Acceleration for 1.0 Second Period with 2% Probability of Exceedance in 50 Years: U.S. Geological Survey Open-File Report 97-131-L, scale 1:7,000,000.

Petersen, M., Bryant, W., Cramer, C., Cao, T., Reichle, M., Frankel, A., Lienkaemper, J., McCrory, P., and Schwartz, D., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Division of Mines and Geology Open-File Report 96-08, 66 p., and U.S. Geological Survey Open-File Report 96-706, 66 p.

Map prepared by U.S. Geological Survey.

FIGURE 4.4 Contours on map of the United States showing maximum considered earthquake ground motion of 1.0-s spectral response acceleration (5% of critical damping), Site Class B. (*Source:* Minimum Design Loads for Buildings and Other Structures, *SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.*)


Explanation
Contour intervals, % g
150
125
100
75
60
50
40
30
25
20
15
10
8
6
4
2
0
Note: contours are irregularly spaced
Areas with a constant spectral response acceleration of 60% g
6.2
Point value of spectral response acceleration expressed as a percent of gravity
10
Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.
Scale 1:13,000,000
100 0 100 200 300 400 500 600 MILES
100 0 100 200 300 400 500 600 KILOMETERS
95°
90°
85°
80°
75°
70°
65°
50°
45°
40°
12.5
13.2
13.0
13.9
1.5
3.3
5.4
10
8
6
4
2
15
20
25



DISCUSSION

The acceleration values contoured are the random horizontal component. For design purposes, the reference site condition for the map is to be taken as NEHRP site class B.

Regional maps should be used when additional detail is required.

REFERENCES

Building Seismic Safety Council, 1998, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, FEMA 302.

Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, National Seismic-Hazard Maps: Documentation June 1996: U.S. Geological Survey Open-File Report 96-532, 110 p.

Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1997, Seismic - Hazard Maps for the Conterminous United States, Map F - Horizontal Spectral Response Acceleration for 0.2 Second Period with 2% Probability of Exceedance in 50 Years: U.S. Geological Survey Open-File Report 97-131-F, scale 1:7,000,000.

Petersen, M., Bryant, W., Cramer, C., Cao, T., Reichle, M., Frankel, A., Lienkaemper, J., McCrory, P., and Schwartz, D., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Division of Mines and Geology Open-File Report 96-08, 66 p., and U.S. Geological Survey Open-File Report 96-706, 66 p.

Map prepared by U.S. Geological Survey.

FIGURE 4.5 Contours on map of the United States showing maximum considered earthquake ground motion of 0.2-s spectral response acceleration (5% of critical damping), Site Class B. (*Source:* Minimum Design Loads for Buildings and Other Structures, *SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.*)

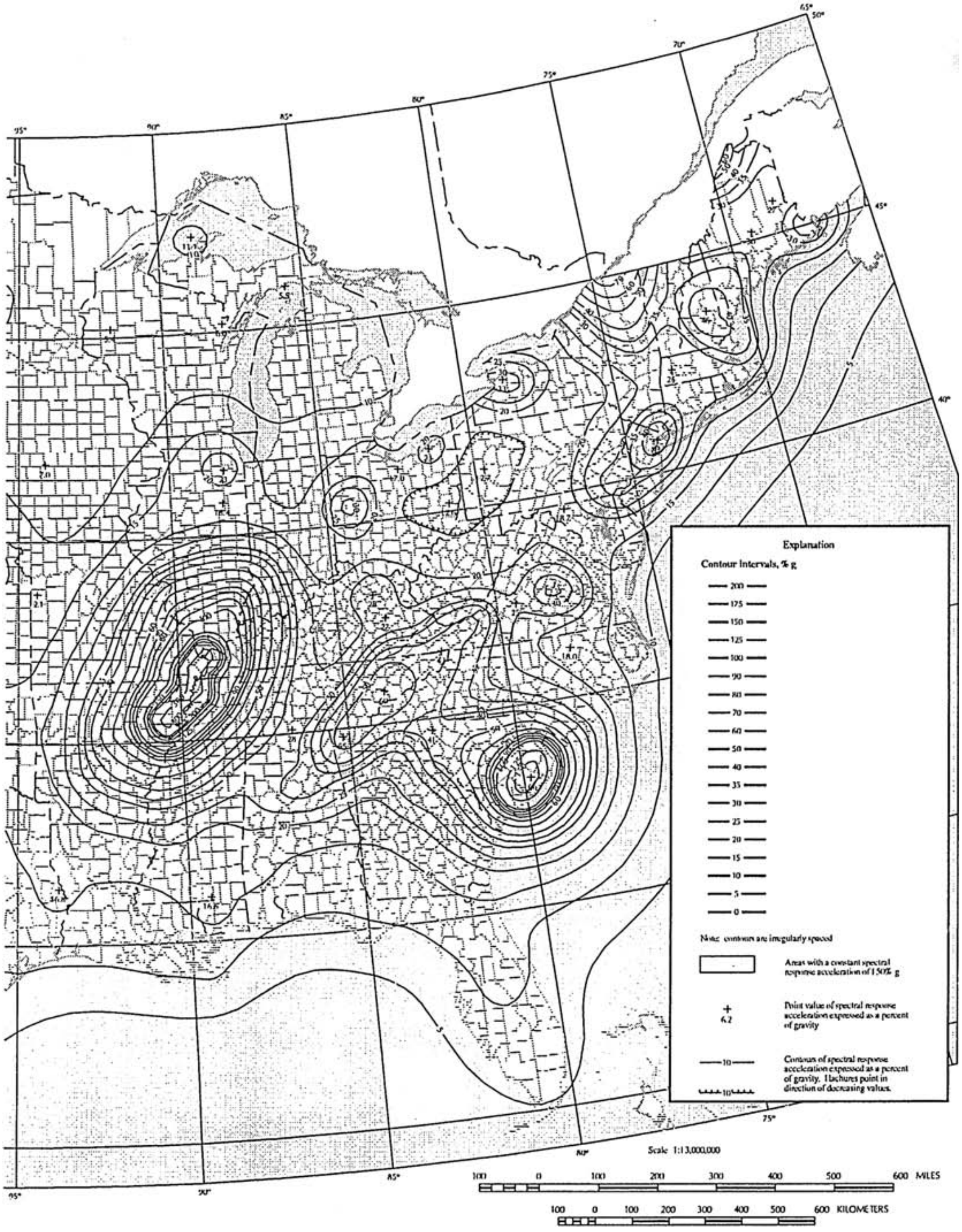

65°
50°
70°
75°
80°
85°
90°
95°
45°
40°
75°
80°
85°
90°
95°
Explanation
Contour intervals, % g
200
175
150
125
100
90
80
70
60
50
40
35
30
25
20
15
10
5
0
Note: contours are irregularly spaced
Areas with a constant spectral response acceleration of 150% g
+ 6.2
Point value of spectral response acceleration expressed as a percent of gravity
10
10
Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.
Scale 1:13,000,000
100 0 100 200 300 400 500 600 MILES
100 0 100 200 300 400 500 600 KILOMETERS

TABLE 4.10 Classification of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Nature of occupancy	Category
Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: Agricultural facilities Certain temporary facilities Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: Buildings and other structures where more than 300 people congregate in one area Buildings and other structures with day care facilities with capacity greater than 150 Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities Jails and detention facilities Power generating stations and other public utility facilities not included in Category IV	III
Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released.	
Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Sec. 1.5.2 that a release of the hazardous material does not pose a threat to the public.	
Buildings and other structures designated as essential facilities including, but not limited to: Hospitals and other health care facilities having surgery or emergency treatment facilities Fire, rescue, ambulance, and police stations and emergency vehicle garages Designated earthquake, hurricane, or other emergency shelters Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response Power generating stations and other public utility facilities required in an emergency Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency Aviation control towers, air traffic control centers, and emergency aircraft hangars Water storage facilities and pump structures required to maintain water pressure for fire suppression Buildings and other structures having critical national defense functions	IV
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.	
Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Sec. 1.5.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.11 Seismic Use Group Designations

		Seismic use group		
		I	II	III
Occupancy category (Table 4.10)	I	X		
	II	X		
	III		X	
	IV			X

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.12 Occupancy Importance Factors

Seismic use group	I_s
I	1.0
II	1.25
III	1.5

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.13 Seismic Site Classification

Site class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{\text{ch}}$	$\bar{s}_u$
A: Hard rock	>5000 ft/s (>1500 m/s)	Not applicable	Not applicable
B: Rock	2500 to 5000 ft/s (760 to 1500 m/s)	Not applicable	Not applicable
C: Very dense soil and soft rock	1200 to 2500 ft/s (370 to 760 m/s)	>50	>2000 lb/ft^2 (>100 kPa)
D: Stiff soil	600 to 1200 ft/s (180 to 370 m/s)	15 to 50	1000 to 2000 lb/ft^2 (50 to 100 kPa)
E: Soil	<600 ft/s (<180 m/s)	<15	<1000 lb/ft^2 (<50 kPa)
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index PI > 20 —Moisture context $w \geq 40\%$ —Undrained shear strength $\bar{s}_u < 500$ lb/ft^2		
F: Soils requiring site-specific evaluation	1. Soils vulnerable to potential failure or collapse 2. Peats and/or highly organic clays 3. Very high plasticity clays 4. Very thick soft/medium clays		

The following definitions apply, where the bar denotes average value for the top 100 ft of soil. See SEI/ASCE 7-02 for specific details.

$\bar{v}_s$ = measured shear wave velocity, ft/s
$\bar{N}$ = standard penetration resistance, blows/ft
$\bar{N}_{\text{ch}}$ = corrected for cohesionless layers, blows/ft
$\bar{s}_u$ = undrained shear strength, lb/ft^2
PI = plasticity index
w = liquid limit

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

TABLE 4.14 Values of F_a as a Function of Site Class and Mapped Short-Period Maximum Considered Earthquake Spectral Acceleration

	Mapped maximum considered earthquake spectral response acceleration at short periods				
Site class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Note: Use straight-line interpolation for intermediate values of S_S.

[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to less than 0.5 s., values of F_a for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Sec. 9.4.1.2.2.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

overstrength, and energy dissipation the seismic force-resisting structural system possesses. Thus, for more ductile systems with larger R, the equivalent lateral seismic design force will be lower than for more seismically vulnerable systems with a smaller R factor. The most conservative, lower-bound R is 1.0, which corresponds to a pure linear-elastic response and the highest possible seismic forces.

The seismic design forces determined in this manner are intended to be used directly with LRFD (strength or limit states) design, but can be reduced for application in allowable strength design, where prescribed in codes and standards.

SEI/ASCE 7-02 defines the seismic base shear coefficient C_s in units of acceleration of gravity g in accordance with the following equations:

$$C_s = \frac{S_{\mathrm{DS}}}{R/I_s} \tag{4.8}$$

where S_{DS} = design spectral response acceleration in the short-period range, g
R = response modification factor for structure (see SEI/ASCE 7-02)
I_s = occupancy importance factor for seismic use group (Table 4.12)

TABLE 4.15 Values of F_v as a Function of Site Class and Mapped 1-s Period Maximum Considered Earthquake Spectral Acceleration

	Mapped maximum considered earthquake spectral response acceleration at 1-s periods				
Site class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

Note: Use straight-line interpolation for intermediate values of S_1.

[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to less than 0.5 s; values of F_v for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Sec. 9.4.1.2.2.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

However, C_s need not be greater than

$$C_s = \frac{S_{D1}}{T(R/I_s)} \tag{4.9a}$$

and shall not be less than

$$C_s = 0.044S S_{DS} I_s \tag{4.9b}$$

or, for buildings in seismic design categories E and F,

$$C_s = \frac{S_1}{R/I_s} \tag{4.9c}$$

where S_{D1} = design spectral acceleration at a period of 1.0 s
T = fundamental period of the structure (s)
S_1 = mapped MCE spectral response acceleration

The two design spectral accelerations, S_{DS} and S_{D1}, are based on the MCE spectral response for short (S_{MS}) and 1-s (S_{M1}) periods, respectively, as determined by the following:

$$S_{DS} = {}^2\!/\!_3 S_{MS} \tag{4.10a}$$

$$S_{D1} = {}^2\!/\!_3 S_{M1} \tag{4.10b}$$

$$S_{MS} = F_a S_s \tag{4.10c}$$

$$S_{M1} = F_v S_1 \tag{4.10d}$$

where S_1 = mapped MCE, 5% of critical damping, spectral response acceleration at a period of 1 s from Fig. 4.4
S_s = mapped MCE, 5% of critical damping, spectral response acceleration at short periods (g), from Fig. 4.5
F_a = site coefficient for short-period earthquake (Table 4.14)
F_v = site coefficient for 1-s-period earthquake (Table 4.15)
S_{M1} = MCE, 5% of critical damping, spectral response acceleration at a period of 1 s, adjusted for site class effects
S_{MS} = MCE, 5% of critical damping, spectral response acceleration at short periods, adjusted for site class effects

A rigorous evaluation of the fundamental elastic period T requires consideration of the intensity of loading and the response of the structure to the loading. To expedite design computations, T may be determined by the following approximation, or by other equations given in SEI/ASCE 7-02:

$$T_a = C_t h_n^{\,x} \tag{4.11}$$

where T_a = approximate fundamental period, sec
C_t = period parameter from Table 4.16
x = period parameter from Table 4.16
h_n = height above the base to the highest level of the building, ft

For vertical distribution of seismic forces, the lateral base shear V should be distributed over the height of the structure as concentrated loads at each floor level or story. The lateral seismic force F at any floor level is determined by the following equation:

$$F_x = C_{vx} V \tag{4.12a}$$

TABLE 4.16 Values of Approximate Period Parameters C_t and x

Structure type	C_t*	x
Moment resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces	0.028 (0.068)	0.8
Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces	0.016 (0.044)	0.9
Eccentrically braced steel frames	0.03 (0.07)	0.75
All other structural systems	0.02 (0.055)	0.75

*Metric equivalents are shown in parentheses.

Source: From *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, American Society of Civil Engineers, Reston, Va., with permission.

where the vertical distribution factor is given by

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{4.12b}$$

where w_x and w_i = portion of the total gravity load of the structure (W) at level x or i
h_x and h_i = height from the base to level x or i
k = 1 for building having period of 0.5 s or less
= 2 for building having period of 2.5 s or more

Use linear interpolation for building periods between 0.5 and 2.5 s.

For horizontal shear distribution, the seismic design story shear in any story, V_x, is determined by

$$V_x = \sum_{i=1}^{n} F_i \tag{4.13}$$

where F_i = the portion of the seismic base shear induced at level *i*. The seismic design story shear is to be distributed to the various elements of the force-resisting system in a story based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

Provisions also should be made in design of structural framing for horizontal torsion, overturning effects, stability, and building drift. More advanced methods of seismic design, using modal, linear, or nonlinear time-history analyses, may be required for taller or irregular structures. Irregularities in mass, stiffness, and geometry should be considered.

(Federal Emergency Management Agency, “NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings,” 2000, Washington, D.C.)

4.8 IMPACT LOADS

The live loads specified in building codes and standards include an allowance for ordinary impact loads. Where structural members will be subjected to unusual vibrations or impact loads, such as those described in Table 4.17, provision must be made for them in design of the members. Most building codes specify a percentage increase in live loads to account for impact loads. Impact loads for cranes are given in Art. 4.9.

TABLE 4.17 Minimum Percentage Increase in Live Load on Structural Members for Impact

Type of member	Source of impact	Percent
Supporting	Elevators and elevator machinery	100
Supporting	Light machines, shaft or motor-driven	20
Supporting	Reciprocating machines or power-driven units	50
Hangers	Floors or balconies	33

4.9 CRANE-RUNWAY LOADS

Design of structures to support cranes involves a number of important considerations, such as determination of maximum wheel loads, allowance for impact, effects due to multiple cranes operating in single or double isles, traction and braking forces, application of crane stops, and cyclic loading and fatigue. In accordance with SEI/ASCE 7-02, the crane live load is its full rated capacity.

The maximum vertical wheel load of powered monorail, cab-operated and remote-controlled overhead cranes, should be increased a minimum of 25% to provide for impact. The maximum vertical load of pendant-operated overhead cranes should be increased a minimum of 10% to account for impact load. Increase in load resulting from impact is not required to be applied to the supporting columns because the impact load effects will not develop or will be negligible.

The lateral force on crane runways with electrically powered trolleys should not be less than 20% of the sum of the crane rated capacity and the trolley and hoist weight. The force should be assumed to be applied by the wheels at the top of the rails, acting in either direction normal to the rails, and should be distributed with due regard for the lateral stiffness of the structure supporting the rails. Bridge or monorail cranes with a hand-geared bridge, trolley, and hoist need not have any vertical load impact increase.

The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, should be a minimum of 10% of the maximum wheel loads due to crane rated capacity, trolley weight, and crane weight. It should be applied at the top of the rail, unless otherwise specified, and parallel to the beam.

The crane runway should be designed for crane stop forces. The velocity of the crane at impact must be taken into account when calculating the crane stop and resulting longitudinal forces. Fatigue and serviceability concerns are extremely important design considerations for structures supporting cranes.

Additional design guidance is given in the Metal Building Manufacturers Association Standard, “Low-Rise Building Systems Manual.” For the design of heavy-duty crane runway systems, AISC Design Guide 7 and AISE Standard No. 13, Association of Iron and Steel Engineers, “Specification for Design and Construction of Mill Buildings,” should be consulted.

4.10 RESTRAINT LOADS

Restraint loads are caused by changes in dimensions or geometry of structures or members due to the behavior of material, type of framing, or details of construction used. Structural effects that may be so induced must be considered where they increase design requirements. They may occur as a result of foundation settlement, or as a result of temperature or shrinkage effects that are restrained by adjoining construction or installations.

4.11 COMBINED LOADS

The types of loads described in Arts. 4.4 through 4.10 may act simultaneously. Maximum stresses or deformations, therefore, may result from some combination of the loads. Building codes specify various combinations that must be investigated, depending on whether allowable strength design (ASD) or load and resistance factor design (LRFD) is used. Load combinations according to SEI/ASCE 7-02, which is the principal load standard, are given here. Note that the most critical load combination may occur when one or more of the loads are not acting.

For ASD, the following load combinations should be investigated:

1. D
2. $D + L + T$
3. $D + (L_r \text{ or } S \text{ or } R)$
4. $0.75[L + (L_r \text{ or S or } R) + T] + D$
5. $0.75(W \text{ or } 0.7E) + D$
6. $0.75[L + (W \text{ or } 0.7E) + (L_r \text{ or } S \text{ or } R)] + D$
7. $0.6D + W$
8. $0.6D + 0.7E$

where D = dead load
L = floor live load, including impact
L_r = roof live load
S = roof snow load
R = rain load (initial rainwater or ice, exclusive of ponding)
W = wind load
E = earthquake load
T = restraint load

Note that because the earthquake load, E, is defined for use with LRFD, it is reduced by a factor of 0.7 in the ASD load combinations. Also, SEI/ASCE 7-02 states that increases in allowable stress shall not be used with the load combinations, unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

For LRFD, the following load combinations should be investigated:

1. $1.4D$
2. $1.2(D + T) + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or S or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or S or } R)$
5. $1.2D + 1.0E + (L \text{ or } 0.2S)$
6. $0.9D + 1.6W$
7. $0.9D + 1.0E$

4.12 FIRE PROTECTION

Building codes play a dominant role in defining the level of structural fire protection that is expected by society. Typically, fire protection is implemented in design through prescriptive code compliance. Alternatively, in situations with unusual conditions, unique designs, or narrower project constraints beyond prescriptive code limits, a more detailed fire engineering or "performance-based design" can be implemented through a qualified consultant. The results need to demonstrate equivalence to the fire safety objectives of the applicable code. As a consequence, a working knowledge of building codes is an important prerequisite for contemporary design.

In the past, keeping abreast of building codes was difficult, even for the largest design offices, since most major cities and a number of states maintained locally developed codes. Today, this impediment is less relevant. In the United States and Canada, the vast majority of cities, states, and provinces now enforce one of the following model codes:

- *International Building Code* (IBC), International Code Council, Falls Church, Va.
- *NFPA 5000, Building Construction and Safety Code*, National Fire Protection Association (NFPA), Quincy, Mass.
- *National Building Code of Canada*, National Research Council of Canada, Ottawa, Ontario

In the United States, some vestiges of the previous three model building codes (National Building Code, Standard Building Code, and Uniform Building Code) may also remain. Regardless of the code used, two fire-related characteristics of materials influence selection and design of structural systems: combustibility and fire resistance.

4.12.1 Combustible and Noncombustible Materials

Most fires are either accidental or caused by carelessness. Fires are usually small when they start, and require fuel and ventilation (air supply) to grow in intensity and magnitude. In fact, many fires either self-extinguish due to a lack of readily available fuel or are extinguished by building occupants. Furthermore, even though most fires involve building contents, a combustible building itself may be the greatest potential source of fuel.

By definition, noncombustible materials such as stone, concrete, brick, and steel do not burn and therefore do not serve as sources of fuel. Although the physical properties of noncombustible materials may be adversely affected by elevated temperature exposures, these materials do not contribute to either the intensity or duration of fires. Wood, paper, and plastics are examples of combustible materials.

Tests conducted by the National Institute for Standards and Technology (formerly the National Bureau of Standards) indicate that an approximate relationship exists between the amount of available combustible material (**fire loading,** pounds of wood equivalent per square foot of floor area) and **fire severity,** hours of equivalent fire exposure (Fig 4.6). Subsequent field surveys measured the fire loads typically found in buildings with different occupancies (Table 4.18). More modern fire load surveys have been expressed in terms of the potential heat energy of the combustible contents.

A reasonable estimate of the structural fire loading for conventional wood-frame construction is $7\frac{1}{2}$ to 10 lb/ft^2. For heavy-timber construction, the corresponding structural fire load may be on the order of $12\frac{1}{2}$ to $17\frac{1}{2}$ lb/ft^2. As a consequence, building codes generally limit the permitted size (allowable height and area) of combustible buildings to a much greater degree than for noncombustible buildings.

However, the potential fuel of the combustible construction or contents is not the only variable that influences fire severity. Ventilation is another major fire parameter, and its effects have been included in more modern analytical models of natural fires.

4.12.2 Fire Resistance

In addition to regulating building construction based on the combustibility or noncombustibility of structures, building codes also specify fire-resistance requirements as a function of building occupancy and size, i.e., height and area. In general, **fire resistance** is defined as the relative ability of construction assemblies, such as, floors, walls, partitions, beams, girders, and columns, to prevent spread of fire to adjacent spaces and to avoid structural collapse when exposed to fire. Fire-resistance requirements are based on laboratory tests conducted in accordance with "Standard Methods of Fire Tests of Building Construction and Materials" (ASTM E 119).

The ASTM E119 test method specifies a "standard" fire exposure that is used to evaluate the relative fire resistance of construction assemblies (Fig. 4.7). Fire-resistance requirements are specified in terms of the time during which an assembly continues to prevent the spread of fire, does not exceed certain temperature limits, and sustains its structural loads without failure, when exposed to

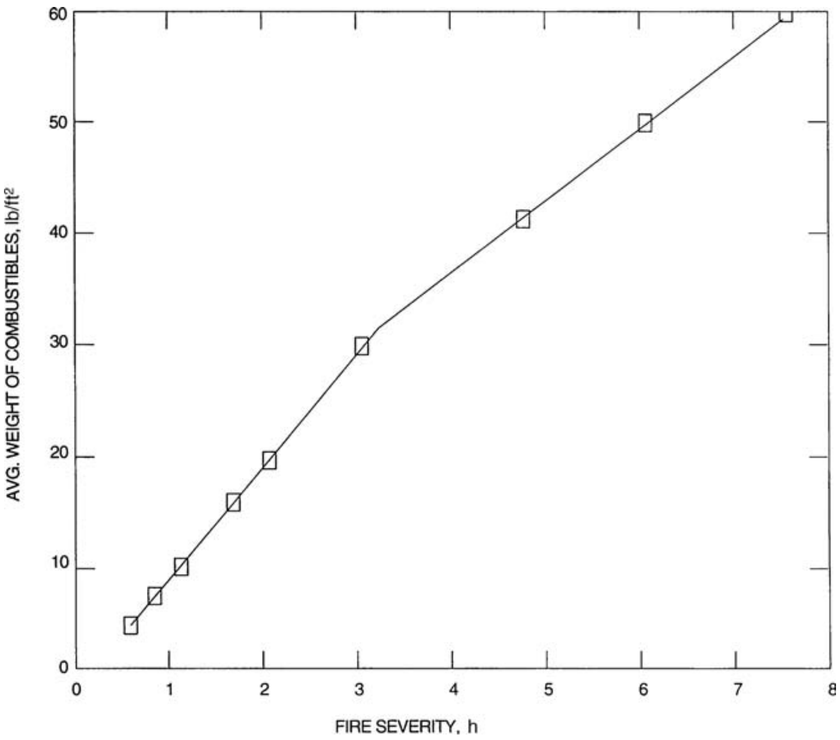


FIGURE 4.6 Curve relating approximate fire severity to average weight of combustibles on floor area in a building.

TABLE 4.18 Typical Occupancy Fire Loads and Fire Severity

Type of occupancy	Occupancy fire load, lb/ft^2	Equivalent fire severity, h
Assembly	5 to 10	$^1/_2$ to 1
Business	5 to 10	$^1/_2$ to 1
Educational	5 to 10	$^1/_2$ to 1
Hazardous	Variable	Variable
Industrial		
Low hazard	0 to 10	0 to 1
Moderate hazard	10 to 25	1 to 2$^1/_2$
Institutional	5 to 10	$^1/_2$ to 1
Mercantile	10 to 20	1 to 2
Residential	5 to 10	$^1/_2$ to 1
Storage		
Low hazard	0 to 10	0 to 1
Moderate hazard	10 to 30	1 to 3

Source: Based on data in *Fire Protection through Modern Building Codes*, American Iron and Steel Institute, Washington, D.C.

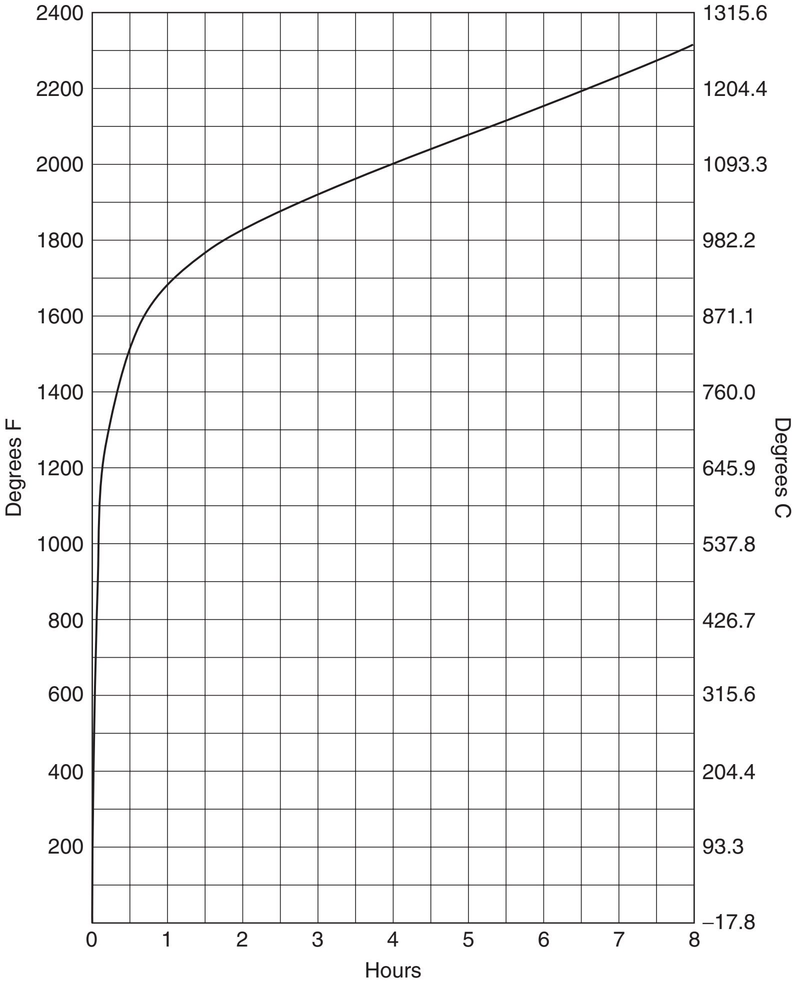


FIGURE 4.7 Variation of temperature with time in the standard fire test specified in ASTM E119. (*Source: American Society for Testing and Materials, Conshohocken, Pa., with permission.*)

the "standard fire." Thus fire-resistance requirements are expressed in terms of hours. The design of free-resistant buildings is typically accomplished in a prescriptive fashion by selecting tested "designs" that meet specific building code requirements.

It is emphasized that ASTM E119-based fire-resistance ratings are comparative in nature. They are conducted on idealized laboratory specimens and are not intended to be a predictor of structural performance of actual buildings in real fires. Special nonstandard fire tests may be desirable in some cases.

Listings of fire-resistance ratings for construction assemblies are available from a number of sources:

- *Fire-Resistance Directory*, Underwriters Laboratories, Northbrook, Ill.
- *Fire-Resistance Ratings*, American Insurance Services Group, New York, N.Y.
- *Fire-Resistance Design Manual*, Gypsum Association, Washington, D.C.

4.12.3 Fireproof Buildings

In the past, the term **fireproof** was frequently used to describe fire-resistant buildings. The use of this and terms such as **fireproofing** is unjustified and should be avoided. Experience has clearly demonstrated that large-loss fires (in terms of both property losses and loss of life) can and do occur in fire-resistant buildings. No building is completely fireproof, given the wide range of uncertainties that influence the development of and effects of real fires, and the previously mentioned limitations of fire-resistance ratings.

(*Fire Protection Handbook*, National Fire Protection Association, Quincy, Mass.)

4.12.4 Effect of Temperature on Steel

The properties of virtually all building materials are adversely affected by the temperatures developed during standard fire tests. Structural steel is no exception. The effect of elevated temperatures on the yield and tensile strengths of steel is described in Art. 1.12. In general, yield strength decreases with large increases in temperature, but structural steels retain about 60% of their ambient-temperature yield strength at 1000°F.

During many real building fires, temperatures in excess of 1000°F develop for relatively brief periods of time, but failures do not occur in structural steel members inasmuch as they are rarely loaded enough to affect full design strength. As a consequence, in some instances, bare structural steel has sufficient load-carrying capacity to withstand the effects of fire. This is not recognized in the standard fire tests, however, because in the standard ASTM E119 tests, the temperatures are continuously increased while structural members are loaded to design capacities. Based on these tests, when building codes specify fire-resistant construction, they require fire-protection materials to "insulate" structural steel elements.

4.12.5 Fire-Protection Materials

A variety of different materials or systems are used to protect structural steel. The performance of these are directly determined during standard fire tests. In addition to the insulation characteristics evaluated in the tests, the physical integrity of fire-protection materials is extremely important and should be preserved during installation. Required fire-protection assemblies should be carefully inspected during and after construction to ensure that they are installed and maintained according to the manufacturers' recommendations and the appropriate fire-resistant designs.

Gypsum. Gypsum, in several forms, is widely used for fire protection (Fig. 4.8). As a plaster, it is applied over metal lath or gypsum lath. In the form of wallboard, gypsum is typically installed over cold-formed steel framing or furring.

The effectiveness of gypsum-based fire protection can be increased significantly by addition of lightweight mineral aggregates, such as vermiculite and perlite, to gypsum plaster. It is important that the mix be properly proportioned and applied in the required thickness and that the lath be correctly installed.

Three general types of gypsum wallboard are readily available: regular, Type X, and proprietary. **Type X** wallboards have specially formulated cores that provide greater fire resistance than conventional wallboard of the same thickness. **Proprietary** wallboards, such as Type C, also are available with even greater fire-resistant characteristics. It is therefore important to verify that the wallboard used is that specified for the desired fire-resistant design. In addition, the type and spacing of fasteners and, when appropriate, the type and support of furring channels should be in accordance with specifications.

("Design Data—Gypsum Products," Gypsum Association, Washington, D.C.)

Spray-Applied Materials. The most widely used fire-protection materials for structural steel are lightweight mineral fiber and cementitious materials that are spray-applied directly to the contours of beams, girders, columns, and floor and roof decks (Fig. 4.9). The spray-applied fire-resistive materials (SFRM) are based on proprietary formulations. Hence it is imperative that the manufacturer's

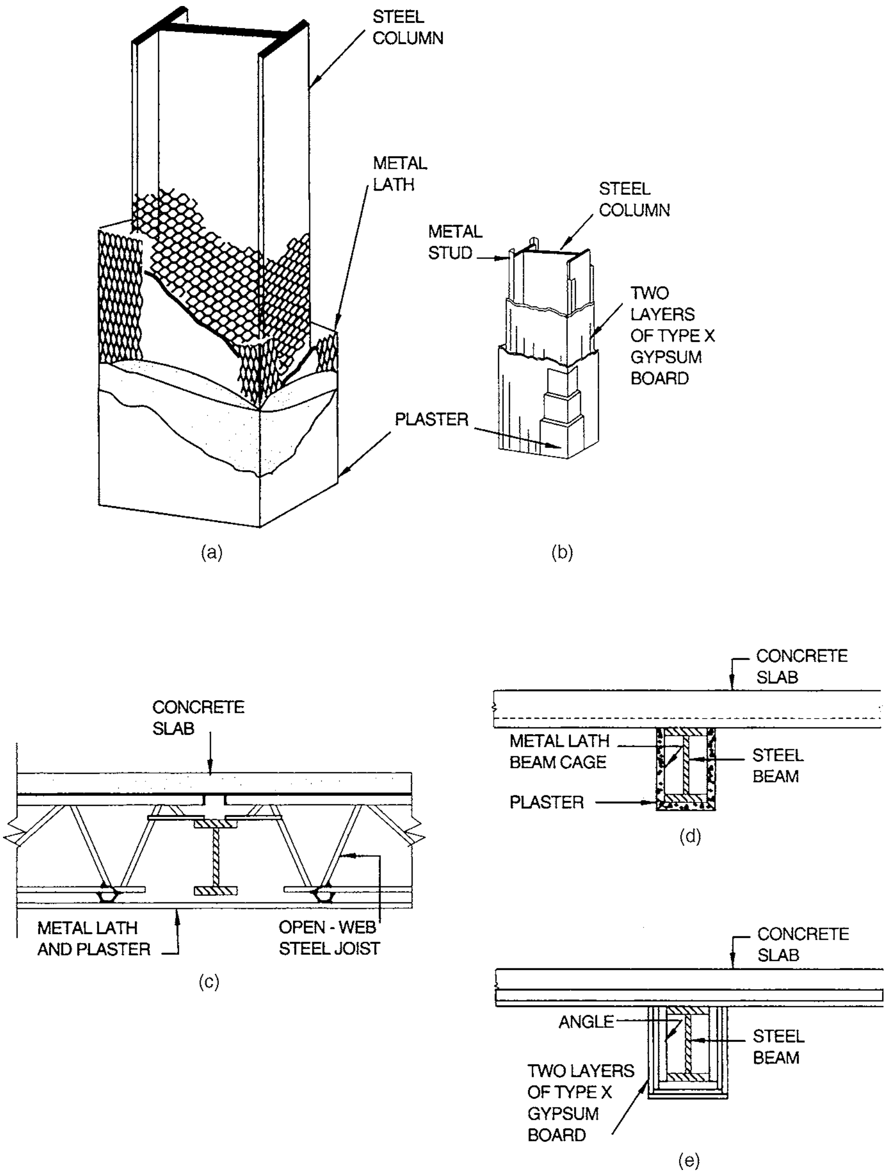


FIGURE 4.8 Some methods for applying gypsum as fire protection for structural steel: (*a*) column enclosed in plaster on metal lath; (*b*) column boxed in with wallboard and plaster; (*c*) open-web joist with plaster ceiling; (*d*) beam enclosed in a plaster cage; (*e*) beam boxed in with wallboard.

recommendations for mixing and application be followed closely. Fire-resistant designs are published annually by Underwriters Laboratories (UL).

Adhesion/cohesion is an important characteristic of spray-applied materials, as covered in ASTM E736. To ensure that it is attained, the structural steel should be free of dirt, oil, and loose scale; generally, the presence of light rust will not adversely affect adhesion. When the steel has been painted, however, field experience and testing have demonstrated that adhesion problems can arise. (Paint and

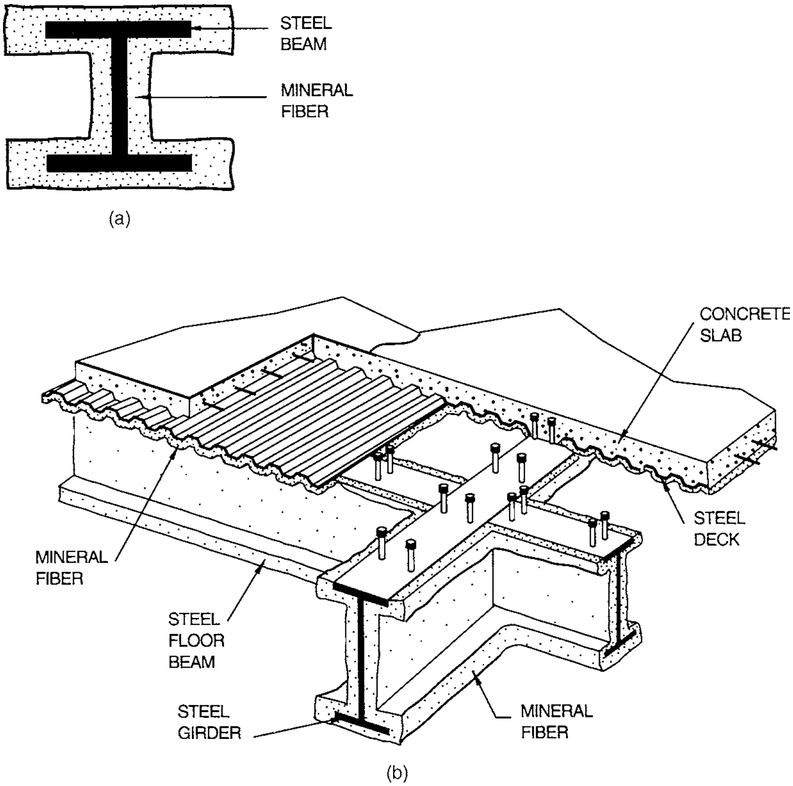


FIGURE 4.9 Mineral fiber spray applied to (*a*) steel beam; (*b*) beam-and-girder floor system, with steel floor deck supporting a concrete slab.

primers are not generally required for corrosion protection when structural steel will be enclosed within a building or otherwise protected from the elements.) If paint is specified for structural steel that will subsequently be protected with spray-applied materials, the specifier should contact the paint and fire-protection material suppliers in advance to ensure that the two materials are compatible. Otherwise, use of expanded metal lath as supplementary mechanical bond may be required to ensure adequate adhesion.

Suspended Ceiling Systems. A wide variety of proprietary suspended ceiling systems are also available for protecting floors and beams and girders (Fig. 4.10). Fire-resistance ratings for such systems are published by Underwriters Laboratories. These systems are specifically designed for membrane fire protection and require careful integration of ceiling tile, grid, and suspension systems. Also, openings for light fixtures, air diffusers, and similar accessories must be adequately limited and protected. As a consequence, manufacturer's installation instructions should be closely followed.

In the case of load-transfer trusses or girders that support loads from more than one floor, building codes may require individual member protection. As a consequence, suspended ceiling systems may not be permitted for this application.

Concrete and Masonry. Concrete, once widely used for fire protecting structural steel, is not particularly efficient for this application because of its weight and relatively high thermal conductivity. As a result, concrete is now rarely used for fire protection only.

Concrete floor slabs are common as fire protection for the tops of flexural members. Concrete or masonry is also sometimes used to encase steel columns for architectural or structural purposes or when more substantial resistance to abrasion and physical damage is required (Fig. 4.11).

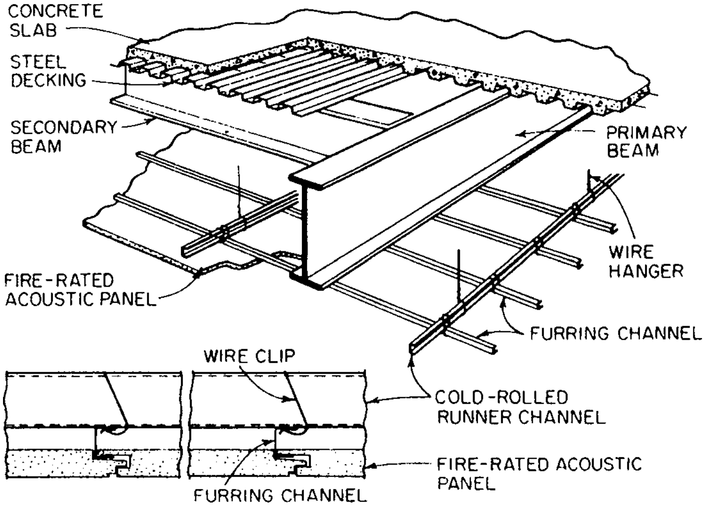


FIGURE 4.10 Steel floor system fire protected on the underside by a suspended ceiling.

Design information on the fire resistance of steel columns encased in concrete or masonry, or protected with precast-concrete column covers, is available in American Concrete Institute, Specification 216.1-97, "Standard Calculation Methods for Structural Fire Protection," 1997; ASCE/SFPE 29-99, 1999; and *International Building Code*, 2003.

4.12.6 Architecturally Exposed Structural Steel (AESS)

This AESS concept involves the architectural expression of structural systems on building exteriors or interiors, in contrast to the general practice of concealing them behind decorative facades. Design of AESS is strongly influenced by esthetics and the building code requirements for fire-resistant construction.

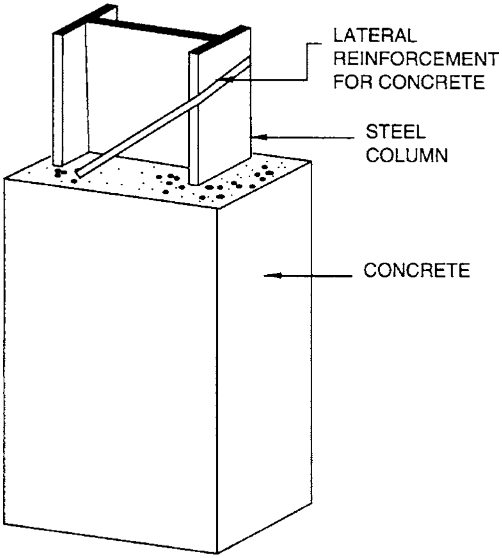


FIGURE 4.11 Concrete-encased steel column.

One approach for meeting code requirements for structural fire protection when the appearance of architecturally exposed steel is desired is illustrated in Fig. 4.12. As shown, flanges of a steel column are fire protected with a spray-applied material, insulation is placed against the web between the flanges, and the assembly is enclosed in a metal cover with the shape of the column.

Another approach is to use tubular columns filled with water (Fig. 4.13). Originally patented in 1884, this system was neglected until the late 1960s, when it was adopted for the 64-story U.S. Steel Building in Pittsburgh, Pa. Since then, several other buildings have been designed using this

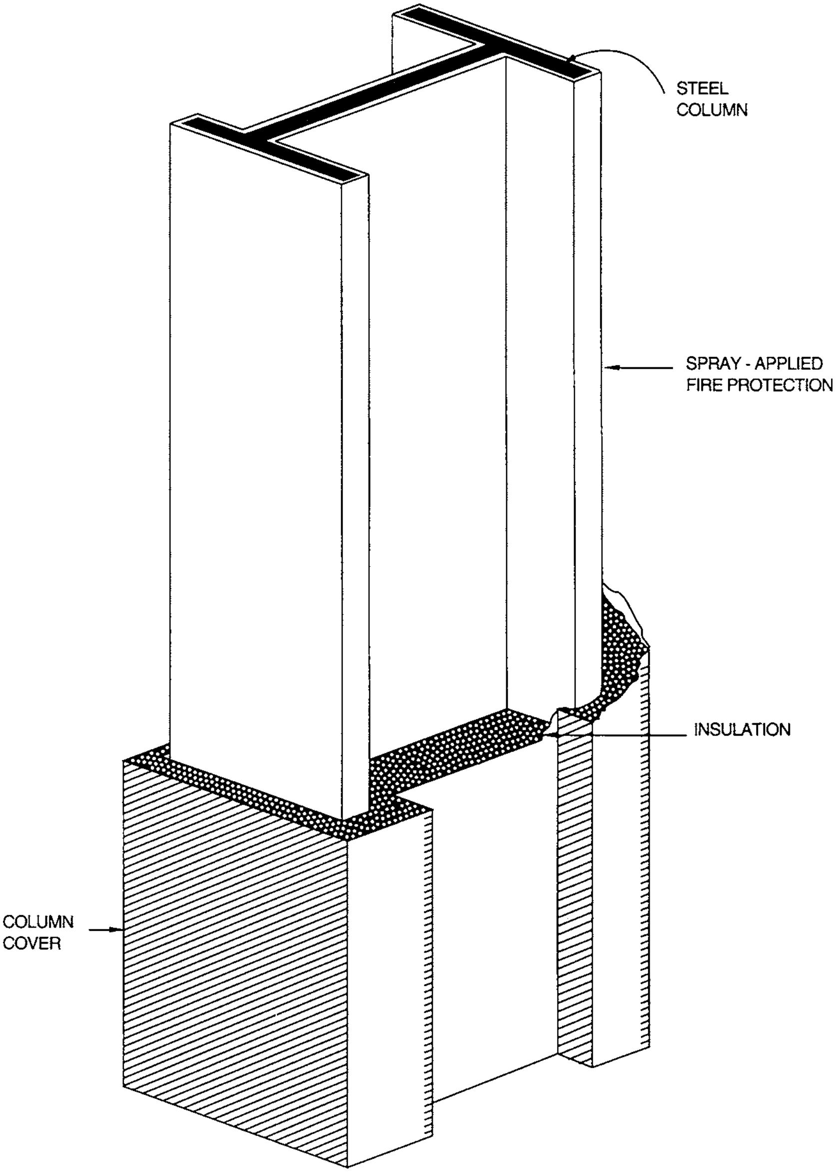


FIGURE 4.12 Fire-protected exterior steel column with exposed metal column covers.

concept. In a fire, the circulating water in a tubular column is expected to limit the temperature rise in the steel. Generally, corrosion inhibitors should be added to the water, and in cold climates, antifreeze solution should be used for exterior columns.

(*Fire Protection through Modern Building Codes,* American Iron and Steel Institute, Washington, D.C.)

In still another approach, sheet-steel covers are applied on the outside of a building to insulated flanges of steel spandrel girders to act as flame shields, as illustrated in Fig. 4.14. These sheet-steel covers not only serve to deflect flames away from the exposed, exterior web of a girder but also provide weather protection for the insulated flanges. As shown, a flame-shielded spandrel girder is protected in the interior of the building in a conventional manner.

As illustrated by full-scale fire tests on flame-shielded spandrel girders, the standard fire test is not representative of the exposure that will be experienced by exterior columns and girders. Research on fire exposure conditions for exterior structural elements has led to development of a comprehensive design method for fire-safe exterior structural steel which has been adopted by some building codes. Likewise, fire engineering principles can be used to determine if less or no protection may be justified for conditions that are beyond the prescriptive code limits, or as an alternative to them. Use of more realistic natural fires, heat-transfer relationships, and structural analyses for these cases can provide the needed rational solutions.

(*Design Guide for Fire-Safe Structural Steel*, American Iron and Steel Institute, Washington, D.C.)

4.12.7 Restrained and Unrestrained Construction

One of the major sources of confusion with respect to design of fire-resistant buildings is the concept of restrained and unrestrained ratings. This concept is peculiar to ASTM E119 and U.S. codes, and is not used in other countries. Fire-resistant design is based on the use of tested assemblies and

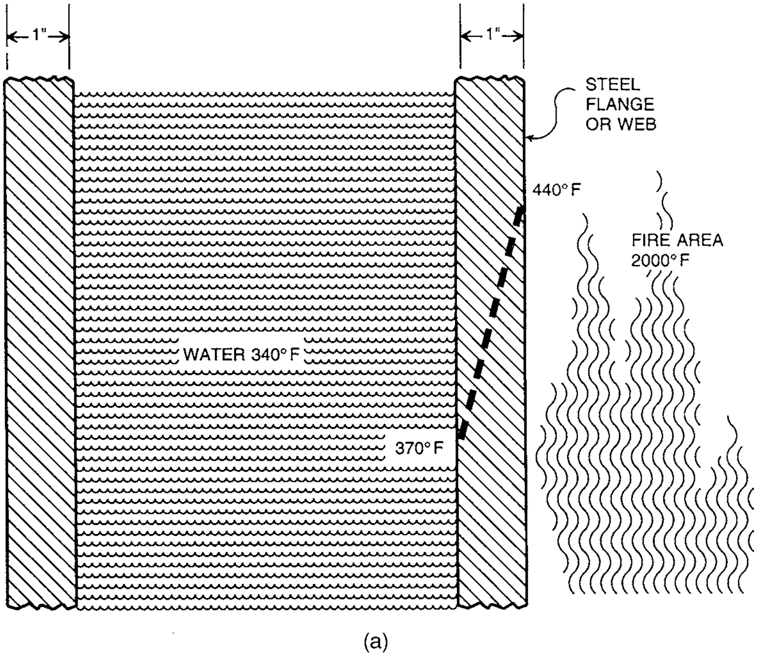
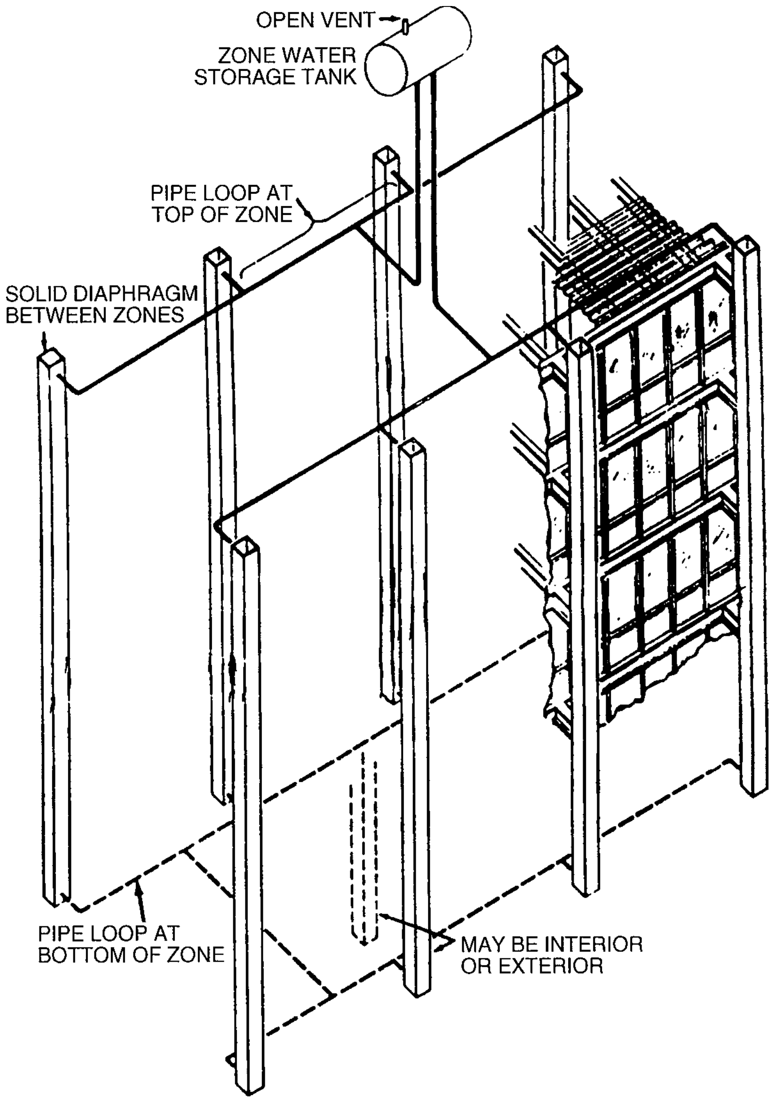


FIGURE 4.13 Tubular steel columns filled with water for fire resistance. (*a*) Temperature variation during exposure to fire. (*b*) Schematic arrangement of fire-protection system.



(b)

FIGURE 4.13 (*Continued*)

is predicated on the assumption that test assemblies are "representative" of actual construction. In reality, this assumption is extremely difficult to implement in laboratory-scale fire tests. The primary difficulty arises from the size of available test furnaces, which typically can only accommodate floor specimens in the range of 15 by 18 ft in area. As a result, a typical test assembly actually represents a relatively small portion of a floor or roof structure. Thus, even though the standard fire test is frequently described as "large scale," it clearly is not "full scale."

In the attempt to model real floor systems in a representative manner, several problems arise. For example, since most floor slabs and roof decks are physically, if not structurally, continuous over beams and girders, real beams and girders are usually much larger than can be accommodated in

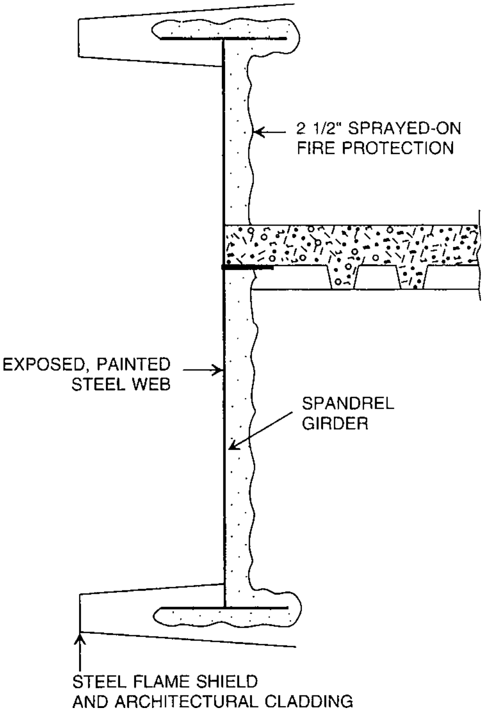


FIGURE 4.14 Flame shields placed on flanges of a spandrel girder to protect the web against flames.

available furnaces. Also, beams frame into columns and girders in a number of different ways. In some cases, connections are designed to resist only shear forces. In other cases, full- or partial-moment connections are provided. In short, given the cost of testing, the complexity of modern structural systems, and the size of available test facilities, it is unrealistic to assume that test assemblies can accurately model real construction systems.

In recognition of the practical difficulties associated with testing, ASTM E119 includes two test conditions, restrained and unrestrained. The restraint that is contemplated in fire testing is restraint against thermal expansion, not structural restraint in the traditional sense. When an assembly is supported or surrounded by construction that is capable of resisting expansion, to some degree, thermal stresses will be induced in the assembly in addition to those due to dead and live loads. Originally, it was thought that thermal stresses would reduce the fire resistance of many assemblies. However, extensive research indicated that restraint actually improved the fire resistance of many common types of floor systems. The two test conditions in E119 recognize the complexity of this issue.

The restrained condition applies when the assembly is supported or surrounded by construction that is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures. Otherwise, the assembly should be considered free to rotate and expand at the supports and should be considered unrestrained. Thus a floor system that is simply supported from a structural standpoint may often be restrained from a fire-resistance standpoint. To provide guidance in the use of restrained and unrestrained ratings, ASTM E119 includes examples in an explanatory appendix (Table 4.19) which indicate that most common types of steel framing systems can be considered to be restrained from a fire-resistance standpoint. More recently, Gewain and Troup (2001) more fully described and justified the use of restrained ratings in steel buildings. (“Restrained Fire Resistance Ratings in Structural Steel Buildings,” *Engineering Journal*, Second Quarter 2001, AISC, Chicago, Ill.) The 2005 AISC “Specification for Structural Steel Buildings” also provides similar guidance.

TABLE 4.19 Example of Restrained and Unrestrained Construction for Use in Fire Tests*

Type of construction	Condition
I. Wall bearing:	
a. Single-span and simply supported end spans of multiple bays.†	
(1) Open-web steel joists or steel beams, supporting concrete slab, precast units, or metal decking	Unrestrained
(2) Concrete slabs, precast units, or metal decking	Unrestrained
b. Interior spans of multiple bays:	
(1) Open-web steel joists, steel beams or metal decking, supporting continuous concrete slab	Restrained
(2) Open-web steel joists or steel beams, supporting precise units or metal decking	Unrestrained
(3) Cast-in-place concrete slab systems	Restrained
(4) Precast concrete where the potential thermal expansion is resisted by adjacent construction‡	Restrained
II. Steel framing:	
(1) Steel beams welded, riveted, or bolted to the framing members	Restrained
(2) All types of cast-in-place floor and roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor or roof system is secured to the framing members	Restrained
(3) All types of prefabricated floor or roof systems where the structural members are secured to the framing members and the potential thermal expansion of the floor or roof system is resisted by the framing system or the adjoining floor or roof construction‡	Restrained
III. Concrete framing:	
(1) Beams securely fastened to the framing members	Restrained
(2) All types of cast-in-place floor or roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor system is cast with the framing members	Restrained
(3) Interior and exterior spans of precast systems with cast-in-place joints resulting in restraint equivalent to that which would exist in condition III	Restrained
(4) All types of prefabricated floor or roof systems where the structural members are secured to such systems and the potential thermal expansion of the floor or roof systems is resisted by the framing system or the adjoining floor or roof construction‡	Restrained
IV. Wood construction:	
All types	Unrestrained

*As recommended by ASTM E119, Appendix X3.

†Floor and roof systems can be considered restrained when they are tied into walls with or without tie beams, the walls being designed and detailed to resist thermal thrust from the floor or roof system.

‡For example, resistance to potential thermal expansion is considered to be achieved when:

(1) Continuous structural concrete topping is used.

(2) The space between the ends of precast units or between the ends of units and the vertical face of supports is filled with concrete or mortar.

(3) The space between the ends of precast units and the vertical faces of supports, or between the ends of solid or hollow-slab units does not exceed 0.25% of the length for normal-weight concrete members or 0.1% of the length for structural lightweight concrete members.

4.12.8 Temperatures of Fire-Exposed Structural Steel Elements

Basic heat-transfer principles indicate that the rate of temperature change of a beam or column varies inversely with mass and directly with the surface area through which heat is transferred to the member. Thus the weight-to-heated perimeter ratio *W/D* of a structural steel member significantly influences the temperature that the member will experience when exposed to fire. *W* is the weight per unit length of the member (lb/ft), and *D* is the inside perimeter of the fire protection material (in). Expressions for calculating *D* are illustrated in Fig. 4.15 for columns and beams with either contour or box protection. In short, the weight-to-heated-perimeter ratio defines the **thermal size** of a structural member.

Since the temperature of a structural steel member is strongly influenced by *W/D*, it therefore follows that the required thickness of fire-protection material is also strongly influenced by *W/D*.

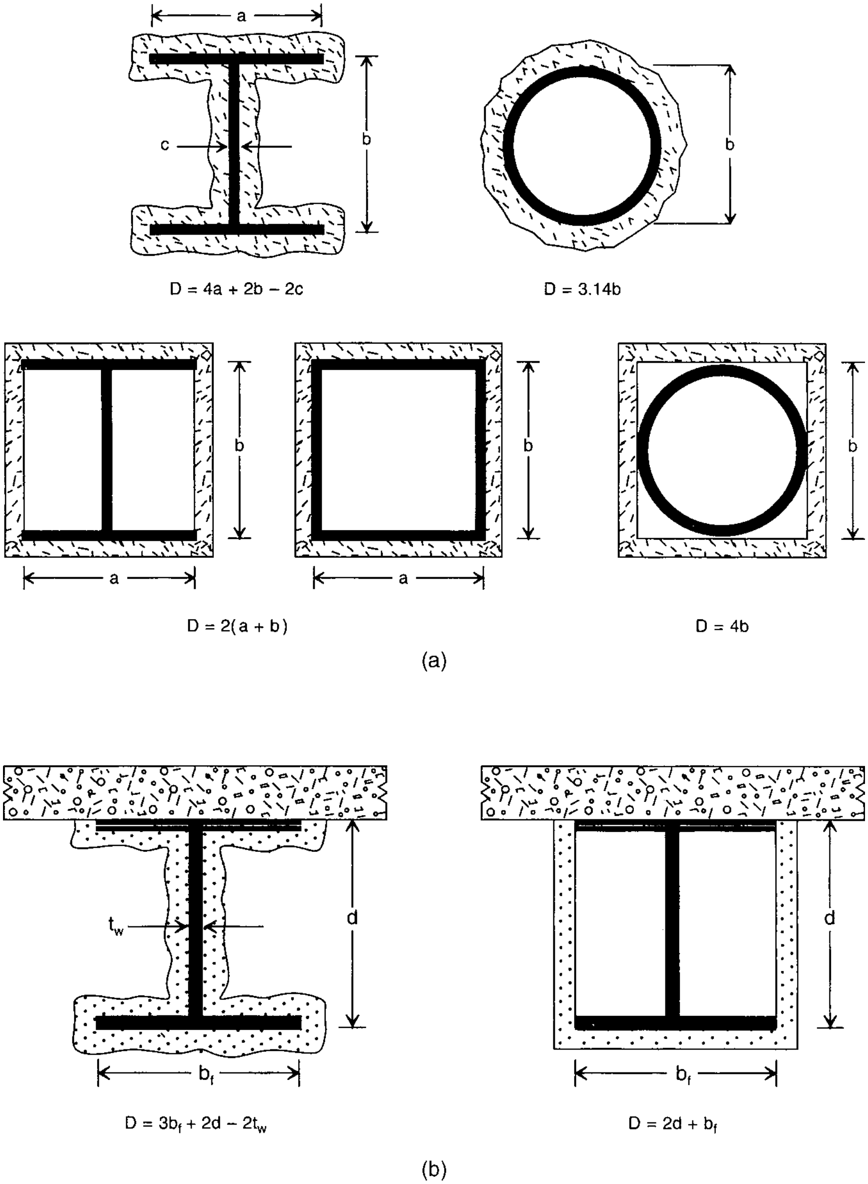


FIGURE 4.15 Equations for determining the heated perimeter *D* of structural steel members (*a*) columns and (*b*) beams.

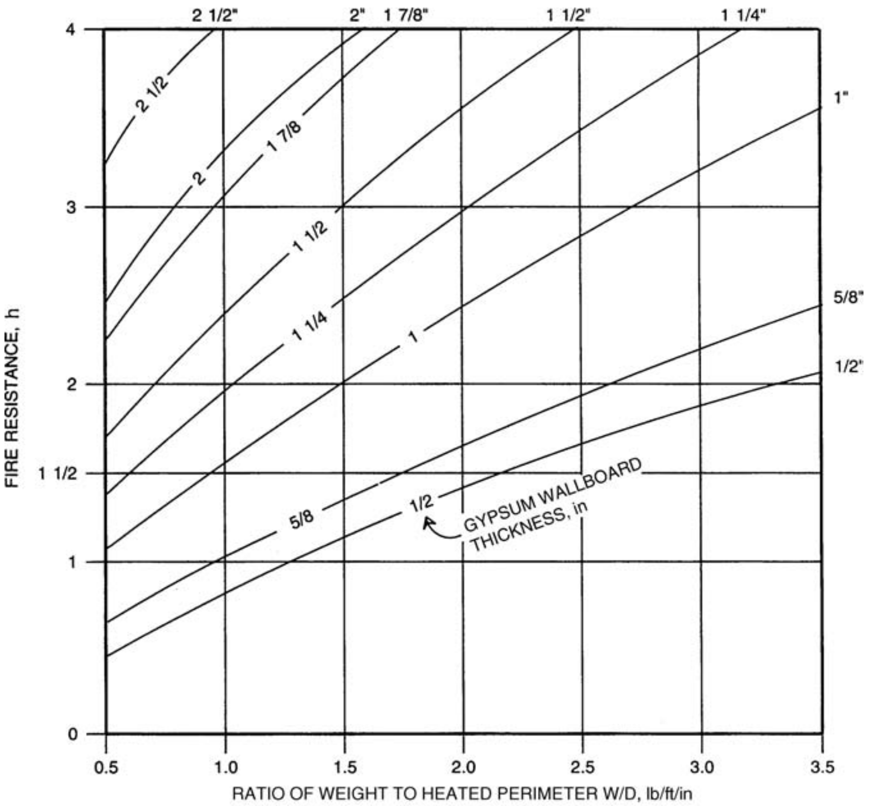


FIGURE 4.16 Variation in fire resistance of structural steel column with weight-to-heated perimeter ratios and gypsum wallboard thickness.

This interrelationship is clearly illustrated in Fig. 4.16, which gives the fire resistance of steel columns protected with different thickness of gypsum wallboard as a function of *W/D*. The curves show that in determination of fire resistance, *W/D* is significant, as is the thickness of the fire-protection material.

In recognition of this basic principle, several semiempirical design equations have been developed for determining the thicknesses of fire protection for structural steel elements as a function of *W/D* for specific fire-resistance ratings. Some of these equations have been incorporated into the Underwriters Laboratories *Fire-Resistance Directory* and are also described in the following publications. ASCE, “Standard Calculation Methods for Structural Fire Protection,” ASCE/SFPE 29-99, 1999; *International Building Code*, 2003; and AISC, “Fire Resistance of Structural Steel Framing,” Design Guide 19, 2003. These calculation methods also have been recognized by model building codes and are widely used in design of cost-effective, fire-resistant steel buildings.

4.12.9 Rational Fire Design

Building code requirements for structural fire protection are generally prescriptive and based on standard (ASTM E119) fire tests. This approach suffers from the following significant limitations:

- The standard fire exposure is arbitrary and does not necessarily represent real building fires. In many cases, real fires result in high temperatures of short duration.

- In effect, the standard fire test presumes that structural members will be fully loaded at the time of a fire. It is unlikely that maximum structural loads will occur simultaneously with an uncontrolled severe fire.
- Given the scale of available laboratory facilities, the structural interaction of the entire framing system cannot be directly evaluated. In effect, ASTM E119 unrestrained ratings presume virtually no structural interaction, i.e., simple supports without continuity. To some degree, structural interaction is indirectly considered in establishing restrained ratings. The boundary conditions are arbitrary, however, and the extension to real buildings is largely based on judgment.

As a consequence of these shortcomings, a more rational engineering design for structural fire protection is desirable. This advanced work can be performed by qualified structural/fire consultants in accordance with performance-based design concepts and standards, using state-of-the-art fire and structural modeling tools. Standards of a more rational type have been developed and are now used routinely in Japan, Australia, and throughout much of Europe (European Convention for Constructional Steelwork, Model Code on Fire Engineering, 2001), and are being developed in the United States by the American Society of Civil Engineers in cooperation with the Society of Fire Protection Engineers. Also, criteria for rational structural design for fire conditions are now provided by the AISC in Appendix 4 of the "Specification for Structural Steel Buildings," 2005. Topics addressed include engineering analysis, determination of heat input, thermal expansion, and degradation of mechanical properties at elevated temperatures.

(*The SFPE Handbook of Fire Protection Engineering*, National Fire Protection Association, Quincy, Mass.)

CHAPTER 5
CRITERIA FOR BUILDING DESIGN

Roger L. Brockenbrough, P.E.
President
R. L. Brockenbrough & Associates, Inc.
Pittsburgh, Pennsylvania

Buildings should be designed to meet the requirements of state or local building codes. For the design of the structural steel framework, most codes refer to the specifications of the American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, IL 60601-1802 (www.aisc.org).

In 2005 the AISC published a new document, "Specification for Structural Steel Buildings." Referred to herein as the AISC Specification, it is known as a **unified** specification because it combines allowable strength design (ASD) and load and resistance factor design (LRFD) in a single specification. This chapter provides a summary of the main provisions of the AISC Specification and insight for design application.

The AISC Specification now provides common requirements for nominal strength, such that the ASD and LRFD design methods now differ mainly in the alternative use of safety factors or load and resistance factors. Also included are design requirements for certain members that were previously addressed in separate documents, such as angle members and hollow structural sections (HSS). The AISC Specification applies to the design, fabrication, and erection of structural steel for buildings, as well as for other structures designed, fabricated, and erected in a manner similar to buildings, i.e., with building-like vertical and lateral load-resisting systems.

The design of structural steel in seismic zones should also comply with the AISC's "Seismic Provisions for Structural Steel Buildings" (2005). The seismic provisions apply when the seismic response modification coefficient R, as specified in the applicable building code, is greater than 3.0.

The design of nuclear structures should comply with the requirements for ASD given in the "Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures in Nuclear Facilities," including Supplement No. 2 (ANSI/AISC N690-94), or with the "Load and Resistance Factor Design of Safety-Related Steel Structures for Nuclear Facilities" (ANSI/AISC N690L-03).

The design of structural members other than HSS that are cold-formed to shape from steel not more than 1 in (25 mm) thick should be based on the "North American Specification for the Design of Cold-Formed Steel Structural Members" (2001), and "Supplement" (2004), published by the American Iron and Steel Institute (AISI), Washington, D.C., as discussed in Chap. 9.

5.1 MATERIALS, DESIGN METHODS, AND OTHER CONSIDERATIONS

5.1.1 Materials

The AISC Specification recognizes the following structural steel materials as designated by ASTM Standards. Information on most of these is given in Chap. 1.

Hot-rolled structural shapes: A36, A529, A572, A588, A709, A913, and A992
Structural tubing: A500, A501, A618, A847
Pipe: A53 (Grade B)
Plates: A36, A242, A283, A514, A529, A572, A588, A709, A852, A1011
Bars: A36, A529, A572, A709
Sheets: A606, A1011 (Grades SS, HSLAS, and HSLAS-F)

The AISC Specification treats structural tubing (round, square, and rectangular) and pipe collectively as HSS, hollow structural sections.

The AISC Specification also specifies the materials to be used for other components such as bolts and nuts, anchor rods, shear studs, filler metal and flux for welding, and steel castings.

To minimize the likelihood of an undesirable premature fracture, Charpy V-notch impact testing and minimum toughness requirements apply to certain thick sections. Included are hot-rolled shapes with a flange thickness exceeding 2 in (50 mm) used as members subject to primary tensile forces due to tension or flexure, and spliced using complete-joint-penetration groove welds that fuse through the thickness. Also included are cross sections built up from plates with a thickness exceeding 2 in (50 mm) subject to similar usage. In such cases, the impact test must meet a minimum average value of 20 ft·lb (27 J) absorbed energy at +70°F (+21°C). Some exceptions apply.

5.1.2 Design Methods

Under the AISC Specification, members and connections may be designed according to the provisions of either load and resistance factor design (LRFD) or allowable strength design (ASD). The strength required of structural members and connections must be determined by structural analysis for the appropriate factored loads and load combinations stipulated in the applicable building code (see Chap. 4). In the absence of a building code, the loads and load combinations should be according to American Society of Civil Engineers (ASCE), ASCE 7, “Minimum Design Loads for Buildings and Other Structures.” Note that the loads, load factors, and load combinations for LRFD and ASD are generally different.

Design is based on the principle that no applicable strength or serviceability limit state is exceeded when the structure is subjected to all appropriate load combinations. Strength limit states are related to safety and maximum load-carrying capacity. Serviceability limit states are related to performance, such as deflection or vibration considerations, under normal service conditions.

LRFD Strength Requirements. A design satisfies the strength requirements of the AISC Specification when the **design strength** of each structural component equals or exceeds the **required strength** determined on the basis of the LRFD load combinations. This is expressed as

$$R_u \leq \phi R_n \tag{5.1}$$

where R_u = required strength (LRFD)
R_n = nominal strength (as given in the Specification)
ϕ = resistance factor
ϕR_n = design strength

ASD Strength Requirements. A design satisfies the strength requirements of the AISC Specification when the **allowable strength** of each structural component equals or exceeds the **required strength,** determined on the basis of the ASD load combinations. This is expressed as

$$R_a \leq \frac{R_n}{\Omega} \tag{5.2}$$

where R_a = required strength (ASD)
R_n = nominal strength (as given in the Specification)
Ω = safety factor
R_n/Ω = allowable strength

5.1.3 Analysis Methods

Elastic, inelastic, or plastic analysis is permitted. However, members designed on the basis of plastic hinging are limited to those with steels having a specified minimum yield stresses not exceeding 65 ksi (450 MPa), and are subject to certain other provisions.

Beams and girders composed of compact sections, and satisfying unbraced length requirements, including composite members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the gravity loading and computed by an elastic analysis, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction, as detailed in App. 1 of the Specification, is not permitted for moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed the following: $0.15\phi_c A_g F_y$ for LRFD or $0.15\phi_c A_g F_y / \Omega_c$ for ASD, where

A_g = gross area (total cross-sectional area), in^2 (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
ϕ_c = resistance factor for compression = 0.90 (LRFD)
Ω_c = safety factor for compression = 1.67 (ASD)

5.1.4 Classification of Connections

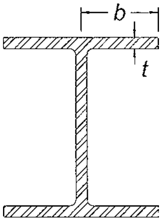
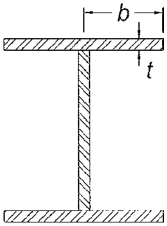
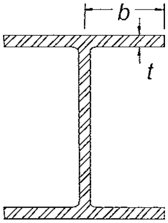
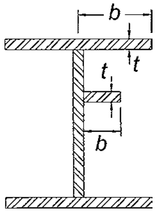
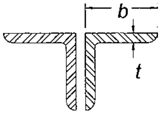
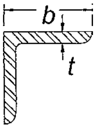
For design purposes, connections are classified as either **simple connections** or **moment connections.** A simple connection is one that transmits negligible moment. Unrestrained rotation (pinned condition) is assumed in design. However, it is important to make sure that details are such that they can withstand the rotation that will develop. Inelastic rotation is permitted.

Moment connections can be one of two types: FR (fully restrained) or PR (partially restrained). A FR connection transfers moment with negligible rotation. It must have sufficient strength and stiffness to maintain the angle between connected members, such as in a rigid frame. A PR connection transfers moment, but the rotation is not negligible. The moment–rotation relationship, generally nonlinear, must be documented in the technical literature or established by analytical or experimental means, and the connection components must have adequate strength, stiffness, and deformation capacity at the limit states.

5.1.5 Classification of Sections for Local Buckling

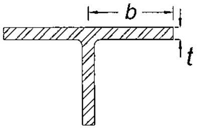
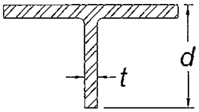
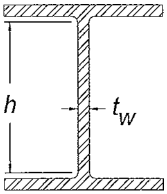
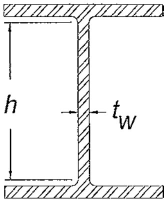
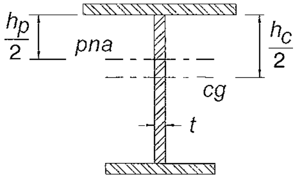
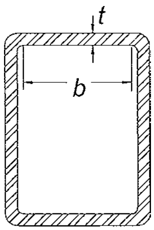
Steel sections subjected to compression from direct forces or flexure may be classified as compact, noncompact, or slender-element sections. For a section (such as an I or a box section) to qualify as compact, the flanges must be continuously connected to the web or webs and the width–thickness ratios of compression elements must not exceed a certain limiting width–thickness ratio, λ_p. If the width–thickness ratio of any compression element exceeds λ_p, but does not exceed the limit λ_r, the section is termed noncompact. Further, if the width–thickness ratio of any element exceeds λ_r, the section is referred to as a slender-element compression section. Values of λ_p and λ_r for numerous cases are given in Table 5.1. Further discussion of local buckling, particularly as applied to cold-formed members, is given in Chap. 9.

TABLE 5.1 Limiting Width–Thickness Ratios for Compression Elements

Case	Description of element	Width–thickness ratio	Limiting width–thickness ratios		Example
			λ_p (compact)	λ_r (noncompact)	
			Unstiffened elements		
1	Flexure in flanges of rolled I-shaped sections and channels in flexure	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_cE/F_L}$ [a] [b]	
3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact, and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_cE/F_y}$ [a]	
5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

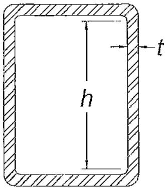
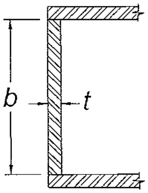
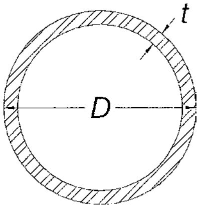
(*Continued*)

TABLE 5.1 Limiting Width–Thickness Ratios for Compression Elements (*Continued*)

Case	Description of element	Width–thickness ratio	Limiting width–thickness ratios λ_p (compact)	 λ_r (noncompact)	Example
			Unstiffened elements		
7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
8	Uniform compression in stems of tees	b/t	NA	$0.75\sqrt{E/F_y}$	
			Stiffened elements		
9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
11	Flexure in webs of singly symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.089\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	

(*Continued*)

TABLE 5.1 Limiting Width–Thickness Ratios for Compression Elements (*Continued*)

Case	Description of element	Width–thickness ratio	Limiting width–thickness ratios		Example
			λ_p (compact)	λ_r (noncompact)	
			Stiffened elements		
13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections: In uniform compression In flexure	 D/t D/t	 NA $0.07E/F_y$	 $1.11E/F_y$ $0.31E/F_y$	

[a] $K_c = 4/\sqrt{h/t_w}$ and $0.35 \leq K_c \leq 0.76$.

[b] $F_L = 0.7F_y$ for minor-axis bending, major-axis bending of slender-web built-up I-shaped members, and major-axis bending of compact- and noncompact-web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_yS_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact- and noncompact-web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

5.2 DESIGN FOR STABILITY

Stability must be provided, both for individual compression elements and for the structure as a whole. Lateral stability of a structure is typically provided by moment frames, braced frames, shear walls, or combined systems.

The AISC Specification requires that the influence of the following be considered: second-order effects (such as P–Δ and P–δ effects); flexural, shear, and axial deformations; geometric imperfections; and member stiffness reduction due to residual stresses. P–Δ indicates the effect of loads acting on the displaced joints, and P–δ indicates the effect of loads acting on the deflected shape of a member between joints. Many structural analysis computer programs can provide second-order elastic analyses.

5.2.1 Determination of Second-Order Effects

In structures designed on the basis of an inelastic analysis, special provisions set forth in App. 7 to the AISC Specification must be met. In structures designed on the basis of elastic analysis, individual member stability and structure stability can be provided by the following.

Moments and forces may be obtained from a general second-order elastic analysis that considers both P–Δ and P–δ effects, where equilibrium is satisfied for the deformed geometry. Alternatively, moments and forces may be obtained by amplification of a traditional first-order elastic analysis, where equilibrium is satisfied for the original or undeformed structure. In low-rise moment frames, the amplification of axial forces is often negligible, but it becomes more significant in high-rise structures. Amplified values of the required flexural strength and axial strength may be calculated from the following equations:

$$M_r = B_1\, M_{\mathrm{nt}} + B_2\, M_{\mathrm{lt}} \tag{5.3}$$

$$P_r = P_{\mathrm{nt}} + B_2 P_{\mathrm{lt}} \tag{5.4}$$

where

$$B_1 = \frac{C_m}{(1-\alpha P_r/P_{e1})} \geq 1 \tag{5.5}$$

$$B_2 = \frac{1}{1-\alpha \sum P_{\mathrm{nt}}/\sum P_{e2}} \geq 1 \tag{5.6}$$

$$\alpha = 1.0 \text{ (LRFD)} = 1.60 \text{ (ASD)}$$

The following definitions apply:

M_r = required second-order flexural strength, kip·in (N·mm)

M_{nt} = first-order moment, assuming no lateral translation of frame, kip·in (N·mm)

M_{lt} = first-order moment as a result of lateral translation of frame only, kip·in (N·mm)

P_r = required second-order axial strength, kips (N)

P_{nt} = first-order axial force, assuming no lateral translation of frame, kips (N)

ΣP_{nt} = total vertical load supported by a story, including gravity column loads, kips (N)

P_{lt} = first-order axial force, as a result of lateral translation of frame only, kips (N)

C_m = coefficient assuming no lateral translation of frame, the value of which is taken as follows:

- For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) \tag{5.7}$$

  where M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when it is bent in single curvature.
- For beam-columns subjected to transverse loading between supports, the value of C_m may be determined either by analysis or taken conservatively as 1.0.

P_{e1} = elastic critical buckling load of the member in the plane of bending, kips (N)

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \tag{5.8}$$

ΣP_{e2} = elastic critical buckling resistance for the story determined by sidesway buckling analysis, kips (N)

- For moment frames where sidesway buckling effective length K_2 factors are determined for the columns, the following equation may be used:

$$\sum P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2} \tag{5.9a}$$

Alternatively, and for all types of lateral load resisting systems, the following equation may be used:

$$\sum P_{e2} = R_M \frac{\sum HL}{\Delta_H} \tag{5.9b}$$

where $R_M = 1.0$ for braced-frame systems and $R_L = 0.85$ for moment-frame and combined systems, unless a larger value is justified by analysis.

E = modulus of elasticity, $E = 29{,}000$ ksi (200,000 MPa)

I = moment of inertia in the plane of bending, in^4 (mm^4)

L = story height, in (mm)

K_1 = effective length factor in the plane of bending, calculated on the basis of no sidesway, set equal to 1.0 unless analysis indicates a smaller value may be used

K_2 = effective length factor in the plane of bending, calculated for a sidesway buckling analysis

Δ_H = first-order interstory drift due to lateral forces, in (mm)

ΣH = story shear produced by lateral forces used to compute Δ_H, kips (N)

5.2.2 Design Requirements

Where required strengths are obtained by second-order analyses, the analyses are subject to the following. For LRFD design, analyses should be made using the LRFD load combinations. For ASD design, make the analyses using 1.6 times the ASD load combinations, and then divide the results by 1.6 to obtain the required strength. Note, however, that the B_1 and B_2 amplifiers used in the amplified first-order analysis (Art. 5.2.1) already include the 1.6 multiplier.

Where B_2 [Eq. (5.6)] > 1.5, required strengths must be determined from a direct analysis method given in App. 7 of the AISC Specification. This method is optional where $B_2 \leq 1.5$. Where $B_2 \leq 1.5$, required strengths can be determined from either a second-order analysis, such as is outlined in Art. 5.2.1, or if conditions 1–3 that follow are satisfied, by a direct first-order analysis.

Where a second-order analysis is used, all gravity load combinations must include a minimum lateral load at each level of 0.002 times the design gravity load at that level. The lateral load must be considered independently in two orthogonal directions. Where the ratio of second-order drift to first-order drift does not exceed 1.1, compression members may be designed using a K factor of 1.0.

The conditions for a direct first-order analysis are as follows.

1. Select a member with yield strength sufficient to satisfy

$$\alpha P_r < 0.5 P_y \tag{5.10}$$

where $\alpha = 1.0$ (LRFD) $= 1.60$ (ASD)
P_r = required axial compressive strength, kips (N)
P_y = member yield strength AF_y, kips (N)

2. Include in all load combinations, applied in combination with other loads at each level of the structure, an additional load N_i, given by

$$N_i = 2.1(\Delta/L)Y_i > 0.0042Y_i \tag{5.11}$$

where Y_i = gravity load (from LRFD load combination or 1.6 times ASD load combination) on level i, kips (N)
L = story height, in (mm)
Δ = first-order interstory drift due to design loads, in (mm). Calculate Δ at strength loads. Where Δ varies over the plan of the structure, take Δ as the average drift weighed in proportion to vertical load or, alternatively, as the maximum drift.
Δ/L = maximum ratio of Δ to L for all stories

The additional load N_i must be considered independently in two orthogonal directions.

3. Apply the nonsway amplification factor B_1 [Eq. (5.5)] to the total member moments.

5.3 DESIGN OF TENSION MEMBERS

For tension member design, the limit states of tensile yielding in the gross section and tensile rupture in the net section must both be considered. Also, it is important to meet these limit states in both the body of the member and at connections. The design tensile strength (LRFD) $\phi_t P_n$ and the allowable tensile strength (ASD) P_n/Ω_t is the lower of these two limit states.

For yielding in the gross section,

$$P_n = F_y A_g \tag{5.12}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

For rupture in the net section,

$$P_n = F_u A_e \tag{5.13}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where A_e = effective net cross-sectional area, in^2 (mm^2)
A_g = gross cross-sectional area of member, in^2 (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
F_u = specified minimum tensile strength, ksi (MPa)

When members without holes are fully connected by welds, the effective net area is as defined in Art. 5.3.2. When holes are present in a member with welded-end connections, and in the case of plug or slot welds, use the net area through the holes.

5.3.1 Net Area

The net area A_n of a member is the sum of the products of the thickness and the net width of each element. In net-width calculations for tension and shear, take the width of a bolt hole as $^1/_{16}$-in (2 mm) greater than the nominal dimension of the hole. For a chain of holes extending across an element in a diagonal or zigzag line, the net width of the part is obtained by deducting from the gross width the sum of the diameters or slot dimensions of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$, where s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in (mm), g = transverse center-to-center spacing (gage) between fastener gage lines, in (mm). For angles, the gage for holes in opposite adjacent legs is taken as the sum of the gages from the back of the angles less the thickness. In the design of splice plates for connections, A_n is limited to $0.85A_g$.

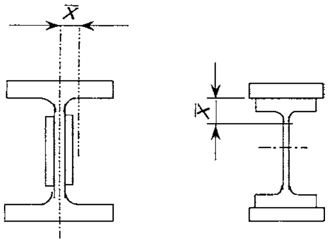
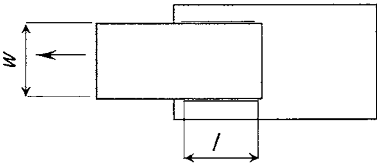
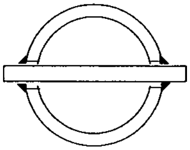
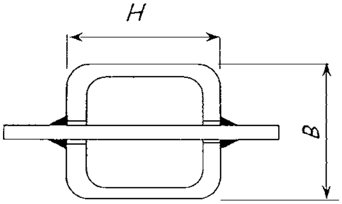
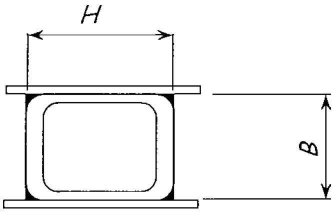
5.3.2 Effective Net Area

Because of a phenomenon known as shear lag, stresses are not distributed uniformly over the cross section at a connection when each element of the cross section is not attached. In such cases, because the net area is not fully effective in transferring tensile forces, an effective net area of tension members must be determined:

$$A_e = A_n U \tag{5.14}$$

where U is the shear lag factor, A_n, Table 5.2. The AISC Specification requires that connections for members such as angles (single or double) and WT sections be proportioned such that U is no less than 0.60, unless the members are designed for the effect of eccentricity.

TABLE 5.2 Shear Lag Factors for Connections to Tension Members

Case	Description of element	Type of connection	Shear lag factor U	Example
1	All tension members	Tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds	$U = 1.0$	—
2	All tension members except plates and HSS (alternatively for W, M, S, and HP, see Case 7)	Tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds	$U = 1 - \bar{x}/l$	
3	All tension members	Tension load is transmitted by transverse welds to some but not all of the cross-sectional elements	$U = 1.0$ A_n = area of directly connected elements	—
4	Plates	Tension load is transmitted by longitudinal welds only	$l \geqq 2w$..........$U = 1.0$ $2w > l \geqq 1.5w$...$U = 0.87$ $1.5w > 1 \geqq w$....$U = 0.75$	
5	Round HSS	Single concentric gusset plate	$l \geqq 1.3D$......$U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ where $\bar{x} = D/\pi$	
6	Rectangular HSS	Single concentric gusset plate	$l \geqq H$.......$U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		Two side gusset plates	$l \geqq H$.......$U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	

(*Continued*)

TABLE 5.2 Shear Lag Factors for Connections to Tension Members (*Continued*)

Case	Description of element	Type of connection	Shear lag factor U	Example
7	W, M, S, or HP shapes (if U is calculated per Case 2, the larger value is permitted to be used)	Flange connected with three or more fasteners per line in direction of loading	$bf \geq {}^{2}/_{3}d$......$U = 0.90$ $bf < {}^{2}/_{3}d$........$U = 0.85$	—
		Web connected with four or more fasteners in the direction of loading	$U = 0.70$	—
8	Single angles (if U is calculated per Case 2, the larger value is permitted to be used)	Four or more fasteners per line in direction of loading	$U = 0.80$	—
		Fewer than four fasteners per line in the direction of loading	$U = 0.60$	—

l = connection length.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

5.3.3 Built-up Members

For tension members built up from plates and shapes, perforated cover plates or tie plates without lacing can be used on the open sides. Tie plates should have a length not less than two-thirds the distance between the lines of welds or bolts connecting them to the member, and a thickness not less than ${}^{1}/_{50}$th of the distance between such lines. Also, the longitudinal spacing of intermittent welds or fasteners of tie plates should not exceed 6 in (150 mm). The longitudinal spacing and edge distance of connectors is further limited by rules that apply to all connections (see Art. 5.9.7). It is considered good practice to limit the slenderness ratio of any component between connections to 300.

5.3.4 Pin-Connected Members

The design tensile strength $\phi_t P_n$ and the allowable tensile strength P_n/Ω_t of pin-connected members is determined as the least of the values calculated for the limit states of tensile rupture, shear rupture, bearing, and yielding. Use $\phi_t = 0.75$ (LRFD) and $\Omega_t = 2.00$ (ASD) for all limit states except yielding, for which $\phi_t = 0.90$ (LRFD) and $\Omega_t = 1.67$ (ASD).

For tensile rupture on the net effective area,

$$P_n = 2tb_{\text{eff}}F_u \tag{5.15}$$

For shear rupture on the effective area,

$$P_n = 0.6A_{sf}F_u \tag{5.16}$$

For bearing on the projected area of the pin,

$$P_n = 1.8A_{\text{pb}}F_y \tag{5.17}$$

For yielding in the gross section,

$$P_n = F_y A_g \tag{5.18}$$

The following definitions apply:

$A_{sf} = 2t(a + d/2)$, in^2 (mm^2)

a = shortest distance from edge of pin hole to edge of member, measured parallel to the direction of the force, in (mm)

$b_{\mathrm{eff}} = 2t + 0.63$, in ($b_{\mathrm{eff}} = 2t + 16$, mm) but not more than actual distance from edge of hole to edge of part, measured normal to the force

d = pin diameter, in (mm)

t = plate thickness, in (mm)

Certain dimensional requirements apply. The pin hole should be located midway between the longitudinal edges of the member. When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole should not be more than $^1/_{32}$ in (1 mm) greater than the diameter of the pin. The width of the plate beyond the pin hole should not be less than $2b_{\mathrm{eff}} + d$ and the minimum extension a beyond the bearing end of the pin hole, parallel to the axis of the member, should not be less than $1.33b_{\mathrm{eff}}$. Plate corners beyond the pin hole can be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

The AISC Specification also gives provisions for the design of eyebars.

5.4 DESIGN OF COMPRESSION MEMBERS

The design compressive strength ϕP_n and the allowable compressive strength P_n/Ω are determined using $\phi_c = 0.90$ (LRFD) and $\Omega_c = 1.67$ (ASD) for all cases. The nominal compressive strength P_n is determined as the least value calculated for the limit states of flexural buckling, torsional buckling, and flexural-torsional buckling, as applicable. Flexural buckling is applicable for doubly symmetric and singly symmetric members. For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, the limit states of torsional or flexural-torsional buckling are also applicable.

An important factor in the design of compression members is the slenderness ratio, KL/r, where L = laterally unbraced length of member, in (mm), r = governing radius of gyration, in (mm), and K = effective length factor. Although the AISC Specification imposes no maximum slenderness limit, it is sometimes considered good practice to limit KL/r to 200 for members designed on the basis of compression.

The buckling coefficient K is the ratio of the effective column length to the unbraced length L. (Also see Art. 5.2.) Values of K depend on the support conditions of the column to be designed. The AISC Specification indicates that K should be taken as unity for columns in braced frames unless analysis indicates that a smaller value is justified. Analysis is required for determination of K for unbraced frames, but K should not be less than unity. Design values for K recommended by the Structural Stability Research Council for use with six idealized conditions of rotation and translation at column supports are illustrated in Fig. 5.1.

The following articles give the compressive strength of members without slender elements. Provisions for the latter may be found in Art. E7 of the AISC Specification.

5.4.1 Compressive Strength for Flexural Buckling

The nominal compressive strength P_n for the limit state of flexural buckling is

$$P_n = A_g F_{cr} \tag{5.19}$$

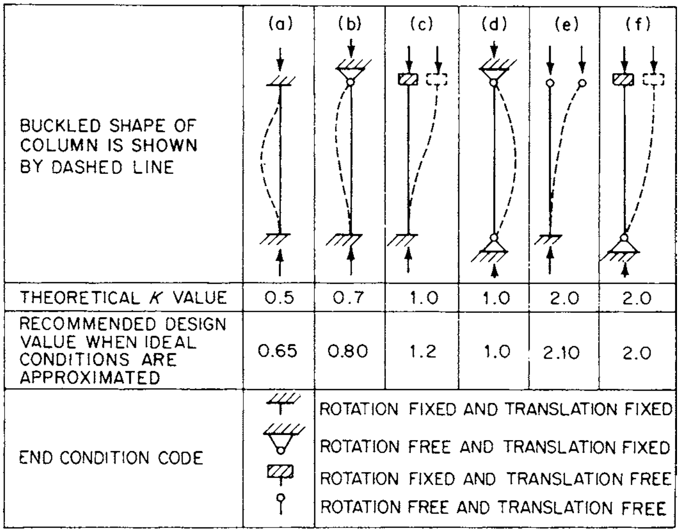
where F_{cr} is the buckling stress, determined according to Eqs. (5.20) and (5.21).

When $(KL/r) \le 4.71\sqrt{E/F_y}$ or $F_e \ge 0.44\, F_y$,

$$F_{cr} = (0.658^{F_y/F_e})F_y \tag{5.20}$$

When $(KL/r) > 4.71\sqrt{E/F_y}$ or $F_e < 0.44F_y$,

$$F_{cr} = 0.877F_e \tag{5.21}$$



	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL *K* VALUE	0.5	0.7	1.0	1.0	2.0	2.0
RECOMMENDED DESIGN VALUE WHEN IDEAL CONDITIONS ARE APPROXIMATED	0.65	0.80	1.2	1.0	2.10	2.0
END CONDITION CODE	ROTATION FIXED AND TRANSLATION FIXED ROTATION FREE AND TRANSLATION FIXED ROTATION FIXED AND TRANSLATION FREE ROTATION FREE AND TRANSLATION FREE					

FIGURE 5.1 Effective length factor *K* for columns.

where F_e = elastic buckling stress determined according to Eq. (5.22) or from the stability analysis:

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \tag{5.22}$$

5.4.2 Compressive Strength for Torsional and Flexural-Torsional Buckling

This article applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns. The elements of the members must have width–thickness ratios for axially compressed elements such that the sections are classified as compact or noncompact (see Art. 5.1.5). For single-angle members, see Art. 5.4.3.

The nominal compressive strength P_n is calculated from Eq. (5.19) based on the limit states of flexural-torsional buckling and torsional buckling. F_{cr} is determined as follows: For double-angle and tee-shaped compression members,

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right] \tag{5.23}$$

where F_{cry} is determined according to Eq. (5.20) or (5.21) for flexural buckling about the y axis of symmetry with $(KL/r) = (KL/r_y)$, and

$$F_{crz} = \frac{GJ}{A\bar{r}_o^2} \tag{5.24}$$

For all other cases, F_{cr} is determined from Eq. (5.20) or (5.21), but with F_e determined as follows:

For doubly symmetric members,

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y} \tag{5.25}$$

For singly symmetric members where y is the axis of symmetry,

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \tag{5.26}$$

For unsymmetric members, F_e is the lowest root of the cubic equation,

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \tag{5.27}$$

The following definitions apply:

K_z = effective length factor for torsional buckling
G = shear modulus of elasticity of steel = 11,200 ksi (77,200 MPa)
C_w = warping constant, in^6 (mm^6)
J = torsional constant, in^4 (mm^4)
I_x, I_y = moment of inertia about the principal axes, in^4 (mm^4)
x_o, y_o = coordinates of shear center with respect to the centroid, in (mm)
$\bar{r}_o$ = Polar radius of gyration about shear center, in (mm)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \tag{5.28}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{5.29}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x L/r_x)^2} \tag{5.30}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y L/r_y)^2} \tag{5.31}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{A\bar{r}_o^2} \tag{5.32}$$

For doubly symmetric I-shaped sections, C_w may be taken conservatively as $I_y d^2/4$. For tees and double angles, take C_w and x_o as 0.

5.4.3 Compressive Strength of Single-Angle Members

The compressive strength of single-angle members can be determined from Eqs. (5.19)–(5.21), using KL/r as given in this article and neglecting eccentricity, provided the angles are (1) loaded at the ends in compression through the same one leg, (2) attached by welding or by two-bolt-minimum connections, and (3) subjected to no intermediate transverse loads. For other conditions, see the AISC Specification. The modified slenderness ratios are intended to account indirectly for bending due to eccentricity of loading and end restraint from truss chords.

For equal-leg angles, or unequal-leg angles connected through the longer leg, which are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

When $0 \le (L/r_x) \le 80$,

$$\frac{KL}{r} = 72 + 0.75\frac{L}{r_x} \tag{5.33}$$

When $(L/r_x) > 80$,

$$\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \le 200 \tag{5.34}$$

For unequal-leg angles with leg length ratios less than 1.7 connected through the shorter leg, the KL/r from Eqs. (5.33) and (5.34) should be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r should not be less than $0.95L/r_z$.

For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

When $0 \le (L/r_x) \le 75$,

$$\frac{KL}{r} = 60 + 0.8\frac{L}{r_x} \tag{5.35}$$

When $(L/r_x) > 75$,

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \le 200 \tag{5.36}$$

For unequal-leg angles with leg length ratios less than 1.7, connected through the shorter leg, the KL/r from Eqs. (5.35) and (5.36) should be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r should not be less than $0.82L/r_z$.

The following definitions apply:

L = length of member between work points at truss chord centerlines, in (mm)

r_z = radius of gyration about minor axis, in (mm)

r_x = radius of gyration about axis parallel to connected leg, in (mm)

b_l = longer leg of angle, in (mm)

b_s = shorter leg of angle, in (mm)

5.4.4 Compressive Strength of Built-up Members

The compressive strength of built-up members comprised of two or more shapes interconnected by stitch bolts or welds should be determined from Eqs. (5.19) to (5.22), but with KL/r replaced by a modified column slenderness ratio $(KL/r)_m$ determined as follows.

For intermediate connectors that are snug-tight bolted,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{a}{r_i}\right)^2} \tag{5.37}$$

For intermediate connectors that are welded or fully tensioned bolted,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \tag{5.38}$$

where $(KL/r)_o$ = slenderness ratio of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, in (mm)

r_i = minimum radius of gyration of individual component, in (mm)

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in (mm)

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, in (mm)

The following dimensional requirements apply for built-up members. Individual components of compression members composed of two or more shapes should be connected to one another at intervals a such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the connectors, does not exceed three-fourths of the governing slenderness ratio of the built-up member. Use the least radius of gyration r_i to compute the slenderness ratio of each component part. The end connection must be welded or fully tensioned bolted with Class A or B faying surfaces. The end connection may be designed for the full compressive load in a bearing-type bolted connection, but the bolts must be fully tightened (see Art. 5.9.5).

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another must be connected by a weld having a length not less than the maximum width of the member, or by bolts spaced longitudinally not more than four diameters apart for a distance equal to $1\frac{1}{2}$ times the maximum width of the member.

Along the length of built-up compression members between the end connections, longitudinal spacing for intermittent welds or bolts should be adequate to transfer required forces. Where a component of a built-up compression member consists of an outside plate, and intermittent welds are provided along the edges of the components or bolts are provided on all gage lines at each section, the maximum spacing should not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 12 in (305 mm). When fasteners are staggered, the maximum spacing on each gage line should not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 18 in (460 mm).

Open sides of compression members built up from plates or shapes should be provided with continuous cover plates perforated with a succession of access holes. According to the AISC Specification, the unsupported width of such plates at access holes contributes to the design strength provided the following requirements are met: (1) the width–thickness ratio conforms to the limitations of Art. 5.1.5, (2) the ratio of hole length in the direction of stress to hole width of hole does not exceed 2, (3) the clear distance between holes in the direction of stress is not less than the transverse distance between the nearest lines of connecting fasteners or welds, and (4) the periphery of the holes has a radius no less than of $1\frac{1}{2}$ in (38 mm).

As an alternative to perforated cover plates, lacing can be used with tie plates at each end and at points where the lacing is interrupted. In members providing design strength, the end tie plates should have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates should have a length not less than one-half this distance. The thickness of tie plates should be not less than $\frac{1}{50}$th the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate should total at least one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates should be not more than six diameters and the tie plates should be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, must be so spaced that L/r of the flange included between their connections does not exceed three-fourths of the governing slenderness ratio for the member as a whole. Lacing must be proportioned to provide a shearing strength normal to the axis of the member equal to 2% of the compressive design strength of the member. The L/r ratio for lacing bars must not exceed 140 for single lacing systems, or 200 for double lacing systems. Join double lacing bars where they intersect. For single lacing bars in compression, take L as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member. For double lacing, take L as 70% of that distance. It is considered good practice to keep the inclination of lacing bars to the axis of the member to not less than 60° for single lacing or 45° for double lacing. Also, use double lacing or lacing made up of angles where distance between the lines of bolts or welds exceeds 15 in (380 mm).

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, and other limitations, see Art. 5.9.7.

5.5 DESIGN OF FLEXURAL MEMBERS

The design flexural strength ϕM_n and the allowable flexural strength M_n/Ω are determined using $\phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD) for all cases. The nominal flexural strength, M_n, is calculated from the applicable equations, which depend on the member cross section and the axis of bending. For cross sections and bending cases not discussed in this article, refer to the provisions given by the AISC Specification.

These design criteria apply to members subject to simple bending, loaded in a plane parallel to a principal axis that passes through the shear center, or restrained against twisting at load points. Also, at all points of support, members must be restrained against twisting (rotation about the longitudinal axis).

A common term used in the strength provisions is C_b, the lateral-torsional buckling modification factor for nonuniform moment diagrams, applicable when both ends of the unsupported segment are braced:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0 \tag{5.39}$$

where $M_{\max}$ = absolute value of maximum moment in the unbraced segment, kip·in (N·mm)
M_A = absolute value of moment at quarter point of the unbraced segment, kip·in (N·mm)
M_B = absolute value of moment at centerline of the unbraced segment, kip·in (N·mm)
M_C = absolute value of moment at three-quarter point of the unbraced segment, kip·in (N·mm)
R_m = cross-section parameter
= 1.0, doubly symmetric members
= 1.0, singly symmetric members subjected to single-curvature bending
= $0.5 + 2(I_{yc}/I_y)^2$, singly symmetric members in reverse-curvature bending
I_y = moment of inertia of section about y axis, in^4 (mm^4)
I_{yc} = moment of inertia about y axis referred to compression flange (use smaller flange if reverse curvature bending), in^4 (mm^4)

In singly symmetric members subjected to reverse-curvature bending, the lateral-torsional buckling strength must be checked for both flanges. The available flexural strength must be greater than or equal to the maximum moment causing compression within the flange under consideration.

Note that important simplifications apply. C_b can conservatively be taken as 1.0 for all cases. Also, for all cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$. For doubly symmetric members with no transverse loading between brace points, C_b reduces to 2.27 for the case of equal end moments of opposite sign, and to 1.67 when one end moment equals zero.

5.5.1 Doubly Symmetric Compact I-Shaped Members and Channels—Major Axis Bending

This article applies to doubly symmetric I-shaped members and channels, subjected to bending about their major axis, and having compact webs and compact flanges (see Art. 5.1.5). All current W, S, M, C, and MC shapes listed in ASTM A6, except W21 × 48, W14 × 99, W14 × 90, W12 × 65, W10 × 12, W8 × 31, W8 × 10, W6 × 15, W6 × 9, W6 × 8.5, and M4 × 6 have compact flanges for steels with $F_y \le 50$ ksi (345 MPa); all current W, S, M, HP, C, and MC shapes listed in ASTM A6 have compact webs for $F_y \ge 65$ ksi (450 MPa).

The nominal flexural strength M_n of the members discussed in this article is the lower of two limit states: yielding (plastic moment) and lateral-torsional buckling. The nominal flexural strength for **yielding (plastic moment)** is

$$M_n = M_p = F_y\, Z_x \tag{5.40}$$

where Z_x is the plastic section modulus about the x axis.

The nominal flexural strength for **lateral-torsional buckling** depends on the length between lateral braces L_b as related to certain limiting lengths, L_p and L_r.

When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

When $L_p < L_b \le L_r$,

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \tag{5.41}$$

When $L_b > L_r$,

$$M_n = F_{cr}S_x \le M_p \tag{5.42}$$

The following definitions apply: The lateral buckling stress F_{cr} is given by

$$F_{cr} = \frac{C_b\pi^2 E}{(L_b/r_{ts})^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{5.43}$$

The limiting lengths are

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} \tag{5.44}$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\,\frac{S_xh_o}{Jc}\right)^2}} \tag{5.45}$$

The term c in Eq. (5.45) is defined as follows:

For a doubly symmetric I-shape,

$$c = 1 \tag{5.46}$$

For a channel,

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \tag{5.47}$$

where h_o = distance between flange centroids.

Conservatively, in lieu of Eq. (5.43), F_{cr} may be taken simply as

$$F_{cr} = \frac{C_b\pi^2 E}{(L_b/r_{ts})} \tag{5.48}$$

and L_r may be taken as

$$L_r = \pi\, r_{ts}\sqrt{\frac{E}{0.7F_y}} \tag{5.49}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{5.50}$$

For doubly symmetric I-shapes with rectangular flanges, $C_w = I_y h_o^2/4$ and thus Eq. (5.50) becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x} \tag{5.51}$$

For other shapes, approximate r_{ts} as the radius of gyration of a section comprising the compression flange plus one-sixth of the web, which is

$$r_{ts} = \frac{b_f}{\sqrt{12[1+(1/6)(ht_w/b_f t_f)]}} \tag{5.52}$$

5.5.2 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges—Major Axis Bending

For doubly symmetric I-shaped members, subjected to bending about their major axis, and having compact webs but noncompact or slender flanges (see Art. 5.1.5), the effects of local buckling must also be considered. The nominal flexural strength M_n of such members is the lower of two limit states: lateral-torsional buckling and compression flange local buckling. The nominal flexural strength for lateral-torsional buckling is the same as given in Art. 5.5.1.

For sections with noncompact flanges, the nominal flexural strength for compression flange local buckling is given by

$$M_n = \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \tag{5.53}$$

For sections with slender flanges, the nominal flexural strength for compression flange local buckling is given by

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \tag{5.54}$$

where $\lambda_{pf} = \lambda_p$ = limiting slenderness for a compact flange (Art. 5.1.5)
$\lambda_{rf} = \lambda_r$ = limiting slenderness for a noncompact flange (Art. 5.1.5)
$\lambda = b_f/2t_f$
$k_c = 4/\sqrt{h/t_w}$ and $0.35 \le k_c \le 0.76$ for calculation purposes

5.5.3 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs—Major Axis Bending

This article applies to doubly symmetric and singly symmetric I-shaped members, subjected to bending about the major axis, with slender webs (see Art. 5.1.5) attached at the mid-width of the flanges. For this case, the nominal flexural strength M_n is the lower of four limit states: compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

For **compression flange yielding,**

$$M_n = R_{pg} F_y S_{xc} \tag{5.55}$$

For **lateral-torsional buckling,**

$$M_n = R_{pg} F_{cr} S_{xc} \tag{5.56}$$

When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.
When $L_p < L_b \le L_r$,

$$F_{cr} = C_b \left[F_y - (0.3F_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le F_y \tag{5.57}$$

When $L_b > L_r$,

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b / r_t)^2} \le F_y \tag{5.58}$$

where L_p is defined by Eq. (5.44) and

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \tag{5.59}$$

$R_{pg} =$ bending strength reduction factor

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w} \left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_{cr}}} \right) \le 1.0 \tag{5.60}$$

and a_w = ratio of two times the web area in compression, due to application of major axis bending moment alone, to the area of the compression flange components, but no more than 10. For I-shapes with a rectangular compression flange,

$$a_w = \frac{h_c t_w}{b_{\text{fc}} t_{\text{fc}}} \tag{5.61}$$

For I-shapes with a rectangular compression flange, the effective radius of gyration for lateral-torsional buckling, r_t, is

$$r_t = \frac{b_{\text{fc}}}{\sqrt{12[(h_o/d) + (1/6)a_w(h^2/h_o d)]}} \tag{5.62}$$

where b_{fc} = width of compression flange
h = clear distance between flanges
d = total member depth

For I-shapes with channel caps or cover plates attached to the compression flange, r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone.

For **compression flange local buckling,**

$$M_n = R_{pg} F_{cr} S_c \tag{5.63}$$

For compact flange sections,

$$F_{cr} = F_y \tag{5.64}$$

For noncompact flange sections,

$$F_{cr} = \left[F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \tag{5.65}$$

For slender flange sections,

$$F_{cr} = \frac{0.9Ek_c}{(b_f/2t_f)^2} \tag{5.66}$$

where

$$k_c = \frac{4}{\sqrt{h/t_w}} \quad \text{and} \quad 0.35 \le k_c \le 0.76 \tag{5.67}$$

and the λ slenderness terms are as defined with Eq. (5.54).

For **tension flange yielding,** when $S_{xt} \ge S_{xc}$, the limit state of tension flange yielding does not apply. When $S_{xt} < S_{xc}$,

$$M_n = F_y S_{xt} \tag{5.68}$$

where S_{xc} is the section modulus about the x axis referred to the compression flange and S_{xt} is the section modulus about the x axis referred to the tension flange.

5.5.4 I-Shaped Members and Channels—Minor Axis Bending

The nominal flexural strength M_n for I-shaped members and channels subjected to bending about the minor axis is the lower of two limit states: yielding (plastic moment) and flange local buckling.

For **yielding (plastic moment),**

$$M_n = M_p = F_y Z_x \le 1.6F_y S_y \tag{5.69}$$

For **flange local buckling,** three cases may arise. When the flanges are compact, this limit state does not apply. When the flanges are noncompact,

$$M_n = \left[M_p - (M_p - 0.7F_y S_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \tag{5.70}$$

When the flanges are classified as slender,

$$M_n = F_{cr} S_y \tag{5.71}$$

$$F_{cr} = \frac{0.69E}{(b_f/2t_f)^2} \tag{5.72}$$

where $\lambda = b/t$ = flange width-to-thickness ratio
λ_{pf} = limiting slenderness for a compact flange (Art. 5.1.5)
λ_{rf} = limiting slenderness for a noncompact flange (Art. 5.1.5)
S_y = minimum section modulus about y axis

5.5.5 Square and Rectangular HSS and Box-Shaped Members

This article applies to square and rectangular hollow structural sections (HSS), and doubly symmetric box-shaped members bent about either axis. The sections may have compact or noncompact webs, and compact, noncompact, or slender flanges (see Art. 5.1.5). The nominal flexural strength, M_n, is the lower of three limit states: yielding (plastic moment), flange local buckling, and web local buckling.

For **yielding (plastic moment),** use Eq. (5.40).

For **flange local buckling,** three cases may arise. When the flanges are compact, this limit state does not apply. When the flanges are noncompact,

$$M_n = M_p - (M_p - F_y S_{\text{eff}})\left(3.57\frac{b}{t}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \tag{5.73}$$

When the flanges are classified as slender,

$$M_n = F_y S_{\text{eff}} \tag{5.74}$$

where S_{eff} is the effective section modulus determined using the effective width b_e of the compression flange of width b, calculated as

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t}\sqrt{\frac{E}{F_y}}\right] \le b \tag{5.75}$$

For **web local buckling** of noncompact sections under pure flexure,

$$M_n = M_p - (M_p - F_y S_x)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \tag{5.76}$$

When the web is compact, the limit state of web local buckling does not apply.

5.5.6 Round HSS

For round HSS that have a diameter-to-thickness ratio D/t less than $0.45E/F_y$, the nominal flexural strength M_n is the lower of two limit states: yielding (plastic moment) and local buckling.

For **yielding (plastic moment),** use Eq. (5.40).

For **local buckling,** three cases may arise. When the section is compact, this limit state does not apply. When the section is noncompact,

$$M_n = \left(\frac{0.021E}{D/t} + F_y\right)S \tag{5.77}$$

When the section has slender walls,

$$M_n = F_{cr}S = \left(\frac{0.33E}{D/t}\right)S \tag{5.78}$$

5.5.7 Rectangular and Round Bars

The nominal flexural strength M_n of rectangular and round bars subjected to bending about either geometric axis is the lower of two limit states: yielding (plastic moment) and lateral-torsional buckling.

For **yielding (plastic moment),** when

$$\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$$

use Eq. (5.69).

For **lateral-torsional buckling,** of rectangular bars in major axis bending:

When $\dfrac{0.08E}{F_y} < \dfrac{L_b d}{t^2} \le \dfrac{1.9E}{F_y}$,

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \tag{5.79}$$

When $\dfrac{L_b d}{t^2} > \dfrac{1.9E}{F_y}$,

$$M_n = \left(\frac{1.9EC_b}{L_b d/t^2}\right)S_x \le M_p \tag{5.80}$$

where t = width of rectangular bar parallel to axis of bending, in (mm)
d = depth of rectangular bar, in (mm)
L_b = distance between points braced against lateral displacement of the compression region, or between points braced to prevent twist of the cross section, in (mm)

When rectangular bars are subjected to minor axis bending, and for all cases of round bars, this limit state does not apply.

5.5.8 Hole Reductions

For flexural members with holes, such as for bolts, the limit state of tensile rupture of the tension flange must also be considered.

When $F_u A_{\text{fn}} \ge Y_t F_y A_{\text{fg}}$, the limit state of tensile rupture does not apply.

When $F_u A_{\text{fn}} < Y_t F_y A_{\text{fg}}$, the nominal flexural strength at the location of the holes in the tension flange is limited to

$$M_n = \frac{F_u A_{\text{fn}}}{A_{\text{fg}}} S_x \tag{5.81}$$

where A_{fg} = gross tension flange area, in^2 (mm^2)
A_{fn} = net tension flange area, in^2 (mm^2) (see Art. 5.3.1)
Y_t = 1.0 for $F_y/F_u \le 0.8$
= 1.1 otherwise

5.5.9 Proportioning Limits for I-Shaped Members

Certain proportioning limits also apply for I-members. Singly symmetric I-shaped members must satisfy the following:

$$0.1 \le \frac{I_{yc}}{I_y} \le 0.9 \tag{5.82}$$

Stiffened, slender web I-shaped members must satisfy the following. When $a/h \le 1.5$,

$$\frac{h}{t_w} \le 11.7\sqrt{\frac{E}{F_y}} \tag{5.83}$$

When $a/h > 1.5$,

$$\frac{h}{t_w} \le \frac{0.42E}{F_y} \tag{5.84}$$

where a = clear distance between transverse stiffeners, in (mm).

For unstiffened girders, h/t_w must not exceed 260, and the ratio of the web area to the compression flange area must not exceed 10.

5.5.10 Cover Plates and Built-up Beams

Where flexural strength requirements vary along the length of a built-up member, it is often more cost effective to vary the flange thickness or width by splicing a series of plates than to use cover plates. With rolled shapes, it may be more effective just to use a stronger section. Nevertheless, where cover plates are used, certain proportioning rules should be followed.

The total cross-sectional area of cover plates of bolted girders should not exceed 70% of the total flange area. High-strength bolts or welds connecting flange to web, or cover plate to flange, must be proportioned to resist the total horizontal shear resulting from bending. The longitudinal distribution of bolts or intermittent welds should be in proportion to the shear. Also, the spacing should not exceed the maximum permitted for compression or tension members (see Arts. 5.3.3 and 5.4.3). Bolts or welds connecting flange to web must also be designed to transmit to the web any loads applied directly to the flange, except when the loads are transferred by direct bearing.

Partial-length cover plates should extend beyond the theoretical cutoff point and the extended portion attached by high-strength bolts in a slip-critical connection (see Art. 5.9.8) or fillet welds. The attachment must develop the cover plate's portion of the flexural design strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder should have continuous welds along both edges of the cover plate in the length a', defined by Eqs. (5.85) to (5.87) in terms of the cover plate width w, and be adequate to develop the cover plate's portion of the available strength in the beam or girder at the distance a' from the end of the cover plate.

When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate,

$$a' = w \tag{5.85}$$

When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate,

$$a' = 1.5w \tag{5.86}$$

When there is no weld across the end of the plate,

$$a' = 2w \tag{5.87}$$

5.6 DESIGN OF MEMBERS FOR SHEAR

This article pertains to the design of webs of singly or doubly symmetric members subject to shear in the plane of the web, to single angles and HSS, and for shear in the weak direction of singly or doubly symmetric shapes. For unsymmetric sections and other cases not discussed in this article, refer to the provisions given by the AISC Specification.

The design shear strength $\phi_v V_n$ and the allowable flexural strength V_n/Ω_v are determined using $\phi_v = 0.90$ (LRFD) and $\Omega_v = 1.67$ (ASD) for all cases except as noted. The nominal shear strength V_n can be calculated by one of two methods. In most cases, such as presented in Art. 5.6.1, it is calculated without utilizing postbuckling strength. Alternatively, for plate girders that meet certain conditions as presented in Art. 5.6.2, advantage may be taken of such strength, usually referred to as tension field action.

Note that the effect of any web openings on the nominal shear strength must be determined by a rational method. Adequate reinforcement must be provided when the required strength exceeds the available strength of the member at the opening. (See David Darwin, *Design of Steel and Composite Beams with Web Openings*, Steel Design Guide Series No. 2, AISC, Chicago, Ill.)

5.6.1 Members with Unstiffened or Stiffened Webs—Tension Field Action Neglected

This article applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web. The nominal shear strength V_n of unstiffened or stiffened webs for the limit states of shear yielding and shear buckling is given by

$$V_n = 0.6F_y A_w C_v \tag{5.88}$$

where A_w = overall depth times web thickness dt_w, in^2 (mm^2), and the coefficient C_v is as defined below.

For webs of rolled W shapes with $h/t_w \le 2.24\sqrt{E/F_y}$, only the limit state of shear yielding applies: $\phi_v = 1.00$ (LRFD), $\Omega_v = 1.50$ (ASD), and $C_v = 1.0$. All current W, S, and HP shapes listed in ASTM A6, except W44 × 230, W40 × 149, W36 × 135, W33 × 118, W30 × 90, W24 × 55, W16 × 26, and W12 × 14, meet this criterion for steels with $F_y \le 50$ ksi (345 MPa).

For webs of all other doubly symmetric shapes (excluding HSS) and singly symmetric sections and channels, $\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD), and C_v is as follows.

When $h/t_w \le 1.10\sqrt{k_v E/F_y}$, the limit state of shear yielding applies:

$$C_v = 1.0 \tag{5.89}$$

All current W, S, M, and HP shapes listed in ASTM A6, except W44 × 230, M12 × 11.8, M12 × 10.8, M12 × 10, M10 × 8, and M10 × 7.5 meet this criteria for steels with $F_y \le 50$ ksi (345 MPa).

When $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$, the limit state of shear buckling applies:

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{5.90}$$

When $h/t_w > 1.37\sqrt{k_v E/F_y}$, the limit state of shear buckling applies:

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \tag{5.91}$$

For unstiffened webs with $h/t_w < 260$,

$k_v = 1.2$ for the stem of tee shapes
$k_v = 5.0$ for all other cases

For stiffened webs,

$$k_v = 5 \qquad \text{when } \frac{a}{h} > 3.0 \text{ or } \frac{a}{h} > \left[\frac{260}{(h/t_w)}\right]^2 \tag{5.92}$$

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad \text{for all other cases} \tag{5.93}$$

where a = distance between transverse stiffeners, in (mm)
h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in (mm)
h = for built-up welded sections, the clear distance between flanges, in (mm)
h = for built-up bolted sections, the distance between fastener lines, in (mm)
h = for tees, the overall depth, in (mm)

Transverse stiffeners are not required where $h/t_w \le 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength, determined from the preceding equations with $k_v = 5$. However, where transverse stiffeners are used to develop the web shear strength, they must have a moment of inertia (about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners) not less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \tag{5.94}$$

When single stiffeners are used with rectangular flange plates, they should be attached to the compression flange to resist torsion. Also, when lateral bracing is attached to stiffeners, singly or in pairs, the stiffeners should be connected to the compression flange to transmit 1% of the total flange stress, unless the flange is composed only of angles.

Transverse stiffeners may be stopped short of the tension flange unless needed to transmit a concentrated load or reaction by bearing. Welds for web attachment of transverse stiffeners should be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. Intermittent fillet welds connecting stiffeners to the girder web should be spaced such that the clear distance between them does not exceed $16t_w$ nor 10 in (250 mm). If bolts are used instead, they should be spaced not more than 12 in (305 mm) on center.

5.6.2 Plate Girders with Tension Field Action

Design of webs on the basis of tension field action typically reduces the required web thickness but increases the number of stiffeners. Thus, possible reductions in material cost must be balanced against increases in fabrication cost. The use of this design method requires that the web plate be supported on all four sides by flanges or stiffeners. Tension field action may not be considered for end panels of members with transverse stiffeners, nor for the following cases:

$a/h > 3.0$

$a/h > [260/(h/t_w)]^2$

$2A_w/(A_{fc} + A_{ft}) > 2.5$

h/b_{fc} or $h/b_{ft} > 6.0$

where a, h, t_w, and A_w are as defined in Art. 5.6.1, A_{fc} and A_{ft} are the cross-section areas of the compression and tension flanges, and b_{fc} and b_{ft} are the widths of the compression and tension flanges. Where tension field action is not applicable, the nominal shear strength must be determined according to Art. 5.6.1.

The nominal shear strength, V_n, for the limit state of tension field yielding is calculated as follows.

When $h/t_w \leq 1.10\sqrt{k_v E/F_y}$,

$$V_n = 0.6F_y A_w \tag{5.95}$$

When $h/t_w > 1.10\sqrt{k_v E/F_y}$,

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \tag{5.96}$$

where k_v and C_v are as defined in Art. 5.6.1.

Transverse stiffeners in girders designed for tension field action must meet the requirements discussed in Art. 5.6.1 and are also subject to the following two limitations:

$$\left(\frac{b}{t}\right)_{st} = \leq 0.56\sqrt{\frac{E}{F_y}} \tag{5.97}$$

$$A_{st} > \frac{F_y}{F_{yst}}\left[0.15D_s h t_w (1 - C_v)\frac{V_r}{V_c} - 18t_w^2\right] \geq 0 \tag{5.98}$$

where $(b/t)_{st}$ = stiffener width–thickness ratio
F_{yst} = specified yield stress of the stiffener material, ksi (MPa)
C_v = coefficient defined in Art. 5.6.1
D_s = 1.0 for stiffeners in pairs
= 1.8 for single-angle stiffeners
= 2.4 for single-plate stiffeners
V_r = required shear strength at the location of the stiffener, kips (N)
V_c = available shear strength; ϕV_n (LRFD) or V_n/Ω (ASD) with V_n as given by Eq. (5.95) or (5.96), as applicable

5.6.3 Shear Strength of Other Members

For **single angles,** calculate the nominal shear strength V_n using Eq. (5.88) with $C_v = 1.0$ and $A_w = bt$, where b = width of the leg resisting the shear force, in (mm). Also, $k_v = 1.2$.

For **rectangular HSS and box members,** calculate V_n from the equations in Art. 5.6.2, with $A_w = 2ht$, where h is the width resisting the shear force, taken as the clear distance between flanges less the inside corner radii; t is the wall thickness, and $k_v = 5$. If the corner radius of a hollow structural section is not known, h may be taken as the corresponding outside dimension minus three times the wall thickness.

For **round HSS** calculate V_n as follows:

$$V_n = \frac{F_{cr} A_g}{2} \tag{5.99}$$

where A_g is the gross cross-section area. The critical buckling stress F_{cr} is the larger of the following two equations, but also must not exceed $0.6F_y$:

$$F_{cr} = \frac{1.60E}{\sqrt{L_v/D}(D/t)^{3/4}} \tag{5.100}$$

$$F_{cr} = \frac{0.78E}{(D/t)^{3/2}} \tag{5.101}$$

where A_g = gross area of section based on design wall thickness, in^2 (mm^3)
D = outside diameter, in (mm)
L_v = distance from maximum to zero shear force, in (mm)
t = design wall thickness, in (mm)

For **singly and doubly symmetric shapes** loaded in the weak axis without torsion, calculate V_n for each shear-resisting element using Eq. (5.88), and the limits in Eqs. (5.89) to (5.91), with $A_w = b_f t_f$ (flange width × flange thickness) and $k_v = 1.2$. Note that for ASTM A6 W, S, M, and HP shapes, when $F_y \leq 50$ ksi (345 MPa), $C_v = 1.0$.

5.7 DESIGN FOR COMBINED FORCES AND TORSION

Where a member is subjected to axial force and flexure concurrently, and possibly torsion as well, interaction equations are used to consider the combined effects. Such equations involve calculation of ratios of required to available strength. This article addresses members subjected to axial force and flexure about one or both axes, with or without torsion, and members subject to torsion only. The AISC Specification gives these equations, but also provides alternative methods for some cases.

5.7.1 Doubly and Singly Symmetric Members under Axial Force and Flexure

Doubly and singly symmetric members subjected to axial force and flexure may be treated as follows. (See Arts. 5.7.2 and 5.7.3 for alternatives.) The interaction of **compression and flexure** in members for which $0.1 \leq (I_{yc}/I_y) \leq 0.9$, constrained to bend about a geometric axis (x and/or y), is limited by the following equations, where I_{yc} is the moment of inertia about the y axis referred to the compression flange, in^4 (mm^4).

For $P_r/P_c \geq 0.2$,

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{5.102}$$

For $P_r/P_c < 0.2$,

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{5.103}$$

The following definitions apply when the axial force causes compression:
For **LRFD design,** using LRFD load combinations,

$P_r = P_u$ = required compression strength, kips (N)
$P_c = \phi_c P_n$ = design compression strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = \phi_b M_n$ = design flexural strength, kip·in (N·mm)
ϕ_c = resistance factor for compression = 0.90
ϕ_b = resistance factor for flexure = 0.90

For **ASD design,** using ASD load combinations,

$P_r = P_a$ = required compression strength, kips (N)
$P_c = P_n/\Omega_c$ = allowable compression strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = M_n/\Omega_b$ = allowable flexural strength, kip·in (N·mm).
Ω_c = safety factor for compression = 1.67
Ω_b = safety factor for flexure = 1.67

Doubly and singly symmetric members subjected to **tension and flexure** are also subject to Eqs. (5.102) and (5.103), but the definitions of terms differ. Also, when determining M_n for doubly symmetric members, the C_b term (see Art. 5.5) may be increased by $(1 + P_u/P_{ey})^{1/2}$ for LRFD and $(1 + 1.5\, P_a/P_{ey})^{1/2}$ for ASD, where the tension acts concurrently with flexure. The elastic critical buckling load is given by $P_{ey} = \pi^2 EI_y/L_b^2$.

The following definitions apply when the axial force causes tension:
For **LRFD design,** using LRFD load combinations,

$P_r = P_u$ = required tensile strength, kips (N)
$P_c = \phi_t P_n$ = design tensile strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = \phi_b M_n$ = design flexural strength, kip·in (N·mm)
ϕ_t = resistance factor for tension
= 0.90 for gross section yielding = 0.75 for net section rupture
ϕ_b = resistance factor for flexure = 0.90

For **ASD design**, using ASD load combinations,

$P_r = P_a$ = required tensile strength, kips (N)
$P_c = P_n/\Omega_t$ = allowable tensile strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = M_n/\Omega_b$ = allowable flexural strength, kip·in (N·mm)
Ω_t = safety factor for tension
= 1.67 for gross section yielding = 2.00 for net section rupture
Ω_b = safety factor for flexure = 1.67

5.7.2 Doubly Symmetric Members under Axial Compression and Single-Axis Flexure

For doubly symmetric members in flexure and compression with moments primarily in one plane, the AISC Specification permits one to consider two separate independent limit states, in-plane instability and out-of-plane buckling or flexural-torsional buckling, as an alternative to the combined approach provided in Art. 5.7.1. For the limit state of in-plane instability, use Eqs. (5.105) and (5.106) with P_c, M_r, and M_c determined in the plane of bending. For the limit state of out-of-plane buckling, use the following:

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 \leq 1.0 \tag{5.104}$$

where P_{co} = available compressive strength out of the plane of bending, kips (N)
M_{cx} = available flexural-torsional strength for strong axis flexure, kip·in (N·mm)

If bending occurs only about the weak axis, the moment ratio in Eq. (5.104) may be neglected. For members with significant biaxial moments, $M_r/M_c \geq 0.05$ in both directions, use Eqs. (5.102) and (5.103).

5.7.3 Unsymmetric and Other Members under Axial Force and Flexure

The method presented here is for unsymmetric shapes, but may be used as an alternative for shapes discussed in Art. 5.7.1. Equation (5.105), absolute value, must be satisfied using the principal bending axes, either by the addition of all the maximum axial and flexural terms, or by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added or subtracted from the axial term as appropriate. Second-order effects must be included, as discussed in Art. 5.2.

$$\left|\frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}}\right| \leq 1.0 \tag{5.105}$$

The subscripts *w* and *z* indicate the major and minor axes of bending, respectively.

The following definitions apply for **LRFD design,** using LRFD load combinations:

f_a = required axial stress, ksi (MPa)
$F_a = \phi_c F_{cr}$ or $\phi_t F_{cr}$ = design axial stress (compression or tension as appropriate), ksi (MPa)
f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section, ksi (MPa)
$F_{bw}, F_{bz} = \phi_b M_n/S$ = design flexural stress, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.
ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for tension = 0.90 for yielding or 0.75 for rupture

ϕ_b = resistance factor for flexure = 0.90

The following definitions apply for **ASD design,** using ASD load combinations:

f_a = required compression, ksi (MPa)

$F_a = \frac{F_{cr}}{\Omega_c} = \frac{F_{cr}}{\Omega_t}$ = allowable axial stress (compression or tension as appropriate), ksi (MPa)

f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section, ksi (MPa)

$F_{bw}, F_{bz} = \frac{M_n}{\Omega_b S}$ = allowable flexural stress, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.

Ω_c = safety factor for compression = 1.67

Ω_t = safety factor for tension = 1.67 for yielding or 2.00 for rupture

Ω_b = safety factor for flexure = 1.67

5.7.4 Round and Rectangular HSS under Torsion

Closed sections are particularly advantageous where a member is required to resist torsion. The design torsional strength $\phi_T T_n$ and the allowable torsional strength T_n/Ω_T for round and rectangular HSS is determined as follows using $\phi_T = 0.90$ (LRFD) and $\Omega_T = 1.67$ (ASD). The nominal torsional strength T_n for the limit states of torsional yielding and torsional buckling is

$$T_n = F_{cr}C \tag{5.106}$$

where C is the HSS torsional constant and F_{cr} is the critical torsional shear stress.

For a round HSS,

$$C = \frac{\pi(D-t)^2 t}{2} \tag{5.107}$$

For rectangular HSS,

$$C = 2(B - t)(H - t)t - 4.5(4 - \pi)t^3 \tag{5.108}$$

For round HSS, F_{cr} is the larger of the following:

$$F_{cr} = \frac{1.23E}{\sqrt{L/D}(D/t)^{5/4}} \leq 0.6F_y \tag{5.109}$$

and

$$F_{cr} = \frac{0.60E}{(D/t)^{3/2}} \leq 0.6F_y \tag{5.110}$$

For rectangular HSS, F_{cr} depends on the h/t ratio, as follows.

For $h/t \leq 2.45\sqrt{E/F_y}$,

$$F_{cr} = 0.6F_y \tag{5.111}$$

For $2.45\sqrt{E/F_y} < h/t \leq 3.07\sqrt{E/F_y}$,

$$F_{cr} = 0.6F_y\left[\frac{2.45\sqrt{E/F_y}}{(h/t)}\right] \tag{5.112}$$

For $3.07\sqrt{E/F_y} < h/t \leq 260$,

$$F_{cr} = \frac{0.458\pi^2 E}{(h/t)^2} \tag{5.113}$$

5.7.5 Round and Rectangular HSS under Torsion and Other Forces

If the required torsional strength T_r does not exceed 20% of the available torsional resistance T_c, check interaction using Art. 5.7.1, neglecting torsion. Otherwise, the interaction of forces is limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \tag{5.114}$$

The following definitions apply for **LRFD design,** using LRFD load combinations:

P_r = required axial strength, kips (N)
$P_c = \phi P_n$ = design tensile or compressive strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = \phi_b M_n$ = design flexural strength, kip·in (N·mm)
V_r = required shear strength, kips (N)
$V_c = \phi_v V_n$ = design shear strength, kips (N)
T_r = required torsional strength, kip·in (N·mm)
$T_c = \phi_T T_n$ = design torsional strength, kip·in (N·mm)

The following definitions apply for **ASD design,** using ASD load combinations:

P_r = required axial strength, kips (N)
$P_c = P_n/\Omega$ = allowable tensile or compressive strength, kips (N)
M_r = required flexural strength, kip·in (N·mm)
$M_c = M_n/\Omega_b$ = allowable flexural strength, kip·in (N·mm)
V_r = required shear strength, kips (N)
$V_c = V_n/\Omega_v$ = allowable shear strength, kips (N)
T_r = required torsional strength, kip·in (N·mm)
$T_c = T_n/\Omega_T$ = allowable torsional strength, kip·in (N·mm)

5.7.6 Members Other Than HSS under Torsion and Other Forces

Members other than HSS under torsion generally develop both normal stress and shear stress. (See "Torsional Analysis of Structural Steel Members," Design Guide No. 9, AISC, 1997). The design torsional strength ϕF_n and the allowable torsional strength F_n/Ω for non-HSS members is taken as the lowest value obtained for the limit states of yielding (under normal stress), shear yielding, and buckling. Use Eqs. (5.115) to (5.117) to determine F_n with $\phi_T = 0.90$ (LRFD) and $\Omega_T = 1.67$ (ASD).

For the limit state of yielding,

$$F_n = F_y \tag{5.115}$$

For the limit state of shear yielding,

$$F_n = 0.6F_y \tag{5.116}$$

For the limit state of buckling,

$$F_n = F_{cr} \tag{5.117}$$

where F_{cr} is the buckling stress for the section as determined by analysis. The AISC Specification permits constrained local yielding adjacent to areas that remain elastic.

5.8 DESIGN OF COMPOSITE MEMBERS

In composite construction, a steel section works together with concrete to resist column forces, flexure, or both. Composite columns involve either an encased steel section or a hollow structural section filled with concrete. Composite flexural members are typically comprised of a steel beam and an overlying concrete slab, with shear connectors (studs) at the interface. In this case, the slab acts as a cover plate and allows the use of a lighter steel section. Additionally, concrete-encased and concrete-filled sections are sometimes used as flexural members. In addition to the provisions summarized below, the AISC Specification gives design requirements for combined axial compression and flexure in composite members.

5.8.1 General Provisions for Composite Members

The AISC Specification requires that, when determining load effects (forces, stresses, and deformations) in members and connections of a structure that includes composite members, consideration be given to the effective sections at the time each increment of load is applied. Design, detailing, and material properties pertaining to the concrete and reinforcing steel should comply with specifications of the American Concrete Institute (ACI 318). Available strength of members can be determined by either the plastic stress distribution method or the strain-compatibility method. The tensile strength of the concrete is neglected.

For the plastic stress distribution method, the available strength is computed assuming that the steel components have reached a stress of F_y in either tension or compression, and the concrete components in compression have reached a stress of $0.85f_c'$. This is the method that would be typically used for regular sections. Equivalent stress blocks are assumed for the stress distributions and moments summed about the neutral axis to calculate the resisting moment. For round HSS filled with concrete, a stress of $0.95f_c'$ may be used for concrete components in compression, to account for the beneficial effects of concrete containment.

For the strain-compatibility method, a linear strain distribution across the section is assumed, and a maximum concrete compressive strain of 0.003. The stress–strain relationship, obtained from tests or published results, is used to determine the stresses over the cross section, and moments summed about the neutral axis to calculate the resisting moment. This method should be used for irregular sections and cases where the steel does not exhibit a typical elastoplastic response.

Certain material limitations apply. For the determination of available strength, concrete must have a compressive strength f_c' of not less than 3 ksi (21 MPa) nor more than 10 ksi (70 MPa) for normal-weight concrete and not less than 3 ksi (28 MPa) nor more than 6 ksi (42 MPa) for lightweight concrete. Higher-strength concrete materials may be used for stiffness calculations but not for strength calculations. The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column must not exceed 75 ksi (525 MPa), unless higher material strengths are justified by testing and analysis.

5.8.2 Encased Composite Columns

To qualify for design as an encased composite column, three conditions must be met: (1) the cross-sectional area of the steel core must comprise at least 1% of the total composite cross section, (2) the concrete encasement of the steel core must be reinforced with continuous longitudinal bars and lateral ties or spirals, with a minimum transverse reinforcement of at least 0.009 in^2/in (0.225 mm^2/mm) of tie spacing, and (3) the reinforcement ratio (ratio of area of continuous longitudinal reinforcing to gross area of composite member) must be at least 0.004. Additional requirements for shear connectors and detailing are discussed subsequently.

To determine the available compressive strength (design compressive strength $\phi_c P_n$ and allowable compressive strength P_n/Ω_c) of an axial-loaded encased column for the limit state of flexural buckling, use $\phi_c = 0.75$ (LRFD) and $\Omega_c = 2.00$ (ASD). First calculate the following:

$$P_o = A_s F_y + A_{sr} F_{\text{yr}} + 0.85 A_c f_c' \tag{5.118}$$

$$P_e = \frac{\pi^2 EI_{\text{eff}}}{(KL)^2} \tag{5.119}$$

$$EI_{\text{eff}} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \tag{5.120}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.3 \tag{5.121}$$

The following definitions apply:

A_s = area of steel section, in^2 (mm^2)
A_c = area of concrete, in^2 (mm^2)
A_{sr} = area of continuous reinforcing bars, in^2 (mm^2)
E_c = modulus of elasticity of concrete
= $w_c^{1.5}\sqrt{f_c'}$ ksi ($5000\sqrt{f_c'}$ MPa)
E_s = modulus of elasticity of steel = 29,000 ksi (210 MPa)
EI_{eff} = effective rigidity of composite section, kip·in^2 (N·mm^2)
f_c' = specified minimum concrete compressive strength, ksi (MPa)
F_y = specified minimum yield stress of steel section, ksi (MPa)
F_{yr} = specified minimum yield strength of reinforcing bars, ksi (MPa)
I_c = moment of inertia of concrete section, in^4 (mm^4)
I_s = moment of inertia of steel shape, in^4 (mm^4)
I_{sr} = moment of inertia of reinforcing bars, in^4 (mm^4)
K = effective length factor (see Art. 5.2)
L = laterally unbraced length of the member, in (mm)
P_e = elastic axial buckling strength, kips (kN)
P_o = nominal axial compressive strength without length effects, kips (kN)
w_c = weight of concrete per unit volume
= $90 \le w_c \le 150$ lb/ft^3($1440 \le w_c \le 2450$ kg/m^3)

The nominal compressive strength is then calculated as

When $P_e \ge 0.44\,P_o$,
$$P_n = P_o\left[0.658^{(P_o/P_e)}\right] \tag{5.122}$$

When $P_e < 0.44\,P_o$,
$$P_n = 0.877\,P_e \tag{5.123}$$

To ensure that axial loads applied to encased composite columns are properly shared between the steel and concrete, shear connectors are required [see Eq. (5.128) for nominal strength]. The following requirements must be met:

When the external force is applied directly to the steel section, shear connectors must be provided to transfer the required shear force V', given by

$$V' = V\left(1 - \frac{A_s F_y}{P_o}\right) \tag{5.124}$$

When the external force is applied directly to the concrete encasement, shear connectors must be provided to transfer the required shear force V', given by

$$V' = V\left(\frac{A_s F_y}{P_o}\right) \tag{5.125}$$

where V = shear force introduced to column, kips (N)
A_s = area of steel section, in^2 (mm^2)
F_y = specified minimum yield stress of steel section, ksi (MPa)
P_o = nominal axial compressive strength without length effects, kips (N)

Also, when load is applied to the encased composite concrete by direct bearing on the concrete, the design bearing strength $\phi_B P_p$ and the allowable tensile strength P_p/Ω_B are determined from

$$P_p = 1.7 f_c' A_B \tag{5.126}$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \phi_B = 2.31 \text{ (ASD)} \tag{5.127}$$

where A_B is the loaded area, in^2 (mm^2).

Additionally, various detailing requirements for encased composite columns must be met. At least four longitudinal bars must be used. Transverse reinforcement must be spaced at the lesser of 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least dimension of the composite section. The encasement must provide at least 1.5 in (38 mm) of clear cover to the steel.

Shear connectors, meeting the requirements of Eqs. (5.124) and (5.125), must be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing is 16 in (405 mm), and connectors must be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes must be interconnected with lacing, tie plates, batten plates, or similar components to prevent buckling of individual shapes as a result of loads applied prior to hardening of the concrete.

For encased composite columns, the nominal shear strength of one stud shear connector is

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \le A_{sc} F_u \tag{5.128}$$

where A_{sc} = cross-sectional area of stud, in^2 (mm^2)
E_c = modulus of elasticity of concrete, ksi (MPa)
F_u = specified minimum tensile strength of stud shear connector, ksi (MPa)

The shear strength of an encased column should be based on the shear strength of the steel section alone, plus the shear strength of any tie reinforcement, or, alternatively, on the shear strength of the reinforced concrete (see ACI 318).

5.8.3 Filled Composite Columns

To qualify for design as a filled composite column, three conditions must be met: (1) the cross-sectional area of the steel HSS shall comprise at least 1% of the total composite cross section; (2) for rectangular HSS filled with concrete, the width–thickness ratio b/t must not exceed $2.26\sqrt{E/F_y}$; and (3) for round HSS, the diameter–thickness ratio D/t must not exceed $0.15E/F_y$. However, higher b/t and D/t ratios may be used when justified by testing and analysis.

The design compressive strength $\phi_c P_n$ and allowable compressive strength P_n/Ω_c for axially loaded filled composite columns are determined for the limit state of flexural buckling using Eqs. (5.118) to (5.123) with $\phi_c = 0.75$ (LRFD) and $\Omega_c = 2.00$ (ASD), but with the following modified definitions:

$$P_o = A_s F_y + A_{sr} F_{\mathrm{yr}} + C_2 A_c f_c' \tag{5.129}$$

$$EI_{\mathrm{eff}} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \tag{5.130}$$

where $C_2 = 0.85$ for rectangular HSS, $C_2 = 0.95$ for circular sections, and

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \tag{5.131}$$

For concrete-filled composite members subjected to tensile forces, the available tensile strength is calculated as follows, ignoring any tensile strength of the concrete. The design tensile strength $\phi_t P_n$ and the allowable tensile strength P_n/Ω_t for the limit state of yielding are determined using $\phi_t = 0.90$ (LRFD) and $\Omega_t = 1.67$ (ASD), where the nominal tensile strength P_n, kips (kN), is

$$P_n = P_t = A_s F_y + A_{sr} F_{\mathrm{yr}} \tag{5.132}$$

The available shear strength for filled composite columns is based on either the shear strength of the steel section alone or the shear strength of the concrete (see ACI 318).

Loads applied to filled composite columns must be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core may be developed from direct bond interaction, shear connection, or direct bearing. Use the force-transfer mechanism providing the largest nominal strength; do not superpose mechanisms. When the load is applied to the concrete core by direct bearing, the design bearing strength $\phi_c P_n$ and the allowable tensile strength P_n/Ω_c for the limit state of concrete crushing are determined using $\phi_c = 0.65$ (LRFD) and $\Omega_c = 2.31$ (ASD). Calculate the nominal bearing strength as $P_n = 1.7 f_c' A_B$, where A_B is the loaded area, in^2 (mm^2). When shear connectors are used to transfer the required shear force, distribute them along the length of the member at least a distance of 2.5 times the width of structural steel HSS or 2.5 times the diameter of round HSS, both above and below the load transfer region. The maximum connector spacing should not exceed 16 in (405 mm).

5.8.4 Composite Beams with Shear Connectors

The most common application of composite construction is a flexural member with shear connectors. See Arts. 5.8.5 and 5.8.6 for shear connector requirements. For such applications, the design may be based on an effective concrete-steel T-beam, where the width of the concrete slab on either side of the beam centerline is limited to the following:

1. One-eighth of the beam span, center-to-center of supports.
2. One-half the distance to the centerline of the adjacent beam.
3. The distance to the edge of the slab.

Temporary supports (shores) during construction are optional. However, when temporary shores are not used during construction, the steel section alone must have adequate strength (see Art. 5.5) to support all loads applied prior to the concrete attaining 75% of its specified strength f_c'.

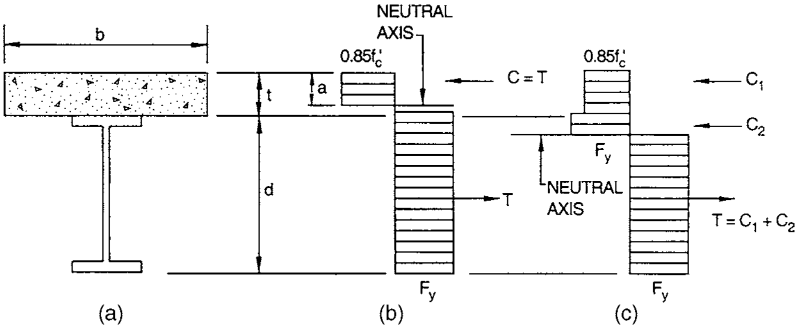


FIGURE 5.2 Plastic distribution of stresses in a composite beam in the positive-moment region. (*a*) Structural steel beam connected to concrete slab for composite action. (*b*) Stress distribution when neutral axis is in the slab. (*c*) Stress distribution when neutral axis is in the web.

The design shear strength and allowable shear strength of composite beams should be based on the steel section alone.

The design positive flexural strength $\phi_b M_n$ and allowable positive flexural strength M_n/Ω_b are determined as follows, using $\phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD). The determination of the nominal flexural strength M_n depends on h/t_w, the web depth–thickness ratio of the steel beam:

When $h/t_W \leq 3.76\sqrt{E/F_{yf}}$, determine M_n for the limit state of yielding (plastic moment) from the plastic stress distribution on the composite section (Fig. 5.2).

When $h/t_W > 3.76\sqrt{E/F_{yf}}$, determine M_n for the limit state of yielding (yield moment) from the superposition of elastic stresses, considering the effects of shoring.

Here E_s = modulus of elasticity of steel = 29,000 ksi (210 MPa), and F_{yf} = specified minimum yield stress of the flange of the steel section, ksi (MPa).

The design negative flexural strength $\phi_b M_n$ and allowable negative flexural strength M_n/Ω_b may be determined based on the steel section alone (see Art. 5.5). Alternatively, they may be determined from the plastic stress distribution on the composite section for the limit state of yielding (plastic moment), using $\phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD), provided that (1) the steel beam is an adequately braced compact section (see Art. 5.1.5), (2) shear connectors connect the slab to the steel beam in the negative moment region, and (3) slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

5.8.5 Composite Beams with Formed Steel Deck

For composite construction consisting of concrete slabs on formed steel deck connected to steel beams, the available flexural strength is determined by the applicable portions of Art. 5.8.4, subject to the conditions:

1. The nominal rib height of the steel deck is limited to 3 in (75 mm) maximum. The average width of concrete rib or haunch w_r must be at least 2 in (50 mm); in calculations, it may be taken as no more than the minimum clear width near the top of the steel deck.

2. The concrete slab must be connected to the steel beam with welded stud shear connectors $^3/_4$ in (19 mm) or less in diameter (see AWS D1.1). Studs may be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, must extend not less than $1^1/_2$ in (38 mm) above the top of the steel deck. There must be at least $^1/_2$ in (13 mm) of concrete cover above the top of the installed studs.

3. The slab thickness above the steel deck must be at least 2 in (50 mm).

4. The deck must be anchored to all supporting members, with anchorage at a maximum spacing of 18 in (460 mm). Stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other suitable approved devices may be used for anchorage.

5. For deck ribs oriented perpendicular to the steel beam, the concrete below the top of the steel deck must be neglected in determining section properties and in calculating the concrete area A_c.

6. For deck ribs oriented parallel to the steel beam, the concrete below the top of the steel deck may be included in determining composite section properties and should be included in calculating A_c. Also, deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch. Further, when the nominal depth of the steel deck is $1\frac{1}{2}$ in (38 mm) or greater, the average width w_r of the supported haunch or rib must be not less than 2 in (50 mm) for the first stud in the transverse row plus four stud diameters for each additional stud.

5.8.6 Shear Connectors for Composite Beams

The horizontal shear at the interface between the steel beam and the concrete slab is assumed to be transferred by shear connectors. For composite action in positive-moment regions (concrete in flexural compression), the total horizontal shear force V' between the point of maximum positive moment and the point of zero moment is the least of the following three limit states:

Concrete crushing $$V' = 0.85 f_c' A_c \tag{5.133}$$

Tensile yielding of steel section $$V' = A_s F_y \tag{5.134}$$

Strength of shear connectors $$V' = \sum Q_n \tag{5.135}$$

where A_c = area of concrete slab within effective width, in^2 (mm^2)
A_s = area of steel cross section, in^2 (mm^2)
ΣQ_n = sum of nominal strengths of shear connectors between point of maximum positive moment and point of zero moment, kips (N)

For composite action in negative moment regions of continuous composite beams where longitudinal reinforcing steel is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and the point of zero moment is the smaller of the following two limit states:

Tensile yielding of slab reinforcement $$V' = A_r F_{\mathrm{yr}} \tag{5.136}$$

Strength of shear connectors $$V' = \sum Q_n \tag{5.137}$$

where A_r = area of adequately developed longitudinal reinforcing steel within effective width of concrete slab, in^2 (mm^2)
F_{yr} = minimum specified yield stress of the reinforcing steel, ksi (MPa)

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment is equal to the horizontal shear force V' divided by the nominal strength of one shear connector.

The nominal strength, Q_n, of one stud shear connector embedded in solid concrete or in a composite slab is

$$Q_n = 0.5 A_{\mathrm{sc}} \sqrt{f_c' E_c} \le R_g R_p A_{\mathrm{sc}} F_u \tag{5.138}$$

where A_{sc} = cross-sectional area of stud shear connector, in^2 (mm^2)
E_c = modulus of elasticity of concrete, ksi (MPa)
F_u = specified minimum tensile strength of a stud shear connector, ksi (MPa)
R_g = 1.0 for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; for any number of studs welded in a row directly to the steel shape; for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth ≥ 1.5
R_g = 0.85 for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth < 1.5
R_g = 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape
R_p = 1.0 for studs welded directly to the steel shape (i.e., not through steel deck or sheet) and having a haunch detail with not more than 50% of the top flange covered by deck or sheet steel closures
R_p = 0.75 for studs welded in a composite slab with the deck oriented perpendicular to the beam and $e_{\text{mid-ht}} \geq 2$ in (51 mm); for studs welded through steel deck, or steel sheet used as girder filler material, and embedded in a composite slab with the deck oriented parallel to the beam
R_p = 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{\text{mid-ht}} < 2$ in (51 mm)
$e_{\text{mid-ht}}$ = distance from edge of stud shank to steel deck web, measured at mid-height of deck rib, and in the load-bearing direction of the stud (i.e., direction of maximum moment for simply supported beam), in (mm)

Channels welded to the steel beam may also be used as shear connectors. The welds must be designed to develop the force Q_n and the effects of eccentricity must be considered. The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f_c'E_c} \tag{5.139}$$

where t_f = flange thickness of channel shear connector, in (mm)
t_w = web thickness of channel shear connector, in (mm)
L_c = length of channel shear connector, in (mm)

5.8.7 Concrete-Encased and Concrete-Filled Flexural Members

The design flexural strength $\phi_b M_n$ and allowable flexural strength M_n/Ω_b of concrete-encased and concrete-filled sections may be determined by one of the following:

1. From the superposition of elastic stresses on the composite section, considering the effects of shoring, for the limit state of yielding (yield moment), using $\phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD).
2. From the plastic stress distribution on the steel section alone, for the limit state of yielding (plastic moment), using $\phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD).
3. From the plastic stress distribution on the composite section or from the strain-compatibility method, if shear connectors are provided and the conditions of Art. 5.8.2 or 5.8.3 are met, using $\phi_b = 0.85$ (LRFD) and $\Omega_b = 1.76$ (ASD).

As with composite slabs, when temporary shores are not used during construction, the steel section alone must have adequate strength (see Art. 5.5) to support all loads applied prior to the concrete attaining 75% of its specified strength f_c'.

The available shear strength should be based on the shear strength of the steel section alone, plus the shear strength of any tie reinforcement, or, alternatively, on the shear strength of the reinforced concrete (see ACI 318).

5.9 DESIGN OF CONNECTIONS

The required strength of connections should be determined by structural analysis for the specified design loads, consistent with the type of construction specified. The design strength ϕR_n and the allowable strength R_n/Ω for the connections can be determined from the provisions of the AISC Specification. Design for static loads is summarized in this article. For design of connections subjected to fatigue, see App. 3 of the AISC Specification. For design of HSS connections, see Chap. K of the AISC Specification. Also, see Chap. 3 for additional information and examples.

5.9.1 General Provisions for Connections

As indicated in Art. 5.1.4, connections are classified for design purposes as either **simple connections** or **moment connections.** The details for simple connections must be designed to be flexible and accommodate end rotations of simple beams. The AISC Specification allows them to be proportioned for the reaction shears only, and self-limiting inelastic deformation in the connection elements is permitted. For moment connections, the design must consider the combined effect of forces resulting from both moment and shear.

Compression members such as columns or chords of trusses are often designed with joints that transmit the compression in bearing. AISC gives the following rules:

> (a) When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.
>
> (b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii), below. It is permissible to use the less severe of the two conditions:
>
> (i) An axial tensile force of 50 percent of the required compression strength of the member; or
>
> (ii) The moment and shear resulting from a transverse load equal to 2 percent of the required compression strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

Splices in heavy sections, defined as rolled sections with a flange thickness greater than 2 in (50 mm) or built-up sections with plates thicker than 2 in (50 mm), must meet additional requirements. When tensile forces due to applied tension or flexure are transmitted through splices in heavy sections, by complete-joint-penetration (CJP) groove welds, the material must provide a Charpy V-notch impact toughness of 20 ft·lb (27 J) at + 70°F (+21°C). Also, beam copes and weld access holes must be shaped to specified criteria and thermal cut surfaces must be ground to bright metal and inspected by magnetic particle or dye-penetrant methods. These requirements are not applicable to splices of plates of built-up shapes that are welded prior to assembling the shape. Also, as an alternative to the requirements, consider splices using partial-joint-penetration (PJP) groove welds for the flanges and fillet-welded web splice plates, or using bolts for some or all of the splice.

Eccentricity, which affects the distribution of resisting forces, must be considered in the design of connections for groups of welds or bolts at the ends of members that transmit axial force, unless the center of gravity of the group coincides with the center of gravity of the member. However, such eccentricity need not be considered for end connections of members such as statically loaded single or double angles.

Bolts used in combination with welds cannot be designed to share the load with the welds except for the following. Shear connections with bolts installed in standard holes or in short slots transverse to the direction of the load may share the load with longitudinally loaded fillet welds, but the available strength of the bolts must not be taken as greater than 50% of the available strength of bearing-type bolts (see Art. 5.9.8). This exception applies to both high-strength bolts and A307 bolts. Also, in alterations or rehab work, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections may be used to carry existing loads, and the welding used to provide the additional required strength.

Bolts used in combination with rivets, such as in alterations or rehab work, cannot be designed to share the load with the rivets except as follows. When high-strength bolts are designed as slip-critical connections, the bolts may be designed to share the load with the existing rivets.

Limitations on bolted and welded connections are as follows. The AISC Specification requires that only pretensioned joints, slip-critical joints, or welds may be used for the following connections:

(1) Column splices in all multi-story structures over 125 ft (38 m) in height.

(2) Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent in structures over 125 ft (38 m) in height.

(3) In all structures carrying cranes of over five-ton (50 kN) capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

(4) Connections for the support of machinery and other live loads that produce impact or reversal of load.

In all other cases, snug-tightened high-strength bolts and A307 bolts are acceptable.

5.9.2 Design Considerations for Welded Connections

Welds used in building construction are generally either **groove welds,** in which the weld is made in a groove between connection elements, or **fillet welds,** in which a weld of generally triangular cross section is made between intersecting surfaces of connection elements. **Plug welds** and **slot welds** are also used for some purposes. In this case, the weld is made in a circular or an elongated hole, partly or completely filling the hole, depending on depth.

For **groove welds,** the effective area for computation of design strength is the length of the weld times the effective throat thickness. The effective throat thickness of a **complete-joint-penetration groove weld** is the thickness of the thinner part joined. The effective throat thickness of a **partial-joint-penetration groove weld** is given in Table 5.3. However, the joint must be designed so the effective throat thickness is no less than the minimum values given in Table 5.4. Generally, the effective throat size of a partial-joint-penetration weld is dependent on the process used and the weld position. The designer should indicate either the effective throat required or the weld strength required, so that the fabricator can detail the joint based on the weld process and position that will be used.

Flare groove welds are welds made in a groove formed by a member with a curved surface in contact with a planar member (flare bevel groove weld) or in a groove formed by two members

TABLE 5.3 Effective Throat of Partial-Joint-Penetration Groove Welds

Welding process	Welding position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove type	Effective throat
Shielded-metal arc (SMAW)	All	J or U groove, 60° V	Depth of groove
Gas-metal arc (GMAW) Flux-cored arc (FCAW)	All	J or U groove, 60° V	Depth of groove
Submerged arc (SAW)	F	J or U groove, 60° bevel, or V	Depth of groove
Gas-metal arc (GMAW) Flux-cored arc (FCAW)	F, H	45° bevel	Depth of groove
Shielded-metal arc (SMAW)	All	45° bevel	Depth of groove minus $^1/_8$ in (3 mm)
Gas-metal arc (GMAW) Flux-cored arc (FCAW)	V, OH	45° bevel	Depth of groove minus $^1/_8$ in (3 mm)

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill,. 2005, with permission.

TABLE 5.4 Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material thickness of thinner part joined, in (mm)	Minimum effective throat thickness, in (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19) to 1 1/2 (38)	5/16 (8)
Over 1 1/2 (38) to 2 1/4 (57)	3/8 (10)
Over 2 1/4 (57) to 6 (150)	1/2 (13)
Over 6 (150)	5/8 (16)

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

with a curved surface (flare V-groove weld). The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a formed section, or a rectangular tube is given in Table 5.5, unless other effective throats are established through testing. The effective size of flare groove welds that are not filled flush should be reduced from that given in Table 5.5 by the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

For **fillet welds,** the effective area is the effective length multiplied by the effective throat. The effective throat is the shortest distance from the root to the nominal face of the weld. The AISC Specification states that "An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables." For fillet welds in holes and slots, the effective length is the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area must not exceed the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

The minimum-size fillet welds, as designated by the nominal leg dimension, must be not less than the size required to transmit calculated forces nor the size as shown in Table 5.6. The maximum size of fillet welds of connected parts is given by the AISC Specification as follows:

> (a) Along edges of material less than 1/4-in (6 mm) thick, not greater than the thickness of the material.
>
> (b) Along edges of material 1/4-in (6 mm) or more in thickness, not greater than the thickness of the material minus 1/16-in (2 mm), unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1/16-in (2 mm) provided the weld size is clearly verifiable.

TABLE 5.5 Effective Weld Sizes of Flare Groove Welds*

Welding process	Flare bevel groove†	Flare V groove
GMAW and FCAW–G	5/8 R	3/4 R
SMAW and FCAW–S	5/16 R	5/8 R
SAW	5/16 R	1/2 R

*R = radius of joint surface, which can be taken as $2t$ for HSS.

†For flare bevel groove with $R < 0.375$ in (10 mm), use only reinforcing fillet weld on filled flush joint.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

TABLE 5.6 Minimum Size of Fillet Welds

Material thickness of thinner part joined, in (mm)	Minimum size of fillet weld,* in (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

*Leg dimension of fillet welds. Single-pass welds must be used.
Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

The minimum effective length of fillet welds designed on the basis of strength is four times the nominal size. Otherwise, the weld size must be taken as no more than one-quarter of the effective length of the weld. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld must be not less than the perpendicular distance between them.

For end-loaded fillet welds with a length up to 100 times the leg dimension, the effective length may be taken as the actual length. When the length of an end-loaded fillet weld exceeds 100 times the weld size, the effective length is determined by multiplying the actual length by the reduction factor β:

$$\beta = 1.2 - 0.002\left(\frac{L}{w}\right) \leq 1.0 \qquad (5.140)$$

where L = actual length of end-loaded weld, in (mm), and w = weld leg size, in (mm). When the length of the weld exceeds 300 times the leg size, the value of β is taken as 0.60.

Intermittent fillet welds may be used to transfer calculated stress when the required strength is less than that for a continuous fillet weld of the smallest permitted size, and also to join components of built-up members. The effective length of any segment of intermittent fillet welding should be not less than four times the weld size, with a minimum of 1½ in (38 mm).

In lap joints, the lap must be at least five times the thickness of the thinner part joined, but not less than 1 in (25 mm). Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only must be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is restrained to prevent opening of the joint under maximum loading.

Fillet welds may be stopped short or extend to the ends or sides of parts, or may be arranged in a box configuration, subject to the following limits for fillet weld terminations given by the AISC Specification:

> For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.
>
> For connections where flexibility of the outstanding elements is required, when end returns are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.
>
> Fillet welds joining transverse stiffeners to plate girder webs shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.
>
> Fillet welds that occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

Where not otherwise limited, it is considered good practice to terminate fillet welds one weld size from the edge of the connection element, to minimize notches in the base metal.

Fillet welds may be used along the periphery of holes or slots to transmit shear in lap joints, to prevent buckling or separation of lapped parts, and to join components of built-up members. Such fillet welds are not considered plug or slot welds. Plug welds and slot welds may also be used for these purposes, but fillet welds are usually preferred. The effective shearing area of plug and slot welds is the nominal cross-sectional area of the hole or slot in the plane of the faying surface. See the AISC Specification for limitations on size and spacing.

5.9.3 Design of Welds for Strength

The design strength ϕR_n and the allowable strength R_n/Ω of welds is the lower value of the base material strength and the weld metal strength. These are determined as follows for the limit states of tensile rupture, shear rupture, or yielding:

For the **base material,**

$$R_n = F_{\mathrm{BM}} A_{\mathrm{BM}} \tag{5.141}$$

For the **weld metal,**

$$R_n = F_w A_w \tag{5.142}$$

where F_{BM} = nominal strength of the base material per unit area, ksi (MPa)
F_w = nominal strength of the weld metal per unit area, ksi (MPa)
A_{BM} = cross-sectional area of the base material, in^2 (mm^2)
A_w = effective area of the weld, in^2 (mm^2)

The values of ϕ, Ω, F_{BM}, and F_w and limitations thereon are given in Table 5.7. The AISC Specification also gives alternative rules that may result in greater strengths.

If two or more types of welds (groove, fillet, plug, or slot) are combined in a single joint, the strength of each should be calculated separately with reference to the axis of the group to determine the strength of the combination.

5.9.4 Filler Metal for Welds

The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area should comply with the requirements for matching filler metals given in specifications of the American Welding Society, AWS D1.1. Also, the AISC Specification requires filler metal with a specified Charpy V-notch (CVN) toughness of 20 ft·lb (27 J) at 40°F (4°C) to be used for the following joints when subject to tension normal to the effective area: (1) CJP groove-welded tee and corner joints with steel backing left in place, unless the joints are designed using the nominal strength and resistance factor or safety factor for a PJP weld, and (2) CJP groove-welded splices in heavy sections (see Art. 5.9.1). When Charpy V-notch toughness is specified, the process consumables for all weld metal deposited in a joint, including tack welds, root pass, and subsequent passes, must be compatible to assure notch toughness in the completed weld.

5.9.5 Design Considerations for Bolted Connections

High-strength bolts should conform to the provisions of the RCSC Specification ("Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001) as well as the AISC Specification, which takes precedence should differences occur. High-strength bolts typically used include those designated as ASTM A325 and A490, as well as ASTM F1852, a "twist-off" tension-control bolt–nut–washer assembly.

Where bolt lengths exceeding 12 diameters or diameters exceeding 1½ in (38 mm) are required, bolts or threaded rods conforming to ASTM A354 or A449 should be used. These are treated as threaded rods for strength calculations. If used in slip-critical connections, the bolt geometry, including head and nut, should be equal to or proportional to that provided by ASTM A325 or A490 bolts. Installation should comply with all applicable requirements of the RCSC Specification with modifications as required for increased diameter or length.

TABLE 5.7 Available Strength of Welded Joints

Load type and direction relative to weld axis	Pertinent metal	Φ and Ω	Nominal strength (F_{bm} or F_w)	Required filler metal strength level[a,b]
Complete-joint-penetration groove welds				
Tension: Normal to weld axis	Strength of the joint is controlled by the base metal.			Matching filler metal shall be used. For tee and corner joints with backing left in place, notch tough filler metal is required.
Compression: Normal to weld axis	Strength of the joint is controlled by the base metal.			Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or compression: Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.			Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal.			Matching filler metal shall be used.[c]
Partial-joint-penetration grove welds including flare *V* groove and flare bevel groove welds				
Tension: Normal to weld axis	Base	$\Phi = 0.90$ $\Omega = 1.67$	F_y	Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\Phi = 0.80$ $\Omega = 1.88$	$0.60F_{Exx}$	
Compression: Column to base plate and column splices designed per J1.4(a)	Compressive stress need not be considered in design of welds joining the parts.			
Compression: Connections of members designed to bear other than columns as described in J1.4(b)	Base	$\Phi = 0.90$ $\Omega = 1.67$	F_y	
	Weld	$\Phi = 0.80$ $\Omega = 1.88$	$0.60F_{Exx}$	
Compression: Connections not finished-to-bear	Base	$\Phi = 0.90$ $\Omega = 1.67$	F_y	
	Weld	$\Phi = 0.80$ $\Omega = 1.88$	$0.90F_{Exx}$	
Tension or compression: Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.			Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Base	See J4, AISC Specification		
	Weld	$\Phi = 0.75$ $\Omega = 2.0$	$0.60F_{Exx}$	
Fillet welds including fillets in holes and slots and skewed tee joints				
Shear	Base	See J4, AISC Specification		Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\Phi = 0.75$ $\Omega = 2.0$	— $0.60F_{Exx}$[d]	
Tension or compression: Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.			

(*Continued*)

TABLE 5.7 Available Strength of Welded Joints (*Continued*)

Load type and direction relative to weld axis	Pertinent metal	Φ and Ω	Nominal strength (F_{bm} or F_w)	Required filler metal strength level[a,b]
Plug and slot welds				
Shear	Base	See J4, AISC Specification		Filler metal with a strength level equal to or less than matching filler metal is permitted.
Parallel to faying surface on the effective area	Weld	$\Phi = 0.75$ $\Omega = 2.0$	$0.60F_{Exx}$	

Notes:

[a] For matching weld metal see AWS D1.1, Sec. 3.3.

[b] Filler metal with a strength level one strength level greater than matching is permitted.

[c] Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\Phi = 0.80$, $\Omega = 1.88$ and $0.60F_{Exx}$ as the nominal strength.

[d] See the AISC Specification for alternatives.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

Depending on design considerations, high-strength bolts are tightened to either a specified bolt tension or to a snug-tight condition. The specified bolt tension, which is given in either Tables 5.8 or 5.9, may be achieved with the turn-of-nut method, a direct tension indicator, a calibrated wrench, or an alternative design bolt such as the twist-off type. The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact.

High-strength bolts may be tightened to the snug-tight condition in bearing-type connections or, for ASTM A325 or A325M bolts only, in tension or combined shear and tension applications, providing that loosening or fatigue due to vibration or load fluctuations are not design considerations. Bolts tightened only to the snug-tight condition must be clearly identified on design and erection drawings. In all other cases, high-strength bolts should be tightened to the specified bolt tension.

Tight mill scale is acceptable, but joint surfaces, including those adjacent to washers, must be free of loose scale.

In slip-critical connections in which the direction of loading is toward an edge of a connected part, the bearing strength must be checked.

TABLE 5.8 Minimum Bolt Pretension, kips*

Bolt size, in	A325 bolts	A490 bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

*Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

TABLE 5.9 Minimum Bolt Pretension, kN*

Bolt size, mm	A325M bolts	A490M bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

When A490 or A490M bolts over 1 in (25 mm) in diameter are used in slotted or oversized holes in external plies, a $^{5}/_{16}$-in-thick (8-mm) hardened washer (conforming to ASTM F436 except for thickness) should be used in lieu of the standard washer.

5.9.6 Holes for Bolted Connections

Maximum nominal sizes of holes for bolts are given in either Tables 5.10 or 5.11, but larger holes are permitted in column base details, such as required for location tolerance of anchor rods. Standard holes or short-slotted holes transverse to the direction of the load should generally be used, unless oversized holes or slotted holes are required and approved. Finger shims up to $^{1}/_{4}$ in (6 mm) thick may be used in slip-critical connections without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but must not be used in bearing-type connections. Hardened washers should be installed over oversized holes in an outer ply when high-strength bolts are used.

Short-slotted holes may be used in both slip-critical or bearing-type connections. Slots may be used for any direction of loading in slip-critical connections, but should have their length normal to the direction of the load in bearing-type connections. Washers should be installed over short-slotted holes in an outer ply and, when high-strength bolts are used, hardened washers should be used.

Long-slotted holes may be used in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes may be used for any

TABLE 5.10 Nominal Hole Dimensions, in

Bolt diameter	Standard hole diameter	Oversized hole diameter	Short-slot (width × length)	Long-slot (width × length)
$^{1}/_{2}$	$^{9}/_{16}$	$^{5}/_{8}$	$^{9}/_{16} \times ^{11}/_{16}$	$^{9}/_{16} \times 1^{1}/_{4}$
$^{5}/_{8}$	$^{11}/_{16}$	$^{13}/_{16}$	$^{11}/_{16} \times ^{7}/_{8}$	$^{11}/_{16} \times 1^{9}/_{16}$
$^{3}/_{4}$	$^{13}/_{16}$	$^{15}/_{16}$	$^{13}/_{16} \times 1$	$^{13}/_{16} \times 1^{7}/_{8}$
$^{7}/_{8}$	$^{15}/_{16}$	$1^{1}/_{16}$	$^{15}/_{16} \times 1^{1}/_{8}$	$^{15}/_{16} \times 2^{3}/_{16}$
1	$1^{1}/_{16}$	$1^{1}/_{4}$	$1^{1}/_{16} \times 1^{5}/_{16}$	$1^{1}/_{16} \times 2^{1}/_{2}$
$\geq 1^{1}/_{8}$	$d + ^{1}/_{16}$	$d + ^{5}/_{16}$	$(d + /_{16}) \times (d + ^{3}/_{8})$	$(d + ^{1}/_{16}) \times (2.5 \times d)$

Note: *d* is nominal bolt diameter, in.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

TABLE 5.11 Nominal Hole Dimensions, mm

Bolt diameter	Standard hole diameter	Oversized hole diameter	Short-slot (width × length)	Long-slot (width × length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27*	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

*Clearance provided allows the use of a 1-in bolt if desirable.

Note: *d* is nominal bolt diameter, mm.

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direction of loading in slip-critical connections, but must be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, should be provided. In high-strength bolted connections, the plate washers or continuous bars must be at least $^5/_{16}$ in (8 mm) thick and of structural-grade material, but generally need not be hardened (see Art. 5.9.5 for exceptions). Where hardened washers are required, they must be placed over the outer surface of the plate washer or bar.

5.9.7 Spacing and Edge Distance in Bolted Connections

The distance between centers of standard, oversized, or slotted holes should not be less than $2^2/_3$ times the nominal diameter, *d*, of the fastener, and preferably should not be less than 3*d*.

The distance in any direction from the center of a standard hole to an edge of a connected part should not be less than either the value from either Tables 5.12 or 5.13, or as required for strength (see Art. 5.9.8). The distance from the center of an oversized or slotted hole to an edge of a connected

TABLE 5.12 Minimum Edge Distance,* in, from Center of Standard Hole† to Edge of Connected Part

Bolt diameter (in)	At sheared edges	At rolled edges of plates, shapes or bars, or thermal-cut edges‡
$^1/_2$	$^7/_8$	$^3/_4$
$^5/_8$	$1^1/_8$	$^7/_8$
$^3/_4$	$1^1/_4$	1
$^7/_8$	$1^1/_2$§	$1^1/_8$
1	$1^3/_4$§	$1^1/_4$
$1^1/_8$	2	$1^1/_2$
$1^1/_4$	$2^1/_4$	$1^5/_8$
Over $1^1/_4$	$1^3/_4 \times d$	$1^1/_4 \times d$

*Lesser edge distances are permitted if bearing strength requirements, Art. 5.9.8 as appropriate, are satisfied.

†For oversized or slotted holes, see Table 5.14.

‡All edge distances in this column may be reduced $^1/_8$ in when the hole is at a point where the required strength does not exceed 25% of the maximum strength in the element.

§These may be $1^1/_4$ in at the ends of beam connection angles and shear end plates.

Note: *d* is nominal bolt diameter, in.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

TABLE 5.13 Minimum Edge Distance,* mm, from Center of Standard Hole[†] to Edge of Connected Part

Bolt diameter (mm)	At sheared edges	At rolled edges of plates, shapes or bars, or thermal-cut edges[‡]
16	28	22
20	34	26
22	38[§]	28
24	42[§]	30
27	48	34
30	52	38
36	64	46
Over 36	1.75*d*	1.25*d*

*Lesser edge distances are permitted if bearing strength requirements, Art. 5.9.8 as appropriate, are satisfied.

†For oversized or slotted holes, see Table 5.15.

‡All edge distances in this column may be reduced 3 mm when the hole is at a point where required strength does not exceed 25% of the maximum strength in the element.

§These may be 32 mm at the ends of beam connection angles and shear end plates.

Note: *d* is nominal bolt diameter, mm.

Source: "Specification for Structural Steel Buildings," American Institute of Steel Construction, Chicago, Ill., 2005, with permission.

part should be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from either Tables 5.14 or 5.15, or as required for strength.

The maximum distance from the center of a bolt to the nearest edge of parts in contact is 12 times the thickness of the connected part, and must also not exceed 6 in (150 mm). The longitudinal spacing of connectors between elements in continuous contact, such as a plate and a shape or two plates, is given by the AISC Specification as follows:

> For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 12 in (305 mm). For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 7 in (180 mm).

5.9.8 Design of Bolts and Threaded Parts for Strength

The design of bolts and threaded parts is based on the nominal stresses given in Table 5.16. The nominal stresses for four cases are treated in the following. Additionally, the bearing strength at bolt holes must be considered, and this is presented as item 5.

TABLE 5.14 Values of Edge Distance Increment C_2, in

		Slotted holes		
		Long axis perpendicular to edge		
Nominal diameter of fastener (in)	Oversized holes	Short slots	Long slots*	Long axis parallel to edge
$\leq {}^{7}/_{8}$	${}^{1}/_{16}$	${}^{1}/_{8}$	${}^{3}/_{4}d$	0
1	${}^{1}/_{8}$	${}^{1}/_{8}$		
$\geq 1{}^{1}/_{8}$	${}^{1}/_{8}$	${}^{3}/_{16}$		

*When length of slot is less than maximum allowable (see Table 5.10), C_2 values may be reduced by one-half the difference between the maximum allowable and actual slot lengths; *d* is nominal bolt diameter, in.

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TABLE 5.15 Values of Edge Distance Increment C_2, mm

Nominal diameter of fastener (mm)	Oversized holes	Slotted holes		
		Long axis perpendicular to edge		Long axis parallel to edge
		Short slots	Long slots*	
≤22	2	3	0.75*d*	0
24	3	3		
≥27	3	5		

*When length of slot is less than maximum allowable (see Table 5.11), C_2 values may be reduced by one-half the difference between the maximum allowable and actual slot lengths; *d* is nominal bolt diameter, mm.

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1. Tension or Shear. The design tension or shear strength, ϕR_n, and the allowable tension or shear strength, R_n/Ω, of a snug-tightened or a pretensioned high-strength bolt or threaded part is determined for the limit states of tension and shear rupture:

$$R_n = F_n A_b \tag{5.143}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

TABLE 5.16 Nominal Stress of Fasteners and Threaded Parts

Description of fastener	Nominal tensile stress, F_{nt}, ksi (MPa)	Nominal shear stress in bearing-type connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310) [a, b]	24 (165) [b, c, f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]
Threaded parts meeting the requirements of Sec. A3.4, AISC Specification, when threads are not excluded from shear planes	$0.75F_u$ [a, d]	$0.40F_u$
Threaded parts meeting the requirements of Sec. A3.4, AISC Specification, when threads are excluded from shear planes	$0.75F_u$ [a, d]	$0.50F_u$

[a] Subject to the requirements of App. 3, Design for Fatigue, AISC Specification.

[b] For A307 bolts the tabulated values must be reduced by 1% for each $^1/_{16}$ in (1.6 mm) over 5 diameters of length in the grip.

[c] Threads permitted in shear planes.

[d] The nominal tensile strength of the threaded portion of an upset rod, based on the cross-sectional area at its major thread diameter A_D, must be larger than the nominal body area of the rod before upsetting times F_y.

[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see App. 3, AISC Specification.

[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in (1270 mm), tabulated values must be reduced by 20%.

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where F_n = nominal tensile strength F_{nt} or shear strength F_{nv} (Table 5.16)
A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote [d], Table 5.16), in^2 (mm^2)

The required tensile strength must include any tension resulting from prying action produced by deformation of the connected parts.

2. Combined Tension and Shear in Bearing-Type Connections. For a bolt subjected to combined tension and shear, the available tensile strength is determined for the limit states of tension rupture and shear rupture:

$$R_n = F'_{\text{nt}} A_b \tag{5.144}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where F'_{nt} is the nominal tensile strength per unit area modified to include the effects of shear, ksi (MPa), as defined by the following equations.

For LRFD,

$$F'_{\text{nt}} = 1.3F_{\text{nt}} - \frac{F_{\text{nt}}}{\phi F_{nv}} f_v \le F_{\text{nt}} \tag{5.145}$$

For ASD,

$$F'_{\text{nt}} = 1.3F_{\text{nt}} - \frac{\Omega F_{\text{nt}}}{F_{nv}} f_v \le F_{\text{nt}} \tag{5.146}$$

where F_{nt} = nominal tensile stress (Table 5.16), ksi (MPa)
F_{nv} = nominal shear stress (Table 5.16), ksi (MPa)
f_v = required shear stress, ksi (MPa)

Also, the available shear stress of the fastener must equal or exceed the required shear stress f_v. Note that when the required stress in either shear or tension is less than or equal to 20% of the corresponding available strength per unit area, the effects of combined stress need not be investigated.

3. High-Strength Bolts in Slip-Critical Connections. High-strength bolts in slip-critical connections may be designed for the limit state of slip at either a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength and bearing strength. The AISC Specification indicates that, unless otherwise designated by the engineer of record, connections with standard holes or slots transverse to the direction of the load should be designed for slip at a serviceability limit state, and connections with oversized holes or slots parallel to the direction of the load should be designed for slip at the required strength. Generally, connections with standard holes or slots transverse to the direction of the load would only be designed for slip at the required strength for special cases where deformations due to slip could cause a structural failure as a result of changes in structure geometry.

The design slip resistance ϕR_n and the allowable slip resistance R_n/Ω is determined from the following equation for the limit state of slip:

$$R_n = \mu D_u h_{\text{sc}} T_b N_s \tag{5.147}$$

For connections designed for slip at a serviceability limit state, $\phi = 1.00$ and $\Omega = 1.50$. For connections designed for slip at the required strength limit state, $\phi = 0.85$ and $\Omega = 1.76$.

The following definitions apply:

μ = mean slip coefficient

μ = 0.35 for Class A surfaces (unpainted clean mill scale on steel surfaces, surfaces with Class A coatings on blast-cleaned steel, and hot-dipped galvanized and roughened surfaces)

μ = 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel),

D_u = multiplier that reflects ratio of mean installed bolt pretension to specified minimum bolt pretension = 1.13

h_{sc} = hole factor

h_{sc} = 1.00 for standard-size holes

h_{sc} = 0.85 for oversize and short-slotted holes

h_{sc} = 0.70 for long-slotted holes

T_b = minimum fastener tension, kips (kN) (Tables 5.8 or 5.9)

N_s = number of slip planes

4. Combined Tension and Shear in Slip-Critical Type Connections. When a slip-critical connection is subjected to an applied tension that reduces the net clamping force, the available slip resistance per bolt must be multiplied by the following reduction factor:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \text{(LRFD)} \tag{5.148a}$$

$$k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} \text{(ASD)} \tag{5.148b}$$

where N_b = number of bolts carrying the applied tension
T_a = tension force due to ASD load combinations, kips (N)
T_b = minimum fastener tension, kips (kN) (see Tables 5.8 or 5.9)
T_u = tension force due LRFD load combinations, kips (N)

5. Bearing Strength at Bolt Holes. Bearing strength must be checked for both bearing-type and slip-critical connections. The available bearing strength at bolt holes for the limit state of bearing is calculated using ϕ = 0.75 (LRFD) and Ω = 2.00 (ASD). The nominal bearing strength R_n is determined as follows.

The bearing resistance of a connection is the sum of the bearing resistances of the individual bolts. For a bolt in a connection with standard, oversized, and short-slotted holes independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force, there are two choices.

When deformation at the bolt hole at service load is a design consideration,

$$R_n = 1.2L_c tF_u \le 2.4dtF_u \tag{5.149a}$$

When deformation at the bolt hole at service load is not a design consideration,

$$R_n = 1.5L_c tF_u \le 3.0dtF_u \tag{5.149b}$$

For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force,

$$R_n = 1.0L_c tF_u \le 2.0dtF_u \tag{5.150}$$

The following definitions apply:

F_y = specified minimum yield stress of the connected material, ksi (MPa)

F_u = specified minimum tensile strength of the connected material, ksi (MPa)

L_c = clear distance, in the direction of the force, between edge of hole and edge of adjacent hole or edge of material, in (mm)

d = nominal bolt diameter, in (mm)

t = thickness of connected material, in (mm)

5.9.9 Design of Connecting Elements

Connecting elements of connections (gusset plates, splice plates, clip angles, etc.), as well as affected elements of the members themselves (flanges or webs at member ends, etc.), must be designed for the forces present. Design for tension, shear, block shear, and compression is treated in the following.

1. Elements in Tension. The available strength (design strength ϕR_n and allowable strength R_n/Ω) of affected member elements and connecting elements loaded in tension is the lower of the values for the limit states of tension yielding and tension rupture.

For tension yielding,

$$R_n = A_g F_y \tag{5.151}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For tension rupture of the connecting element,

$$R_n = A_e F_u \tag{5.152}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where A_e is the effective net area, which includes the shear lag effect. (See Art. 5.3.2.) Also, for bolted splice plates, A_e must not exceed $0.85A_g$.

2. Elements in Shear. The available shear yield strength of affected member elements and connecting elements in shear is the lower of the values for the limit states of shear yielding and shear rupture.

For shear yielding,

$$R_n = 0.60 A_g F_y \tag{5.153}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

For shear rupture,

$$R_n = 0.6 F_u A_{nv} \tag{5.154}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where A_{nv} = net area subject to shear, in^2 (mm^2)

3. Elements Subjected to Block Shear. Block shear rupture is a limit state defined by one or more shear failure paths and a perpendicular tension failure path. The available strength for this limit state is given by

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{\text{nt}} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{\text{nt}} \tag{5.155}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where A_{gv} = gross area subject to shear, in^2 (mm^2)
A_{nt} = net area subject to tension, in^2 (mm^2)
A_{nv} = net area subject to shear, in^2 (mm^2)
U_{bs} = 1.0 where the tension stress is uniform
= 0.5 where the tension stress is nonuniform (such as from bending)

4. Elements in Compression. The available strength of connecting elements in compression is determined for the limit states of yielding and buckling as follows.

For $KL/r \leq 25$, limit state of yielding,

$$P_n = F_y A_g \tag{5.156}$$

$$\phi = 0.9 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For $KL/r > 25$, limit state of buckling, see Art. 5.4.

5.9.10 Design of Fillers

Plates used to build up the thickness of one component being spliced to another are known as fillers. The AISC Specification gives the applicable design requirements as follows.

> In welded construction, any filler $^1\!/_4$-in (6 mm) or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than $^1\!/_4$-in (6 mm) thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.
>
> When a bolt that carries load passes through fillers that are equal to or less than $^1\!/_4$-in (6 mm) thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than $^1\!/_4$-in (6 mm) thick, one of the following requirements shall apply:
>
> (1) For fillers that are equal to or less than $^3\!/_4$-in (19 mm) thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.4(t - 0.25)]$ [S.I.: $[1 - 0.0154(t - 6)]$], where t is the total thickness of the fillers up to $^3\!/_4$-in (19 mm);
>
> (2) The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers;
>
> (3) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or
>
> (4) The joints shall be designed to prevent slip at required member strength levels in accordance with Sec. J3.8. [See Art. 5.9.8.]

5.9.11 Design of Splices

Groove-welded splices in plate girders and beams should develop the nominal strength of the smaller section spliced. Other types of splices in cross sections of plate girders and beams, such as bolted splices, should develop the strength required by the forces at the splice point.

5.9.12 Design of Components in Bearing

The available bearing strength (design bearing strength ϕR_n and allowable bearing strength R_n/Ω) for surfaces in contact is determined for the limit state of bearing (local compressive yielding) as given in the following. Use $\phi = 0.75$ (LRFD) and $\Omega = 2.00$ (ASD) for all cases.

The nominal bearing strength for milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners, is

$$R_n = 1.8F_yA_{\text{pb}} \tag{5.157}$$

where F_y = specified minimum yield stress, ksi (MPa)
A_{pb} = projected bearing area, in^2 (mm^2)

The nominal bearing strength for expansion rollers and rockers depends on the diameter d:
If $d \le 25$ in (635 mm),

$$R_n = \frac{1.2(F_y - 13)ld}{20} \tag{5.158a}$$

$$\text{SI: } R_n = \frac{1.2(F_y - 90)ld}{20} \tag{5.158b}$$

If $d > 25$ in (635 mm),

$$R_n = \frac{6.0(F_y - 13)l\sqrt{d}}{20} \tag{5.159a}$$

$$\text{SI: } R_n = \frac{30.2(F_y - 90)l\sqrt{d}}{20} \tag{5.159b}$$

where d = diameter, in (mm)
l = length of bearing, in (mm)

For bearing strength in bolt holes, see Art. 5.9.8.

5.9.13 Bearing on Concrete

The available bearing strength (design bearing strength ϕP_p and allowable bearing strength P_p/Ω) for column bases on concrete is determined as follows for the limit state of concrete crushing. Use ϕ_c = 0.60 (LRFD) and $\Omega = 2.50$ (ASD).

When the base bears on the full area of a concrete support, the nominal bearing strength, P_p, is

$$P_p = 0.85 f'_c A_1 \tag{5.160}$$

When the base bears on less than the full area of a concrete support, the nominal bearing strength is

$$P_p = 0.85 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7 f'_c A_1 \tag{5.161}$$

where A_1 = area of steel concentrically bearing on a concrete support, in^2 (mm^2) and A^2 = maximum area of the portion of the supporting surface geometrically similar to and concentric with the loaded area, in^2 (mm^2). Also, $A_2 \leq 4A_1$.

5.9.14 Design of Flanges and Webs for Concentrated Forces

This article addresses the design of single- and double-concentrated forces acting normal to the flange of wide flange sections and similar built-up shapes. A single concentrated force can be either tensile, such as from a hanger, or compressive, such as from an end reaction. Double-concentrated forces treated are one tensile and one compressive, oriented so as to form a couple on the same side of the member, such as the forces applied to a column flange by a beam in a moment connection.

When the required strength exceeds the available strength as determined for each limit state in this article, stiffeners and/or doublers (plates welded to and parallel with webs to increase resistance to concentrated forces) must be provided and designed for the difference between the required strength and the available strength. Stiffeners and doublers must also meet certain additional design requirements presented in Art. 5.9.15.

Design for various limit states is treated in the following. Local flange bending applies only for tensile forces, local web yielding applies for both tensile and compressive forces, and the other limit states apply only to compressive forces.

1. Local Flange Bending. This limit state applies to tensile single-concentrated forces and the tensile component of double-concentrated forces.

The available strength (design strength ϕR_n and allowable strength R_n/Ω) is determined for the limit state of local flange bending from

$$R_n = 6.25 t_f^2 F_{yf} \tag{5.162}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where F_{yf} = specified minimum yield stress of the flange, ksi (MPa), and t_f = thickness of the loaded flange, in (mm). When the concentrated force to be resisted is applied at a distance from the member end less than $10t_f$, R_n must be reduced by 50%. If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Eq. (5.162) need not be checked. Use a pair of transverse stiffeners if the available strength is inadequate.

2. Local Web Yielding. This limit state applies to single-concentrated forces, tension or compression, and both components of double-concentrated forces. The available strength for the limit state of local web yielding is determined using $\phi = 1.00$ (LRFD) and $\Omega = 1.50$ (ASD). Use a pair of transverse stiffeners or a doubler plate if the available strength is inadequate.

When the concentrated force to be resisted is applied at a distance from the member end greater than the depth of the member d, the nominal strength, R_n, is

$$R_n = (5k + N)F_{yw}t_w \tag{5.163}$$

When the concentrated *force* to be resisted is applied at a distance from the member end less than or equal to the depth of the member d,

$$R_n = (2.5k + N)F_{yw}t_w \tag{5.164}$$

where F_{yw} = specified minimum yield stress of the web, ksi (MPa)
N = length of bearing (not less than k for end beam reactions), in (mm)
k = distance from outer face of the flange to the web toe of the fillet, in (mm)
t_w = web thickness, in (mm)

3. Local Web Crippling. This limit state applies to compressive single-concentrated forces or the compressive component of double-concentrated forces. The available strength for the limit state of local web crippling is determined using $\phi = 0.75$ (LRFD) and $\Omega = 2.00$ (ASD). If the available strength is inadequate, use a transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending at least one-half the depth of the web.

When the concentrated compressive force to be resisted is applied at a distance from the member end greater than or equal to $d/2$, the nominal strength R_n is

$$R_n = 0.80t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{5.165}$$

When the concentrated compressive force to be resisted is applied at a distance from the member end less than $d/2$, the nominal strength R_n depends on the N/d ratio.

If $N/d \le 0.2$,

$$R_n = 0.40t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{5.166}$$

If $N/d > 0.2$,

$$R_n = 0.40t_w^2\left[1+\left(\frac{4N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{5.167}$$

where d = overall depth of the member, in (mm), and t_f = flange thickness, in (mm).

4. Web Sidesway Buckling. This limit state applies to compressive single-concentrated forces acting on members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force. The available strength for the limit state of web sidesway buckling is determined using $\phi = 0.85$ (LRFD) and $\Omega = 1.76$ (ASD). The nominal strength, R_n, depends on the flange rotational restraint and the $(h/t_w)/(l/b_f)$ ratio.

When the compression flange is restrained against rotation:

If $(h/t_w)/(l/b_f) \leq 2.3$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{5.168}$$

If $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

For the case with flange rotational restraint, if the required strength of the web exceeds the available strength, provide local lateral bracing at the tension flange, a pair of transverse stiffeners, or a doubler plate. The stiffeners should extend from the loaded flange for at least half the member depth and, here, designed to carry the full load.

When the compression flange is not restrained against rotation:

If $(h/t_w)/(l/b_f) \leq 1.7$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{5.169}$$

If $(h/t_w)/(l/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.

For the case without flange rotational support, if the required strength of the web exceeds the available strength, provide local lateral bracing at both flanges at the point of application of the concentrated forces.

The following definitions apply in Eqs. (5.168) and (5.169):

l = largest laterally unbraced length along either flange at the point of load, in (mm)

b_f = flange width, in (mm)

t_f = flange thickness, in (mm)

t_w = web thickness, in (mm)

h = clear distance between flanges less fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, in (mm)

C_r = 960,000 ksi (6.62×10^6 MPa) when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force

C_r = 480,000 ksi (3.31×10^6 MPa) when $M_u \geq M_y$ (LRFD) or $1.5M_a \geq M_y$ (ASD) at the location of the force

M_u = required flexural strength (LRFD); M_a = required flexural strength (ASD); and M_y = yield moment

5. Local Web Buckling. This limit state applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location. An example is the web of a column where forces are applied by bottom flanges of beams in a moment connection. If the required strength of the web exceeds the available strength, provide a single transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending the full depth of the web.

The available strength for the limit state of local web buckling is determined using $\phi = 0.90$ (LRFD) and $\Omega = 1.67$ (ASD). The nominal strength is

$$R_n = \frac{24 t_w^3 \sqrt{E F_{\mathrm{yw}}}}{h} \tag{5.170}$$

where symbols are as previously defined. When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, reduce R_n by 50%.

6. Web Panel Zone Shear. This limit state applies to double-concentrated forces applied to one or both flanges of a member at the same location. A typical case is the web of a column having a moment connection to its flanges from intersecting beams. The available strength of the web panel zone for the limit state of shear yielding is determined using $\phi = 0.90$ (LRFD) and $\Omega = 1.67$ (ASD) for all cases. If the required strength of the web exceeds the available strength, provide a doubler plate (on one side of the web or both) or a pair of diagonal stiffeners within the boundaries of the rigid connection where the webs lie in a common plane.

When the effect of panel-zone deformation on frame stability is not considered in the analysis, the nominal strength R_v is as follows.

If $P_r \le 0.4P_c$,

$$R_v = 0.60F_y d_c t_w \tag{5.171}$$

If $P_r > 0.4P_c$,

$$R_v = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \tag{5.172}$$

When frame stability, including plastic panel-zone deformation, is considered in the analysis, the nominal strength R_v is as follows.

If $P_r \le 0.75P_c$,

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{\text{cf}}t_{\text{cf}}^2}{d_b d_c t_w}\right) \tag{5.173}$$

If $P_r > 0.75P_c$,

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{\text{cf}}t_{\text{cf}}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \tag{5.174}$$

The following definitions apply in Eqs. (5.171) to (5.174):

P_r = required strength
$P_c = P_y$, kips (N) (LRFD)
$P_c = 0.6P_y$, kips (N) (ASD)
$P_y = F_y A$ = axial yield strength of the column, kips (N)
A = column cross-sectional area, in^2 (mm^2)
t_w = column web thickness, in (mm)
b_{cf} = width of column flange, in (mm)
t_{cf} = thickness of the column flange, in (mm)
d_b = beam depth, in (mm)
d_c = column depth, in (mm)
F_y = specified minimum yield stress of the column web, ksi (MPa)

5.9.15 Additional Stiffener and Doubler Plate Requirements

The AISC Specification gives additional requirements that must be met for stiffeners and doubler plates.

1. Unframed Ends of Beams and Girders. At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, must be provided.

2. Stiffener Requirements for Concentrated Forces. Stiffeners required to resist tensile concentrated forces must be designed as tension members and welded to the loaded flange and the web. Size the flange welds for the difference between the required strength and the applicable available limit state strength. Size the web welds to transfer to the web the algebraic difference in tension force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces must be designed in accordance with the requirements for compression elements in Art. 5.9.9 and the dimensional requirements in Art. 5.4.3, must either bear on or be welded to the loaded flange, and must be welded to the web. The welds to the flange must be sized for the difference between the required strength and the applicable available limit state strength. The weld to the web must be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, the bearing strength given in Art. 5.9.12 applies.

Transverse full-depth bearing stiffeners for compressive forces applied to a beam or plate girder flange must be designed as axially compressed members (columns) in accordance with the requirements for compression elements in Art. 5.9.9 and the dimensional requirements in Art. 5.4.3. Determine member properties using an effective length of $0.75h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ (at interior stiffeners) or $12t_w$ (at the ends of members). The weld connecting full-depth bearing stiffeners to the web should be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and diagonal stiffeners must also comply with the following:

- The width of each stiffener plus one-half the thickness of the column web must not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.
- The thickness of a stiffener must not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and greater than or equal to the width divided by 15.
- Transverse stiffeners must extend a minimum of one-half the depth of the member, except that full-depth stiffeners are required when used to resist web local buckling and at unframed ends.

3. Doubler-Plate Requirements for Concentrated Forces. Size the doubler plate to provide the additional material necessary to equal or exceed strength requirements. Weld the doubler plate to develop the proportion of the total force transmitted to the doubler plate. Doubler plates required for tension strength must be designed in accordance with the requirements for tension members (Art. 5.3). Doubler plates required for compression strength must be designed in accordance with the requirements for compression members (Art. 5.4). Doubler plates required for shear strength must be designed in accordance with the requirements for members in shear (Art. 5.6).

CHAPTER 6

DESIGN OF BUILDING MEMBERS

Ali A. K. Haris, Ph.D., P.E.
President
Haris Engineering, Inc.
Overland Park, Kansas

Kaise Haris
Structural Engineer
Haris Engineering, Inc.
Overland Park, Kansas

Steel members in building structures can be part of the floor framing system to carry gravity loads, the vertical framing system, the lateral framing system to provide lateral stability to the building and resist lateral loads, or two or more of these systems. Floor members are normally called **joists, purlins, beams,** or **girders.** Roof members are also known as **rafters.**

Purlins, which support floors, roofs, and decks, are relatively close in spacing. Beams are floor members supporting the floor deck. Girders are steel members spanning between columns and usually supporting other beams. Transfer girders are members that support columns and transfer loads to other columns. The primary stresses in joists, purlins, beams, and girders are due to flexural moments and shear forces.

Vertical members supporting floors in buildings are designated **columns.** The most common steel shapes used for columns are wide-flange sections, pipes, and tubes. Columns are subject to axial compression and also often to bending moments. Slenderness in columns is a concern that must be addressed in the design.

Lateral framing systems may consist of the floor girders and columns that support the gravity floor loads but with rigid connections. These enable the flexural members to serve the dual function of supporting floor loads and resisting lateral loads. Columns, in this case, are subject to combined axial loads and moments. The lateral framing system also can consist of vertical diagonal braces or shear walls whose primary function is to resist lateral loads. Mixed bracing systems and rigid steel frames are also common in tall buildings.

Most steel floor framing members are considered simply supported. Most steel columns supporting floor loads only are considered as pinned at both ends. Other continuous members, such as those in rigid frames, must be analyzed as plane or space frames to determine the members' forces and moments.

Other main building components are steel trusses used for roofs or floors to span greater lengths between columns or other supports, built-up plate girders and stub girders for long spans or heavy loads, and open-web steel joists. See also Chap. 7.

This chapter addresses the design of these elements, which are common to most steel buildings. Design is based on the "Specification for Structural Steel Buildings," American Institute of Steel

Construction (AISC). This unified specification covers both allowable strength design (ASD) and load and resistance factor design (LRFD), as reviewed in detail in Chap. 5. Generally, the nominal strength equations are the same for both methods, but the nominal strength is multiplied by a resistance factor to determine the "design strength" for LRFD and divided by a safety factor to determine the "allowable strength" for ASD. LRFD uses greater (factored) loads, but the final results of the two methods are about the same. The examples in this chapter are for LRFD, but are easily adapted to ASD.

6.1 TENSION MEMBERS

Members subject to tension loads only include hangers, diagonal braces, truss members, and columns that are part of the lateral bracing system with significant uplift loads.

The AISC "Specification for Structural Steel Buildings" (hereafter AISC Specification) gives the nominal strength P_n (kips) of a cross section subject to tension only as the smaller of the capacity of yielding in the gross section,

$$P_n = F_y A_g \tag{6.1}$$

or the capacity at fracture in the net section,

$$P_n = F_u A_e \tag{6.2}$$

For LRFD, factored load may not exceed either of the following:

$$P_u = \phi F_y A_g \qquad \phi = 0.9 \tag{6.3a}$$

$$P_u = \phi F_u A_e \qquad \phi = 0.75 \tag{6.3b}$$

where F_y and F_u are, respectively, the yield stress and the tensile strength (ksi) of the material. For ASD, the load may not exceed

$$P = \frac{F_y A_g}{1.67} \tag{6.4a}$$

$$P = \frac{F_u A_e}{2.00} \tag{6.4b}$$

A_g is the gross area (in^2) of the member, and A_e is the effective cross-sectional area at the connection.

The effective area A_e is given by

$$A_e = UA_n \tag{6.5}$$

where A_n = net area
U = reduction coefficient (see Art. 5.3)

6.2 EXAMPLE—LRFD FOR DOUBLE-ANGLE HANGER

A composite floor framing system is to be designed for sky boxes of a sports arena structure. The sky boxes are located about 15 ft below the bottom chord of the roof trusses. The sky-box framing is supported by an exterior column at the exterior edge of the floor and by steel hangers 5 ft from the inside edge of the floor. The hangers are connected to either the bottom chord of the trusses or to the steel beams spanning between trusses at roof level. The reactions due to service dead and live loads at the hanger locations are $P_{\text{DL}} = 55$ kips and $P_{\text{LL}} = 45$ kips. Hangers supporting floors and balconies should be designed for additional impact factors representing 33% of the live loads. Assume load factors of 1.2 for live load and 1.6 for dead load, in combination; and 1.4 for live load alone.

The factored axial tension load is the larger of

$$P_{UT} = 55 \times 1.2 + 45 \times 1.6 \times 1.33 = 162 \text{ kips (governs)}$$

$$P_{UT} = 55 \times 1.4 = 77 \text{ kips}$$

Double angles of A36 steel with one row of three bolts at 3-in spacing will be used ($F_y = 36$ ksi and $F_u = 58$ ksi). The required area of the section is determined as follows. From Eq. (6.3a), with $P_U = 162$ kips,

$$A_g = \frac{162}{0.9 \times 36} = 5.00 \text{ in}^2$$

From Eq. (6.3b),

$$A_e = \frac{162}{0.75 \times 58} = 3.72 \text{ in}^2$$

Try two angles, $5 \times 3 \times {}^3/_8$ in, with $A_g = 5.72 \text{ in}^2$. For 1-in-diameter A325 bolts in the 5-in leg with hole size $1^1/_{16}$ in, the net area of the angles is

$$A_n = 5.72 - 2 \times {}^3/_8 \times {}^{17}/_{16} = 4.92 \text{ in}^2$$

and

$$U = 1 - \left(\frac{\bar{x}}{L}\right) = 1 - \frac{0.698}{9} = 0.92$$

The effective area is

$$A_e = UA_n = 0.92 \times 4.92 = 4.53 \text{ in}^2 > 3.72 \text{ in}^2 \qquad \text{OK}$$

6.3 EXAMPLE—LRFD FOR WIDE-FLANGE TRUSS MEMBERS

One-way, long-span trusses are to be used to frame the roof of a sports facility. The truss span is 300 ft. All members are wide-flange sections. (See Fig. 6.1 for the typical detail of the bottom-chord splice of the truss.)

Connections of the truss diagonals and verticals to the bottom chord are bolted. Slip-critical, the connections serve also as splices, with $1^1/_8$-in-diameter A325 bolts, in oversized holes to facilitate truss assembly in the field. The holes are $1^7/_{16}$ in in diameter. The bolts are placed in two rows in each flange. The number of bolts per row is more than two. The web of each member is also spliced with a plate with two rows of $1^1/_8$-in-diameter A325 bolts.

The structural engineer analyzes the trusses as pin-ended members. Therefore, all members are considered to be subject to axial forces only. Members of longspan trusses with significant deflections and large, bolted, slip-critical connections, however, may have significant bending moments.

The factored axial tension in the bottom chord at midspan due to combined dead, live, theatrical, and hanger loads supporting sky boxes is $P_u = 2280$ kips.

With a wide-flange section of Grade 50 steel ($F_y = 50$ ksi and $F_u = 65$ ksi), the required minimum gross area, from Eq. (6.3a), is

$$A_g = \frac{P_u}{\phi F_y} = \frac{2280}{0.9 \times 50} = 50.67 \text{ in}^2$$

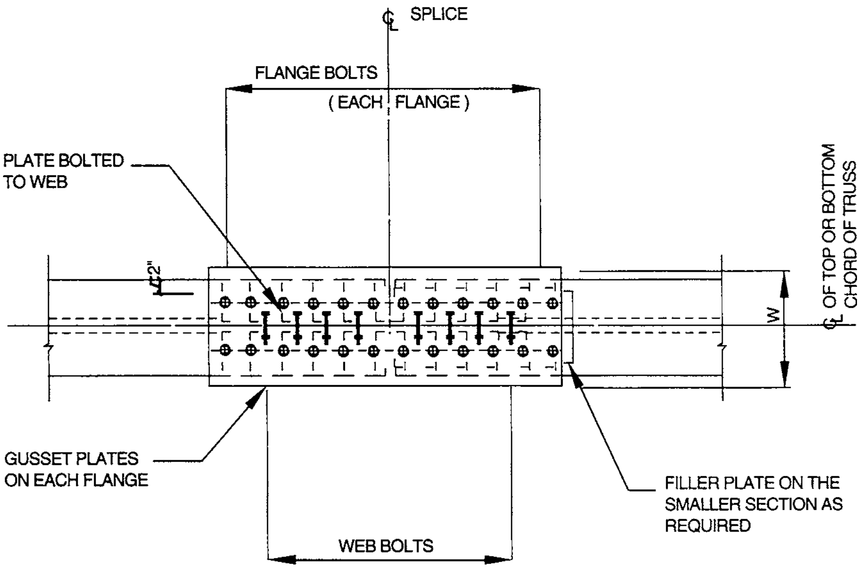


FIGURE 6.1 Detail of a splice in the bottom chord of a truss.

Try a W14 × 176 section with $A_g = 51.8\ \text{in}^2$, flange thickness $t_f = 1.31$ in, and web thickness $t_w = 0.83$ in. The net area is

$$A_n = 51.8 - (2 \times 1.31 \times 1.4375 \times 2 + 2 \times 0.83 \times 1.4375) = 41.88\ \text{in}^2$$

Since all parts of the wide-flange section are connected at the splice connection, $U = 1$ for determination of the effective area from Eq. (6.5). Thus $A_e = A_n = 41.88\ \text{in}^2$. From Eq. (6.3b), the design strength is

$$\phi P_n = 0.75 \times 65 \times 41.88 = 2042\ \text{kips} < 2280\ \text{kips} \qquad \text{NG}$$

Try a W14 × 193 with $A_g = 56.8\ \text{in}^2$, $t_f = 1.44$ in, and $t_w = 0.89$ in. The net area is

$$A_n = 56.8 - (2 \times 1.44 \times 1.4375 \times 2 + 2 \times 0.89 \times 1.4375)$$
$$= 45.96\ \text{in}^2$$

From Eq. (6.3b), the design strength is

$$\phi P_n = 0.75 \times 65 \times 45.96 = 2241\ \text{kips} < 2280\ \text{ksi} \qquad \text{NG}$$

Use the next size, W14 × 211.

6.4 COMPRESSION MEMBERS

Steel members in buildings subject to compressive axial loads include columns, truss members, struts, and diagonal braces. Slenderness is a major factor in design of compression members. Most suitable steel shapes are pipes, tubes, or wide-flange sections, as designated for columns in the AISC "Manual of Steel Construction." Double angles, however, are commonly used for diagonal braces and truss members. Double angles can be easily connected to other members with gusset plates and bolts or welds.

The AISC Specification gives the nominal strength P_n (kips) for a steel compact or noncompact section without slender elements in compression as

$$P_n = A_g F_{\text{cr}} \tag{6.6}$$

The factored load for LRFD, P_u (kips), may not exceed

$$P_u = \phi P_n \qquad \phi = 0.90 \tag{6.7}$$

The flexural buckling stress, F_{cr}, is a function of the material strength and slenderness. First, the elastic critical buckling stress F_e (ksi) is determined as follows.

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \tag{6.8}$$

When $(KL/r) \le 4.71\sqrt{E/F_y}$ (or $F_e \ge 0.44 F_y$),

$$F_{\text{cr}} = (0.658^{F_y/F_e})F_y \tag{6.9}$$

When $(KL/r) > 4.71\sqrt{E/F_y}$ (or $F_e < 0.44 F_y$),

$$F_{\text{cr}} = 0.877 F_e \tag{6.10}$$

where A_g = gross area of the member, in^2
K = effective length factor (see Art. 5.4)
L = unbraced length of the member, in
F_y = yield stress of steel, ksi
E = modulus of elasticity of steel, ksi
r = radius of gyration corresponding to the plane of buckling, in

6.5 EXAMPLE—LRFD FOR STEEL PIPE IN AXIAL COMPRESSION

Pipe sections of A36 steel are to be used to support framing for the flat roof of a one-story factory building. The roof height is 18 ft from the tops of the steel roof beams to the finish of the floor. The steel roof beams are 16 in deep, and the bases of the steel-pipe columns are 1.5 ft below the finished floor. A square joint is provided in the slab at the steel column. Therefore, the concrete slab does not provide lateral bracing. The effective height of the column, from the base of the column to the center line of the steel roof beam, is

$$h = 18 + 1.5 - \frac{16}{2 \times 12} = 18.83 \text{ ft}$$

The dead load on the column is 30 kips. The live load due to snow at the roof is 36 kips. The factored axial load is the larger of the following:

$$P_u = 30 \times 1.4 = 42 \text{ kips}$$
$$P_u = 30 \times 1.2 + 36 \times 1.6 = 93.6 \text{ kips} \qquad \text{(governs)}$$

With the factored load known, the required pipe size may be obtained from a table in the AISC "Manual of Steel Construction." For KL = 19 ft, a standard 6-in pipe (weight 18.97 lb per linear ft) offers the least weight for a pipe with a compression-load capacity of at least 93.6 kips. For verification of this selection, the following computations for the column capacity were made based on a radius of gyration r = 2.25 in.

Because

$$\left(\frac{KL}{r} = \frac{19\times 12}{2.25} = 101.3\right) \le \left(4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{36}} = 133.7\right)$$

The elastic critical stress, from Eq. (6.8), is

$$F_e = \frac{\pi^2 \times 29{,}000}{101.3^2} = 27.87 \text{ ksi}$$

Equation (6.9) yields the critical stress

$$F_{\text{cr}} = (0.658^{36/27.87}) \times 36 = 20.97 \text{ ksi}$$

The design strength of the 6-in pipe, then, from Eqs. (6.6) and (6.7), is

$$\phi P_n = 0.90 \times 5.58 \times 20.97 = 105.3 \text{ kips} > 93.6 \text{ kips} \qquad \text{OK}$$

6.6 EXAMPLE—LRFD FOR WIDE-FLANGE SECTION WITH AXIAL COMPRESSION

A wide-flange section is to be used for columns in a five-story steel building. A typical interior column in the lowest story will be designed to support gravity loads. (In this example, no eccentricity will be assumed for the load.) The effective height of the column is 18 ft. The axial loads on the column from the column above and from the steel girders supporting the second level are dead load 420 kips and live load (reduced according to the applicable building code) 120 kips.

The factored axial load is the larger of the following:

$$P_u = 420 \times 1.4 = 588 \text{ kips}$$

$$P_u = 420 \times 1.2 + 120 \times 1.6 = 696 \text{ kips} \qquad \text{(governs)}$$

Use of the column design tables of the AISC "Manual of Steel Construction" presents the following options:

For a column of A36 steel, select a W14 × 99, with a design strength $\phi P_n = 745$ kips.

For a column of A992 steel, select a W12 × 87, with a design strength $\phi P_n = 758$ kips.

Both steels cost about the same, so the W12 × 87 of A992 steel is the most economical wide-flange section.

6.7 EXAMPLE—LRFD FOR DOUBLE ANGLES WITH AXIAL COMPRESSION

Double angles are the preferred steel shape for a diagonal in the vertical bracing part of the lateral framing system in a multistory building (Fig. 6.2). Lateral load on the diagonal in this example is due to wind only and equals 65 kips. The diagonals also support the steel beam at midspan. As a result, the compressive force on each brace due to dead loads is 15 kips, and that due to live loads is 10 kips. The maximum combined factored load is $P_u = 1.2 \times 15 + 1.3 \times 65 + 0.5 \times 10 = 107.5$ kips.

The length of the brace is 19.85 ft, neglecting the size of the joint. A36 steel is selected because slenderness is a major factor in determining the nominal capacity of the section. Selection of the size of double angles is based on trial and error, which can be assisted by load tables in the AISC "Manual

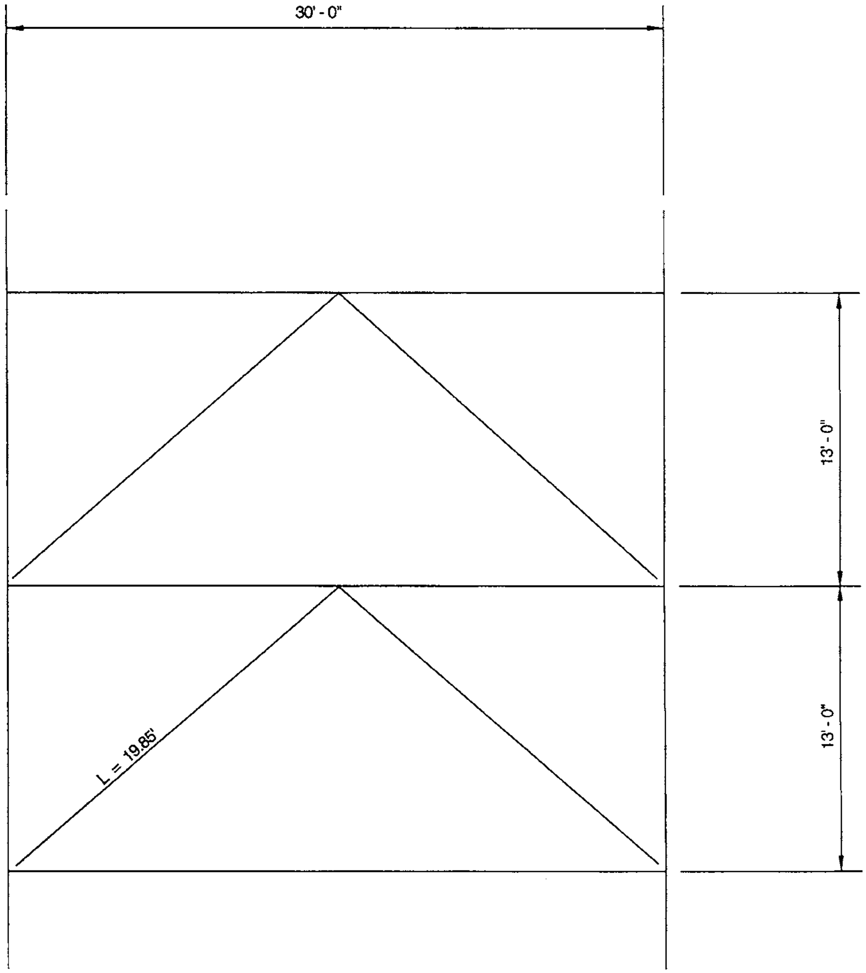


FIGURE 6.2 Inverted V-braces in a lateral bracing bent.

of Steel Construction" for columns of various shapes and sizes. For the purpose of illustration of the step-by-step design, double angles 6 × 4 × $^5/_8$ in with $^3/_8$-in spacing between the angles are chosen. Section properties are as follows: gross area $A_g = 11.7$ in^2 and the radii of gyration are $r_x = 1.90$ in and $r_y = 1.67$ in.

First, the slenderness effect must be evaluated to determine the corresponding critical compressive stresses. The effect of the distance between the spacer plates connecting the two angles is a design consideration in LRFD. Assuming that the connectors are fully tightened bolts, the system slenderness is calculated as follows:

The AISC Specification defines the following modified column slenderness for a built-up member where intermediate connectors are welded or fully tension bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\left(\frac{\alpha^2}{1+\alpha^2}\right)\left(\frac{a}{r_{ib}}\right)^2} \tag{6.11}$$

where $(KL/r)_o$ = column slenderness of built-up member acting as a unit
α = separation ratio = $h/2r_{ib}$
h = distance between centroids of individual components perpendicular to member axis of buckling, in
a = distance between connectors, in
r_{ib} = radius of gyration of individual angle relative to its centroidal axis parallel to member axis of buckling, in

In this case, $h = 1.03 + 0.375 + 1.03 = 2.44$ in and $\alpha = 2.44/(2 \times 1.13) = 1.08$. Assume maximum spacing between connectors is $a = 80$ in. With $K = 1$, substitution in Eq. (6.11) yields

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{19.85\times 12}{1.66}\right)^2 + 0.82\left(\frac{1.08^2}{1+1.08^2}\right)\left(\frac{80}{1.13}\right)^2} = 150$$

The elastic critical stress buckling stress, from Eq. (6.8), is

$$F_e = \frac{\pi^2 \times 29{,}000}{[(19.85\times 12)/1.66]^2} = 13.90 \text{ ksi}$$

For the determination of the critical stress F_{cr}, since

$$\left(\frac{KL}{r} = \frac{19.85\times 12}{1.67} = 143.5\right) > \left(4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{36}} = 133.7\right)$$

$$F_e = \frac{\pi^2 \times 29{,}000}{143.5^2} = 13.90 \text{ ksi}$$

The critical stress, from Eq. (6.10), is

$$F_{\text{cr}} = 0.877 \times 13.90 = 12.19$$

From Eqs. (6.6) and (6.7), the design strength is

$$\phi P_n = 0.90 \times 11.7 \times 12.19 = 128.4 \text{ kips} > 107.5 \text{ kips} \qquad \text{OK}$$

6.8 STEEL BEAMS

According to the AISC Specification, the nominal capacity M_p (in·kips) of a steel section in flexure is equal to the plastic moment:

$$M_p = ZF_y \tag{6.12}$$

where Z is the plastic section modulus (in^3) and F_y is the steel yield strength (ksi). However, this applies only when local or lateral torsional buckling of the compression flange is not a governing criterion. The nominal capacity M_p is reduced when the compression flange is not braced laterally for a length that exceeds the limiting unbraced length for full plastic bending capacity L_p. Also, the nominal moment capacity is less than M_p when the ratio of the compression-element width to its thickness exceeds limiting slenderness parameters for compact sections. The same is true for the effect of the ratio of web depth to thickness. (See Chap. 5.)

In addition to strength requirements for design of beams, serviceability is important. Deflection limitations defined by local codes or standards of practice must be maintained in selecting member sizes. Dynamic properties or the beams are also important design parameters in determining the vibration behavior of floor systems for various uses.

The shear forces in the web of wide-flange sections should be calculated, especially if large concentrated loads occur near the supports. For LRFD, the AISC Specification requires that the factored shear V_v (kips) not exceed

$$V_v = \phi_v V_n \qquad \phi_v = 0.90 \tag{6.13}$$

The nominal shear strength V_n (kips) is calculated as follows:

$$V_n = 0.6F_{yw}A_wC_v \tag{6.14}$$

where h = clear distance between flanges (less the fillet or cover radius for rolled shapes, in
t_w = web thickness, in
F_{yw} = yield strength of the web, ksi
A_w = web area, in^2
C_v = web shear coefficient

For $h/t_w \le 1.10\sqrt{k_vE/F_{yw}}$, or $h/t_w \le 2.34\sqrt{E/F_{yw}}$ for rolled I-shaped members ($k_v = 5$),

$$C_v = 1.0 \tag{6.15}$$

For $1.10\sqrt{k_vE/F_{yw}} < h/t_w \le 1.37\sqrt{k_vE/F_{yw}}$,

$$C_v = \frac{1.10\sqrt{k_vE/F_{yw}}}{h/t_w} \tag{6.16}$$

For $h/t_w > 1.37\sqrt{k_vE/F_{yw}}$,

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_{yw}} \tag{6.17}$$

$$\begin{aligned} k_v &= 5 + \frac{5}{(a+h)^2} \\ &= 5 \qquad \text{when } a/h > 3 \text{ or } a/h > [260/(h/t)]^2 \\ &= 5 \qquad \text{if no stiffeners are used} \end{aligned}$$

where a = distance between transverse stiffeners

6.9 EXAMPLE—LRFD FOR SIMPLE-SPAN FLOOR BEAM

Floor framing for an office building is to consist of open-web steel joists with a standard corrugated metal deck and 3-in-thick normal-weight concrete fill. The joists are to be spaced 3 ft center to center. Steel beams spanning 30 ft between columns support the joists. A bay across the building floor is shown in Fig. 6.3.

Floor beam *AB* in Fig. 6.3 will be designed for this example. The loads are listed in Table 6.1. The live load is reduced in Table 6.1, as permitted by the applicable building code. The reduction factor R is given by the smaller of

$$R = 0.0008(A - 150) \tag{6.18}$$

$$R = 0.231\left(1 + \frac{D}{L}\right) \tag{6.19}$$

$$R = 0.4 \qquad \text{for beams} \tag{6.20}$$

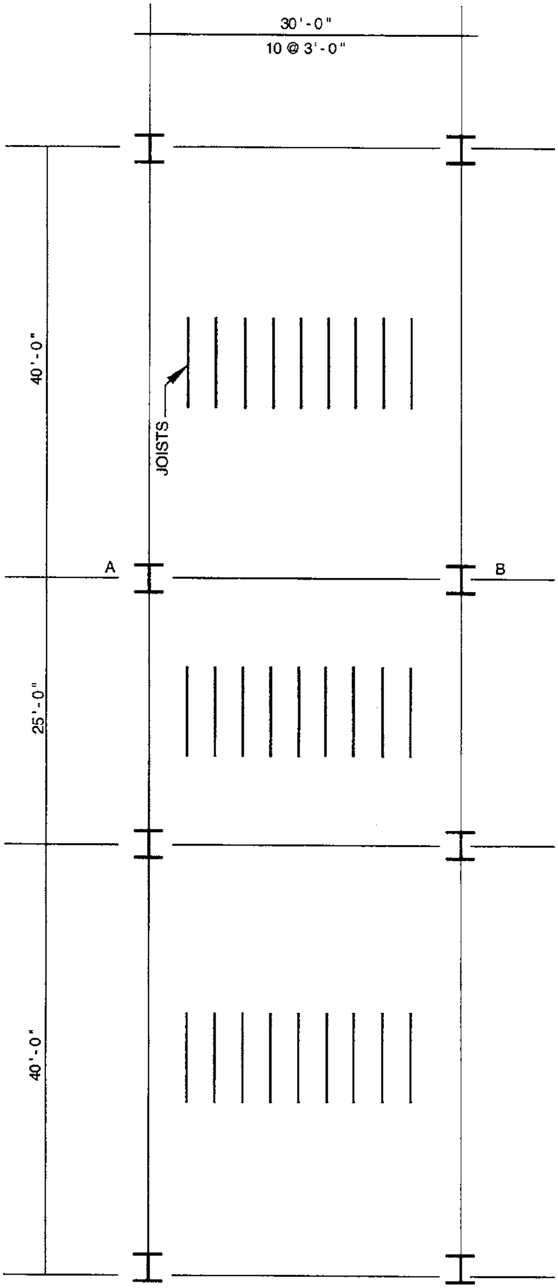


FIGURE 6.3 Part of the floor framing for an office building.

where D = dead load
L = live load
A = area supported = 30(40 + 25)/2 = 975 ft^2

From Eq. (6.18), $R = 0.0008(975 - 150) = 0.66$.
From Eq. (6.19), $R = 0.231\ (1 + {}^{73}/_{50}) = 0.568$.
From Eq. (6.20), R = 0.4 (governs), and the reduced live load is 50(10.4) = 30 lb per ft^2, as shown in Table 6.1.

If the beam's self-weight is assumed to be 45 lb/ft, the factored uniform load is the larger of the following:

TABLE 6.1 Loads on Floor Beam *AB* in Fig. 6.3

Dead loads, lb/ft^2	
Floor deck	45
Ceiling and mechanical ductwork	5
Open-web joists	3
Partitions	20
Total dead load (exclusive of beam weight)	73
Live loads, lb/ft^2	
Full live load	50
Reduced live load: 50(1 − 0.4)	30

$$W_u = 1.4\left[\frac{73(40+25)}{2} + 45\right] = 3384.5 \text{ lb/ft}$$

$$W_u = 1.2\left[\frac{73(40+25)}{2} + 45\right] + 1.6 \times \frac{30(40+25)}{2}$$

$$= 4461 \text{ lb/ft} \qquad \text{(governs)}$$

The factored moment then is

$$M_u = \frac{4.461(30)^2}{8} = 501.9 \text{ kip}\cdot\text{ft}$$

To select for beam *AB* a wide-flange section with F_y = 50 ksi, the top flange being braced by joists, the required plastic modulus Z_x is determined as follows.

The factored moment M_u may not exceed the design strength of ϕM_r, and

$$\phi M_r = \phi Z_x F_y \tag{6.21}$$

Therefore, from Eq. (6.21),

$$Z_x = \frac{501.9 \times 12}{0.9 \times 50} = 133.8 \text{ in}^3$$

A wide-flange section W24 × 55 with Z = 135 in^3 is adequate.

Next, criteria are used to determine if deflections are acceptable. For the live-load deflection, the span L is 30 ft, the moment of inertia of the W24 × 55 is I = 1360 in^4, and the modulus of elasticity E = 29,000 ksi. The live load is W_L = 30(40 + 25)/2 = 975 lb/ft. Hence the live-load deflection is

$$\Delta_L = \frac{5W_L L^4}{384EI} = \frac{5 \times 0.975 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 1{,}360} = 0.451 \text{ in}$$

This value is less than L/360 = 30 × ${}^{12}/_{360}$ = 1 in, as specified in the applicable building code. The code requires that deflections due to live load plus a factor K times dead load not exceed L/240. The K value, however, is specified as zero for steel. [The intent of this requirement is to include the long-term effect (creep) due to dead loads in the deflection criteria.] Hence the live-load deflection satisfies this criterion.

The immediate deflection due to the weight of the concrete and the floor framing is also commonly determined. If excessive deflections due to such dead loads are found, it is recommended that steel members be cambered to produce level floors and to avoid excessive concrete thickness during finishing the wet concrete.

In this example, the load due to the weight of the floor system is from Table 6.1 with the weight of the beam added:

$$W_{wt} = \frac{(45+3)(40+25)}{2} + 55 = 1615 \text{ lb/ft}$$

The deflection due to this load is

$$\Delta_{wt} = \frac{5 \times 1.615 \times 30^4 \times 12}{384 \times 29,000 \times 1,360} = 0.746 \text{ in}$$

Therefore, cambering the beam 3/4 in at midspan is recommended.

For review of the shear capacity of the section, the depth/thickness ratio of the web is

$$\frac{h}{t_w} = 54.6 < \left(2.45\sqrt{\frac{29,000}{50}} = 59.0 \right)$$

From Eq. (6.14), the design shear strength is

$$\phi V_n = 0.9 \times 0.6 \times 50 \times 23.6 \times 0.395 = 252 \text{ kips}$$

The factored shear force near the support is

$$V_u = 4.461 \times \frac{30}{2} = 66.92 \text{ kips} < 252 \text{ kips} \qquad \text{OK}$$

As illustrated in this example, it usually is not necessary to review the design of each simple beam with uniform load for shear capacity.

6.10 EXAMPLE—LRFD FOR FLOOR BEAM WITH UNBRACED TOP FLANGE

A beam of A992 steel with a span of 20 ft is to support the concentrated load of a stub pipe column at midspan. The factored concentrated load is 55 kips. No floor deck is present on either side of the beam to brace the top flange, and the pipe column is not capable of bracing the top flange laterally. The weight of the beam is assumed to be 50 lb/ft.

The factored moment at midspan is

$$M_u = 55 \times \frac{20}{4} + 0.050 \times \frac{20^2}{8} = 277.5 \text{ kip} \cdot \text{ft}$$

A beam size for a first trial can be selected from a load-factor design table for steel with $F_y = 50$ ksi in the AISC "Steel Construction Manual." The table lists several properties of wide-flange shapes, including plastic moment capacities ϕM_p. For example, an examination of the table indicates that the lightest beam with ϕM_p exceeding 277.5 kip·ft is a W18 × 40 with $\phi M_p = 294$ kip·ft. Whether this beam can be used, however, depends on the resistance of its top flange to buckling. The manual table also lists the limiting laterally unbraced lengths for full plastic bending capacity L_p and inelastic torsional buckling L_r. For the W18 × 40, $L_p = 4.5$ ft and $L_r = 12.0$ ft (Table 6.2).

In this example, then, the 20-ft unbraced beam length exceeds L_r, so the design strength will be less than M_p. From the table in the manual for the W18 × 40 (Grade 50), design strength $\phi M_r = 205$ kip·ft < 277.5 kip·ft. Therefore, a larger size is necessary.

TABLE 6.2 Properties of Selected W Shapes for LRFD

Property	W18 × 35 Grade 50	W18 × 40 Grade 50	W21 × 50 Grade 50	W21 × 62 Grade 50
φM_p, kip·ft	249	294	416	540
L_p, ft	4.31	4.49	4.59	6.25
L_r, ft	11.5	12.0	12.5	16.7
φM_r, kip·ft	173	205	285	381
S_x, in^3	57.6	68.4	94.5	127
I_y, in^4	15.3	19.1	24.9	57.5
h_o, in	17.28	17.38	20.28	20.39
r_y, in	1.22	1.27	1.30	1.77
J, in^4	0.506	0.81	1.14	57.5
C_w	1140	1440	2560	5970

The next step is to find a section that if its L_r is less than 20 ft, its M_r exceeds 277.5 kip·ft. The manual table indicates that a W21 × 50 has the required properties (Table 6.2). With the aid of Table 6.2, the nominal flexural design strength M_n can be computed from

$$M_n = F_{\text{cr}} S_x$$

The buckling stress is defined as follows:

$$F_{\text{cr}} = \text{buckling stress} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{6.22}$$

The effective radius of gyration r_{tx} (in) is

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$$

where h_o = distance between the flange centroids, in
J = torsional constant, in^4
C_w = warping constant
c = 1.0 for doubly symmetric I-shape

Therefore,

$$r_{tx} = \left(\frac{\sqrt{24.9 \times 2560}}{94.5}\right)^{1/2} = 1.635 \text{ in}$$

Also,

$$F_{\text{cr}} = \frac{1.0 \times \pi^2 \times 29{,}000}{[(20 \times 12)/1.635]^2}\sqrt{1 + 0.078\frac{1.14 \times 1.0}{94.5 \times 20.28}\left(\frac{20 \times 12}{1.635}\right)^2} = 18.77 \text{ ksi}$$

Thus, the nominal flexural strength is

$$M_n = 18.77 \times 94.5 = 1774 \text{ kip·in} = 147.9 \text{ kip·ft} < 277.5 \text{ kip·ft}$$

The W21 × 50 does not have adequate flexural strength. Therefore, trials to find the lowest-weight larger size must be continued. This trial-and-error process can be eliminated by using beam-selector charts in the AISC manual. These charts give the beam design moment corresponding to unbraced length for various rolled sections. Thus, for $\phi M_r < 277.5$ kip·ft and $L = 20$ ft, the charts indicate that a W21 × 62 of A992 steel satisfies the criteria (Table 6.2). As a check, the following calculation is made with the properties of the W21 × 62 given in Table 6.2.

Application of Eq. (6.22) using

$$r_{tx} = \left(\frac{\sqrt{57.5 \times 5970}}{127} \right)^{1/2} = 2.148 \text{ in}$$

Therefore,

$$F_{\text{cr}} = \frac{1.0 \times \pi^2 \times 29{,}000}{[(20 \times 12)/2.148]^2} \sqrt{1 + 0.078 \frac{1.83 \times 1.0}{127 \times 20.39} \left(\frac{20 \times 12}{1.214} \right)^2} = 29.79 \text{ ksi}$$

Thus, the nominal moment capacity is

$$M_n = 29.79 \times 127 = 3783 \text{ kip·in} = 315.2 \text{ kip·ft} > 277.5 \text{ kip·ft} \qquad \text{OK}$$

6.11 EXAMPLE—LRFD FOR FLOOR BEAM WITH OVERHANG

A floor beam of A36 steel carrying uniform loads is to span 30 ft and cantilever over a girder for 7.5 ft (Fig. 6.4). The beam is to carry a dead load due to the weight of the floor plus assumed weight of beam of 1.5 kips/ft and due to partitions, ceiling, and ductwork of 0.75 kips/ft. The live load is 1.5 kips/ft.

Negative Moment. The cantilever is assumed to carry full live and dead loads, while the back span is subjected to the minimum dead load. This loading produces maximum negative moment and maximum unbraced length of compression (bottom) flange between the support and points of zero moment. The maximum factored load on the cantilever (Fig. 6.4*a*) is

$$W_{uc} = 1.2(1.5 + 0.75) + 1.6 \times 1.5 = 5.1 \text{ kips/ft}$$

The factored load on the back span from dead load only is

$$W_{ub} = 1.2 \times 1.5 = 1.8 \text{ kips/ft}$$

Hence the maximum factored moment (at the support) is

$$-M_u = 5.1 \times \frac{7.5^2}{2} = 143.4 \text{ kip} \cdot \text{ft}$$

From the bending moment diagram in Fig. 6.4*b*, the maximum factored moment in the back span is 137.1 kip·ft, and the distance between the support of the cantilever and the point of inflection in the back span is 5.3 ft. The compression flange is unbraced over this distance. The beam will be constrained against torsion at the support. Therefore, since the 7.5-ft cantilever has a longer unbraced length and its end will be laterally braced, design of the section should be based on $L_b = 7.5$ ft.

A beam size for a first trial can be selected from a load-factor design table in the AISC "Steel Construction Manual." The table indicates that the lightest-weight section with ϕM_p exceeding 143.4 kip·ft and with potential capacity to sustain the large positive moment in the back span is a W18 × 35. Table 6.2 lists section properties needed for computation of the design strength. The table indicates that the limiting unbraced length L_r for inelastic torsional buckling is 11.5 ft > L_b. The nominal flexural strength should be computed by

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p \qquad (6.23)$$

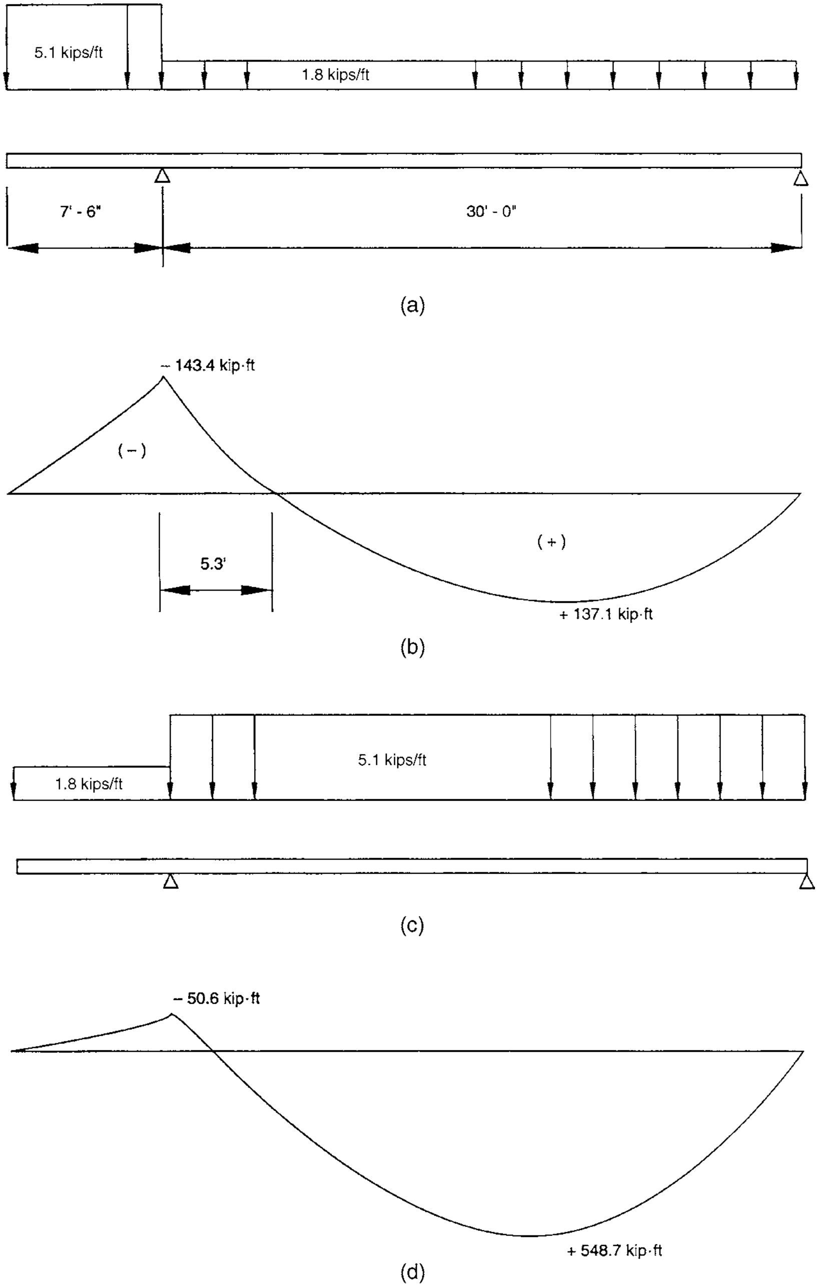


FIGURE 6.4 Loads and moments for a floor beam with an overhang. (*a*) Placement of factored loads for maximum negative moment. (*b*) Factored moments for the loading in (*a*). (*c*) Placement of factored loads for maximum positive moment. (*d*) Factored moments for the loading in (*c*).

For an unbraced cantilever, the moment gradient C_b is unity. Therefore, the design strength at the support is

$$\begin{aligned}\phi M_n &= 0.90 \times 1.0 \left[249 - \left(249 - 0.7 \times 36 \times \frac{57.6}{12} \right) \left(\frac{7.5 - 4.31}{11.5 - 4.31} \right) \right] \\ &= 191.8 \text{ kip} \cdot \text{ft} > 143.4 \text{ kip} \cdot \text{ft} \qquad \text{OK}\end{aligned}$$

Positive Moment. For maximum positive moment, the cantilever carries minimum load, whereas the back span carries full load (Fig. 6.4*c*). Dead load is the minimum for the cantilever:

$$W_{uc} = 1.2 \times 1.5 = 1.8 \text{ kips/ft}$$

Maximum factored load on the back span is

$$W_{ub} = 1.2(1.5 + 0.75) + 1.6 \times 1.5 = 5.1 \text{ kips/ft}$$

Corresponding factored moments are (Fig. 6.4*d*)

$$-M_u = 1.8 \times \frac{7.52^2}{2} = 50.6 \text{ kip} \cdot \text{ft}$$

$$+M_u = 5.1 \times \frac{30^2}{8} - \frac{50.6}{2} + \frac{1}{2 \times 5.1} \left(\frac{50.6}{30} \right)^2 = 548.7 \text{ kip} \cdot \text{ft}$$

Since the top flange of the beam is braced by the floor deck, the design strength of the section is ϕM_p. For the W18 × 35 selected for negative moment, Table 6.2 shows $\phi M_p = 249 < 548.7$ kip·ft. Hence this section is not adequate for the maximum positive moment. The least-weight beam with $\phi M_p > 548.7$ kip·ft is a W24 × 62 ($\phi M_p = 578$ kip·ft).

6.12 COMPOSITE BEAMS

Composite steel beam construction is common in multistory commercial buildings. Utilizing the concrete deck as the top (compression) flange of a steel beam to resist maximum positive moments produces an economical design. In general, composite floor-beam construction consists of the following:

- Concrete over a metal deck, the two acting as one composite unit to resist the total loads. The concrete is normally reinforced with welded wire mesh to control shrinkage cracks.
- A metal deck, usually 1½, 2, or 3 in deep, spanning between steel beams to carry the weight of the concrete until it hardens, plus additional construction loads.
- Steel beams supporting the metal deck, concrete, construction, and total loads. When unshored construction is specified, the steel beams are designed as noncomposite to carry the weight of the concrete until it hardens, plus additional construction loads. The steel section must be adequate to resist the total loads acting as a composite system integral with the floor slab.
- Shear connectors, studs, or other types of mechanical shear elements welded to the top flange of the steel beam to ensure composite action and to resist the horizontal shear forces between the steel beam and the concrete deck.

The effective width of the concrete deck as a flange of the composite beam is defined in Chap. 5. The compression force C (kips) in the concrete is the smallest of the values given by Eqs. (6.24)–(6.26). Equation (6.24) denotes the design strength of the concrete:

$$C_c = 0.85 f_c' A_c \tag{6.24}$$

where $f_c' =$ concrete compressive strength, ksi
$A_c =$ area of the concrete within the effective slab width, in^2 (if the metal deck ribs are perpendicular to the beam, the area consists only of the concrete above the metal deck. If, however, the ribs are parallel to the beam, all the concrete, including the concrete in the ribs, comprises the area.)

Equation (6.25) gives the yield strength of the steel beam:

$$C_t = A_s F_y \tag{6.25}$$

where $A_s =$ area of the steel section (not applicable to hybrid sections), in^2
$F_y =$ yield stress of the steel, ksi

Equation (6.26) expresses the strength of the shear connectors:

$$C_s = \sum Q_n \tag{6.26}$$

where $\sum Q_n$ is the sum of the nominal strength of the shear connectors between the point of maximum positive moment and zero moment on either side.

For full composite design. three locations of the plastic neutral axis are possible. The location depends on the relationship of C_c to the yield strength of the web, $P_{yw} = A_w F_y$, and C_t. The three cases are as follows (Fig. 6.5):

Case 1. The plastic neutral axis is located in the web of the steel section. This case occurs when the concrete compressive force is less than the web force, $C_c \leq P_{yw}$.

Case 2. The plastic neutral axis is located within the thickness of the top flange of the steel section. This case occurs when $P_{yw} < C_c < C_t$.

Case 3. The plastic neutral axis is located in the concrete slab. This case occurs when $C_c \geq C_t$. (When the plastic axis occurs in the concrete slab, the tension in the concrete below the plastic neutral axis is neglected.)

See Art. 5.8 for restrictions on shear connector spacing and location.

The total horizontal shear force C at the interface between the steel beam and the concrete slab is assumed to be transmitted by shear connectors. Hence the number of shear connectors required for composite action is

$$N_s = \frac{C}{Q_n} \tag{6.27}$$

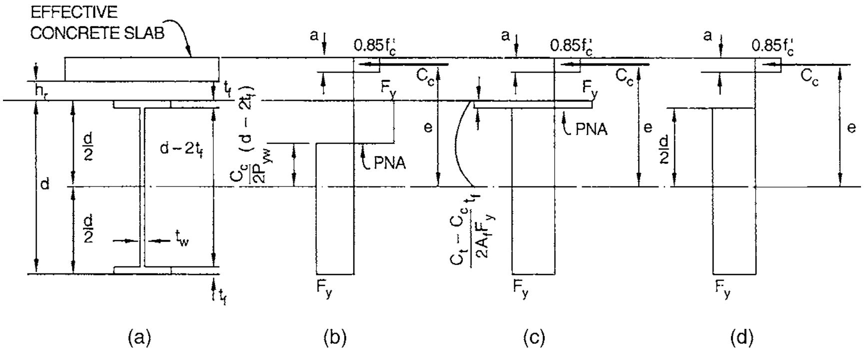


FIGURE 6.5 Stress distributions assumed for plastic design of a composite beam. (*a*) Cross section of composite beam. (*b*) Plastic neutral axis (PNA) in the web. (*c*) PNA in the steel flange. (*d*) PNA in the slab.

where Q_n = nominal strength of one shear connector, kips
N_s = number of shear studs between maximum positive moment and zero moment on each side of the maximum positive moment

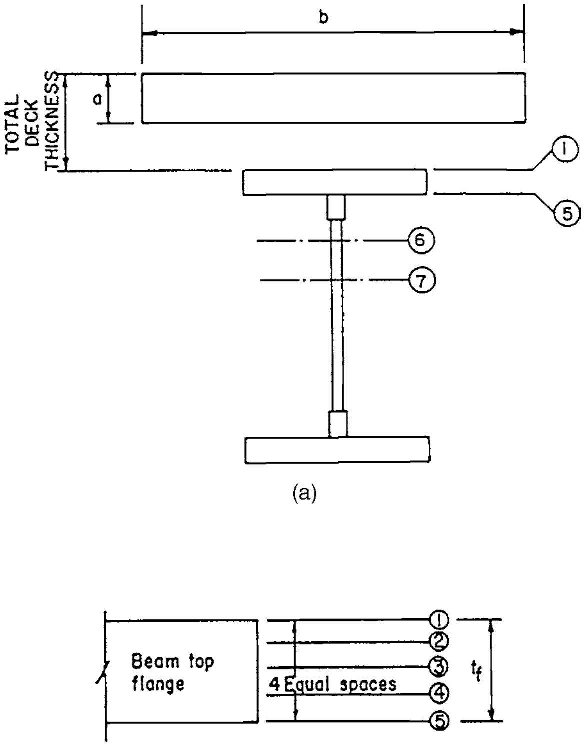
The nominal strength of a shear stud connector embedded in a solid concrete slab may be computed from

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u \tag{6.28}$$

where A_{sc} = cross-sectional area of stud, in^2
f'_c = specified compressive strength of concrete, ksi
E_c = modulus of elasticity of the concrete, ksi
$= w^{1.5}\sqrt{f'_c}$
w = unit weight of the concrete, lb/ft^3
F_u = specified minimum tensile strength of a stud, ksi

R_g and R_p are strength reduction factors, Art. 5.8.6.

For a beam with nonsymmetrical loading, the distances between the maximum positive moment and point of zero moment (inflection point) on either side of the point of maximum moment will not be equal. Or if one end of a beam has negative moment, then the inflection point will not be at that end.



(a)

(b)

FIGURE 6.6 Seven locations of the plastic neutral axis used for determining the strength of a composite beam. (*a*) For cases 6 and 7, the PNA lies in the web. (*b*) For cases 1 through 5, the PNA lies in the steel flange.

TABLE 6.3 Q_n for Partial Composite Design (kips)

Location of PNA	Q_n and concrete compression
(1)	A_xF_y
(2)–(5)	$A_sF_y–2\Delta A_fF_y^*$
(6)	$0.5[C(5) + C(7)]^\dagger$
(7)	$0.25A_sF_y$

*ΔA_f = area of the segment of the steel flange above the plastic neutral axis (PNA).

†$C(n)$ = compressive force at location (n).

When a concentrated load occurs on a beam, the number of shear connectors between the concentrated load and the inflection point should be adequate to develop the maximum moment at the concentrated load.

When the moment capacity of a fully composite beam is much greater than the applied moment, a partially composite beam may be utilized. It requires fewer shear connectors and thus has a lower construction cost. A partially composite design also may be used advantageously when the number of shear connectors required for a fully composite section cannot be provided because of limited flange width and length.

Figure 6.6 shows seven possible locations of the plastic neutral axis (PNA) in a steel section. The horizontal shear between the steel section and the concrete slab, which is equal to the compressive force in the concrete C, can be determined as illustrated in Table 6.3.

6.13 LRFD FOR COMPOSITE BEAM WITH UNIFORM LOADS

The typical floor construction of a multistory building is to have composite framing. The floor consists of 3¼-in-thick lightweight concrete over a 2-in-deep steel deck. The concrete weighs 115 lb/ft^3 and has a compressive strength of 3.0 ksi. An additional 30% of the dead load is assumed for equipment load during construction. The deck is to be supported on steel beams with stud shear connectors on the top flange for composite action (Art. 6.12). Unshored construction is assumed. Therefore, the beams must be capable of carrying their own weight, the weight of the concrete before it hardens, deck weight, and construction loads. Shear connectors will be ¾ in in diameter and 3½ in long. The floor system should be investigated for vibration, assuming a damping ratio of 5%.

A typical beam supporting the deck is 30 ft long. The distance to adjacent beams is 10 ft. Ribs of the deck are perpendicular to the beam. Uniform dead loads on the beam are, for construction, 0.50 kip/ft, plus 30% for equipment loads, and for superimposed load, 0.25 kip/ft. Uniform live load is 0.50 kip/ft.

Beam Selection. Initially, a beam of A992 steel that can support the construction loads is selected. It is assumed to weigh 22 lb/ft. Thus, the beam is to be designed for a service dead load of 0.5 × 1.3 + 0.022 = 0.672 kip/ft.

$$\text{Factored load} = 0.672 \times 1.4 = 0.941 \text{ kip/ft}$$

$$\text{Factored moment} = M_u = 0.941 \times \frac{30^2}{8} = 105.8 \text{ kip}\cdot\text{ft}$$

The plastic section modulus required therefore is

$$Z = \frac{M_u}{\phi F_y} = \frac{105.8 \times 12}{0.9 \times 50} = 28.2 \text{ in}^3$$

Use a W14 × 22 (Z = 33.2 in^3 and moment of inertia I = 199 in^4).

The beam should be cambered to offset the deflection due to a dead load of 0.50 + 0.022 = 0.522 kip/ft.

$$\text{Camber} = \frac{5 \times 0.522 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 199} = 1.6 \text{ in}$$

Camber should be specified on the drawings as 1.5 in.

Strength of Fully Composite Section. Next, the composite steel section is designed to support the total loads. The live load may be reduced in accordance with area supported (Art. 6.9). The reduction factor is $R = 0.0008(300 - 150) = 0.12$. Hence the reduced live load is $0.5(1 - 0.12) = 0.44$ kip/ft. The factored load is the larger of the following:

$$1.2(0.50 + 0.25 + 0.022) + 1.6 \times 0.44 = 1.63 \text{ kips/ft}$$

$$1.4(0.5 + 0.25 + 0.022) = 1.081 \text{ kips/ft}$$

Hence the factored moment is

$$M_u = 1.63 \times \frac{30^2}{8} = 183.4 \text{ kip} \cdot \text{ft}$$

The concrete flange width is the smaller of $b = 10 \times 12 = 120$ in or $b = 2(30 \times {}^{12}\!/_{8}) = 90$ in (governs).

The compressive force in the concrete C is the smaller of the values computed from Eqs. (6.24) and (6.25).

$$C_c = 0.85 f_c' A_c = 0.85 \times 3 \times 90 \times 3.25 = 745.9 \text{ kips}$$

$$C_t = A_s F_y = 6.49 \times 50 = 324.5 \text{ kips} \qquad \text{(governs)}$$

The depth of the concrete compressive-stress block (Fig. 6.5) is

$$a = \frac{C}{0.85 f_c' b} = \frac{324.5}{0.85 \times 3.0 \times 90} = 1.414 \text{ in}$$

Since $C_c > C_t$, the plastic neutral axis will lie in the concrete slab (case 3, Art. 6.12). The distance between the compression and tension forces on the W14 × 22 (Fig. 6.5*d*) is

$$e = 0.5d + 5.25 - 0.5a$$

$$= 0.5 \times 13.7 + 5.25 - 0.5 \times 1.414 = 11.393 \text{ in}$$

The design strength of the W14 × 22 is

$$\phi M_n = 0.9 C_t e = 0.9 \times 324.5 \times \frac{11.393}{12} = 277.3 \text{ kip} \cdot \text{ft} > 183.9 \text{ kip} \cdot \text{ft} \qquad \text{OK}$$

Partial Composite Design. Since the capacity of the full composite section is more than required, a partial composite section may be satisfactory. Seven values of the composite section (Fig. 6.6) are calculated as follows, with the flange area $A_f = 5 \times 0.335 = 1.675$ in^2.

1. Full composite:

$$\sum Q_n = A_s F_y = 324.5 \text{ kips}$$
$$\varphi M_n = 277.2 \text{ kip} \cdot \text{ft}$$

2. Plastic neutral axis $\Delta A_f = A_f/4 = 0.4188$ in below the top of the top flange. From Table 6.3, $\Sigma Q_n = A_s F_y - 2\Delta A_f F_y$.

$$\sum Q_n = 324.5 - 2 \times 0.4188 \times 50 = 282.6 \text{ kips}$$
$$a = \frac{282.6}{0.85 \times 3.0 \times 90} = 1.2315 \text{ in}$$
$$e = \frac{13.7}{2} + 5.25 - \frac{1.2315}{2} = 11.484 \text{ in}$$
$$M_n = 282.6 \times 11.484 + 0.5(324.5 - 282.6)\left(13.7 - 0.335\frac{324.5 - 282.6}{2 \times 1.675 \times 50}\right)$$
$$= 3531 \text{ kip} \cdot \text{in}$$
$$\varphi M_n = 0.90 \times \frac{3531}{12} = 264.8 \text{ kip} \cdot \text{ft}$$

3. PNA $\Delta A_f = A_f/2 = 0.8375$ in below the top of the top flange.

$$\sum Q_n = 324.5 - 2 \times 0.8375 \times 50 = 240.8 \text{ kips}$$
$$a = \frac{240.8}{0.85 \times 3.0 \times 90} = 1.0490 \text{ in}$$
$$e = \frac{13.7}{2} + 5.25 - \frac{1.0490}{2} = 11.575 \text{ in}$$
$$M_n = 240.8 \times 11.575 + 0.5(324.5 - 240.68)\left(13.7 - 0.335\frac{324.5 - 240.8}{2 \times 1.675 \times 50}\right)$$
$$= 3353 \text{ kip} \cdot \text{in}$$
$$\varphi M_n = 0.90 \times \frac{3353}{12} = 251.5 \text{ kip} \cdot \text{ft}$$

4. PNA $\Delta A_f = 3A_f/4 = 1.2563$ in below the top of the top flange.

$$\sum Q_n = 324.5 - 2 \times 1.2563 \times 50 = 198.9 \text{ kips}$$
$$a = \frac{198.9}{0.85 \times 3.0 \times 90} = 0.8665 \text{ in}$$
$$e = \frac{13.7}{2} + 5.25 - 0.8665 = 11.667 \text{ in}$$
$$M_n = 198.9 \times 11.667 + 0.5(324.5 - 198.9)\left(13.7 - 0.335\frac{324.5 - 198.9}{2 \times 1.675 \times 50}\right)$$
$$= 3165 \text{ kip} \cdot \text{in}$$
$$\varphi M_n = 0.90 \times \frac{3165}{12} = 237.4 \text{ kip} \cdot \text{ft}$$

5. PNA at the bottom of the top flange ($\Delta A_f = A_f$).

$$\begin{aligned}
\sum Q_n &= 324.5 - 2 \times 1.675 \times 50 = 157.0 \text{ kips} \\
a &= \frac{157.0}{0.85 \times 3.0 \times 90} = 0.68410 \text{ in} \\
e &= \frac{13.7}{2} + 5.25 - 0.6841 = 11.758 \text{ in} \\
M_n &= 157.0 \times 11.758 + 0.5(324.5 - 157.0)\left(13.7 - 0.335\frac{324.5 - 157.0}{2 \times 1.675 \times 50}\right) \\
&= 2965 \text{ kip} \cdot \text{in} \\
\varphi M_n &= 0.90 \times 2965 = 222.4 \text{ kip} \cdot \text{ft}
\end{aligned}$$

6. Plastic neutral axis within the web: $\Sigma Q_n = 104$ kips is the average of items 5 and 7. (See Table 6.3.):

$$\begin{aligned}
\sum Q_n &= \frac{157.0 + 81.1}{2} = 119.1 \text{ kips} \\
a &= \frac{119.1}{0.85 \times 3.0 \times 90} = 0.5187 \text{ in} \\
e &= \frac{13.7}{2} + 5.25 - 0.5187 = 11.841 \text{ in} \\
M_n &= 119.1 \times 11.5187 + 0.5(324.5 - 119.1)\left(13.7 - 0.335\frac{324.5 - 119.1}{2 \times 1.675 \times 50}\right) \\
&= 2775 \text{ kip} \cdot \text{in} \\
\varphi M_n &= 0.90 \times 2775 = 208.1 \text{ kip} \cdot \text{ft}
\end{aligned}$$

7.

$$\begin{aligned}
\sum Q_n &= C_c = 0.25 \times 324.5 = 81.1 \text{ kips} \\
a &= \frac{81.1}{0.85 \times 3.0 \times 90} = 0.3535 \text{ in} \\
\frac{C_c}{2P_{\text{yw}}}(d - 2t_f) &= \frac{81.1}{2 \times 0.23 \times 50} \\
&= 3.53 \text{ in} = \text{distance from PNA to the center of gravity of the beam} \\
y &= \frac{0.335 \times 5.0 \times (13.7/2 + 3.53 - 0.335/2) + 0.23[(13.7/2 + 3.53 - 0.335)/2]^2}{0.335 \times 5.0 + 0.23[(13.7/2) + 3.53 - 0.335]} \\
&= 7.20 \text{ in} = \text{distance from centroid tensile portion of beam to the} \\
&\quad\ \text{center of gravity of the beam} \\
e &= \frac{13.7}{2} - 3.53 + 7.20 + 5.25 - \frac{0.3535}{2} \\
e &= 15.60 \text{ in} = \text{distance from PNA to } C_c \\
y' &= \frac{13.7}{2} - 3.53 + 7.20 + 5.25 - \frac{0.335}{2} \\
y' &= 10.36 \text{ in} = \text{distance from PNA to } C_{\text{flange}}
\end{aligned}$$

$$y'' = \frac{(13.7/2) - 3.53 + 0.335}{2} + 7.20$$

$$y'' = 8.70 \text{ in} = \text{distance from PNA to } C_{\text{web}}$$

$$M_n = C_c \times e + C_{\text{flange}} \times y' + C_{\text{web}} \times \ y''$$

$$= 81.1 \times 15.60 + 50\{0.335 \times 5.0 \times 10.36 + 0.23[(13.7/2) - 3.53 - 0.355](8.70)\}$$

$$M_n = 2{,}431 \text{ kip} \cdot \text{ft}$$

$$\phi M_n = \frac{0.90 \times 2331}{2}$$

$$\phi M_n = 182.3 \text{ kip} \cdot \text{ft}$$

From the partial composite values 2 to 7, value 6 is greater than $M_u = 183.4$ kip·ft.

The AISC "Manual of Steel Construction" includes design tables for composite beams that greatly simplify the calculations. For example, the table for a W14 × 22, A992, composite beam gives ϕM_n for the seven positions of the PNA and for several values of the distance Y_2 (in) from the concrete compressive force C to the top of the steel beam. For the preceding example,

$$Y_2 = Y_{\text{con}} - \frac{a}{2} \tag{6.29}$$

where Y_{con} = total thickness of floor slab, in
a = depth of the concrete compressive-stress block, in

From the table for case 6, $\sum Q_n = 119$ kips.

$$a = \frac{119}{0.85 \times 3.0 \times 90} = 0.519 \text{ in}$$

Substitution of a and $Y_{\text{con}} = 5.25$ in in Eq. (6.29) gives

$$Y_2 = 5.25 - \frac{0.519}{2} = 4.99 \text{ in}$$

The manual table gives the corresponding moment capacity for case 6 and $Y_2 = 4.99$ in as

$$\phi M_n = 195 \text{ kip} \cdot \text{ft} > 183.9 \text{ kip} \cdot \text{ft} \qquad \text{OK}$$

The number of shear studs is based on $C = 119.1$ kips. The nominal strength Q_n of one stud is given by Eq. (6.28). For a ¾-in stud, with shearing area $A_{sc} = 0.442$ in^2 and tensile strength $F_u = 60$ ksi, the limiting strength without reduction factors is $A_{sc}\ F_u = 0.442 \times 60 = 26.5$ kips. With concrete unit weight $w = 115$ lb/ft^3 and compressive strength $f_c' = 3.0$ ksi, and modulus of elasticity $E_c = 2136$ ksi, the nominal strength given by Eq. (6.28) is

$$Q_n = 0.5 \times 0.442\sqrt{3.0 \times 2136} = 17.7 \text{ kips} < R_g R_p \ 26.5 \text{ kips}$$

$R_g = 1.0$ for deck perpendicular to beam, and $R_p = 0.75$ when $e_{\text{mid-ht}} \geq 2$ in (see Art. 5.8.6). Thus, $R_g R_p A_{sc} F_u = 1.00 \times 0.75 \times 26.5 = 19.9$ kips and $Q_n = 17.7$ kips.

The number of shear studs required is 2 × 119.1/17.7 = 13.5. Use 14. The total number of metal deck ribs supported on the steel beam is 30. Therefore, only one row of shear studs is required.

Deflection Calculations. Deflections are calculated based on the partial composite properties of the beam. First, the properties of the transformed full composite section (Fig. 6.7) are determined.

The modular ratio E_s/E_n is $n = 29{,}000/2136 = 13.6$. This is used to determine the transformed concrete area $A_1 = 3.25 \times 90/13.6 = 21.52$ in^2. The area of the W14 × 22 is 6.49 in^2, and its moment of

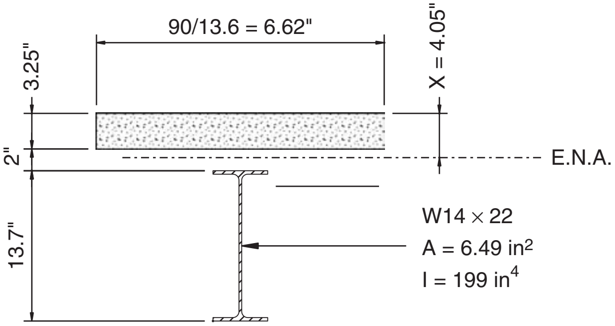


FIGURE 6.7 Transformed section of a composite beam.

inertia $I_s = 199$ in^4. The location of the elastic neutral axis is determined by taking moments of the transformed concrete area and the steel area about the top of the concrete slab:

$$X = \frac{21.52 \times (3.25/2) + 6.49(0.5 \times 13.7 + 5.25)}{21.52 + 6.49} = 4.05 \text{ in}$$

The elastic transformed moment of inertia for full composite action is

$$\begin{aligned} I_{\text{tr}} &= \frac{90 \times 3.25^3}{13.6 \times 12} + 21.52\left(4.05 - \frac{3.25}{2}\right)^2 + 6.49\left(\frac{13.7}{2} + 5.25 - 4.05\right)^2 + 199 \\ &= 765.0 \text{ in}^4 \end{aligned}$$

Since partial composite construction is used, the effective moment of inertia is determined from

$$I_{\text{eff}} = I_s + (I_{\text{tr}} - I_s)\sqrt{\frac{\Sigma Q_n}{C_f}} \tag{6.30}$$

where C_f = concrete compression force based on full composite action

$$I_{\text{eff}} = 199 + (765 - 199)\sqrt{\frac{119.1}{324.5}} = 541.8 \text{ in}^4$$

I_{eff} is used to calculate the immediate deflection under service loads (without long-term effects).

For long-term effect on deflections due to creep of the concrete, the moment of inertia is reduced to correspond to a 50% reduction in E_c. Accordingly, the transformed moment of inertia with full composite action and 50% reduction in E_c is $I_{\text{tr}} = 652.6$ in^4 and is based on a modular ratio $2n = 27.2$. The corresponding transformed concrete area is $A_1 = 10.76$ in^2.

The reduced effective moment of inertia for partial composite construction with long-term effect is determined from Eq. (6.30):

$$I_{\text{eff}} = 199 + (652.6 - 199)\sqrt{\frac{119.1}{324.5}} = 473.8 \text{ in}^4$$

Since unshored construction is specified, the deflection under the weight of concrete when placed and the steel weight is compensated for by the camber specified. Long-term effect due to these weights need not be considered because the concrete is not stressed by them.

Deflection due to long-term superimposed dead load is

$$D_1 = \frac{5 \times 0.25 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 473.8} = 0.331 \text{ in}$$

Deflection due to short-term (reduced) live load is

$$D_2 = \frac{5 \times 0.44 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 541.8} = 0.510 \text{ in}$$

Total deflection is

$$D = D_1 + D_2 = 0.331 + 0.510 = 0.842 \text{ in} = \frac{L}{428} \qquad \text{OK}$$

Vibration Investigation. The vibration study of composite beams is based on the following report: T. M. Murray et al., "Floor Vibrations due to Human Activity," *AISC Steel Design Guide*, No. 11, 1997. Utilization of lightweight concrete and longer girder spans has resulted in lower natural frequencies in similar structural floor systems. A more detailed analysis for a floor-vibration design criterion is recommended due to a walking excitation or impact "heel drop" force to determine the peak acceleration a_p. The peak acceleration, presented as the percentage of the acceleration of gravity, $(a_p/g) \times 100\%$, will be the governing acceleration limit to satisfy, as calculated in the following formula:

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \tag{6.31}$$

where P_o = a constant force representing the excitation
f_n = fundamental natural frequency of a beam or joist, girder, or combination, Hz
β = modal damping ratio
W = effective weight supported by the beam or joist, girder, or combination

The suggested limiting values for the constant force, damping ratio, and acceleration limit are shown in Table 6.4. A damping ratio of 0.02 can be used for floors with few nonstructural components (ceilings, ducts, partitions, etc.), as can occur in open work areas and churches. For floors with nonstructural components and furnishings but with only small demountable partitions, typical of many modular office areas, a damping ratio of 0.03 is recommended. For floors with full-story-height partitions, a damping ratio of 0.05 is suggested.

The fundamental frequency of a uniformly loaded simply supported joist, beam, or girder is

$$f_n = 0.18\sqrt{\frac{g}{\Delta}} \tag{6.32}$$

where g = acceleration due to gravity, 386 in/s^2
Δ = midspan deflection relative to the supports due to the weight supported

TABLE 6.4 Recommended Values of Parameters

Type of occupancy	Constant force P_o (lb)	Damping ratio β	Acceleration limit $(a_o/g) \times 100\%$
Offices, residences, churches	65	0.02–0.05	0.5%
Shopping malls	65	0.02	1.5%
Footbridges—indoor	92	0.01	1.5%
Footbridges—outdoor	92	0.01	5.0%

The effective supported weight of joist, beam, or girder is

$$W = wBL \tag{6.33}$$

where w = supported weight per unit area
B = effective width
L = member span

For the vibration analysis, additional parameters pertaining to the entire floor system must be established first for the best estimation of peak accelerations due to a heel-drop forcing function. The entire floor dimensions are 180 ft × 90 ft. The girders on all four sides are W24 × 55 Grade 50. The interior girders are also W24 × 55 Grade 50 sections spaced at 30 ft in the longitudinal and latitudinal directions.

The loading must be adjusted to estimate the least loading scenario which is most critical to extreme vibrations. A dead load of 46.7 lb/ft^2 is estimated by considering 3.5 in of concrete, 1-in effective concrete thickness of deck, 2 lb/ft^2 metal deck, and 4 lb/ft^2 for ceiling, mechanical, and minimal partition loads. The live load is reduced to a magnitude of 11 lb/ft^2.

For transformed composite moment of inertia calculations, there are two modifications that differ from traditional composite calculations. First, a dynamic modulus of elasticity is considered because the stiffness of concrete is greater under a dynamic load as compared to a static load. The dynamic modular ratio is defined as

$$n = \frac{E_s}{1.35 \times E_s} \tag{6.34}$$

For this example,

$$n = \frac{29{,}000}{1.35 \times 2136} = 10.06$$

The effective width of the slab is the minimum of the member spacing or 0.4 × (member span) for an interior member and 0.20 × (member span) for an exterior member. The concrete flange width is the smaller of $b = 10 \times 12 = 120$ in or $b = 0.4 \times 30 \times 12 = 144$ in.

Interior Beam Vibration Investigation. The beam vibration calculations will be based on full composite action, and the shear deformations will not be included.

The concrete flange width is the smaller of $b = 120$ in or $b = 144$ in. The transformed moment of inertia $I_t = 843.2$ in^4 for the W14 × 22 beam, not considering the 1-in effective concrete in the deck.

The total load is

$$(46.7 + 11.0) \times 10 + 22 = 0.6 \text{ kip/ft}$$

and the beam deflection is calculated as

$$\Delta_j = \frac{5 \times 0.6 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 843.2} = 0.447 \text{ in}$$

$$f_n = 0.18\sqrt{\frac{386}{0.447}} = 5.29 \text{ Hz}$$

The effective width of a joist or beam is

$$B_j = C_j \left(\frac{D_s}{D_j}\right)^{1/4} L_j < \frac{2}{3} \times \text{ floor width} \tag{6.35}$$

where C_j = 2.0 for joists and beams in most areas
= 1.0 for joists and beams parallel to an interior edge
D_s = transformed slab moment of inertia per unit width
= $d_e^3/(12n)$, in^4/ft
d_s = effective depth of the concrete slab, usually taken as the depth of the concrete above the form deck plus one-half the depth of the form deck
n = dynamic modular ratio, see Eq. (6.34)
D_j = beam or joist transformed moment of inertia per unit width
= I_t/S, in^4/ft
L_j = joist or beam span above the form deck plus one-half the depth of the form deck,

$$D_s = \frac{12 \times 4.25^3}{12 \times 10.06} = 7.63 \text{ in}^4/\text{ft}$$

$$D_j = \frac{843.2 \times 12}{120} = 84.32 \text{ in}^4/\text{ft}$$

For this beam, $C_j = 2.0$.

$$B_j = 2 \times \left(\frac{843.2 \times 12}{120}\right)^{1/4} \times 30 = 32.91 \text{ ft} < \frac{2}{3} \times \text{ floor width}$$

Since 0.7 × girder spacing = 21 ft, less than the beam span, an increase of 50% of the effective weight must be applied due to continuous action.

$$W_j = 1.5 \times \frac{0.559 \times 12}{120} \times 32.91 \times 30 = 88.76 \text{ kips}$$

Interior Girder Vibration Investigation. The girder vibration calculations will be based on full composite action and the shear deformations will not be included. The concrete flange width is the smaller of $b = 10 \times 12 = 120$ in or $b = 0.4 \times 30 \times 12 = 144$ in. The transformed moment of inertia $I_t = 4315$ in^4 for the W24 × 55 beam considering the 1-in effective concrete in the deck.

The total load is

$$30 \times \frac{0.5593 \times 12}{120} + 0.055 = 1.853 \text{ kips/ft}$$

and the girder deflection is calculated as

$$\Delta_g = \frac{5 \times 0.5993 \times 30^4 \times 12^3}{384 \times 29{,}000 \times 4315} = 0.269 \text{ in}$$

$$f_{\text{comb}} = 0.18\sqrt{\frac{386}{0.246}} = 6.808 \text{ Hz}$$

The effective width of a girder is

$$B_g = C_g \left(\frac{D_s}{D_g}\right)^{1/4} L_g < \frac{2}{3} \times \text{floor length} \tag{6.36}$$

where C_g = 1.6 for girders supporting joists connected to the girder flange (e.g., joist seats)
= 1.8 for girders supporting beams connected to the girder web
D_g = girder transformed slab moment of inertia per unit width
= $d_e^3/12n$, in^4/ft
D_j = beam or joist transformed moment of inertia per unit width
= I_g/S, in^4/ft
L_g = girder span

$$D_j = \frac{4315 \times 12}{30} = 143.85 \text{ in}^4/\text{ft}$$

For this girder, $C_g = 1.8$.

$$B_g = 1.8 \times \left(\frac{84.32}{4315}\right)^{1/4} \times 30 = 47.245 \text{ ft} < \frac{2}{3} \times \text{floor length}$$

The girders are not affected by continuous action.

$$W_g = \frac{1.853}{30} \times 47.25 \times 30 = 87.56 \text{ kips}$$

Combined Vibration Investigation. The girder vibration calculations will be based on full composite action and the shear deformations will not be included.

For the combined mode, the equivalent weight is

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g \tag{6.37}$$

where Δ_j, Δ_g = beam, joist, or girder maximum deflection

If the girder span L_g is greater than the joist or beam effective width B_j, then the following adjustment to the girder deflection is applicable but $L_g/B_j \geq 0.5$:

$$\Delta'_g = \frac{L_g}{B_j}(\Delta_g) \tag{6.38}$$

Since $L_g/B_j = 30/32.91 = 0.912 \leq 1$, then

$$\Delta'_g = \frac{30}{32.91}(0.269) = 0.246 \text{ in}$$

$$W = \frac{0.447}{0.447 + 0.246} 88.76 + \frac{0.246}{0.447 + 0.246} 87.56 = 88.327 \text{ kips}$$

The combined natural frequency is calculated as follows:

$$f_{\text{comb}} = 0.18\sqrt{\frac{386}{0.447 + 0.246}} = 4.249 \text{ Hz}$$

For a typical modular office with small demountable partitions, $\beta = 0.03$ and $P_o = 65$ lb.

$$\frac{a_p}{g} = \frac{65 \times \exp(-0.35 \times 4.249)}{0.03 \times 88.327 \times 1000} \times 100\% = 0.55\% > 0.5\% \qquad \text{NG}$$

The W14 × 22 beam did not satisfy the vibration criteria. It is suggested that the W14 × 22 be replaced with a W16 × 26 beam that results in an acceleration limit of 0.49%, even though the flexural strength of this section is adequate.

6.14 EXAMPLE—LRFD FOR COMPOSITE BEAM WITH CONCENTRATED LOADS AND END MOMENTS

The general information for design of a floor system is the same as that given in Art. 6.14. In this example, a girder of A992 steel is to support the floorbeams. (Deck ribs are parallel to the girder.) The girder loads and span are shown in Fig. 6.8 and Table 6.5. The spacing to the left adjacent girder is 30 ft and to the right girder is 20 ft.

Dead-Load Moment for Unshored Beam. The steel girder is to support construction dead loads, nonshored, with 30% additional dead load assumed applied during construction. The girder is assumed to weigh 44 lb/ft. The negative end moments are neglected for this phase of the design since the concrete may be placed over the entire span between the supports but not over the cantilever.

The factored dead loads are

$$P_u = 14.85 \times 1.30 \times 1.4 = 27.03 \text{ kips}$$

$$W_{Lu} = 0.5 \times 1.30 \times 1.4 = 0.910 \text{ kips/ft}$$

$$W_{Ru} = 0.2 \times 1.30 \times 1.4 = 0.364 \text{ kips/ft}$$

$$W_{Gu} = 0.044 \times 1.4 = 0.062 \text{ kips/ft}$$

For the girder acting as a simple beam with a 30-ft span, the factored dead-load moment is $M_u = 328.0$ ft·kips, and the plastic modulus required is $Z = M_u/0.9F_y = 328 \times 12/(0.9 \times 50) = 87.5$ in^3. The least-weight section with larger modulus is a W21 × 44, with $Z = 95.4$ in^3.

Camber. This is computed for maximum deflection attributable to full-construction dead loads. For this computation, the dead-load portion of the end moments is included. The loads are listed under construction dead load in Table 6.5.

The corresponding deflection is 1.09 in. A camber of 1 in may be specified.

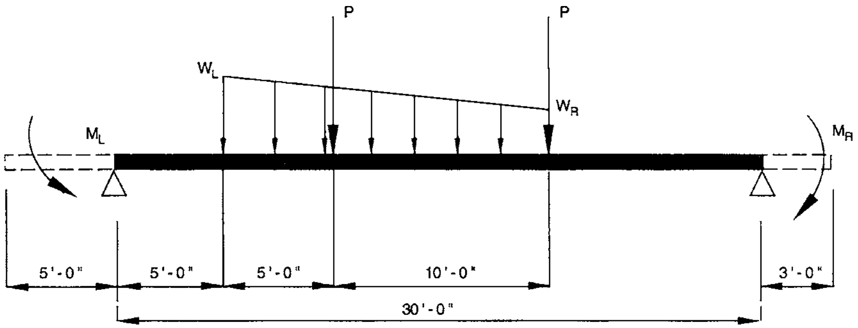


FIGURE 6.8 Composite beam with overhang carries two concentrated loads and a uniformly decreasing load over part of the span. Cantilever carries uniform loads.

TABLE 6.5 Concentrated and Partial Loads on Composite Beam

Type of load	Construction dead load	Superimposed dead load	Live load
Concentrated load P, kips	14.85	7.5	15.0
Negative moment M_L, kip·ft	22.5	7.5	20.0
Negative moment M_R, kip·ft	7.5	2.5	7.0
Partial-load start w_L, kips/ft	0.50	0.75	0.50
Partial-load end w_R, kips/ft	0.20	0.30	0.20

Design for Maximum End Moment. This takes into account the unbraced length of the girder. For the maximum possible unbraced length of the bottom (compression) flange of the steel section, only the dead loads act between supports. The factored dead loads are

$$\begin{aligned}
P_u &= 1.2 \times 14.85 = 17.82 \text{ kips}\\
W_{Lu} &= 1.2 \times 0.5 = 0.60 \text{ kips/ft}\\
W_{Ru} &= 1.2 \times 0.2 = 0.24 \text{ kips/ft}\\
W_{Gu} &= 1.2 \times 0.044 = 0.053 \text{ kips/ft}\\
M_{Lu} &= 1.2(22.5 + 7.5) + 1.6 \times 20 = 68.0 \text{ kip} \cdot \text{ft}\\
M_{Ru} &= 1.2(7.5 + 2.50) + 1.6 \times 7 = 23.2 \text{ kip} \cdot \text{ft}
\end{aligned}$$

The unbraced length of the bottom flange is 2.9 ft. The cantilever length is 5 ft (governs). The design strength ϕM_n for a wide-flange section of A992 steel may be obtained from curves in the AISC "Manual of Steel Construction." A curve indicates that the W21 × 44 with an unbraced length of 5 ft has a design strength $\phi M_n = 356$ kip·ft.

Design for Positive Moment. For this computation, the load factor used for the negative dead-load moments is 1.2, with only dead load on the cantilevers. The load factor for live loads is 1.6.

The factored loads, with live loads reduced 40% for the size of areas supported, are

$$\begin{aligned}
P_u &= 1.2(14.85 + 7.5) + 1.6 \times 9.0 = 41.22 \text{ kips}\\
W_{Lu} &= 1.2(0.5 + 0.75) + 1.6 \times 0.30 = 1.98 \text{ kips/ft}\\
W_{Ru} &= 1.2(0.20 + 0.30) + 1.6 \times 0.12 = 0.792 \text{ kips/ft}\\
W_{Gu} &= 1.2 \times 0.044 = 0.053 \text{ kips/ft}\\
M_{Lu} &= 1.2 \times 22.5 = 27.0 \text{ kip} \cdot \text{ft}\\
M_{Ru} &= 1.2 \times 7.5 = 9.0 \text{ kip} \cdot \text{ft}
\end{aligned}$$

For these loads, the factored maximum positive moment is $M_u = 509.6$ kip·ft.

For determination of the capacity of the composite beam, the effective concrete flange width is the smaller of

$$b = \frac{12(30 + 20)}{2} = 300 \text{ in}$$

$$b = 12 \times \frac{30}{4} = 90 \text{ in} \qquad \text{(governs)}$$

Design tables for composite beams in the AISC manual greatly simplify calculation of design strength. For example, the table for the W21 × 44 Grade 50 beam gives ϕM_n for seven positions of the plastic neutral axis and for several values of the distance Y_2 from the top of the steel beam to the centroid of the effective concrete flange force (ΣQ_n) (see Art. 5.8). Try $\Sigma Q_n = 260$ kips. The corresponding depth of the concrete compression block is

$$a = \frac{260}{0.85 \times 3.0 \times 90} = 1.133 \text{ in}$$

From Eq. (6.29), $Y_2 = 5.25 - 1.133/2 = 4.68$ in. The manual table gives the corresponding design strength for Case 6 and $Y_2 = 4.68$ in, by interpolation, as

$$\phi M_n = 546 \text{ kip·ft} > (M_u = 509.6 \text{ kip·ft})$$

The maximum positive moment M_u occurs 13.25 ft from the left support (Fig. 6.8). The inflection points occur 0.49 and 0.19 ft from the left and right supports, respectively.

Shear Connectors. Next, the studs required to develop the maximum positive moment and the moments at the concentrated loads are determined. Welded studs $^3/_4$ in in diameter are to be used. As in Art. 6.13, the nominal strength of a stud is $Q_n = 17.7$ kips.

For development of the maximum positive moment on both sides of the point of maximum moment, with $\Sigma Q_n = 260$ kips, at least 260/17.7 = 14.69 studs are required. Since the negative-moment region is small, it is not practical to limit the stud placement to the positive-moment region only. Therefore, additional studs are required for placement of connectors over the entire 30-ft span.

Stud spacing on the left of the point of maximum moment should not exceed

$$S_L = \frac{12(13.25 - 0.49)}{14.69} = 10.42 \text{ in}$$

Stud spacing on the right of the point of maximum moment should not exceed

$$S_R = \frac{12(30 - 13.25 - 0.19)}{14.69} = 13.53 \text{ in}$$

For determination of the number of studs and spacing required between the concentrated load P 10 ft from the left support (Fig. 6.8) and the left inflection point, the maximum moment at that load is calculated to be $M_{Lu} = 502.1$ kip·ft. For the W21 × 44 Grade 50 beam, the manual table indicates that for $\Sigma Q_n = 260$ kips and $Y_2 = 4.68$ in, as calculated previously, the design strength is $\phi M_n = 546$ kip·ft. For $^3/_4$-in studs and $\Sigma Q_n = 260$ kips, the required number of studs is 14.69. Spacing of these studs, which may not exceed 10.42 in, is also limited to

$$S_{PL} = \frac{12(10 - 0.49)}{14.69} = 7.77 \text{ in}$$

Hence the number of studs to be placed in the 10 ft between P and the left support is 10 × 12/7.77 = 15.4 studs. Use 16 studs.

For determination of the number of studs and spacing required between the concentrated load P 10 ft from the right support (Fig. 6.8) and the right inflection point, the maximum moment at that load is calculated to be $M_{Ru} = 481.2$ kip·ft. For the W21 × 44, the manual table indicates that, for Case 7, $\Sigma Q_n = 163$ kips and $\phi M_n = 486$ kip·ft. The required number of studs for $\Sigma Q_n = 163$ kips is 163/17.7 = 9.21 studs. Spacing of these studs, which may not exceed 13.53 in, is also limited to

$$S_{PR} = \frac{12(10 - 0.19)}{9.21} = 12.78 \text{ in}$$

The number of studs to be placed in the 10 ft between P and the right support is 10 × 12/12.78. Use 10 studs.

The number of studs required between the two concentrated loads equals the sum of the number required between the point of maximum moment and P on the left and right. On the left, the required number of studs is 13.25 × 12/10.42 − 16 = −0.74. Since the result is negative, use on the left the maximum permissible stud spacing of 36 in. On the right, the required number of studs is 16.75 × 12/13.53 − 10 = 4.85. Use 5 studs. The spacing should not exceed 12(16.75 − 10)/5 = 16.2 in. Specification of one spacing for the middle segment, however, is more practical. Accordingly, the number of studs between the two concentrated loads would be based on the smallest spacing on either side of the point of maximum moment: 10 × 12/16.2 = 7.4. Use 8 studs spaced 15 in center to center.

It may be preferable to specify the total number of studs placed on the beam based on one uniform spacing. The spacing required to develop the maximum moment on either side of its location and between each concentrated load and a support is 7.77 in, as calculated previously. For this spacing over the 30-ft span, the total number of studs required is 30 × 12/7.77 = 46.3. Use 48 studs (the next even number).

Deflection Computations. The elastic properties of the composite beam, which consists of a W21 × 44 and a concrete slab 5.25 in deep (an average of 4.25 in deep) and 90 in wide, are as follows:

$$E_c = 115^{1.5}\sqrt{3.0} = 2136 \text{ ksi}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{2136} = 13.58$$

$$\frac{b}{n} = \frac{90}{13.58} = 6.63 \text{ in}$$

$$I_{\text{tr}} = 2496 \text{ in}^4$$

For determination of the effective moment of inertia I_{eff} at the location of the maximum moment, a reduced value of the transformed moment of inertia I_{tr} is used based on the partial-composite construction assumed in the computation of shear-connector requirements. For use in Eq. (6.30), the moment of inertia of the W21 × 44 is $I_s = 843$ in^4, $Q_n = 260$ kips, and C_f is the smaller of

$$C_f = 0.85 f_c' A_c = 0.85 \times 3.0 \times 4.25 \times 90 = 975.4 \text{ kips}$$

$$C_f = A_s F_y = 13.0 \times 50 = 650 \text{ kips} \qquad \text{(governs)}$$

$$I_{\text{eff}} = 843 + (2496 - 843)\sqrt{\frac{260}{650}} = 1888 \text{ in}^4$$

A reduced moment of inertia I_r due to long-time effect (creep of the concrete) is determined based on a modular ratio $2n = 2 \times 13.58 = 27.16$ and effective slab width $b/n = 90/27.16 = 3.31$ in. The reduced transformed moment of inertia is 2088 in^4 and the reduced effective moment of inertia is

$$I_r = 843 + (2088 - 843)\sqrt{\frac{260}{650}} = 1630 \text{ in}^4$$

The deflection computations for unshored construction exclude the weight of the concrete slab and steel beam. Whether the steel beam is adequately cambered or not, the assumption is made that the concrete will be finished as a level surface. Hence the concrete slab is likely to be thicker at midspan of the beams and deck.

For computation of the midspan deflections, the cantilevers are assumed to carry only dead load. From Table 6.5, the superimposed dead loads are $P_s = 7.5$ kips, $w_{Ls} = 0.75$ kips/ft, and $w_{Rs} = 0.30$ kips/ft. The dead-load end moments are $M_L = 22.5$ kip·ft and $M_R = 7.5$ kip·ft. For $I_r = 1630$ in^4, the maximum deflection due to these loads is

$$D = \frac{15{,}865{,}000}{29{,}000 \times 1630} = 0.336 \text{ in}$$

The deflection at the left concentrated load P is 0.296 in, and at the second load it is 0.288 in.

From Table 6.5, the live loads with a 40% reduction for size of area supported are $P_L = 9.0$ kips, $w_{LL} = 0.30$ kips/ft, and $w_{RL} = 0.12$ kips/ft. The maximum deflection due to these loads and with an effective moment of inertia of 1888 in^4 is 0.319 in. The deflection at the left load is 0.282 in and at the second load is 0.275 in.

Total deflections due to superimposed dead loads and live loads are

$$\text{Maximum deflection} = 0.336 + 0.319 = 0.655 \text{ in}$$

$$\text{Deflection at left load } P = 0.295 + 0.282 = 0.577 \text{ in}$$

$$\text{Deflection at right load } P = 0.288 + 0.275 = 0.563 \text{ in}$$

6.15 EXAMPLE—LRFD FOR WIDE-FLANGE COLUMN IN A MULTISTORY RIGID FRAME

Columns at the ninth level of a multistory building are to be part of a rigid frame that resists wind loads. Typical floor-to-floor height is 13 ft.

In the ninth story, a wide-flange column of A992 steel is to carry loads from a transfer girder, which supports an offset column carrying the upper levels. Therefore, the lower column discontinues at the ninth level. The loads on that column are as follows: dead load, 750 kips; superimposed dead load, 325 kips; and live load, 250 kips. The moments due to gravity loads at the beam–column connection are

$$\text{Dead-load major-axis moment} = 180 \text{ kip}\cdot\text{ft}$$

$$\text{Live-load major-axis moment} = 75 \text{ kip}\cdot\text{ft}$$

$$\text{Dead-load major-axis moment} = 75 \text{ kip}\cdot\text{ft}$$

$$\text{Live-load major-axis moment} = 40 \text{ kip}\cdot\text{ft}$$

The column axial loads and moments due to service lateral loads with P–Δ effect included are

$$\text{Axial load} = 600 \text{ kips}$$

$$\text{Major-axis moment} = 1050 \text{ kip}\cdot\text{ft}$$

$$\text{Minor-axis moment} = 0.0$$

The beams attached to the flanges of the column with rigid welded connections are part of the rigid frame and have spans of 30 ft. The following beam sizes and corresponding stiffnesses at the top and bottom ends of the column apply.

The beams at both sides of the column at the floor above and the floor below are W36 × 300. The sum of the stiffnesses I_b/L_b of the beam is

$$\sum (I_b/L_b) = 20{,}300 \times \frac{2}{30 \times 12} = 112.8 \text{ in}^3$$

where I_b is the beam moment of inertia (in^4).

The effective length factor K_x corresponding to the case of frame with sidesway permitted is used in determining the axial-load capacity and the moment magnifier B_1. The moment magnifier B_2 is considered unity inasmuch as the P–Δ effect is included in the analysis.

Axial-Load Capacity. Since the column is part of a wind-framing system, the K values should be computed based on column and beam stiffnesses. To determine the major-axis K_x, assume that a

W14 × 426 with $I_{cx} = 6600$ in^4 will be selected for the column. At the top of the column, where there is no column above the floor, the relative column–beam stiffness is

$$G_A = \frac{\Sigma(I_c/L_c)}{\Sigma(I_b/L_b)} = \frac{6600/12(13-3)}{112.8} = 0.49$$

At the column bottom, with a W14 × 426 column below,

$$G_B = \frac{\Sigma(I_c/L_c)}{\Sigma(I_b/L_b)} = \frac{2 \times 6600/12(13-3)}{112.8} = 0.98$$

From a nomograph for the case when sidesway is permitted (Fig. 6.9*b*), $K_x = 1.23$ (at the intersection with the K axis of a straight line connecting 0.49 on the G_A axis with 0.98 on the G_B axis).

Since the connection of beams to the column web is a simple connection with inhibited sidesway, $K_y = 1.0$.

The effective lengths to be used for determination of axial-load capacity are

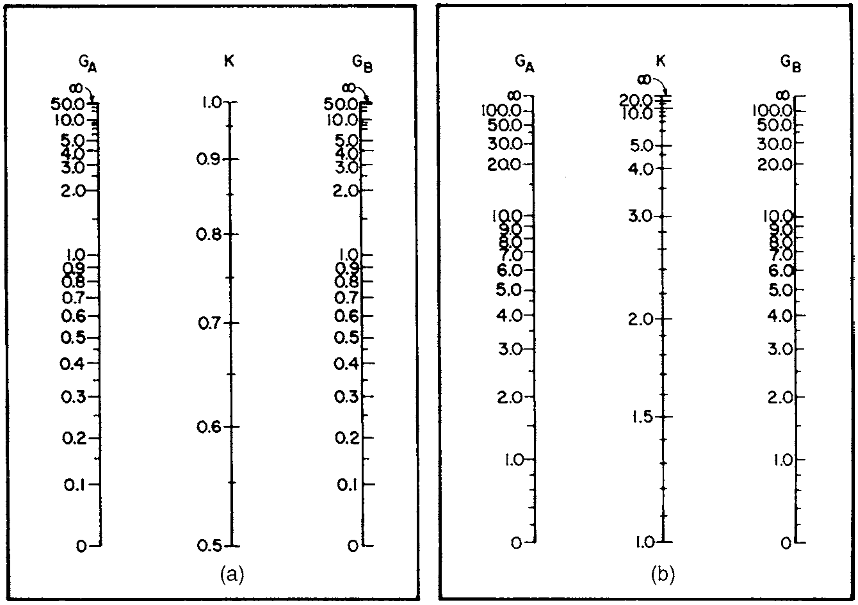
$$K_x L_x = 1.23(13 - 3) = 12.3 \text{ ft}$$

$$K_y L_y = 1.0 \times 13 = 13 \text{ ft}$$

The W14 × 426 has radii of gyration $r_x = 7.26$ in and $r_y = 4.34$ in. Therefore, the slenderness ratios for the column are

$$\frac{K_x L_x}{r_x} = \frac{12.3 \times 12}{7.26} = 20.3$$

$$\frac{K_y L_y}{r_y} = \frac{13 \times 12}{4.34} = 35.9 \qquad \text{(governs)}$$



(a) (b)

FIGURE 6.9 Nomographs for determination of the effective length factor for a column. (*a*) For use when sidesway is prevented. (*b*) For use when sidesway may occur.

Use of the AISC "Manual of Steel Construction" tables for design axial strength of compression members simplifies evaluation of the trial column size. For the W14 × 426, A992 section, a table indicates that for $K_yL_y = 13$ ft, $\phi P_n = 4830$ kips.

Moment Capacity. Next, the nominal bending-moment capacities are calculated. For strong-axis bending moment, $K_yL_y = 13$ ft is assumed for the flange lateral buckling state. The limiting lateral unbraced length L_p (in) for plastic behavior for the W14 × 426 is

$$L_p = \frac{300r_y}{\sqrt{F_y}} = \frac{300 \times 4.34}{\sqrt{50}} = 184 \text{ in} = 15.3 \text{ ft} > 13 \text{ ft}$$

Since the unbraced length is less than L_p,

$$\phi M_{nx} = 0.9 \times 869 \times \frac{50}{12} = 3259 \text{ kip} \cdot \text{ft}$$

$$\phi M_{ny} = 0.9Z_yF_y = 0.9 \times 434 \times \frac{50}{12} = 1628 \text{ kip} \cdot \text{ft}$$

Interaction Equation for Dead Load. For use in the interaction equation for axial load and bending [see Art. 5.7.1, Eq. (5.102)], the factored dead load is

$$P_u = 1.4(750 + 325 + 0.426 \times 13) = 1513 \text{ kips}$$

The factored moments applied to columns due to any general loading conditions should include the second-order magnification. When the frame analysis does not include second-order effects, the factored column moment can be determined from Eq. (5.3).

Computer analysis programs usually include the second-order analysis (P–Δ effects). Therefore, the values of B_2 for moments about both column axes can be assumed to be unity. However, B_1 should be determined for evaluation of the nonsway magnifications. For a braced column (drift prevented), the slenderness coefficient K_x is determined from Fig. 6.9*a* with $G_A = 0.49$ and $G_B = 0.98$, calculated previously. The nomograph indicates that $K_s = 0.73$.

For determination of B_1, the column when loaded is assumed to have single curvature with end moments $M_1 = M_2$. Hence $C_m = 1$.

Determine the elastic buckling load P_{ex} for the column moment of inertia $I_x = 6600$ in^4:

$$P_{ex} = \frac{\pi^2 \times 29{,}000 \times 6600}{[0.73 \times 12(13-3)]^2} = 247{,}000 \text{ kips}$$

With these values, the magnification factor for M_{ux} is

$$B_{ex} = \frac{C_m}{1 - P_u/P_{ex}} = \frac{1.0}{1 - 1513/247{,}000} = 1.006$$

The elastic buckling load P_{ey} with respect to the y axis is

$$P_{ex} = \frac{\pi^2 \times 29{,}000 \times 2360}{(1 \times 13 \times 12)^2} = 247{,}000 \text{ kips}$$

Application of the magnification factor of the dead-load moments due to gravity loads yields

$$M_{ux} = 1.006 \times 1.4 \times 180 = 253.5 \text{ kip} \cdot \text{ft}$$

$$M_{uy} = 1.058 \times 1.4 \times 75 = 111.1 \text{ kip} \cdot \text{ft}$$

The interaction result, which may be considered a section efficiency ratio, is $P_u/\phi P_n = 1513/4830 = 0.313 > 0.2$,

$$R = 0.312 + \frac{8}{9}\left(\frac{253.5}{3259} + \frac{111.1}{1628}\right)$$
$$= 0.312 + \frac{8}{9}(0.0778 + 0.682) = 0.443 < 1.0$$

Interaction Equation for Full Gravity Loading. For use in the interaction equation based on factored loads and moments due to 1.2 times the dead load plus 1.6 times the live load,

$$P_u = 1.2(750 + 325 + 0.426 \times 13) + 1.6 \times 250 = 1697 \text{ kips}$$

Determined in the same way as for the dead load, the magnification factors are

$$B_{1x} = \frac{1.0}{1 - 1697/247{,}000} = 1.007$$
$$B_{1y} = \frac{1.0}{1 - 1697/27{,}700} = 1.065$$

Application of the magnification factors to the factored moments yields

$$M_{ux} = 1.007(1.2 \times 180 + 1.6 \times 75) = 338.4 \text{ kip} \cdot \text{ft}$$
$$M_{uy} = 1.065(1.2 \times 75 + 1.6 \times 40) = 164.0 \text{ kip} \cdot \text{ft}$$

With $P_u/\phi P_n = 1697/4830 = 0.351 > 0.2$, substitution of the preceding values in the interaction equation (Eq. 5.102) yields

$$R = 0.351 + \frac{8}{9}\left(\frac{338.4}{3259} + \frac{164.0}{1628}\right)$$
$$= 0.351 + \frac{8}{9}(0.1038 + 0.1008) = 0.533 < 1$$

Interaction Equation with Wind Load. For use in the interaction equation based on factored loads and moments due to 1.2 times the dead load plus 0.5 times the live load plus 1.3 times the wind load of 600 kips, including the P–Δ effect,

$$P = 1.2(750 + 325 + 0.426 \times 13) + 0.5 \times 250 + 1.3 \times 600 = 2202 \text{ kips}$$

Under wind action, double curvature may occur for strong-axis bending. For this condition, with $M_1 = M_2$,

$$C_{mx} = 0.6 - 0.4 \times 1 = 0.2$$

In this case, the magnification factor for strong-axis bending is

$$B_{1x} = \frac{0.2}{1 - 2202/247{,}000} = 0.202 < 1$$

Use $B_{1x} = 1.0$. The magnification factor for minor-axis bending is, with $C_m = 1$ for single-curvature bending,

$$B_{1y} = \frac{1.0}{1 - 2202/27{,}700} = 1.0864$$

Application of the magnification factors to the factored moments yields

$$M_{ux} = 1.0(1.2 \times 180 + 0.5 \times 75 + 1.3 \times 1050) = 1618 \text{ kip-ft}$$

$$M_{uy} = 1.0864(1.2 \times 75 + 0.5 \times 40) = 119.5 \text{ kip}\cdot\text{ft}$$

With $P_u/\phi P_n^* = 2202/4830 = 0.456 > 0.2$, substitution of the preceding values in the interaction equation [Eq. (5.102)] yields

$$R = 0.456 + \frac{8}{9}\left(\frac{1618}{3259} + \frac{119.5}{1628}\right)$$

$$= 0.456 + \frac{8}{9}(0.496 + 0.0734) = 0.96 < 1$$

This is the governing R value, and since it is less than unity, the column selected, W14 × 426, is adequate.

CHAPTER 7

FLOOR AND ROOF SYSTEMS

Daniel A. Cuoco, P.E.
President
Thornton-Tomasetti Group, Inc.
New York, New York

Structural steel framing provides designers with a wide selection of economical systems for floor and roof construction. Steel framing can achieve longer spans more efficiently than other types of construction. This minimizes the number of columns and footings thereby increasing speed of erection. Longer spans also provide more flexibility for interior-space planning.

Another advantage of steel construction is its ability to readily accommodate future structural modifications, such as openings for tenants' stairs and changes for heavier floor loadings. When reinforcement of existing steel structures is required, it can be accomplished by such measures as addition of framing members connected to existing members and field welding of additional steel plates to strengthen existing members.

FLOOR DECKS

The most common types of floor-deck systems currently used with structural steel construction are concrete fill on metal deck, precast-concrete planks, and cast-in-place concrete slabs.

7.1 CONCRETE FILL ON METAL DECK

The most prevalent type of floor deck used with steel frames is concrete fill on metal deck. The metal deck consists of cold-formed profiles made from steel sheet, usually having a specified minimum yield strength of at least 33 ksi, with 40 ksi becoming more common. Design requirements for metal deck are contained in the American Iron and Steel Institute's "North American Specification for the Design of Cold-Formed Steel Structural Members." (See Chap. 9.)

The concrete fill is usually specified to have a 28-day compressive strength of at least 3000 psi. Requirements for concrete design are contained in the American Concrete Institute Standard ACI 318, "Building Code Requirements for Reinforced Concrete."

Sheet thicknesses of metal deck usually range between 24 and 16 ga, although thicknesses outside this range are sometimes used. The design thicknesses corresponding to typical gage designations are shown in Table 7.1.

Metal deck is commonly available in depths of $1\frac{1}{2}$, 2, and 3 in. Generally, it is preferable to use a deeper deck that can span longer distances between supports and thereby reduce the number of beams

TABLE 7.1 Equivalent Thicknesses for Cold-Formed Steel

Gage designation	Design thickness, in
28	0.0149
26	0.0179
24	0.0239
22	0.0299
20	0.0359
18	0.0478
16	0.0598

required. For example, a maximum beam spacing of about 15 ft can be achieved with 3-in deck. However, each project must be evaluated on an individual basis to determine the most efficient combination of deck depth and beam spacing.

For special long-span applications, metal deck is available with depths of $4^1/_2$, 6, and $7^1/_2$ in from some manufacturers.

Composite versus Noncomposite Construction. Ordinarily, composite metal-deck construction is used with structural-steel framing. In this case, the deck acts not only as a permanent form for the concrete slab but also, after the concrete hardens, as the positive bending reinforcement for the slab. To achieve this composite action, deformations are formed in the deck to provide a mechanical interlock with the concrete (Fig. 7.1). Although not serving a primary structural purpose, welded wire fabric is usually placed within the concrete slab about 1 in below the top surface to minimize cracking due to concrete shrinkage and thermal effects. This welded wire fabric also provides, to a limited degree, some amount of crack control in negative-moment regions of the slab over supporting members.

In cases where the long-term reliability of composite metal deck will be questionable, the metal deck should be used solely as a form, and reinforcement should be placed within the slab to resist all design loadings. For example, in regions where deicing chemicals are applied to streets, metal deck used in parking structures is susceptible to corrosion and may eventually be ineffective unless special precautions are taken.

Noncomposite metal deck is used solely as a form to support the concrete until it hardens. Reinforcement should be placed within the slab to resist all design loadings.

Noncellular versus Cellular Deck. Though relatively uncommon in today's steel-framed construction, it is possible to distribute a building's electrical wiring within the floor deck system, in which case cellular metal deck can be used in lieu of noncellular deck. However, in cases where floor depth is not critical, maximum wiring flexibility and capacity can be attained by using a raised access floor above the structural floor deck.

Cellular deck is essentially noncellular deck, such as that shown in Fig. 7.1, with a flat sheet added to the bottom of the deck to create cells (Fig. 7.2). Electrical, power, and telephone wiring is placed within the cells for distribution over the entire floor area. In many cases, a sufficient number of cells is obtained by combining alternate panels of cellular deck and noncellular deck, which is called a blended system (Fig. 7.3). When cellular deck is used, the 3-in depth is the minimum preferred because it provides convenient space for wiring. The $1^1/_2$-in depth is rarely used.

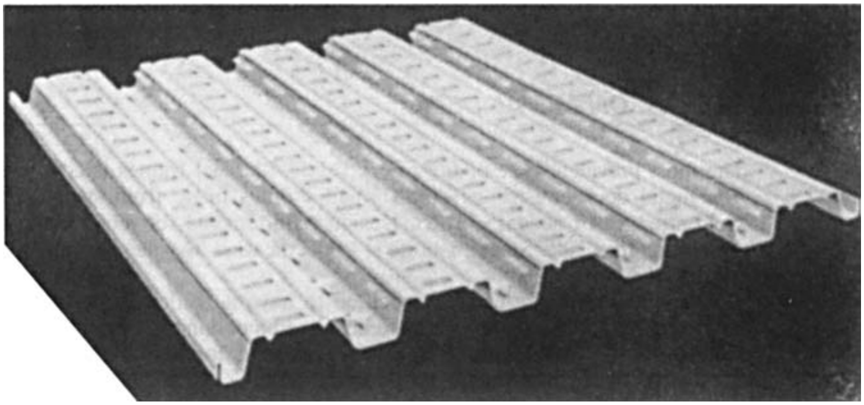


FIGURE 7.1 Cold-formed steel decking used in composite construction with concrete fill.

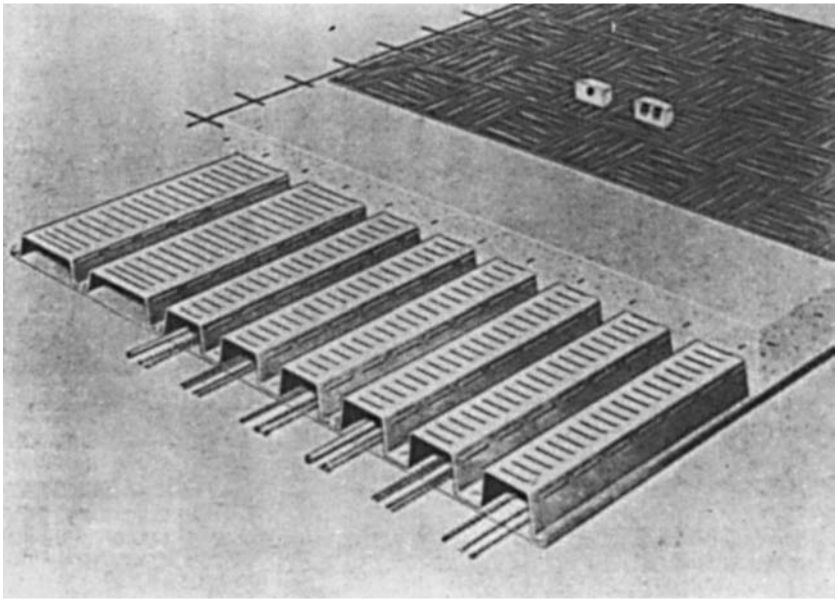


FIGURE 7.2 Cellular steel deck with concrete slab.

For feeding wiring into the cells, a trench header is placed within the concrete above the metal deck, in a direction perpendicular to the cells (Fig. 7.4). Special attention should be given to the design of the structural components adjacent to the trench header, since composite action for both the floor deck and beams is lost in these areas. Where possible, the direction of the cells should be selected to minimize the total length of trench header required. Generally, by running the cells in the longitudinal direction of the building, the total length of trench header is significantly less than if the cells were run in the transverse direction (Fig. 7.5).

If a uniform grid of power outlets is desired, such as 5 ft by 5 ft on centers, preset outlets can be positioned above the cells and cast into the concrete fill. However, in many cases the outlet locations will be dictated by subsequent tenant layouts. In such cases, the concrete fill can be cored and after-set outlets can be installed at any desired location.

Shored versus Unshored Construction. To support the weight of newly placed concrete and the construction live loads applied to the metal deck, the deck can either be shored or be designed to span between supporting members. If the deck is shored, a shallower-depth or thinner-gage deck can be used. The economy of shoring, however, should be investigated, inasmuch as the savings in deck cost is frequently more than offset by the cost of the shoring. Also, slab deflections that will occur after the shoring is removed should be evaluated, as well as concrete cracking over supporting members.

FIGURE 7.3 Blended deck, alternating cellular and noncellular panels, in composite construction.

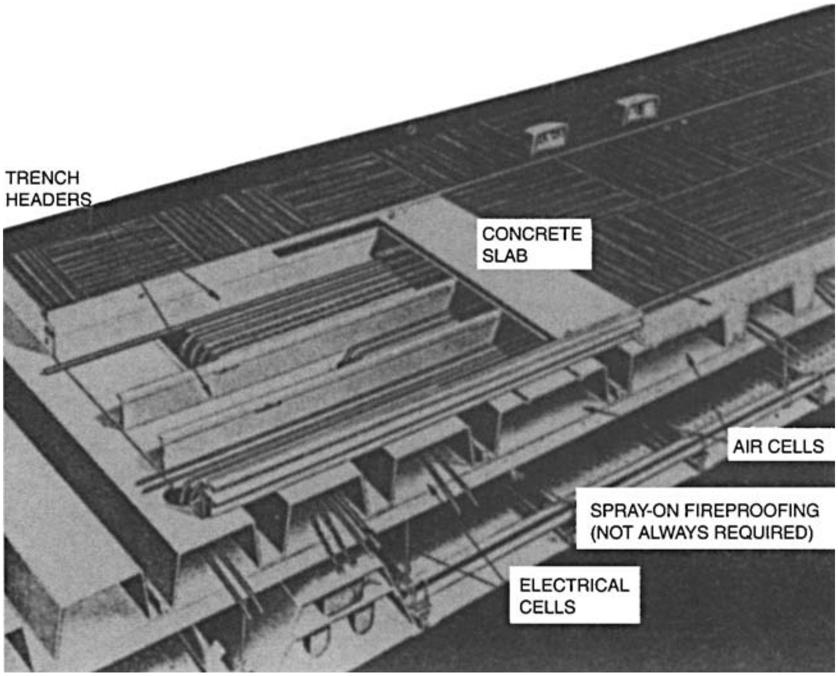


FIGURE 7.4 Cellular steel deck with trench header placed within the concrete slab to feed wiring to cells.

Another consideration is that use of shoring can sometimes affect the construction schedule, since the shoring is usually kept in place until the concrete fill has reached at least 75% of its specified 28-day compressive strength. In addition, when shoring is used, the concrete must resist the stresses resulting from the total dead load combined with all superimposed loadings.

When concrete is cast on unshored metal deck, the weight of the concrete causes the deck to deflect between supports. This deflection is usually limited to the lesser of $^1/_{180}$ the deck span or $^3/_4$ in. If the

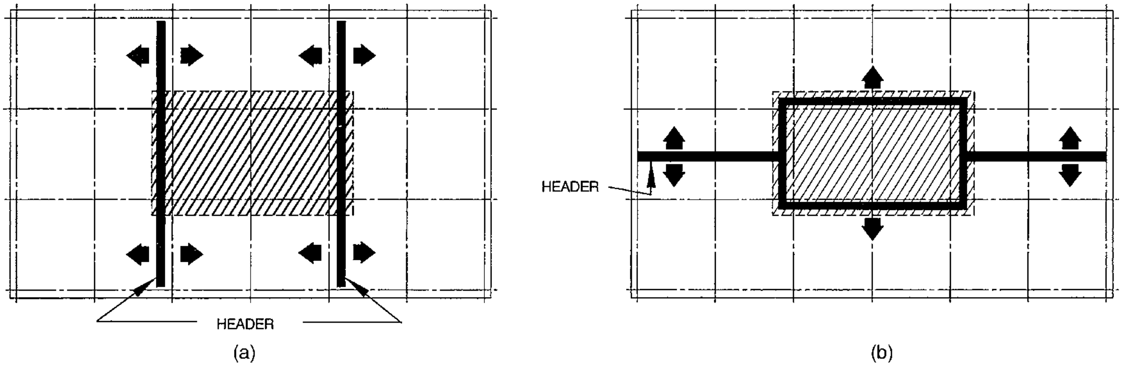


FIGURE 7.5 Floor layout for cellular deck with cells in different directions. Length of trench header serving them is less for (*a*) cells in the longitudinal direction than for (*b*) cells in the transverse direction.

resulting effect on floor levelness is objectionable, the top surface can be finished level, but this will result in additional concrete being placed to compensate for the deflection. The added weight of this additional concrete must be taken into account in design of the metal deck to ensure adequate strength. The concrete fill, however, need only resist the stresses resulting from superimposed loadings.

Unshored metal-deck construction is the system most commonly used. The additional cost of the deeper or thicker deck is generally much less than the cost of shoring. To increase the efficiency of the unshored deck in supporting the weight of the unhardened concrete and construction live loads, from both a strength and deflection standpoint, the deck is normally extended continuously over supporting members for two or three spans, in lieu of single-span construction. However, for loadings once the concrete is hardened, the composite slab is designed for the total load, including slab self-weight, with the slab treated as a single span, unless negative-moment reinforcement is provided over supports in accordance with conventional reinforced-concrete-slab design (disregarding the metal deck as compressive reinforcement).

In cases where the metal deck is designed to span between supporting members without shoring, but there are a few isolated locations with excessive spans, it may be more economical to provide shoring in those isolated locations rather than increase the depth or thickness of the metal deck for the entire floor. Although shoring is normally supported from the floor slab below, an alternative approach is to support shoring from adjacent floor beams and girders, which would provide less interference with ongoing construction operations on the floor slab below the metal deck.

Lightweight versus Normal-Weight Concrete. Either lightweight or normal-weight concrete can serve the structural function of the concrete fill placed on the metal deck. Although there is a cost premium associated with lightweight concrete, sometimes the savings in steel framing and foundation costs can outweigh the premium. Also, lightweight concrete in sufficient thickness can provide the necessary fire rating for the floor system and thus eliminate the need for additional slab fire protection (see “Fire Protection” below). For exposed floor slabs, the potential for increased shrinkage and corresponding cracking of lightweight concrete should be considered. The tradeoffs in use of lightweight concrete versus normal-weight concrete plus fire protection should be evaluated on a project-by-project basis.

Fire Protection. Most applications of concrete fill on metal deck in buildings require that the floor-deck assembly have a fire rating. For noncellular metal deck, the fire rating is usually obtained either by providing sufficient concrete thickness above the metal deck or by applying spray-on fire protection to the underside of the metal deck. For cellular metal deck, which utilizes outlets that penetrate the concrete fill, the fire rating is usually obtained by the latter method. As an alternative, a fire-rated ceiling system can be installed below the cellular or noncellular deck.

When the required fire rating is obtained by concrete-fill thickness alone, lightweight concrete requires a lesser thickness than normal-weight concrete for the same rating. For example, a 2-hour rating can be obtained by using either $3^1/_4$ in of lightweight concrete or $4^1/_2$ in of normal-weight concrete above the metal deck. The latter option is not often used, since the additional thickness of heavier concrete penalizes the steel tonnage (i.e., heavier beams, girders, and columns) and the foundations; however, for situations involving high, nonreducible live loads, such as public assembly areas, the improved composite action of the beams and girders with the thicker floor deck can result in little or no penalty to the beam and girder weight.

If spray-on fire protection is used on the underside of the metal deck, the thickness of concrete above the deck can be the minimum required to resist the applied floor loads. This minimum thickness is usually $2^1/_2$ in, and the less expensive normal-weight concrete may be used instead of lightweight concrete. Therefore, the two options that are frequently considered for a 2-hour-rated, noncellular floor-deck system are $3^1/_4$-in lightweight concrete above the metal deck without spray-on fire protection and $2^1/_2$-in normal-weight concrete above the metal deck with spray-on fire protection (Fig. 7.6). Since the dead load of the floor deck for the two options is essentially the same, the steel framing and foundations will also be the same. Thus, the comparison reduces to the cost of the more expensive lightweight concrete versus the cost of the normal-weight concrete plus the spray-on fire protection. Since the costs, and contractor preferences, vary with geographical location, the evaluation must be made on an individual project basis. (See also Art. 4.12.)

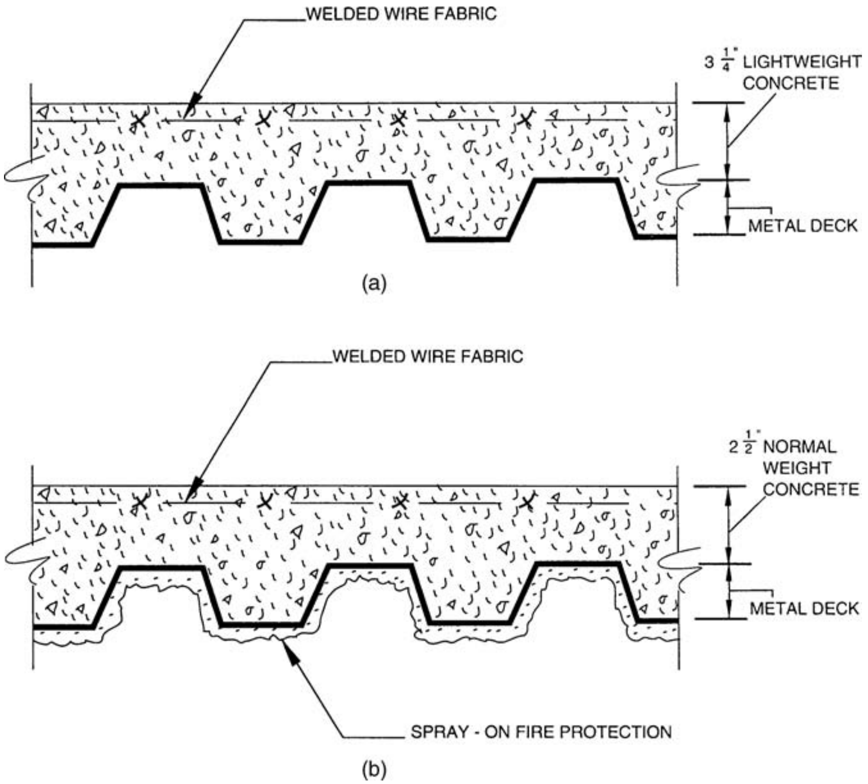


FIGURE 7.6 Two-hour fire-rated floor systems, with cold-formed steel deck. (*a*) With lightweight concrete fill; (*b*) with normal-weight concrete fill.

Diaphragm Action of Metal-Deck Systems. Concrete fill on metal deck readily serves as a relatively stiff diaphragm that transfers lateral loads, such as wind and seismic forces, at each floor level through in-plane shear to the lateral load-resisting elements of the structure, such as shear walls and braced frames. The resulting shear stresses can usually be accommodated by the combined strength of the concrete fill and metal deck, without need for additional reinforcement. Attachment of the metal deck to the steel framing, as well as attachment between adjacent deck units, must be sufficient to transfer the resulting shear stresses (see "Attachment of Metal Deck to Framing" below).

Additional shear reinforcement may be required in floor decks with large openings, such as those for stairs or shafts, with trench headers for electrical distribution, or with other shear discontinuities. Also, floors in multistory buildings in which cumulative lateral loads are transferred from one lateral load-resisting system to another (for example, from perimeter frames to interior shear walls), may be subjected to unusually large shear stresses that require a diaphragm strength significantly greater than that for a typical floor.

Attachment of Metal Deck to Framing. Metal deck can be attached to the steel framing with puddle (arc spot) welds, screws, powder-driven fasteners, or when composite floor framing is used (Art. 7.8), by means of shear connectors welded through the metal deck onto the top flange of the framing member. These attachments provide lateral bracing for the steel framing and, when applicable, transfer shear stresses resulting from diaphragm action. The maximum spacing of attachments to steel framing is generally 12 in.

Attachment of adjacent deck units to each other, that is, sidelap connection, can be made with welds, screws, or button punches. Generally, the maximum spacing of sidelap attachments is 36 in. In addition to diaphragm or other loading requirements, the type, size, and spacing of attachments is sometimes dictated by insurance (Factory Mutual or Underwriters' Laboratories) requirements.

Weld sizes generally range between $^1/_2$-in and $^3/_4$-in minimum visible diameter. When metal deck is welded to steel framing, welding washers should be used if the deck thickness is less than 22 ga to minimize the possibility of burning through the deck. Sidelap welding is not recommended for deck thicknesses of 22 ga and thinner.

Screws can be either self-drilling or self-tapping. Self-drilling screws have drill points and threads formed at the screw end. This enables direct installation without the need for predrilling of holes in the steel framing or metal deck. Self-tapping screws require that a hole be drilled prior to installation. Typical screw sizes are No. 12 and No. 14 (with 0.216-in and 0.242-in shank diameter, respectively) for attachment of metal deck to steel framing. No. 8 and No. 10 screws (with 0.164-in and 0.190-in shank diameter, respectively) are frequently used for sidelap connections.

Powder-driven fasteners are installed through the metal deck into the steel framing with pneumatic or powder-actuated equipment. Predrilled holes are not required. These types of fasteners are not used for sidelap connections.

Button punches can be used for sidelap connections of certain types of metal deck that utilize upstanding seams at the sidelaps. However, since uniformity of installation is difficult to control, button punches are not usually considered to contribute significantly to diaphragm strength.

The diaphragm capacity of various types and arrangements of metal deck and attachments is given in the Steel Deck Institute *Diaphragm Design Manual.*

7.2 PRECAST-CONCRETE PLANK

Precast-concrete plank is another type of floor deck that is used with steel-framed construction (Fig. 7.7). The plank is prefabricated in standard widths, usually ranging between 4 and 8 ft, and is normally prestressed with high-strength steel tendons. Shear keys formed at the edges of the plank are subsequently grouted, to allow loads to be distributed between adjacent planks. Voids are usually placed within the thickness of the plank to reduce the deadweight without causing significant reduction in plank strength. The inherent fire resistance of the precast concrete plank obviates the need for supplementary fire protection.

Topped versus Untopped Planks. Precast planks can be structurally designed to sustain required loadings without need for a cast-in-place concrete topping. However, in many cases, it is advantageous to utilize a topping to eliminate differences in camber and elevation between adjacent planks at the joints and thus provide a smooth slab top surface. When a topping is used, the top surface of the plank may be intentionally roughened to achieve composite action between topping and plank. Thereby, the topping also serves as a structural component of the floor-deck system.

A cast-in-place concrete topping can be used for embedment of conduits and outlets that supply electricity and communication services. Voids within the planks can also be used as part of the

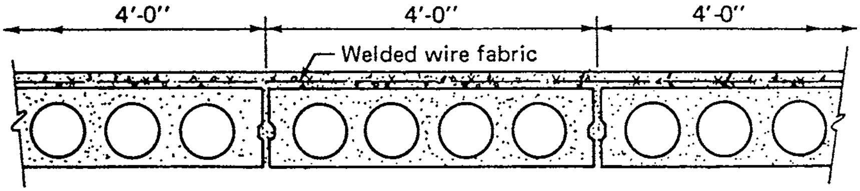


FIGURE 7.7 Precast-concrete plank floor with concrete topping.

distribution system. When the topping is designed to act compositely with the plank, however, careful consideration must be given to the effects of these embedded items.

Dead-Load Deflection of Concrete Plank. In design of prestressed-concrete planks, the prestressing load balances a substantial portion of the dead load. As a result, relatively small dead-load deflections occur. For planks subjected to significant superimposed dead-load conditions of a sustained nature, for example, perimeter plank supporting an exterior masonry wall, additional prestressing to compensate for the added dead load, or some other stiffening method, is required to prevent large initial and creep deflections of the plank.

Diaphragm Action of Concrete-Plank Systems. The diaphragm action of a floor deck composed of precast-concrete planks can be enhanced by making field-welded connections between steel embedments located intermittently along the shear keys of adjacent planks. Additional diaphragm strength may be required in certain situations, such as when large floor-deck openings are present (see "Diaphragm Action of Metal-Deck Systems" in Art. 7.1).

Attachments of Concrete Plank to Framing. Precast-concrete planks are attached to and provide lateral bracing for supporting steel framing. A typical method of attachment is a field-welded connection between the supporting steel and steel embedments in the precast planks.

7.3 CAST-IN-PLACE CONCRETE SLABS

Use of cast-in-place concrete for floor decks in steel-framed construction is a traditional approach that is seldom used today because of the advent of metal deck and spray-on fire protection. For one of the more common types of cast-in place concrete floors, the formwork is configured to encase the steel framing, to provide fire protection and lateral bracing for the steel (see Fig. 7.8). If the proper confinement details are provided, this encasement can also serve to achieve composite action between the steel framing and the floor deck.

Dead-load deflections should be calculated and, for long spans with large deflections, the formwork should be cambered to provide a level deck surface after removal of the formwork shoring. Diaphragm action is readily attainable with cast-in-place concrete floor decks. As previously indicated, additional diaphragm strength may be required in certain situations, such as when large floor-deck openings are present (see "Diaphragm Action of Metal-Deck Systems" in Art. 7.1).

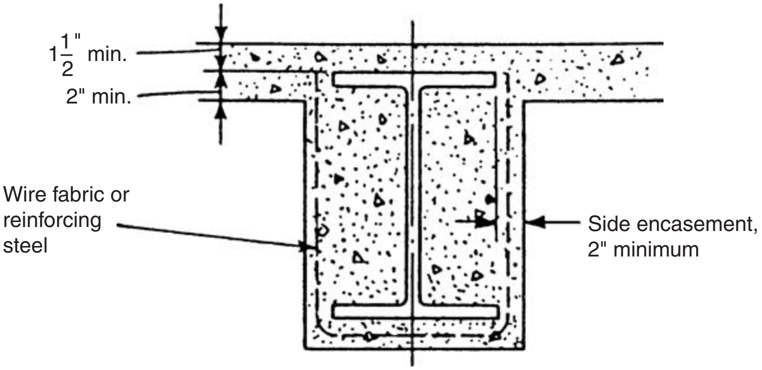


FIGURE 7.8 Minimum requirements for composite action with concrete-encased steel framing.

ROOF DECKS

The systems used for floor decks (Arts. 7.1 to 7.3) can also be used for roof decks. When used as roof decks, these systems are overlaid by roofing materials, to provide a weathertight enclosure. Other roof-deck systems are described in Arts. 7.4 to 7.7.

7.4 METAL ROOF DECK

Steel-framed buildings often utilize a roof deck composed simply of metal deck. When properly sloped for drainage, the metal deck itself can serve as a watertight enclosure. Alternatively, roofing materials can be placed on top of the deck. In either case, diaphragm action can be achieved by proper sizing and attachment of the metal deck. A fire rating can be provided by applying spray-on fire protection to the underside of the roof deck, or by installing a fire-rated ceiling system below the deck.

Metal roof deck usually is used for noncomposite construction. It is commonly available in depths of 1½, 2, and 3 in. Long-span roof deck is available with depths of 4½, 6, and 7½ in from some manufacturers. Cellular roof deck is sometimes used to provide a smooth soffit. When a lightweight insulating concrete fill is placed over the roof deck, the deck should be galvanized and also vented (perforated) to accelerate the drying time of the insulating fill, and prevent entrapment of water vapor. Acoustical deck is also available for use in applications where enhanced sound absorption is desired.

Standing-Seam System. When the metal roof deck is to serve as a weathertight enclosure, connection of deck units with standing seams offers the advantage of placing the deck seam above the drainage surface of the roof, thereby minimizing the potential for water leakage (Fig. 7.9). The seams can simply be snapped together or, to enhance their weathertightness, can be continuously seamed by mechanical

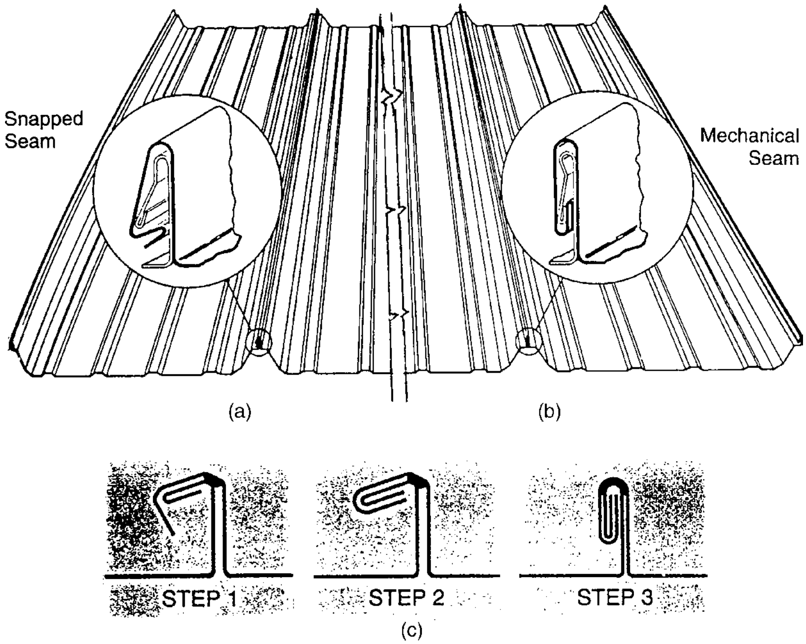


FIGURE 7.9 Standing-seam roof deck. (*a*) With snapped seam; (*b*) with mechanical seam; (*c*) steps in forming a seam.

means with a field-operated seaming machine provided by the deck manufacturer. Some deck types utilize an additional cap piece over the seam, which is mechanically seamed in the field (Fig. 7.10). Frequently, the seams contain a factory-applied sealant for added weather protection.

Thicknesses of standing-seam roof decks usually range between 26 and 20 ga. Typical spans range between 3 and 8 ft. A roof slope of at least $^{1}/_{4}$ in per ft should be provided for drainage of rainwater.

Standing-seam systems are typically attached to the supporting members with concealed anchor clips (Fig. 7.11) that allow unimpeded longitudinal thermal movement of the deck relative to the supporting structure. This eliminates buildup of stresses within the system and possible leakage at connections. However, the effect on the lateral bracing of supporting members must be carefully evaluated, which may result in a need for supplementary bracing. An evaluation method is presented

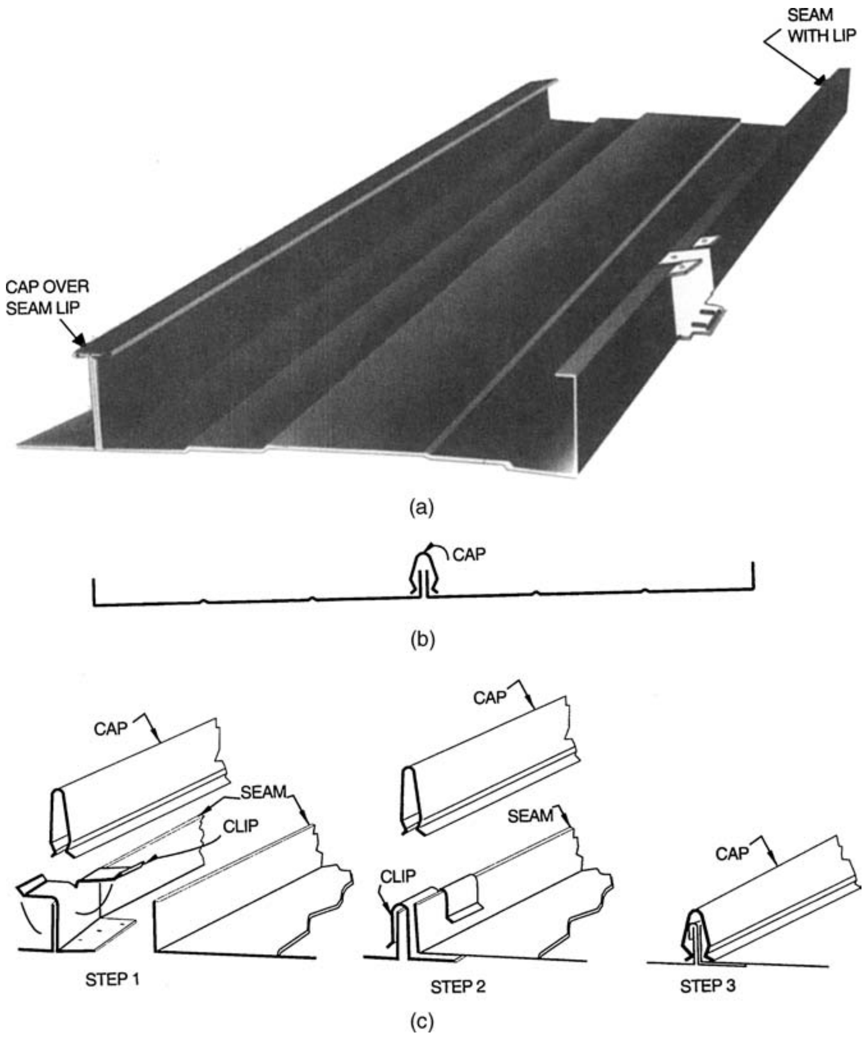


FIGURE 7.10 Standing-seam roof deck with cap installed over the seams. (*a*) Channel cap with flanges folded over lip of seam. (*b*) U-shaped cap clamps over clips on seam. (*c*) Steps in forming a seam with clamped cap.

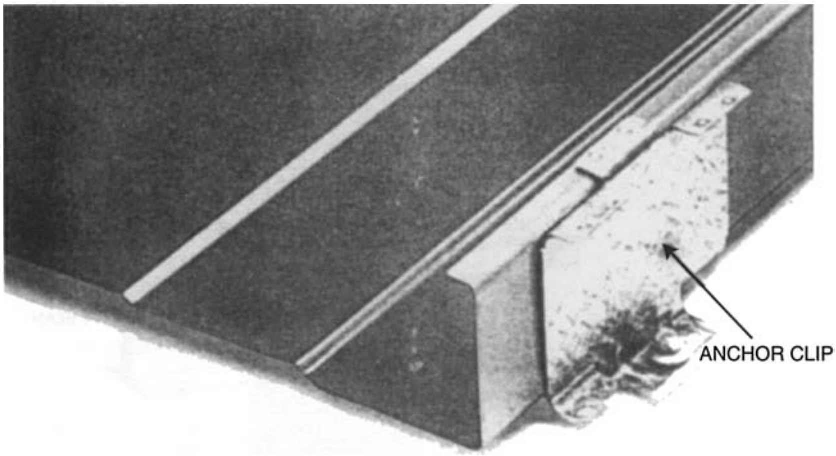


FIGURE 7.11 Typical anchor clip for standing-seam roof deck.

in the American Iron and Steel Institute's "North American Specification for the Design of Cold-Formed Steel Structural Members." (See Art. 9.12.4.)

7.5 LIGHTWEIGHT PRECAST-CONCRETE ROOF PANELS

Roof decks of lightweight precast-concrete panels typically span 5 to 10 ft between supports. Panel thicknesses range from 2 to 4 in, and widths are usually 16 to 24 in. Depending on the product, concrete density can vary from 50 to 115 lb per ft^3. Certain types of panels have diaphragm capacities depending on the edge and support connections used. Many panels can achieve a fire rating when used as part of an approved ceiling assembly.

The panels are typically attached to steel framing with cold-formed-steel clips (see Fig. 7.12). The joints between panels are cemented on the upper side, usually with an asphaltic mastic compound. Insulation and roofing materials are normally placed on top of the panels. Some panels are nailable for application of certain types of roof finishes, such as slate, tile, and copper.

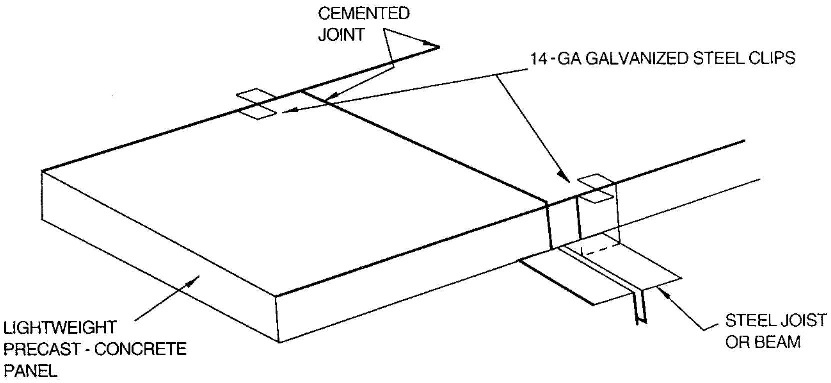


FIGURE 7.12 Typical clips for attachment of lightweight precast-concrete panels to steel framing. The clips are driven into place for a wedge fit at diagonal corners of the panels. Minimum flange width for supporting member is preferably 4 in.

7.6 WOOD-FIBER PLANKS

Planks formed of wood fibers bonded with portland cement provide a lightweight roof deck with insulating and acoustical properties. The typical density of this material ranges between 30 and 40 lb/ft^3. Some plank types have diaphragm capacities. When used as part of an approved ceiling assembly, many planks can achieve a fire rating. This type of roof deck system is commonly used for gymnasiums and similar facilities because of its superior acoustical properties.

The planks are usually supported by steel bulb tees (Fig. 7.13), which are nominally spaced 32 to 48 in on center. The joint over the bulb tee is typically grouted with a gypsum-concrete grout and roofing materials are applied to the top surface of the planks.

7.7 GYPSUM-CONCRETE DECKS

Although they are not in common use today, poured gypsum concrete can be used in conjunction with steel bulb tees, formboards, and galvanized reinforcing mesh (Fig. 7.14). Drainage slopes can be readily built into the roof deck by varying the thickness of gypsum.

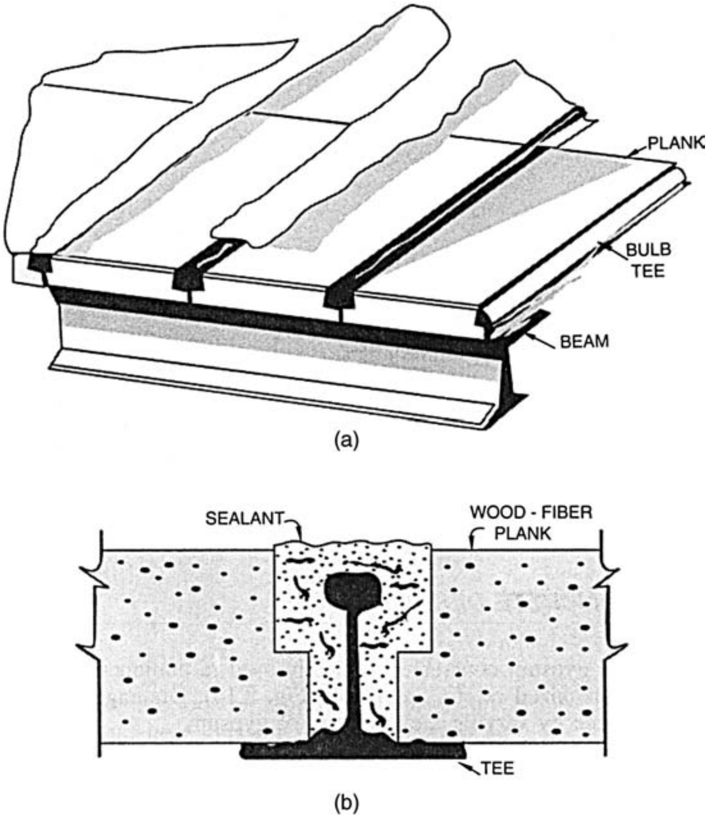


FIGURE 7.13 (*a*) Wood-fiber planks from roof deck. (*b*) Plank is supported by a steel bulb tee.

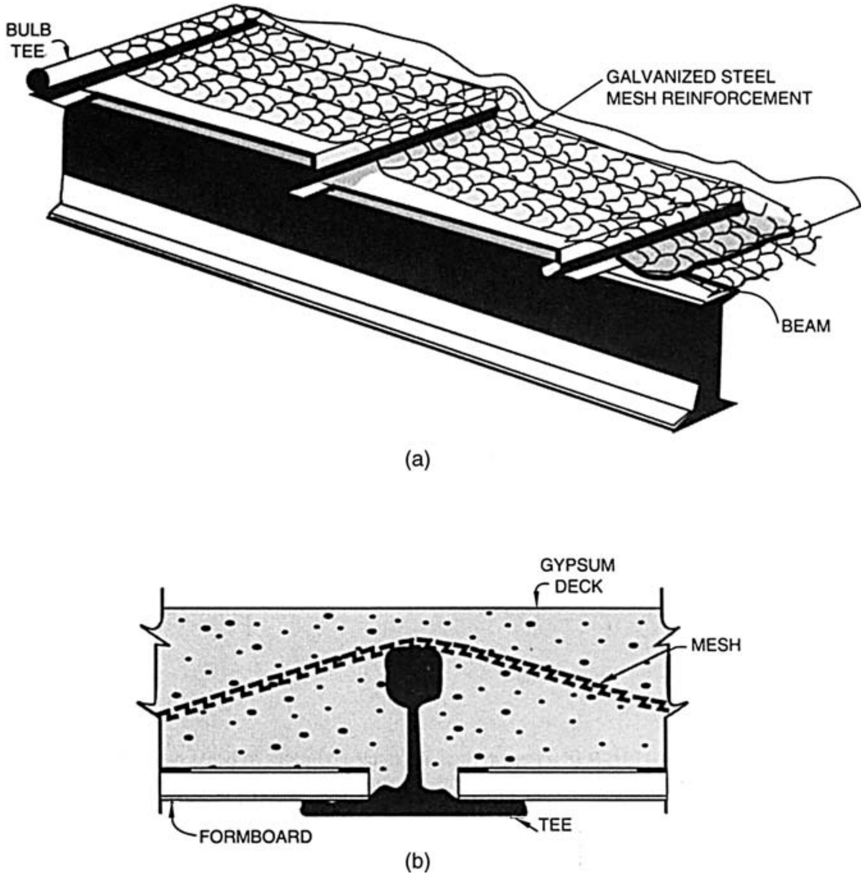


FIGURE 7.14 (*a*) Gypsum-concrete roof deck. (*b*) Cast on formboard, the deck is supported by a steel bulb tee.

FLOOR FRAMING

With a large variety of structural steel floor-framing systems available, designers frequently investigate several systems during the preliminary design stage of a project. The lightest framing system, although the most efficient from a structural engineering standpoint, may not be the best selection from an overall project standpoint, since it may have such disadvantages as high fabrication costs, large floor-to-floor heights, and difficulties in interfacing with mechanical ductwork.

Spandrel members are frequently subjected to torsional loadings induced by facade elements and thus require special consideration. In addition, design of these members is frequently governed by deflection criteria established to avoid damage to, or to permit proper functioning of, the facade construction.

7.8 ROLLED SHAPES

Hot-rolled, wide-flange steel shapes are the most commonly used members for multistory steel-framed construction. These shapes, which are relatively simple to fabricate, are economical for beams and girders with short to moderate spans.

In general, wide-flange shapes are readily available in several grades of steel. Currently, the preferred material specification for wide-flange shapes is ASTM A992, having a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi, rather than the previously used designations of ASTM A36 and ASTM A572 Grade 50. Higher strengths can be obtained by specifying ASTM A572 Grade 60 or 65, or ASTM A913 Grade 60, 65, or 70. Other ASTM designations and grades of steel are also available, though less commonly used.

Interfacing with mechanical ductwork is usually accomplished in one of two ways. First, the steel framing can be designed to incorporate the shallowest members that provide the required strength and stiffness, and the mechanical ductwork can be routed beneath the floor framing. As an alternative, deeper beams and girders than would otherwise be necessary can be used, and these members can be fabricated with penetrations, or openings, that allow passage of ductwork and pipes. A variation of this approach is to run the ductwork between floor beams, thereby requiring penetrations only in the girders. Openings can be either unreinforced, when located in zones subjected to low stress levels, or reinforced with localized steel plates, pipes, or angles (Fig. 7.15).

("Steel and Composite Beams with Web Openings," Steel Design Guide Series no. 2, American Institute of Steel Construction, Chicago, Ill.)

Composite versus Noncomposite Construction. Wide-flange beams and girders are frequently designed to act compositely with the floor deck. This enables the use of lighter or shallower members. Composite action is readily achieved through the use of shear connectors welded to the top flange of the beam or girder (Fig. 7.16). When the floor deck is composed of concrete fill on metal deck, the shear connectors are field-welded through the metal deck and onto the top flange of the beam or girder, prior to concrete placement.

Composite strength is usually controlled by shear transfer or by bottom flange tension. In cases where increased future loadings are likely, such as file storage loading in office areas, additional

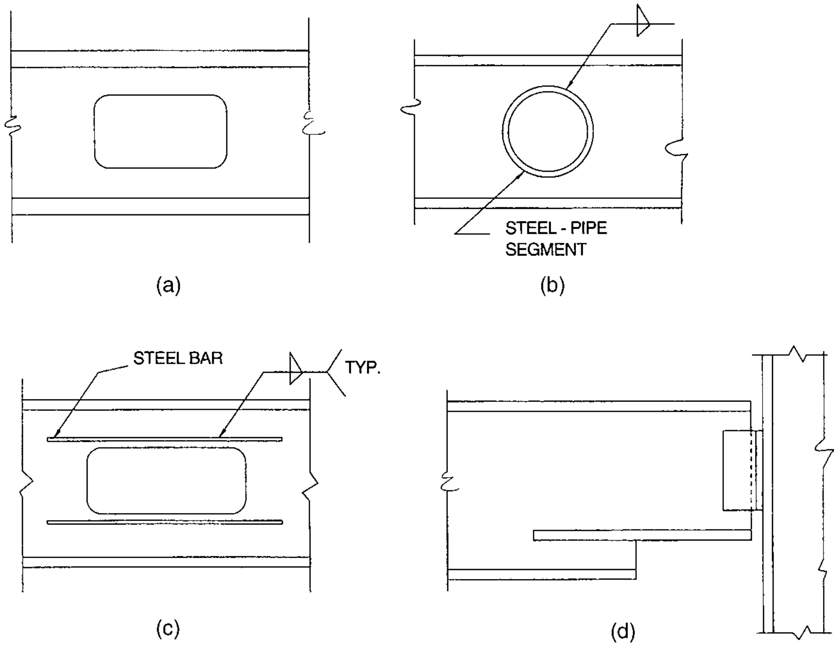


FIGURE 7.15 Penetrations for ducts and pipes in beam or girder webs. (*a*) Rectangular opening, unreinforced. (*b*) Circular opening reinforced with a steel-pipe segment. (*c*) Rectangular penetration reinforced with steel bars welded to the web. (*d*) Reinforced cope at a column.

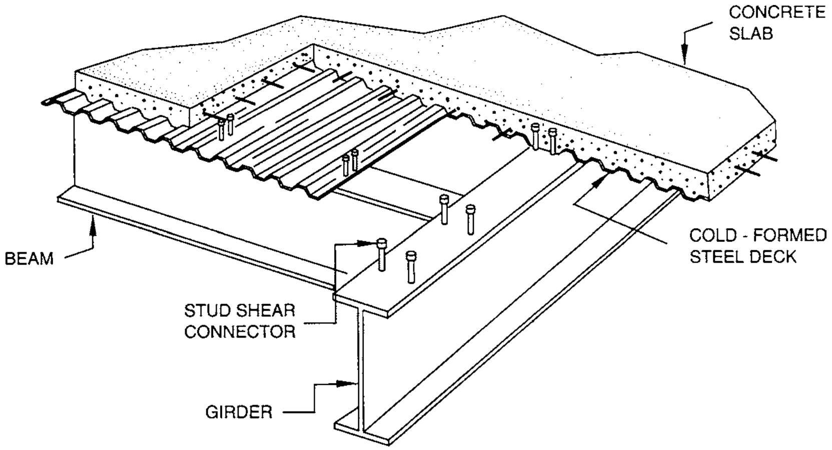


FIGURE 7.16 Beam and girder with shear connectors for composite action with concrete slab.

shear connectors can be provided in the original design at minimal additional cost. When the increased loadings must be accommodated, reinforcement plates need only be welded to the easily accessible bottom flange of the beams and girders, since the added shear connectors have already been installed.

Noncomposite design is generally found to be more economical for relatively short spans, where the added cost of shear connectors tends not to justify the savings in steel framing, or for small projects where the cost of mobilization to provide shear connectors is not warranted.

Shored versus Unshored Construction. Composite floor framing can be designed as being either shored or unshored during construction. In most cases, unshored construction is used, which allows dead-load deflections to occur during the concrete placement and the floors to be finished with a level surface. In such cases, the additional concrete dead load must be taken into account when designing the beams and girders, and other components of the structure.

When unshored construction is used for moderate spans with relatively large dead-load deflections, the beams and girders can be cambered for the dead-load deflection, thereby resulting in a level floor surface after placement of the concrete. Maximum and minimum values of camber, which are dependent on the depth and length of the beam or girder, and permissible camber tolerances, are given in the "Standard Mill Practice" section of the AISC *Manual of Steel Construction*.

When camber is specified, however, careful consideration should be given to the end restraint of the beam (for example, whether the beam frames into girders or into columns), even if simple connections are used throughout. End restraint reduces deflections, and camber that exceeds the actual dead-load deflection can sometimes be troublesome, since it may affect the fire rating (because of insufficient concrete-fill thickness over metal deck), the elevation of preset inserts in an electrified floor system, or installation of interior finishes. It should also be kept in mind that permissible camber tolerances are always positive, that is, will result in a camber greater than that specified.

Shored construction will result in lighter or shallower beams and girders than unshored construction, since the flexural members will act compositely with the floor deck in resisting the weight of the concrete when the shores are removed. However, consideration must be given to the deflections that will occur after shore removal, and whether the resulting floor levelness will be acceptable. Also, the effect on the construction schedule should be considered, since the shoring must be kept in place until the concrete fill has reached sufficient strength. This is usually at least 75% of its specified minimum 28-day compressive strength.

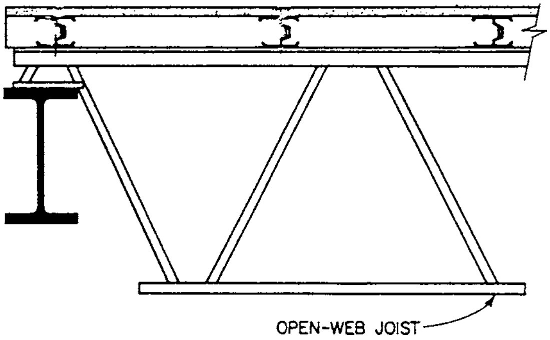


FIGURE 7.17 Open-web steel joist supports gypsum deck.

7.9 OPEN-WEB JOISTS

Although they are more frequently used for moderate- to long-span roof framing, open-web steel joists (Fig. 7.17) are sometimes used for floor framing in multistory buildings, particularly for small office buildings. Joists as floor members subjected to gravity loadings represent an efficient use of material, particularly since net uplift loadings that are sometimes applicable for roof joist design are not applicable for floor joist design. Also, the open webs of joists provide an effective means of routing mechanical ductwork throughout the floor.

Joists can be designed to act compositely with the floor deck by adding shear connectors to the top chord. In cases where increased future loadings are likely, such as file storage loading in office areas, the web members and their connections can be oversized and additional shear connectors can be provided in the original design at minimal additional cost. At the time when the increased loadings must be accommodated, reinforcement plates need only be welded to the easily accessible bottom chord of the joists, since the added shear connectors and increased web sizes have already been provided.

7.10 LIGHTWEIGHT STEEL FRAMING

Cold-formed steel structural members can provide an extremely lightweight floor framing system. These members, usually C or Z shapes, are normally spaced on 24 in centers and can span up to about 30 ft between supports. Because of their light weight, these members can be handled and installed easily and quickly. Connections of cold-formed members are usually accomplished by welding or by the use of self-drilling screws.

This type of floor-framing system is frequently used in conjunction with cold-formed steel load-bearing wall studs for low-rise construction. Spans are usually short to keep depth of floor system small. This depth has a direct bearing on the overall height of structure to which costs of several building components are proportional.

Space in residential buildings often is so arranged that beams and columns can be confined, hidden from view, within walls and partitions. Since parallel walls or partitions usually are spaced about 12 ft apart, joists that span between beams located in those dividers can be short-span.

In Fig. 7.18, the joists span in the short direction of the panel to obtain the least floor depth. They are supported on beams of greater depth hidden from view in the walls. With moment connections to the columns, these beams are designed to resist lateral forces on the building as well as vertical loading. (Depth of the beams may be dictated by lateral-force design criteria.) As part of moment-resisting frames, the beams usually are oriented to span parallel to the narrow dimension of the structure. In that case, the joists are set parallel to the long axis of the building. When beam and joist spans are nearly equal, framing costs generally will be lower if the joists are oriented to span between wind girders,

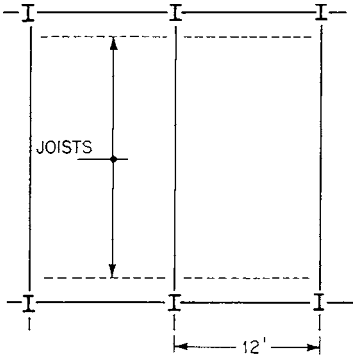


FIGURE 7.18 Typical short-span floor framing for a high-rise residential building.

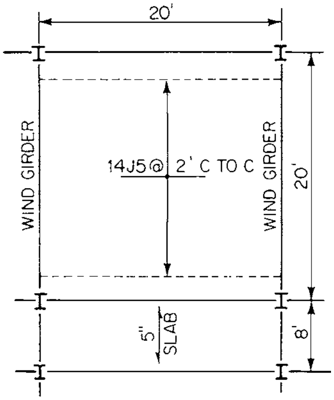


FIGURE 7.19 For economical framing, joists are supported on wind girders.

regardless of their orientation (Fig. 7.19). This arrangement takes advantage of the substantial members required for lateral-force resistance without appreciably increasing their sizes to carry the joists.

The service core of a high-rise residential building, containing stairs, elevators, and shafts for ducts and pipes, usually is framed with lightweight, shallow beams. These are placed around openings to provide substantial support for point loading.

Because of lighter dead and live loads, columns in residential buildings are much smaller than columns in office buildings and usually are less visible. Orientation of columns usually is determined by wind criteria and often is oriented as indicated in Fig. 7.20. However, seismic loads (if applicable) and/or P–Δ effects may control in the longitudinal direction, and in that case, additional lateral-load resisting elements such as frame bracing or shear walls can be added.

7.11 TRUSSES

When relatively long spans are involved, trusses are frequently selected for the floor-framing system. As for open-web joists, mechanical ductwork can be easily routed through the web openings. Shear connectors can be added for composite action with the floor deck. Increased future loadings can be accommodated at a minimal cost premium by oversizing the web members and their connections and providing additional shear connectors in the original design.

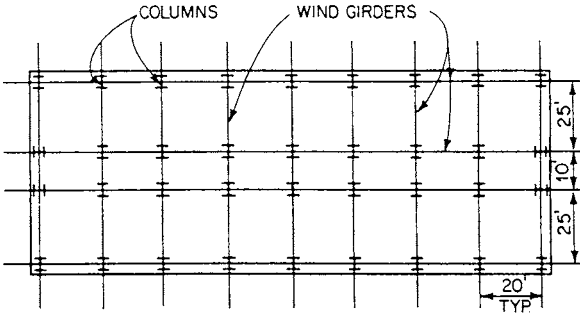


FIGURE 7.20 Typical framing plan for narrow, high-rise building orients columns for strong-axis resistance to lateral forces in the narrow direction.

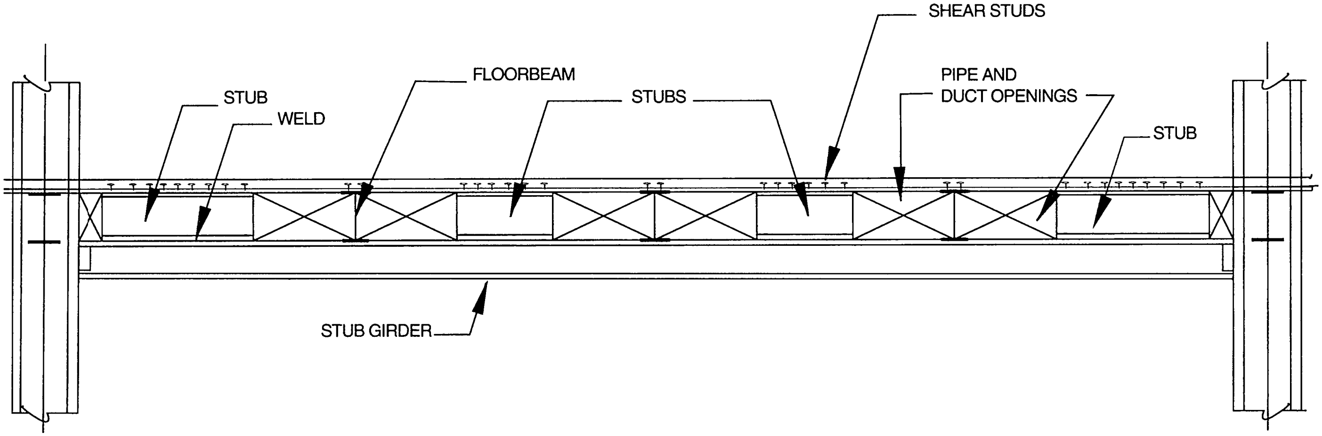


FIGURE 7.21 Stub girder supports floor beams on top flange.

7.12 STUB GIRDERS

The primary advantage of the stub-girder system is that it provides ample space for routing mechanical ductwork throughout a floor while achieving a reduced floor construction depth as compared to conventional steel framing.

This system utilizes floorbeams that are supported on top of, rather than framed into, stub girders. Thus, the floorbeams are designed as continuous members, which results in steel savings and reduced deflections. A stub girder consists of a shallow wide-flange member directly beneath the floorbeams and intermittent wide-flange stubs having the same depth as the floorbeams. The stubs are placed perpendicular to and between the floorbeams, leaving space for the passage of mechanical ductwork (Fig. 7.21). The stubs are welded to the top of the stub girder and connect to the floor deck, which is typically concrete fill on metal deck, thereby enabling the stub girder to act compositely with the floor deck.

7.13 STAGGERED TRUSSES

In an effort to provide a structural-steel framing system with a minimum floor-to-floor height for multistory residential construction, the staggered truss system was developed. This system consists of story-high trusses spanning the full width of a building. They are placed at alternate column lines in alternate stories, thus resulting in a staggered arrangement of trusses (Fig. 7.22). The trusses span about 60 ft between exterior columns, resulting in a column-free interior space. In addition to the simple checkerboard pattern, alternative stacking patterns are possible in order to accommodate varied interior layouts (Fig. 7.23).

At a typical floor, the deck spans between the top chord of one truss and the bottom chord of the adjacent truss. Since the staggered trusses are typically spaced 20 to 30 ft on centers, a long-span floor deck system is required. Precast-concrete plank, with or without topping, is frequently used, since, in addition to accommodating the span, the plank underside can be finished to provide an acceptable ceiling. An alternative system consists of long-span composite metal deck, having a depth of up to $7^1/_2$ in, with concrete fill. The top and bottom chords of the trusses are usually wide-flange shapes to efficiently resist the bending stresses induced by the floor loadings.

Diagonal web members of the trusses are deleted at corridor openings. This results in bending stresses in the truss chords due to Vierendeel action. Consequently, corridors are typically located near the building centerline, that is, near midspan of the trusses, at points of minimum truss shear, thereby minimizing the chord bending stresses.

Lateral loads in the transverse direction are transferred to the truss top chords via diaphragm action of the floor deck. These loads are transmitted through the depth of the trusses to the bottom chords and are then transferred through the floor deck at that level to the adjacent-truss top chords. The overturning couple produced by the transfer of lateral load from the top chord to the bottom chord is resisted by a vertical couple at the ends of the truss. Only axial forces are induced in the exterior columns. Therefore, transverse lateral loads are transmitted down through the structure without creating bending stresses in the trusses or columns, except at truss openings.

In the longitudinal direction, lateral loads are transferred via floor diaphragm action to the exterior columns. These resist the loads by conventional means, such as rigid frames or braced bents. To provide added strength and stiffness, the exterior columns are usually oriented so that the strong axis assists in resisting lateral loads in the longitudinal direction.

To achieve the necessary structural interaction between the trusses and the floor deck and to provide the necessary continuity of the floor diaphragm, adequate connection by such means as weld plates or shear connectors must be provided between the various structural elements. Floor decks with large openings or other shear discontinuities may require additional reinforcement.

Although the staggered-truss system resists gravity and lateral loads primarily by axial stresses, consideration must be given to the bending stresses in the exterior columns that result from the truss

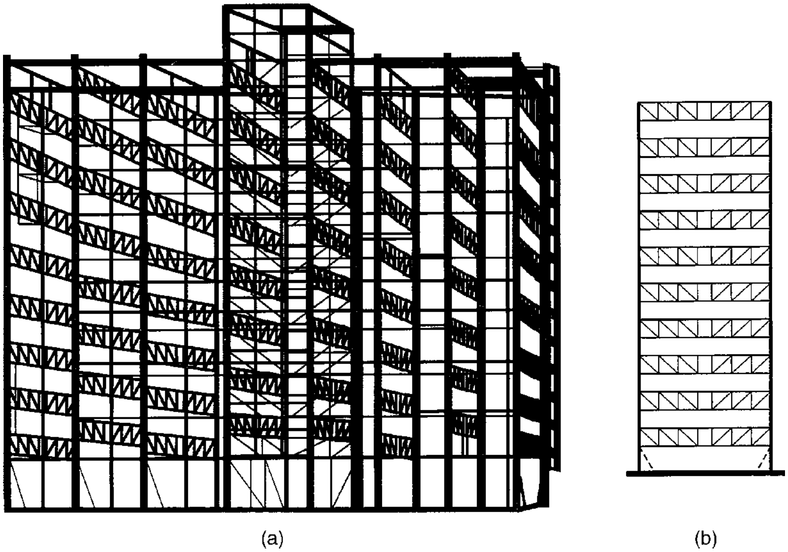


FIGURE 7.22 Staggered-truss system. (*a*) Story-high trusses are erected in alternate stories along alternate column lines. (*b*) Typical vertical section through building.

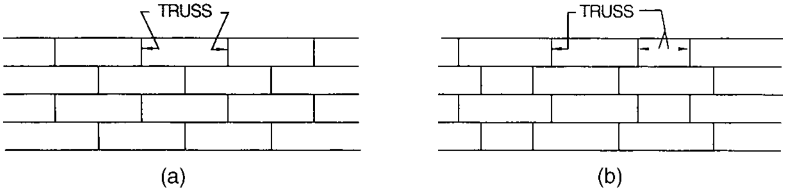


FIGURE 7.23 Stacking of trusses in staggered-truss systems. (*a*) Checkerboard pattern; (*b*) an alternative arrangement.

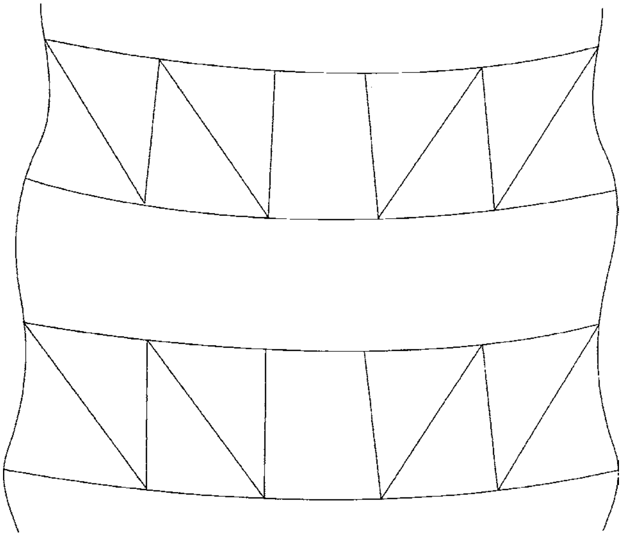


FIGURE 7.24 Deformations of staggered trusses induce bending in exterior columns.

deformations under gravity loads (Fig. 7.24). These bending stresses can be significantly reduced by cambering the trusses, thereby preloading the columns. An alternative is to provide slotted bottom-chord connections that are tensioned or welded after dead load is applied.

7.14 CASTELLATED BEAMS

A special fabrication technique is applied to wide-flange shapes to produce castellated beams. This technique consists of cutting the web of a wide-flange shape along a corrugated pattern, separating and shifting the upper and lower pieces, and rewelding the two pieces along the middepth of the newly created beam (Fig. 7.25). The result is a beam with hexagonal openings having a depth, strength, and stiffness greater than the original wide-flange shape, but that maintains the same weight per foot as the original wide-flange shape. A similar technique, with some additional trimming of the web, can be used to create a beam with round openings. The numerous openings, or castellations, that are formed in the beam web can accommodate mechanical ductwork, thereby reducing the overall floor depth.

Castellated beams can be designed to act compositely with the floor deck. Economical spans typically range from about 35 to 70 ft. For composite design, it is structurally more efficient to fabricate the beam from a heavier wide-flange shape for the lower portion than for the upper portion. As a rule of thumb, the deflection of a castellated beam is about 25% greater than the deflection of an equivalent beam with the same depth but without web openings, primarily due to increased shear deformations.

The load capacity of a castellated beam is frequently dictated by the local strength of the web posts and the tee portions above and below the openings. Therefore, these beams are more efficient for supporting uniform loadings than for concentrated loadings. The latter produce web-shear distributions that tend to be less favorable because the perforated web has less capacity than the solid web.

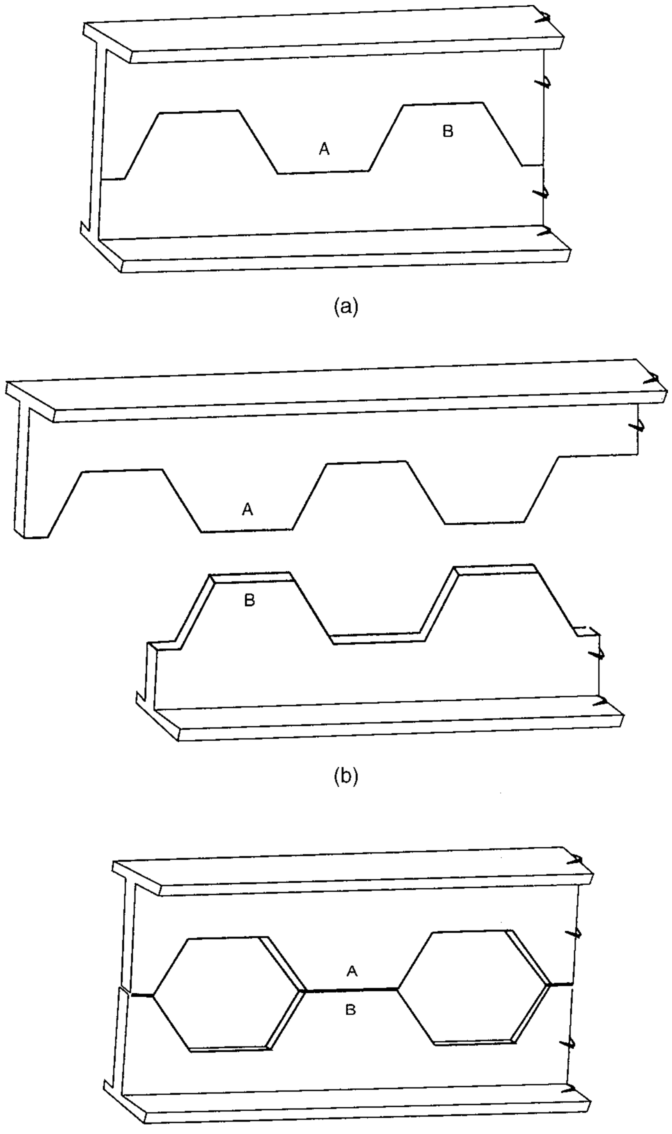
7.15 LRFD EXAMPLES FOR COMPOSITE FLOORS

Examples of composite beam and girder designs for the floors of typical interior bays of office building floor systems are shown in Fig. 7.26 (30-ft by 30-ft bay) and 7.27 (30-ft by 45-ft bay). The designs are based on the following:

- Beams and girders are ASTM A992 steel (50-ksi yield stress).
- Floor is 3-in, 20-ga composite steel deck with 3.25 in of lightweight concrete fill with a total weight of 47 lb/ft^2.
- Total dead load (floor slab, partitions, ceiling, and mechanical) is 77 lb/ft^2.
- Live load is 80 lb/ft^2, with live-load reductions in accordance with size of loaded areas supported (Art 4.4.3).
- Cambering compensates for approximately 75% of dead-load deflections.
- Live-load deflections are limited to 1/360 of the span.
- Percentage of full composite action is limited to 90.

For the 30-ft by 30-ft bay shown in Fig. 7.26, beams are W14 × 22 and girders are W18 × 40. However, the design shown would not meet acceptable vibration criteria for an open-office layout having a damping ratio of 2.5% (see Art. 7.18). This would require that the typical beam be increased from W14 × 22 to W16 × 26, and the typical girder be increased from W18 × 40 to W18 × 46. The steel weight would increase from 3.5 to 4.1 lb/ft^2.

For the 30-ft by 45-ft bay shown in Fig. 7.27, beams are W18 × 35 and girders are W21 × 50. The steel weight is 4.6 lb/ft^2. In this case, the design shown would meet acceptable vibration criteria for an open-office layout having a damping ratio of 2.5%.



(a)

(b)

(c)

FIGURE 7.25 Steps in formation of a castellated beam. (*a*) Corrugated cut is made longitudinally in a wide-flange beam. (*b*) Half of the beam is moved longitudinally with respect to the other half and (*c*) welded to it.

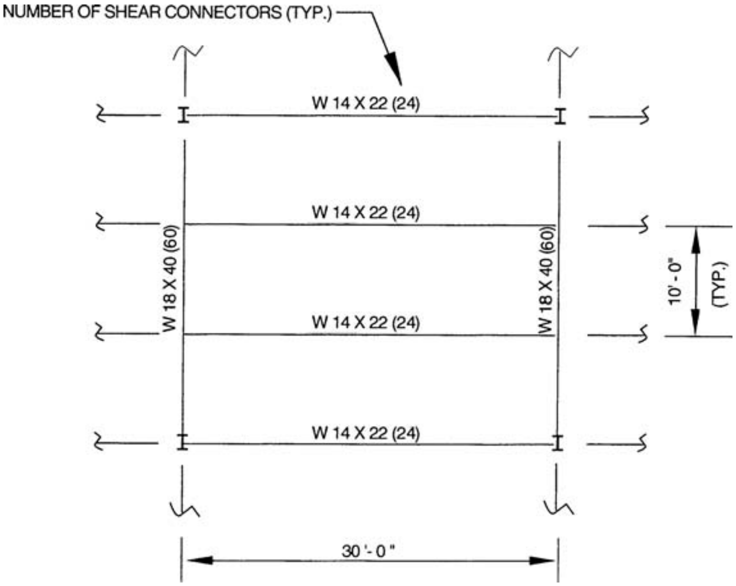


FIGURE 7.26 Sizes computed by LRFD for beams and girders for a 30-ft × 30-ft interior bay of an office building.

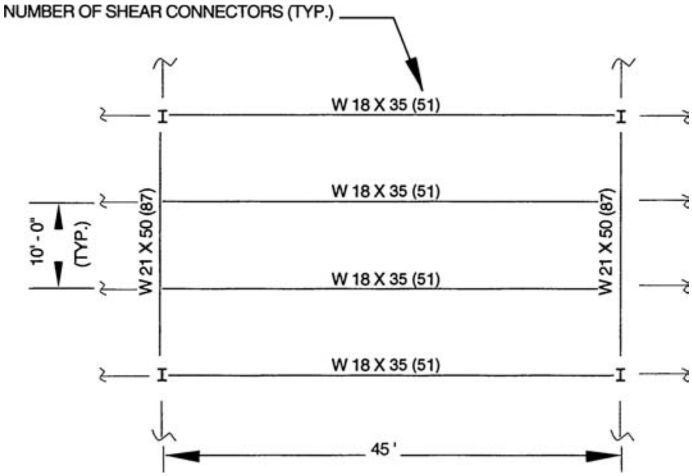


FIGURE 7.27 Sizes computed by LRFD for floor framing for a 30-ft × 45-ft interior bay of an office building.

7.16 DEAD-LOAD DEFLECTION

Although, in general, building codes restrict the magnitude of live-load deflections, they do not contain criteria or limitations relating to dead-load deflections.

The dead-load deflection of the floor framing system will not affect the levelness of the floor surface if the concrete is finished level despite the deflection or if the floor framing members are cambered for deflection due to the concrete dead load. In cases where the concrete is finished with a level surface, the slab will be thicker at midspan due to ponding and, hence, the floor framing members should be designed for the additional concrete dead load. In cases where the floor framing members are cambered, care must be taken to avoid providing too much camber. (See “Shored versus Unshored Construction” in Art 7.8.) Some designers find it prudent to establish a maximum camber limit, such as 1/240 of the span, above which a heavier or deeper beam should be selected in order to avoid potential serviceability problems.

When shored construction is used, or when the concrete floor thickness is kept constant, that is, the top surface follows the deflected shape of the framing members to avoid the placement of additional concrete, the dead-load deflection of the floor-framing system should be evaluated to determine whether the resulting floor levelness will be acceptable.

7.17 FIRE PROTECTION

There are several methods by which fire ratings can be readily achieved for structural-steel floor framing systems. These methods include application of spray-on fire protection, encasement of the framing members in a fire-rated assembly, or installation of a fire-rated ceiling system below the framing. For open-web joists and lightweight steel framing, the last two options are usually more practical because spray-on fire protection of such members tends to be difficult. (See also Art. 4.12.)

7.18 VIBRATIONS

Although a floor system may be adequately designed from a strength standpoint, a serviceability problem will result if unacceptable vibrations occur during normal usage of the floor. Perceptibility to vibrations is significantly affected by the amount of damping, or energy-dissipating capability, provided by the structural and nonstructural components of a floor system.

The anticipated performance of floor systems designed prior to the 1980s could be analyzed by computing the first natural frequency and the amplitude, that is, deflection when subjected to a heel-drop impact, of the floor framing member and plotting the result on a modified Reiher-Meister scale to determine the degree of perceptibility to vibrations. This method was generally accurate for concrete slab (including concrete fill on metal deck) floor systems framed with steel joists or steel beams. However, these floor systems typically had much higher damping than modern floor systems. For example, full-height partitions used in older office spaces resulted in damping ratios of 5–7% compared to modern open-office layouts, which result in damping ratios of only 2–3%. As a result, the modified Reiher-Meister scale is not recommended for the evaluation of modern floor systems with low damping characteristics, such as open-office layouts.

Modern floor systems can be evaluated for walking-induced vibration by using AISC Steel Design Guide Series No. 11, “Floor Vibrations Due to Human Activity.” Design Guide 11 provides recommended damping ratios and actual live loadings for various functional uses, and a procedure for computing the predicted acceleration of the floor system and comparing it to a tolerable acceleration. (See example in Art. 6.13.)

It is sometimes prudent to design for a more conservative condition, such as a lower damping ratio corresponding to an electronic office as opposed to a paper office, to account for a potential change in usage in the future. It is generally much more difficult and costly to rectify a vibration problem in an existing structure than to eliminate the problem in the original design.

ROOF FRAMING

The systems used for floor framing (Arts. 7.8 to 7.14) can also be used for roof framing. Other roof framing systems are described below.

7.19 PLATE GIRDERS

For long spans or heavy loadings that exceed the capacity of standard rolled shapes, plate girders can be used. Plate girders are composed of individual steel plates that can vary in width, thickness, and grade of steel along their length to optimize the cross section. However, it is important to recognize that a minimum weight design is not always the most cost effective design. For example, it is often more economical to use a thicker web plate, rather than a thinner one with multiple transverse stiffeners, because of the reduced fabrication costs. Also, the material savings obtained from splicing flange plates to change thickness may be offset by the cost of the welded splice. (See Art. 10.16.)

7.20 SPACE FRAMES

Space frames represent one of the more efficient uses of structural materials. Space frames are three-dimensional lattice-type structures that span in more than one direction. It is common practice to apply the "space frame" designation to structures that would more accurately be categorized as "space trusses," that is, assemblies of members pin-connected at the joints, or nodes.

In addition to providing great rigidity and inherent redundancy, space frames can span large areas economically, providing exceptional flexibility of usage within the structure by eliminating interior columns. Space frames possess a versatility of shape and form. They can utilize a standard module to generate flat grids, barrel vaults, domes, and free-form shapes.

The most common example of a space frame is the double-layer grid, which consists of top- and bottom-chord layers connected by web members. Various types of grid orientations can be utilized. Top- and bottom-chord members can be either parallel or skewed to the edges of the structure, and can be either parallel or skewed to one another (see Fig. 7.28). One of the advantages of having top and bottom chords skewed relative to one another is that the top-chord members have shorter lengths, thereby resulting in a more economical design for compressive forces. Also, the longer bottom chords have fewer pieces and connections.

Space frames spanning over large column-free areas are generally supported along the perimeter or at the corners. Overhangs are employed where possible to provide some amount of stress counteraction

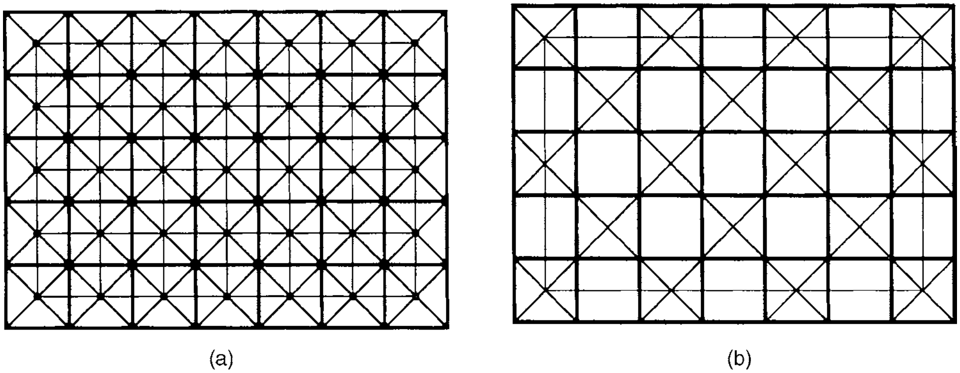


FIGURE 7.28 Types of space-frame grids. (*a*) Top and bottom chords parallel to edges of the structure. (*b*) Top and bottom chords skewed to each other.

to relieve the interior chord forces and to provide a greater number of "active" diagonal web members to distribute the reactions at supports into the space frame. In cases where the reactions are very large, space-frame members near the supports are sometimes extended beneath the bottom chord, in the form of inverted pyramids, to the top of the columns. This effectively produces a column capital, which facilitates distribution of forces into the space frame.

The depth of a space frame is generally 4–8% of its span. To effectively utilize the two-way spanning capability of a space frame, the aspect (length-to-width) ratio should generally not exceed 1.5:1.0. For a 1.5:1.0 ratio, about 70% of the gravity loads are carried by the short span.

Types of members used for space frames may be structural steel hot-rolled shapes, or round or rectangular tubes, or cold-formed steel sections. Many space frames are capable of utilizing two or more different member types.

For some space-frame roof structures, the top chords also act as purlins to directly support the roofing system. In these cases, the top chords must be designed for a combination of axial and bending stresses. For other roof structures, a separate subframing system is utilized for the roofing system, and an interface connection to the space frame is provided at the top chord nodes. In these cases, the roofing system does not transmit bending stresses to the top chord members.

Regardless of the type of space frame, the essence of any such system is its node. Most space frame systems have concentric nodes; that is, the centroidal axes of all members framing into a node project to a common working point at the center of the node. Some systems, however, have eccentric joints. For these, local bending of the members must be considered in addition to the basic joint and member stresses.

Most space frames are assembled either in-place on a piece-by-piece basis, or in portions on the ground and then lifted into place. In some cases, where construction sequencing permits, the entire space frame can be preassembled on the ground and then lifted into place.

7.21 ARCHED ROOFS

Arched roofs are advantageous for long bays, especially if large clearances are desirable along the center. Such braced barrel vaults have been used for hangars, gymnasiums, and churches. While these roofs can be supported on columns, they also can be extended to the ground, thus eliminating the need for walls (Fig. 7.29). The roofs usually are relatively lightweight, though spans are large, because they can be shaped so that load is transmitted to the foundations almost entirely by axial compressive stresses.

Designers have a choice of a wide variety of structural systems for cylindrical arches. Basically, they may be formed with structural framing of various types and a roof deck, or they may be of stressed-skin construction.

Framing may consist of braced arch ribs (Fig. 7.29*a*), curved grids, or space frames. Depending on foundation and other conditions, arch ribs may be fixed-end, single-hinged, double-hinged (pinned),

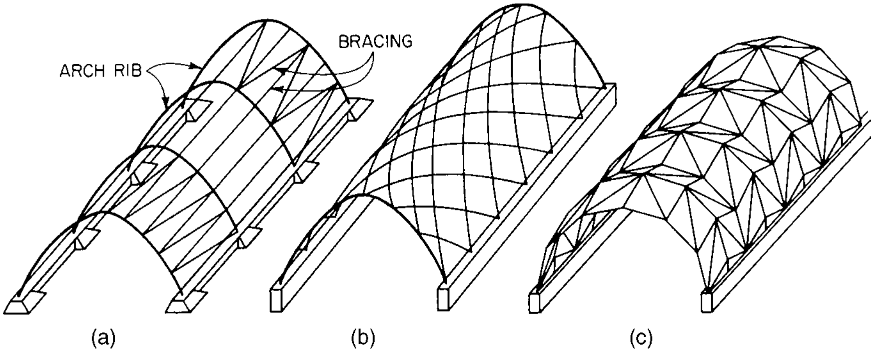


FIGURE 7.29 Cylindrical arches. (*a*) Ribbed; (*b*) diagonal grid (lamella); (*c*) pleated barrel.

or triple-hinged (statically determinate). Much lighter members can be employed for a diagonal grid, or lamella, system (Fig. 7.29*b*), but many more members must be handled.

With stressed-skin construction, the roof deck acts integrally with the framing in carrying the load. As in folded-plate construction, the stiffness can be increased by pleating or undulating the surface (Fig. 7.29*c*).

Regardless of the type of structural system selected, provision must be made for resisting the arch thrust. If ground conditions permit, the thrust may be resisted entirely by the foundations. Otherwise, ties must be used. Arches supported above grade may be buttressed or tied.

7.22 DOME ROOFS

Dome roofs are preferable to arches where the large column-free area to be covered is circular, elliptical, or approximately an equal-sided polygon. They often have been used for the roofs of exhibition buildings, arenas, planetariums, water reservoirs, and gas tanks. Also, the feasibility of covering large stadiums with domes has been demonstrated. Domes are relatively lightweight, despite long spans, because they can be shaped so that loads induce mainly axial stresses.

Domes may be readily supported on columns, without ties or buttresses, because they can be shaped to produce little or no thrust. For a shallow dome, a tension ring usually is provided around the base to resist thrusts. If desired, however, domes may be extended to grade, thus eliminating the need for walls (Fig. 7.30). If an opening is left at the crown, for example, for a lantern (Fig. 7.30*b*),

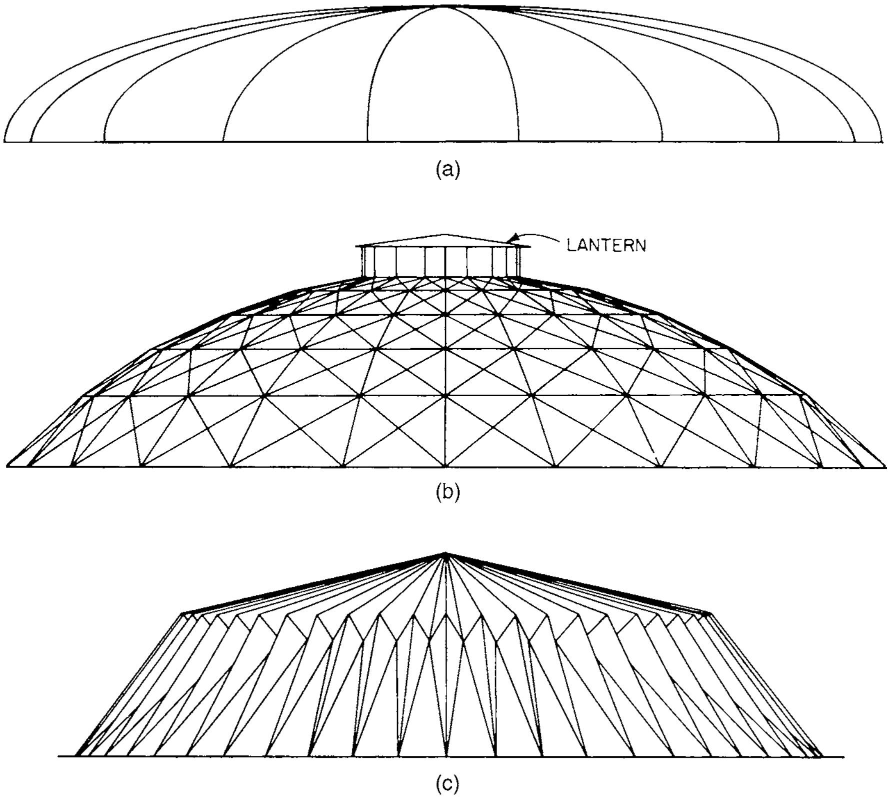


FIGURE 7.30 Steel-framed domes. (*a*) Arch rib; (*b*) Schwedler; (*c*) pleated rib.

a compression ring is installed around the opening to resist the thrusts. Also, if desired, portions of a dome may be made movable, to expose the building interior.

Designers have a choice of a wide variety of structural systems for domes. In general, dome construction may be categorized as single-layer framing (Fig. 7.30*a* and 7.30*b*); double-layer (truss) framing, or space frame, for greater resistance to buckling; and stressed skin, with the roof deck acting integrally with structural framing. Greater stiffness can be obtained by dimpling, pleating (Fig. 7.30*c*), or undulating the surface.

Figure 7.30*a* shows a ribbed dome. Its principal components are half arches. They are shown connected at the crown, but usually, to avoid a cramped joint with numerous members converging there, the ribs are terminated at a small-diameter compression ring circumscribing the crown. The opening may be used for light and ventilation. If the connections at the top and bottom of the ribs permit rotation in the plane of each rib, the system is statically determinate for all loads.

Figure 7.30*b* shows a Schwedler dome, which offers more even distribution of the dead load and reduces the unbraced length of the ribs. Principal members are the arch ribs and a series of horizontal rings with diameter increasing with distance from the crown. The ribs transmit loads to the base mainly by axial compression, and the rings resist hoop stresses. With simplifying assumptions, this system can also be considered statically determinate. For spherical domes of this type, an economical rise-span ratio is 0.13, achieved by making the radius of the dome equal to the diameter of its base.

7.23 CABLE STRUCTURES

High-strength steel cables are very efficient for long-span roof construction. They resist loads solely by axial tension. While the cables are relatively low cost for the load-carrying capacity provided, other necessary components of the system must be considered in making cost comparisons. Costs of these components increase slowly with increasing span. Consequently, the larger the column-free area required, the greater the likelihood that a cable roof will be the lowest-cost system for spanning the area.

Components other than cables that are needed are vertical supports and anchorages. Vertical supports are needed to provide required vertical clearances within the structure, because cables sag below their supports. Usually the cables are supported on posts, or towers, or on walls.

Anchorages are required to resist the tension in the cables. Means employed for the purpose include heavy foundations, pile foundations, part of the building (Fig. 7.31*a*), perimeter compression rings, and interior tension rings (Fig. 7.31*b*). For attachment to the anchorages, each cable usually comes equipped with end fittings, often threaded to permit a jack to grip and tension the cable and to allow use of a nut for holding the tensioned cable in place. In addition, bearing plates generally are needed for distributing the cable reaction.

Cable roofs may be classified as cable-stayed or cable-suspended. In a cable-stayed roof, the deck is carried by girders or trusses, which, in turn, are supported at one or more points by cables. This type of construction is advantageous where long-span cantilevers are needed, for example, for hangars (Fig. 7.31*a*). In a cable-suspended roof, the roof deck and other loads are carried directly by the cables (Fig. 7.31*b*).

The single-layer cable roof structure in Fig. 7.31*b* is composed of radial cables, a central tension ring, and a perimeter compression ring. Since this system is extremely lightweight, it is susceptible to wind uplift and wind-induced oscillations unless a heavy roof deck, such as precast-concrete panels, is utilized. Uplift and oscillation can be eliminated with the use of a double-layer cable roof (Fig. 7.31*c*) in which the primary and secondary cables are pretensioned during erection.

For a double-layer system with diagonal struts between the primary and secondary cables, truss action can be developed. If pretension is sufficiently high in the compression chord, compression induced by increasing load only decreases the tension in that chord but cannot cause stress reversal.

For both single- and double-layer systems, circular or elliptical layouts minimize bending in the perimeter compression ring and are thus more efficient than square or rectangular layouts.

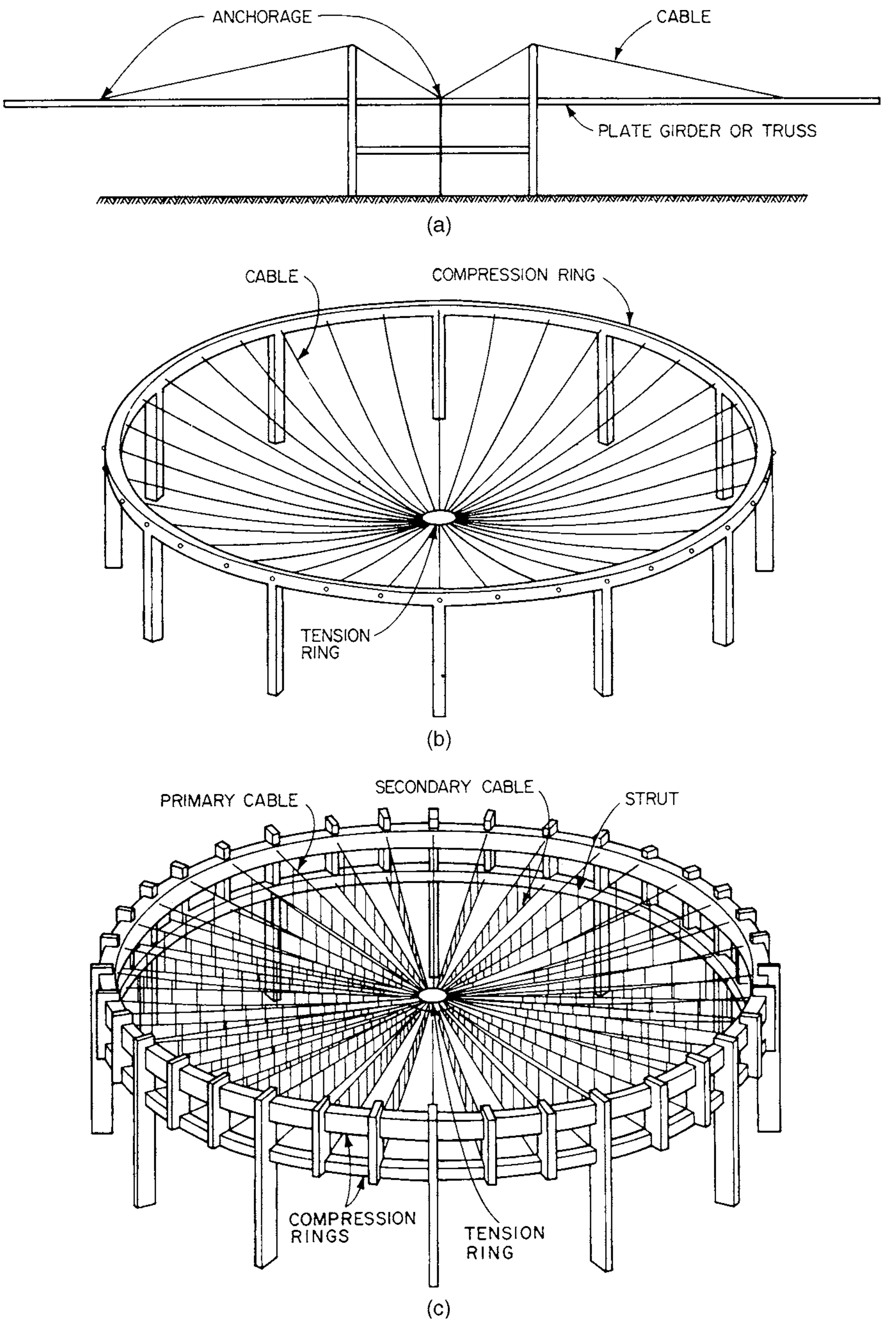


FIGURE 7.31 Cable roofs. (*a*) Cable-stayed cantilever roof; (*b*) single-layer cable-suspended roof; (*c*) double-layer cable-suspended roof.

Since the number of anchorages and connections does not increase linearly with increasing span, cable structures with longer spans can cost less per square foot of enclosed area than those with shorter spans. This is contrary to the economics of most other structural systems, which increase in cost per square foot of enclosed area as the span increases.

Another type of cable structure is the cable-truss dome, or “tensegrity” dome. It consists of a series of radial cable trusses, concentric cable hoops, a central tension ring, and a perimeter compression ring. The dome is prestressed during erection and is typically covered with fabric roofing.

Cable spacing depends on type of roof deck. Close spacing up to a maximum of 10 ft is generally economical.

For watertightness and to avoid potential problems due to roof movements at points where cables penetrate a roof, it is desirable to place cables either completely below or completely above the roof surface. If cables must penetrate a roof, the joints should be caulked and sealed with a metal-protected, rubber-like collar.

In design of cable roofs, special consideration should be given to roof movement, especially if the roof deck does not offer a significant contribution to rigidity. Care should be taken that joints in a flexible roof do not open or that a concrete deck does not develop serious cracks, destroying the watertightness of the roof. Insulation may be necessary to prevent large thermal movements. Consideration should also be given to fire resistance. Sprinklers may be required or desirable. If the cables are galvanized, corrosion usually is unlikely, but the possibility should be investigated, especially for chemically polluted atmospheres.

CHAPTER 8
LATERAL-FORCE DESIGN

Charles W. Roeder, Ph.D., P.E.
Professor of Civil Engineering
University of Washington
Seattle, Washington

Design of buildings for lateral forces requires a greater understanding of the load mechanism than many other aspects of structural design. To fulfill this need, this section provides a basic overview of current practice in seismic and wind design. It also discusses recent changes in design provisions and recent developments that will have an impact on future design.

There are fundamental differences between design methods for wind and earthquake loading. Wind-loading design is concerned with safety, but occupant comfort and serviceability is a dominant concern. Wind loading does not require any greater understanding of structural behavior beyond that required for gravity and other loading, although it is noted that complex, large, or aerodynamically sensitive structures frequently require wind-tunnel testing or more sophisticated dynamic analysis to assure occupant comfort during wind storms. As a result, the primary emphasis of the treatment of wind loading in this chapter is on the loading and the distribution of loading.

Design for seismic loading also is primarily concerned with structural safety during major earthquakes, but increasing emphasis is placed on economic loss and serviceability through performance-based design. These different design goals are achieved by permitting a range of different structural performance levels. During large infrequent seismic events, collapse prevention and life-safety performance limits are economically achieved by permitting large but controlled inelastic deformations of the structure. Inelastic deformation of the structure during severe earthquakes results in more detailed structural design requirements, which are needed to assure system ductility and performance. Therefore, discussion of seismic design also requires discussion of the inelastic behavior of steel structures and design requirements needed to achieve acceptable inelastic performance. Serviceability and economic loss limitations are assured by requiring smaller elastic deformations during the appropriate design events. As a consequence of these differences, seismic design requires a more detailed understanding of elastic and inelastic dynamic analysis and evaluation of a wider range of structural behaviors than are required for most other design loads.

Refer to Chap. 4 for further information on wind and seismic loadings.

8.1 DESCRIPTION OF WIND FORCES

The magnitude and distribution of wind velocity are the key elements in determining wind design forces. Mountainous or highly developed urban areas provide a rough surface, which slows wind velocity near the surface of the earth and causes wind velocity to increase rapidly with height above the earth's surface. Large, level open areas and bodies of water provide little resistance to the surface wind speed, and wind velocity increases more slowly with height. Wind velocity increases with height in all cases but does not increase appreciably above the critical heights of about 950 ft for open terrain to 1500 ft for rough terrain. This variation of wind speed over height has been modeled as a power law:

$$V_2 = V\left(\frac{z}{z_g}\right)^n \tag{8.1}$$

where V is the basic wind velocity, or velocity measured at a height z_g above ground and V_z is the velocity at height z above ground. The coefficient n varies with the surface roughness. It generally ranges from 0.33 for open terrain to 0.14 for rough terrain. The wind speed used in this evaluation procedure has varied over time. Early wind-load predictions were based on fastest-mile wind speeds, which are effectively the maximum average wind speed measured over a distance of 1 mile, at a given height above ground. (ASCE Task Committee on Wind Forces, Committee on Loads and Stresses, "Wind Forces on Structures," *Transactions, ASCE*, vol. 126 part 2, pp. 1124–1198, 1961.) Current structural design codes use a 3-s-gust wind speed for their wind design requirements. ("Minimum Design Loads for Buildings and Other Structures," SEI/ASCE Standard 7-02, American Society of Civil Engineers, Reston, Va., 2002.) While there is variation in the definition of the basic wind speeds that have been used in wind-load estimation, there is clearly a relationship between the various definitions. However, it is important to note that wind speeds established by different definitions should not be arbitrarily combined or compared. Design loads are based on a statistical analysis of the basic wind speed, and maps such as the one shown in Fig. 4.1 have been developed. Typical design limits may be based on maximum wind speeds with 2% annual probability of exceedance or approximately a 50-year event, but winds associated with higher- or lower-probability occurrences may be appropriate for some structures. The statistical wind-speed design maps normally exclude the occurrence of tornadoes and hurricanes. Further, extreme local variations in wind speed are possible in some regions because of climatic and geographic variations. As a result, wind-speed design maps typically require additional consideration of these events. The wind-speed design data are normally maintained for open sites, and the wind speeds must be corrected for other site conditions.

Wind speeds V_w are translated into pressure q by the equation

$$q = C_D \frac{\rho}{2} V_w^2 \tag{8.2}$$

where C_D is a drag coefficient and ρ is the density of air at standard atmospheric pressure. The drag coefficient C_D depends on the shape of the body or structure. It is less than 1 if the wind flows around the body, but it may be significantly greater than 1.0 if the wind is forced to reverse its direction. The pressure q is the stagnation pressure q_s if $C_D = 1.0$, since the structure effectively stops the forward movement of the wind. Thus, on substitution in Eq. (8.2) of $C_D = 1.0$ and air density at standard atmospheric pressure,

$$q_s = 0.00256 V_w^2 \tag{8.3}$$

where the wind speed is in mi/h and the pressure is in lb/ft^2.

The drag coefficient and the shape and geometry of the structure have substantial effects on wind pressure, because the shape of the body may merely divert the direction of the wind, stop the wind, or reverse its direction. These characteristics are illustrated in Fig. 8.1. Large inward pressures develop on the windward walls of enclosed buildings as illustrated in Fig. 8.1*a*. Negative pressures may develop on the leeward side of these enclosed buildings, and this may result in an additional outward pressure on the leeward walls of the structure. Buildings with openings on the windward side will allow air flow into the building, and internal pressures may develop as depicted in Fig. 8.1*b*. These internal pressures cause loads on the overall structure and structural frame. More important, these internal pressures place great demands on the attachment of roofing, cladding, and other nonstructural elements. Openings in a side wall or leeward wall may cause an

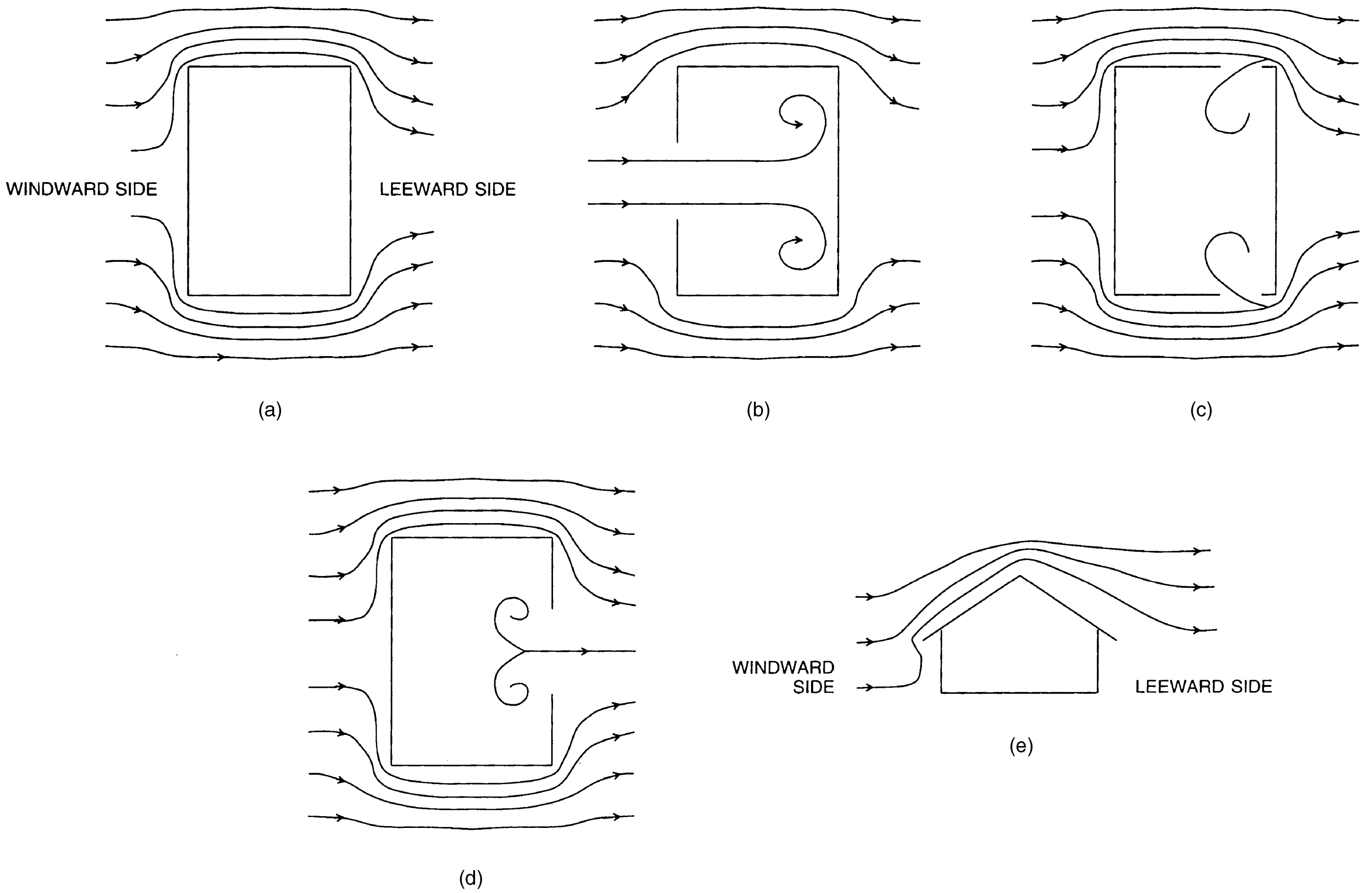


FIGURE 8.1 Plan view of a building indicating the wind loading on it with changes in velocity and direction of wind. (*a*) High pressure on a solid wall on the windward side but outward or reduced inward pressure on the leeward side. (*b*) Wind entering through an opening in the windward wall induces outward pressure on the interior of the walls. (*c*), (*d*) Wind entering through openings in a side wall or a leeward wall produce internal pressures in the building. (*e*) On a sloping roof, high inward pressure develops on the windward side, outward or reduced inward pressure on the leeward side.

internal pressure or a pressure reduction as illustrated in Fig. 8.1*c* and *d*. This internal pressure change depends on the size of the openings for all walls and the geometry of the structures. Slopes of roofs may affect the pressure distribution as illustrated in Fig. 8.1*e*. In general, downward or inward pressures are expected on the windward side, while upward or outward pressure differential may occur on the leeward side of the roof. Projections and overhangs (Fig. 8.2) may also restrict the air flow and accumulate local pressure increases. These local pressures may be very large, and must be considered in design.

Wind speed varies widely with time because of gusting and other short-duration effects. The fastest-mile wind speed is smaller than the short-duration wind speed due to this gusting effect. Historically, gust-factor corrections are made to the fastest-mile wind speed to account for this effect. Gust factors are affected by the roughness of the terrain, and they decrease with increasing height. Gusts are of short duration, but they may cause dynamic vibration or buffeting of the structure.

The velocity used in the pressure calculation is the velocity of the wind relative to the structure. Thus, vibrations or movements of the structure occasionally may affect the magnitude of the relative velocity and pressure. Structures with vibration characteristics which cause significant changes in the relative velocity and pressure distribution are regarded as sensitive to aerodynamic effects. They may be susceptible to dynamic instability due to vortex shedding and flutter. These may occur where local airflow around the structure causes dynamic amplification of the structural response because of the interaction of the structural response with the air flow. These undesirable conditions require special analysis that takes into account the shape of the body, air flow around the body, dynamic characteristics of the structure, wind speed, and other related factors.

Other structures may be aerodynamically stable, but the structural vibration due to wind load may be sufficiently large that occupant comfort becomes a dominant concern. These structures may require consideration of the dynamic interaction of the wind and the building structure. More sophisticated dynamic analysis is needed, and wind-tunnel testing is commonly employed. As a result, structures with dynamic instability or wind-induced vibrations that cause occupant discomfort require higher levels of design and analysis. The discussion on wind-load design in this section will focus on simplified methods of design, which are appropriate for average, modest-sized structural systems.

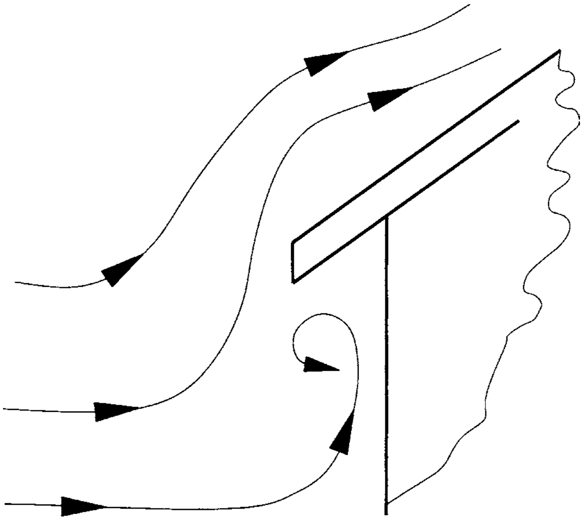


FIGURE 8.2 Roof overhang restricts airflow, creates large local forces on the structure.

8.2 DETERMINATION OF WIND LOADS

Wind loading as described in Art. 8.1 is the basis for design wind loads specified in American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-02. (The 2005 version of this specification was not available at the time of preparation of this chapter, but only modest changes in the wind-load provisions are expected.) The International Building Code (IBC) is the standard model building code commonly used for buildings in the United States, and the IBC currently adopts ASCE 7 wind provisions by reference. Thus, the wind-load discussion herein will focus on the ASCE 7 design provisions. ASCE 7-02 provides three methods for establishing design wind loads. Method 1 is a simplified procedure, which can be applied to relatively simple low-rise buildings of regular shape and geometry that are not susceptible to wind-induced vibration. Method 2 may be applied to buildings which are aerodynamically stable but that may have limited vibration and discomfort issues. This method is analytically more complex, and it requires greater consideration of the dynamic response of the building and the local variation of wind pressures around the building. Method 3 employs wind-tunnel testing. This method must be used for any buildings that have irregular characteristics, extreme local variation in wind velocity, or may be susceptible to aerodynamic instability or excessive vibration. Many larger buildings may employ Method 3 for the economic advantage that may be achieved with greater certainty regarding the local wind loading. All three methods have similar considerations of wind pressure and local variations of the wind pressure, but Methods 2 and 3 are more complex. The primary discussion of wind-load design provisions will focus on Method 1 of the ASCE 7-02 provisions.

8.2.1 ASCE 7-02 Method 1 Provisions

The basic wind speeds specified in the ASCE 7-02 provisions are shown in Fig. 4.1. The contours on the map indicate wind speeds that have a 2% probability of being exceeded in a year at a height of 10 m (33 ft) above ground at an open site. The wind speed includes estimated local gust velocity effects. (These wind speeds are approximately the maximum wind speed expected during an average 50-year period.) The effects of extreme conditions, such as tornadoes, hurricanes, or local wind currents in mountainous regions, are not covered by this map. Further, special wind regions are identified in the map where local wind velocity may significantly exceed the indicated values for the location. The possibility of occurrence of these local variations should be considered in design.

As noted in the background discussion, wind pressure depends on wind velocity, and the wind velocity varies with height and the characteristics of the local terrain. In addition, large variations in local wind velocity and pressure are noted on specific locations of the building as summarized in Figs. 8.1 and 8.2. As a result, the ASCE 7-02 Method 1 wind-design pressures are established by the equation

$$p_s = \lambda I p_{s30} \tag{8.4}$$

The simplified design wind pressure, p_{s30}, is effectively computed by relationships such as Eqs. (8.1) and (8.3). Also, values are tabulated for various wind velocities, locations on the building, and wind exposure conditions in the ASCE provisions (see Fig. 4.2). The importance factor, I, is based on the building category assignments and the characteristics of the wind velocity used in design. (See Table 4.9.) Regions with hurricane wind velocities greater than or equal to 100 mi/h are considered to be high-wind-velocity regions. The factor λ is an adjustment factor for building height and exposure. (See inset table in Fig. 4.2.) The factor accounts for the fact that the wind velocity increases with building height and is a function of the local terrain, as suggested by theory in Eq. (8.1). Exposure B is obstructed terrain typical of wooded areas or suburban or urban areas. Exposure C represents moderate terrain with scattered obstructions, and Exposure D is a smooth open terrain.

The simplified Method 1 procedure can be applied to simple, regular, modest-sized buildings. By this method, maps such as those shown in Fig. 4.1 establish the basic wind speed for the region. The importance factor I of the building must be established based on the function and characteristics of the building. (See Table 4.9.) The adjustment coefficient λ is determined as a function of height and exposure (see Fig. 4.2). Finally, the simplified design wind pressure, p_{s30}, is determined from the wind velocity. Different p_{s30} values are computed for a wide range of structural geometries, and for different locations on the structure and wind velocities as shown in Fig. 4.2. This determination is based on the variation in the direction and influence in wind velocity as illustrated in Figs. 8.1 and 8.2.

The Method 1 procedure requires that the wind-force design be accomplished in each of the primary structural directions as depicted in Fig. 4.2. In addition, two separate load cases may be required in each of the two directions based on the geometry of the structure. Figure 4.2 shows the direction of wind pressure on the structure for the longitudinal and transverse building axes, and it defines the pressure at different locations such as the windward and leeward areas and the walls and roof of the structure. Values of p_{s30} are established for each of these regions. Some locations may have wind pressures acting in either direction. For building geometries which may develop wind pressures in either direction, two separate values are specified for these regions of the building. Separate load cases are then required to accommodate the two different pressure conditions. The pressure distributions defined in Eq. (9.4) and Fig. 4.2 are used to establish the design wind loads on the walls, roof, and structural framing.

The Method 1 procedure then requires an additional local pressure, p_{net}, which is used to design local components and cladding. This pressure is defined by the equation

$$p_{\text{net}} = \lambda I p_{\text{net30}} \tag{8.5}$$

where the values of λ and I are established as described earlier. The pressure, p_{net30}, is defined in terms of interior zones, end zones and corner zones in ASCE 7-02. (See Fig. 4.3.) The three zones of behavior capture the extreme local variation in wind pressure caused by building geometry as noted in Fig. 8.1.

8.2.2 ASCE 7-02 Methods 2 and 3

As noted earlier, the engineer has the option of using Methods 2 and 3 for defining the design wind loads on the building. These methods will frequently offer some economic advantage through reduced wind forces, but they require significantly more complex analysis and evaluation of the wind load. Method 2 is also an analytical method. It uses the same basic wind speed, pressure variations, and locations or zones of the structure. However, the local wind pressure variation is computed based on a combination of pressure coefficients, which are computed based on the specific design parameters. These coefficients require considerable time and evaluation, but they are expected to lead to a more realistic estimate of the design wind pressures on the various components of the building structure. This method is required for some buildings with greater complexity and greater dynamic sensitivity to wind load. They may be used for very simple buildings, but they require additional time and effort for evaluation of the design wind loads.

Method 3 is based on wind-tunnel testing. Wind-tunnel testing is essential for buildings with great dynamic sensitivity to wind-induced vibration and for major buildings in urban areas where the local wind pressures are strongly influenced by surrounding structures. The method clearly gives a far better indication of the response of the building due to wind loading, but requires the time and cost of the wind-tunnel test.

8.3 SEISMIC LOADS IN MODEL CODES

The Uniform Building Code (UBC) of the International Conference of Building Officials has been the primary source of seismic design provisions for the United States in past years. The UBC historically adopted provisions based on recommendations of the Structural Engineers Association of California (SEAOC). The UBC and SEAOC defined design forces and established detailed requirements for seismic design of many structural types. However, another document, "National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings," of the Building Seismic Safety Council (BSSC), Federal Emergency Management Agency (FEMA), Washington, D.C., has maintained a parallel set of seismic load provisions since the 1970s. There has been considerable similarity between the UBC and NEHRP recommendations, since the rationale is similar for both documents and many engineers participate in the development of both documents. However, there have also been differences in the detailed approach used by the UBC and NEHRP provisions. In recent years, efforts have been made to resolve these differences. Today, structural engineers have moved toward a unified national code, which is the International Building Code (IBC) maintained by the International Code Council, Falls Church, Virginia. The IBC today bases its seismic provisions on the NEHRP provisions, and it effectively adopts these provisions by reference to the "Minimum Design Loads for Buildings and Other Structures," ASCE 7-02. Today this is the primary method for establishing earthquake design forces for buildings. The ASCE 7-05 seismic design provisions will be based on the 2003 NEHRP provisions.

Seismic design is based on the concept of permitting significant inelastic deformations during large infrequent earthquakes while preventing building collapse and loss of life for building occupants. Careful detailing requirements are needed to assure this structural ductility and inelastic performance. The American Institute of Steel Construction (AISC) promulgates "Seismic Design Provisions for Structural Steel Buildings (2005)" to assure satisfactory inelastic performance. This document provides detailed design requirement for steel structures, which are then used in conjunction with seismic design forces and deformations provided in the applicable building code, or ASCE 7 standard. ASCE 7-05 was not available at the time of preparation of this chapter. Therefore, the seismic load procedures described here are based on the 2003 NEHRP provisions, which will provide the basis for ASCE 7-05, and AISC 2005 seismic provisions for structural detailing.

8.4 SEISMIC DESIGN LOADS

The NEHRP and ASCE 7 provisions offer two methods for determining and distributing seismic design loads. One is the dynamic method, which is required to be used for a structure that is irregular or of unusual proportions. The other specifies equivalent static forces and is the most widely used, because of its relative simplicity. The methods are based on the equal displacement hypothesis as depicted in Fig. 8.3. This figure shows that the maximum elastic base shear and displacement that is

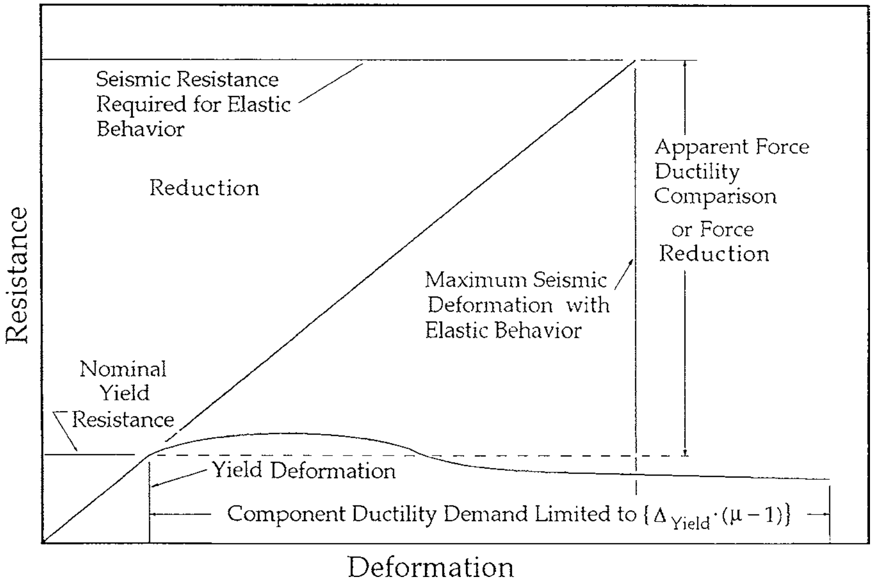


FIGURE 8.3 Schematic of the equal displacement hypothesis.

predicted for a building for a given earthquake acceleration. The structure is initially elastic, and so the linear-elastic force-deflection behavior is plotted in the figure. Large infrequent earthquakes require large seismic design forces for the elastic condition, and it is more economical to design a building that provides reasonable ductility but significantly reduced building resistance. This nominal building performance is plotted as a pushover force–deflection curve in the figure. The equal-displacement hypothesis postulates that the maximum inelastic displacement will be no larger than the maximum displacement expected if the structure remains elastic. This hypothesis has been examined by numerous research studies, and it has been shown to be generally valid if the building is not excessively weak, does not have a very short period, and does not exhibit poor inelastic performance. As a consequence of this observation, seismic design provisions establish reduced seismic design forces and ductile detailing requirements. The design procedure gives all appearances of being a force-based design method, but it is really an inelastic deformation design method based on the equal-displacement hypothesis combined with the detailing requirements of the AISC Seismic Design Provisions.

Seismic Base Shear. The equivalent static force method is the more commonly used method and is described here. The dynamic method is discussed in greater detail in Art. 8.5. The equivalent static force method defines the static shear, V, at the base of the building as

$$V = C_s W \tag{8.6}$$

where W is the total dead load, including permanent equipment, plus a minimum of 10 lb/ft^2 for partition loads, snow loads exceeding 30 lb/ft^2, and at least 25% of floor live loads in storage and warehouse occupancies. The base is the level at which seismic motions are imparted to the building. The seismic response coefficient, C_s, is

$$C_s = \frac{S_{DS} I}{R} \tag{8.7a}$$

but C_s need not exceed

$$C_s = \frac{S_{D1} I}{TR} \tag{8.7b}$$

and C_s should not be less than

$$C_s = 0.044 I S_{DS} \tag{8.7c}$$

In these equations, I is the importance factor for the building, T is the fundamental period of the building, and R is the response modification factor, which is assigned based on the perceived ductility of the structural system. Steel structural systems can offer significant ductility, and therefore R values between 5 and 8 are common. S_{D1} and S_{DS} are the design spectral response acceleration at 1-s period and in the short-period range, respectively. Equations (8.7a) and (8.7b) combine to provide a basic design spectrum as shown in Fig. 8.4. However, the design spectrum may be reduced for very-short-period structures as shown in the figure. The NEHRP provisions divide structures into Seismic Design Categories A through E. The division is based on the importance factor of the building and the severity of the design earthquake ground motion at the site. For structures in Seismic Design Categories E and F, C_s should not be less than

$$C_s = \frac{0.5 S_1 I}{R} \tag{8.7d}$$

The fundamental period of the structure, T, may be computed by a dynamic analysis or by approximate equations such as

$$T_a = C_r\, h_n^x \tag{8.8}$$

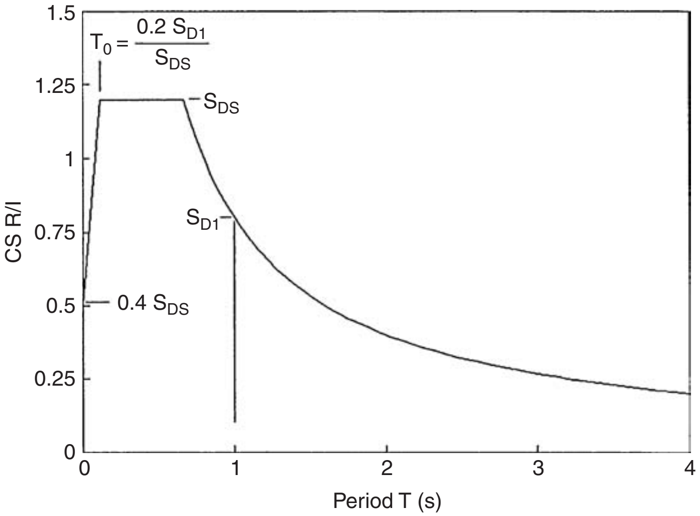


FIGURE 8.4 Resulting seismic-design response spectrum.

where h_n is the height, ft, from the base of the building to level n, which is the uppermost level in the main portion of the structure. The parameter C_r and the exponent x depend on the structural type. C_r varies between 0.016 and 0.03 when h_n is in dimensions of feet, and x varies between 0.76 and 0.9. Equation (8.8) yields periods that are shorter than those computed for some steel-frame structures by an elastic dynamic analysis. Hence, when T is computed from the structural properties and deformation characteristics of the resisting elements, the period used in Eq. (8.7) for determining the seismic base shear cannot exceed the predictions of Eq. (8.8) by more than 40–70%, depending on the design spectral acceleration for the building site. If the computed period exceeds this value, the limiting period is used to establish the seismic design forces. This places an upper-bound limit on the period and a lower-bound limit on the seismic design forces for a given building type.

The design spectral response accelerations, S_{D1} and S_{DS}, depend on the soil conditions and seismicity at the building site. The basic seismicity depends on the maximum considered earthquake ground-motion response spectra, S_1 and S_S, at the 1-s period and short-period zone. These maximum considered response spectra values (see Fig. 8.4) are determined from maps such as those illustrated in Figs. 4.4 and 4.5, and they are generally approximated as the maximum response spectra with a 2% probability of exceedance in 500 years. These maximum considered values are adjusted for the soil site conditions by the soil site coefficients, F_a and F_v, which depend on the soil site class and the site earthquake spectral acceleration. In general, soft soils have significantly larger soil site coefficients than stiff soils, but the increase is greater for sites with lower expected accelerations than sites with larger acceleration levels. Therefore,

$$S_{MS} = F_a S_S \tag{8.9a}$$

and

$$S_{M1} = F_v S_1 \tag{8.9b}$$

The design response spectra values, S_{D1} and S_{DS}, are then determined by arbitrarily using two-thirds of these site values. That is,

$$S_{DS} = \frac{2}{3} S_{MS} \tag{8.10a}$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \tag{8.10b}$$

The coefficient R in Eq. (8.7) reduces the seismic design forces in recognition of the ductility achieved by the structural system during a major earthquake. A measure of the ductility and inelastic

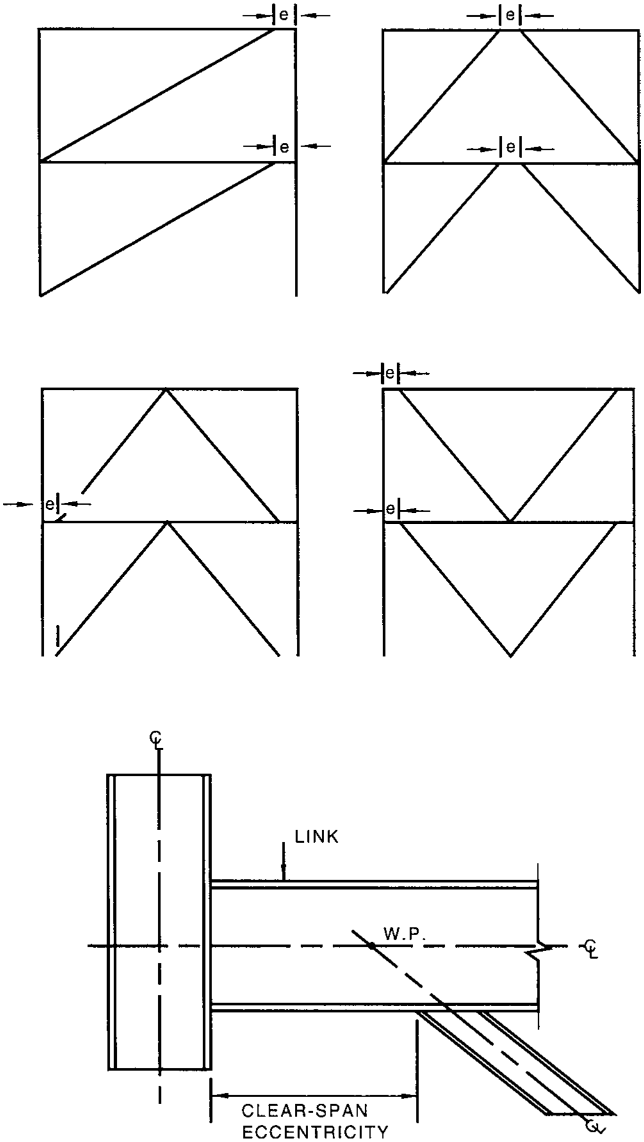


FIGURE 8.5 Typical configurations of eccentric braced frames. See also Fig. 8.15.

behavior of the structure, R ranges from 1.25 to 8. The largest values of R are used for ductile structural systems that can dissipate large amounts of energy and can sustain large inelastic deformations. The smallest values are intended to assure nearly elastic behavior when the overstrength normally achieved in design is considered.

Special steel moment-resisting frames have historically been regarded as one of the most ductile structural systems and are assigned $R = 8$. Moment-resisting steel frames are three-dimensional frames in which the members and joints are capable of resisting lateral forces on the structure primarily by flexure. While this structural system is still highly regarded, the performance of special-moment frames during the January 17, 1994, Northridge earthquake raised serious questions as to the performance of these structures, and this will be discussed in some detail in Art. 8.6. Ordinary moment frames are designed to less stringent ductility criteria, and $R = 3.5$.

For steel eccentric braced frames (Fig. 8.5), at least one end of each diagonal brace intersects a beam at a point away from the column–girder joint or from an adjacent brace–girder joint. This eccentric intersection forms a link beam, which must be designed to yield in shear or bending to prevent buckling of the brace. This system is also quite ductile, and values as large as $R = 8.0$ are also permitted.

Concentrically braced frames (Fig. 8.6) have concentric joints for brace, beam, and column, and the inelastic seismic behavior is dominated by buckling of the brace. The ductility achieved with buckling systems is limited because brace fracture may occur during the inelastic deformation, and as a result $R = 5$ for these ordinary concentrically braced systems. Fracture of the brace is less likely to occur if the connections of the system are designed to avoid deterioration and fracture during brace buckling. As a result, for special concentrically braced frames with these enhanced details and connections, $R = 6$.

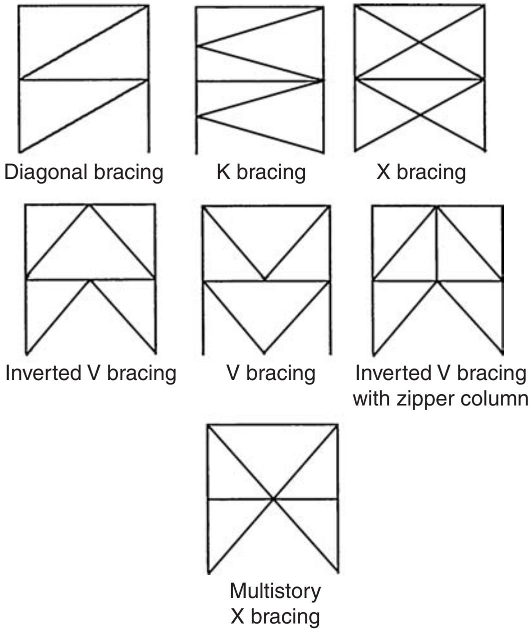


FIGURE 8.6 Typical configurations of concentric braced frames.

Finally, dual systems have improved inelastic performance over the individual systems acting alone. These include special steel moment-resisting frames capable of resisting at least 25% of the design base shear, V, combined with steel eccentric braced frames, special concentrically braced frames, or ordinary braced frames. Such dual systems are consistently permitted larger R values, because of the improved inelastic performance anticipated with the added moment-resisting connections.

Force Distribution. The seismic base shear V, Eq. (8.6), is distributed throughout the structure in accordance with its mass and stiffness. This is accomplished by distributing the forces to individual stories over the building height by the equation

$$F_x = C_{vx}V \tag{8.11a}$$

where

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{8.11b}$$

F_x and w_x are the seismic force and floor weight at the xth level, and h_x is the height from the base to the ith floor. The coefficient, k, varies between 1 and 2, and k is equal to 1.0 for short-period structures and increases to 2.0 for structures with periods longer than 2.5 s. The larger values of k cause larger story forces in the upper floors of a building, and this accounts for contributions of higher modes that are expected with longer-period structures. The force F_x at each floor is distributed horizontally in proportion to the distribution of the mass of the floor. The stiffness of the floor diaphragm must be evaluated to determine whether the diaphragm satisfies the rigid or flexible diaphragm stiffness requirements. With rigid diaphragms, the horizontal forces are distributed to vertical frames with consideration of the horizontal mass distribution (including minimum torsion) and the relative stiffness of the frames. With flexible diaphragms, the horizontal forces are distributed to vertical frames with consideration of the mass distribution and the tributary area of each frame. Floor slabs and their attachments between floor diaphragms and lateral load frames must have adequate strength to distribute these inertial forces. The frames must be designed for a minimum torsion that is produced by a mass eccentricity of 5% of the normal maximum base dimension. This minimum is in addition to torsion due to the computed eccentricity between the centers of mass and gravity.

Deflections and Element Design Forces. Story drifts and element forces must be adjusted to account for P–Δ effects where appropriate. Basic elastic deflections for the seismic design forces, δ_{xe}, are computed by performing an elastic analysis on the structure. These deflections do not represent the seismic story drifts and deformations expected during the design earthquake because they do not consider the inelastic deformation that occurs. These inelastic story drifts, δ_x, are then estimated by the equation

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{8.12a}$$

where C_d is the deflection amplification factor and I is the importance factor. C_d is related to the R factor, but it is invariably smaller than R because of the overstrength that is inherent in the structural design process. Calculation of a stability coefficient, θ, is required:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{8.12b}$$

where P_x is the total vertical design load at and above the level x, h_{sx} is the story height below level x, V_x is the shear force acting between levels x and $x - 1$, and Δ is the design story drift. If θ is greater than 0.1, the forces and deformations of the frame must be adjusted for P–Δ effects by a rational analysis method.

8.5 DYNAMIC METHOD OF SEISMIC LOAD DISTRIBUTION

The equivalent static-force method (Art. 8.4) is based on a single-mode response with approximate load distributions and corrections for higher-mode response. These simplifications are appropriate for simple, regular structures. However, they do not consider the full range of seismic behavior in complex structures. The dynamic method of seismic analysis is required for many structures with unusual or irregular geometry, since it results in distributions of seismic design forces that are consistent with the distribution of mass and stiffness of the frames, rather than arbitrary and empirical rules. Irregular structures include frames with any of the following characteristics:

- The lateral stiffness of any story is less than 70% of that of the story above or less than 80% of the average stiffness for the three stories above, that is, soft stories.
- The mass of any story is more than 150% of the effective mass for an adjacent story, except for a light roof above.
- The horizontal dimension of the lateral-force-resisting system in any story is more than 130% of that of an adjacent story.
- The story strength is less than 80% of the story above.
- The in-plane offset of the lateral-force-resisting elements is greater than the length of these elements.

Frames with horizontal irregularities place great demands on floors acting as diaphragms and the horizontal load-distribution system. Special care is required in their design when any of the following conditions exist:

- The maximum story drift due to torsional irregularity is more than 1.2 times the average story drift for the two ends of the structure.
- There are reentrant corners in the plan of the structure with projections more than 15% of the plan dimension.
- The diaphragms are discontinuous or have cutouts or openings totaling more than 50% of the enclosed area or changes in effective diaphragm stiffness of more than 50%.
- There are discontinuities in the lateral-force load path.

Irregular structures commonly require use of a variation of the dynamic method of seismic analysis, since it provides a more appropriate distribution of design loads. Many of these structures should also be subjected to a step-by-step dynamic analysis (linear or nonlinear) for specific accelerations to check the design further. Nonlinear pushover analyses are also used with increasing frequency to evaluate the inelastic behavior and inelastic seismic design in irregular or unusual structures.

The dynamic method is based on equations of motion for linear-elastic seismic response. The equation of motion for a single-degree-of-freedom system subjected to a seismic ground acceleration a_g may be expressed as

$$m\frac{d^2x}{dt^2} + c\frac{dx}{dt} + kx = -ma_g \tag{8.13}$$

where d^2x/dt^2 is the acceleration of the structure, dx/dt is the velocity relative to the ground motion, and x is the displacement from an equilibrium position. The coefficients m, c, and k are the mass, damping, and stiffness of the system, respectively. Equation (8.13) can be solved by a number of methods.

The maximum acceleration is often expressed as a function of the fundamental period of vibration of the structure in a response spectrum. The response spectrum depends on the acceleration record. Since response varies considerably with acceleration records and structural period, smoothed response spectra are commonly used in design to account for the many uncertainties in future earthquakes and actual structural characteristics.

Most structures are multidegree-of-freedom systems. The n equations of motion for a system with n degrees of freedom are commonly written in matrix form as

$$[\mathbf{M}]\{\ddot{\mathbf{x}}\}+[\mathbf{C}]\{\dot{\mathbf{x}}\}+[\mathbf{K}]\{\mathbf{x}\}=-[\mathbf{M}]\{\mathbf{B}\}a_g \tag{8.14}$$

where $[\mathbf{M}]$, $[\mathbf{C}]$, and $[\mathbf{K}]$ are $n \times n$ square matrices of the mass, damping, and stiffness, and $\{\ddot{\mathbf{x}}\}$, $\{\dot{\mathbf{x}}\}$, and $\{\mathbf{x}\}$ are column vectors of the acceleration, relative velocity, and relative displacement. The column vector $\{\mathbf{B}\}$ defines the direction of the ground acceleration relative to the orientation of the mass matrix. The multidegree-of-freedom equations are coupled. They can be solved simultaneously by a number of methods. However, the single-degree-of-freedom response spectrum method is also commonly used for multidegree-of-freedom systems. The solution is assumed to be separable and the n eigenvalues (natural frequencies) ω_i and eigenvectors (mode shapes) $\{\Phi_i\}$ are found. The solutions for the relative displacements, relative velocities, and accelerations are then for i equals 1 to n:

$$\{\mathbf{x}_i\}=\sum_{j=1}^{n}\{\Phi_j\}\, f_j(t) \tag{8.15a}$$

$$\{\dot{\mathbf{x}}_i\}=\sum_{j=1}^{n}\{\Phi_j\}\, \dot{f}_j(t) \tag{8.15b}$$

$$\{\ddot{\mathbf{x}}_i\}=\sum_{j=1}^{n}\{\Phi_j\}\, \ddot{f}_j(t) \tag{8.15c}$$

The mode shapes are orthogonal with respect to the mass and the stiffness matrix. This orthogonality uncouples the equations of motion if the damping matrix is a diagonal matrix or proportional to a combination of the mass and stiffness matrix; that is, $\{\Phi_j\}^T[M]\{\Phi_i\}$ and $\{\Phi_j\}^T[K]\{\Phi_i\}$ are zero if $i \neq j$ and scalar numbers if $i = j$.

The response-spectrum technique can then be used to find the maximum values of $f_j(t)$ for each mode of vibration. Figure 8.4 shows a typical design response spectrum as produced by ASCE 7. The response is based on calculations of the single-degree-of-freedom elastic response for a range of earthquake acceleration records. Given the frequencies of the modes of vibration for a multidegree-of-freedom system, a spectral acceleration for each mode, S_{ai}, can be determined from the response spectrum. The base shear V_i acting in each mode can then be determined from

$$V_i=\frac{(\{\Phi_j\}^T[\mathbf{M}]\{\mathbf{B}\})^2}{\{\Phi_i\}^T[\mathbf{M}]\{\Phi_i\}}S_{ai} \tag{8.16}$$

The distribution of this maximum base shear over the structure is

$$\{\mathbf{F}_i\}=\frac{(\{\Phi_i\}^T[\mathbf{M}]\{\mathbf{B}\})}{\{\Phi_i\}^T[\mathbf{M}]\{\Phi_i\}}[\mathbf{M}]\{\mathbf{B}\}S_{ai} \tag{8.17}$$

Other response characteristics for each mode can be calculated from similar equations.

The maximum response in each mode does not occur at the same time for all modes. So some form of modal combination technique is used. The complete quadratic combination (CQC) method is one commonly used method for rationally combining these modal contributions. (E. L. Wilson et al., “A Replacement for the SRSS Method in Seismic Analysis,” *Earthquake Engineering and Structural Dynamics*, vol. 9, pp. 187–194, 1981.) The method degenerates into a variation of the square root of the sum-of-the-squares (SRSS) method when the modes of vibration are well separated. The summation must include an adequate number of modes to assure that at least 90% of the mass of the structure is participating in the seismic loading.

The total seismic design force and the force distribution over the height and width of the structure for each mode can be determined by this method. The combined force distribution takes into account the variation of mass and stiffness of the structure, unusual aspects of the structure, and the dynamic response in the full range of modes of vibration, rather than the single mode used in the static-force method. The combined forces are used to design the structure, and are often reduced by *R* in accordance with the ductility of the structural system. In many respects, the dynamic method is much more rational than the static-force method, which involves many more assumptions for computing and distributing design forces. The dynamic method sometimes permits smaller seismic design forces than the static-force method. However, while it offers many rational advantages, the dynamic method is still a linear-elastic approximation to an inelastic-design method. As a result, it assumes that the inelastic response is distributed throughout the structure in the same manner as predicted by the elastic-mode shapes. This assumption may be inadequate if there is a brittle link in the system.

Other analytical methods have been used to overcome some of these latter limitations. Designers are increasingly using inelastic-analysis methods including inelastic dynamic time–history and inelastic pushover analysis methods. These options will be briefly discussed in Art. 8.8.2.

8.6 STRUCTURAL STEEL SYSTEMS FOR SEISMIC DESIGN

Since seismic loading is an inertial loading, the forces are dependent on the dynamic characteristics of the acceleration record and the structure. Seismic design codes use a response spectrum as shown in Fig. 8.4 to model these dynamic characteristics. These forces are usually reduced in accordance with the ductility of the structure. This reduction is accomplished by the *R* factor in the static-force method, and the reduction may be quite large (Art. 8.5). The designer must ensure that the structure is capable of developing the required ductility, as it is well-known that the available ductility varies with different structural systems. Therefore, the structural engineer must ensure that the structural system selected for a given application is capable of achieving the ductility required for the *R* value used in the design. The engineer also must complete the details of the design of members and connections so that the structure lives up to these expectations.

Evaluation of Ductility. Two major factors may affect evaluation of the ductility of structural systems. First, the ductility is often measured by the hysteretic behavior of the critical components. The hysteretic behavior is usually examined by observing the cyclic force-deflection (or moment-rotation) behavior as shown in Fig. 8.7. The slope of the curves represents the stiffness of the structure or component. The enclosed areas represent the energy that is dissipated, and this can be large, because of the repeated cycles of vibration. These enclosed areas are sometimes full and fat (Fig. 8.7*a*), or they may be pinched or distorted (Fig. 8.7*b*). The hysteretic curves also show the inelastic deformation that can be tolerated at various resistance levels. Structural framing with curves enclosing a large area representing large dissipated energy, and structural framing which can tolerate large inelastic deformations without excessive loss in resistance, are regarded as superior systems for resisting seismic loading. As a result, these systems are commonly designed with larger *R* values and smaller seismic loads.

Special steel moment-resisting frames and eccentric braced frames, defined in Art. 8.4, are capable of developing large plastic deformations and large hysteretic areas. As a result, they are designed for larger values of *R*, thus smaller seismic forces and greater inelastic deformation. This hysteretic behavior is important, since it dampens the inelastic response and improves the seismic performance of the structure without requiring excessive strength or deformation in the structure. This is illustrated in Fig. 8.8, which shows the inelastic dynamic response of two steel moment-resisting frames, which had identical mass, stiffness, and seismic excitation (1979, Imperial Valley College), but different seismic resistance. The story drift and inelastic deformation cycles are larger for the elastic structure than for the ductile structure with lateral resistance equal to approximately 40% of that required for elastic response. However, the structure with the smaller resistance has larger maximum deflections and sustains permanent inelastic offset during the earthquake excitation. This shows

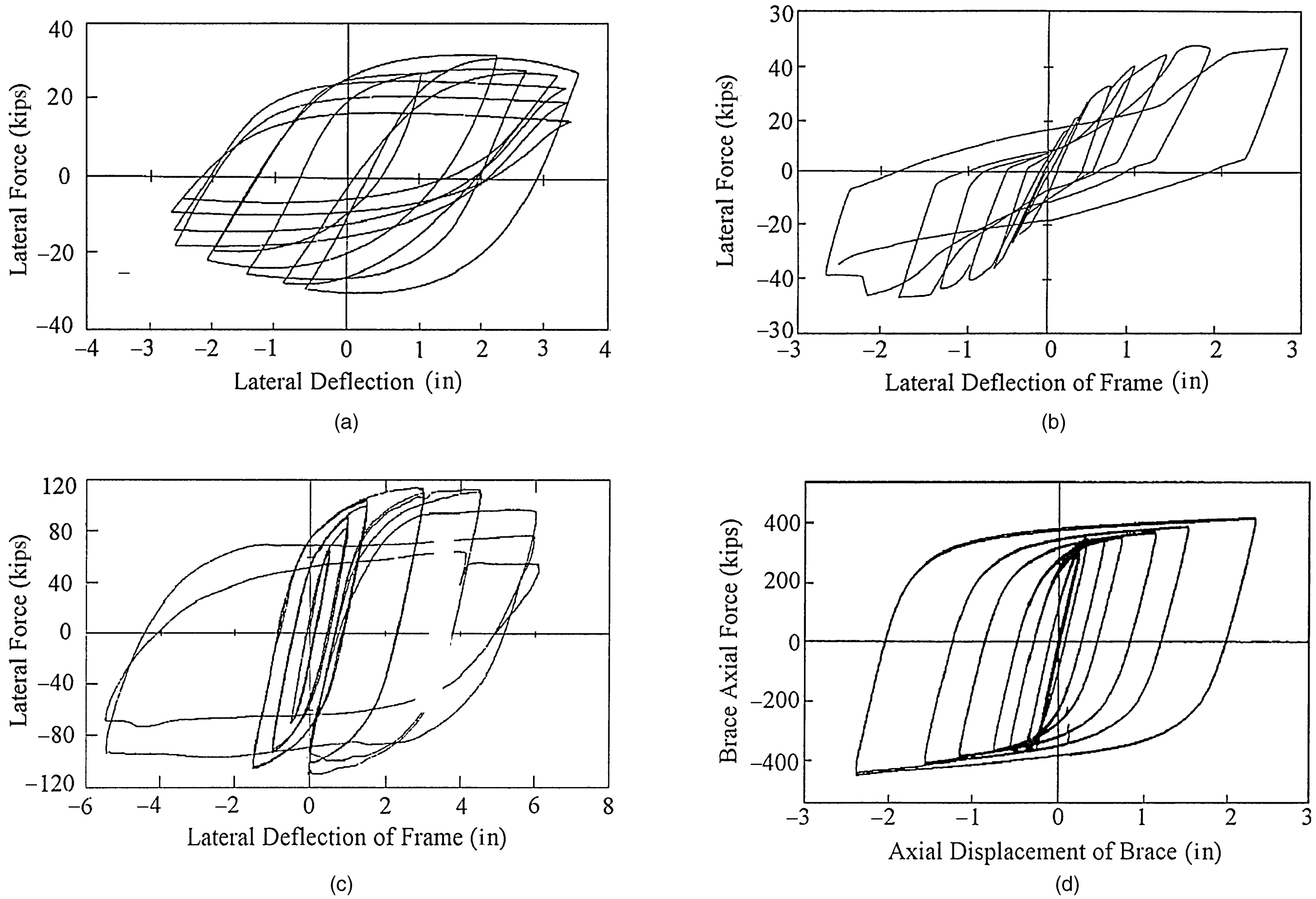


FIGURE 8.7 Hysteretic behavior of three steel frames. (*a*) Moment-resisting frame. (*b*) Concentric braced frame. (*c*) Eccentric braced frame. (*d*) Buckling-restrained brace.

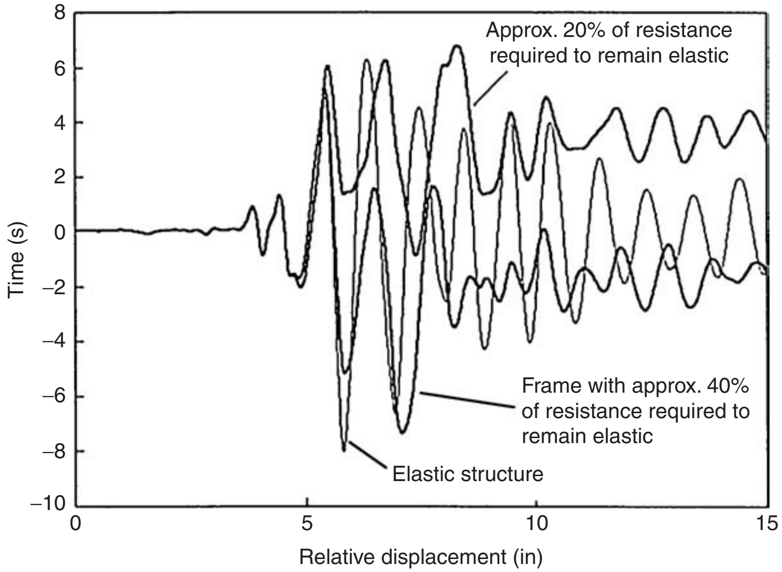


FIGURE 8.8 Comparison of the elastic and inelastic response of three frames with identical mass and stiffness but different resistance.

that structures with smaller design force (larger *R*) require the structure to have the ability to maintain its integrity through larger inelastic deformations than if a larger design force (smaller *R*) were employed.

While some steel structures are very ductile, not all structures have this great ductility. Fracture of the connections has a very detrimental effect on the structural performance, since it may cause a significant loss in both resistance and deformational capacity. Local and global buckling may also change the hysteretic behavior from that of Fig. 8.7*a* to Fig. 8.7*b*. The combined effects of these potential problems means that the structural engineer must pay particular attention to the design details in the seismic design of buildings, since those details are essential to ensuring good seismic performance.

Effects of Inelastic Deformations. The distribution of inelastic deformation is a second factor that can effect the inelastic seismic performance of a structural system. Some structural systems concentrate the inelastic deformation (ductility demand) into a small portion of the structure. This can dramatically increase the ductility demand for that portion of the structure. This concentration of damage is sometimes related to factors that cause pinched hysteretic behavior, since buckling may change the stiffness distribution as well as affect the energy dissipation.

Ductility demand, however, can also be related to other factors. Figure 8.9 shows the computed inelastic response of two steel moment-resisting frames that have identical mass and nearly identical strength and stiffness and are subject to the same acceleration record as that in Fig. 8.8. The frames differ, however, in that one is designed to yield in the beams while the other is designed to yield in the columns. This difference in design concept results in a significant difference in seismic response and ductility demand. Design codes attempt to assure greater ductility from structures designed for smaller seismic forces, but attaining this objective is complicated by the fact that ductility and ductility demand are not fully understood.

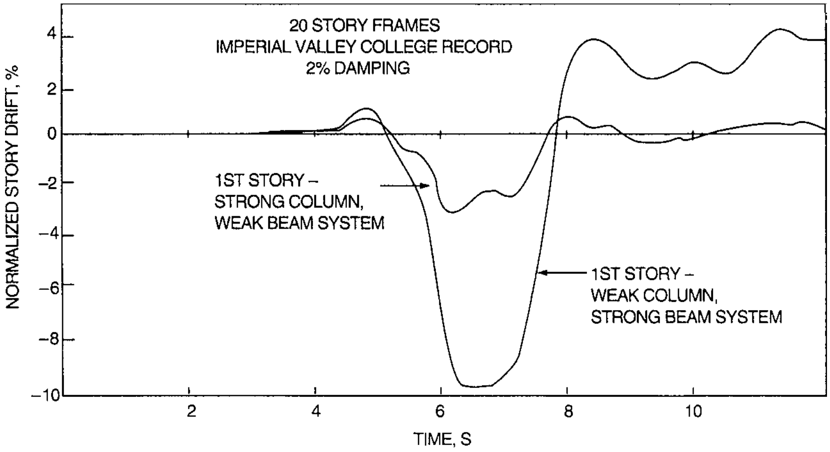


FIGURE 8.9 Curves show inelastic dynamic response of two steel frames with identical mass and nearly identical strength and stiffness but designed with two different strategies for determining inelastic deformations.

Steel moment-resisting frames have historically been regarded as the most ductile structural system for seismic design, but a number of special steel moment frames sustained damage during the January 17, 1994, Northridge earthquake. In these buildings, cracks were initiated near the flange weld. Some of the cracks penetrated into the column and panel zone of the beam–column connections as illustrated in the photo of Fig. 8.10, but others penetrated into the beam flange or the flange welds and the heat-affected zones of these welds. None of these buildings collapsed and there was no loss of life, but the economic loss was considerable. This unexpected damage led to a new evaluation of the design of moment-frame connections through the SAC Steel Project. SAC is a joint venture of SEAOC, ATC (Applied Technology Council), and CUREE (California Universities for Research in Earthquake Engineering), and the joint venture is funded by the Federal Emergency Management Agency (FEMA). This work is summarized in several reports prepared by FEMA.

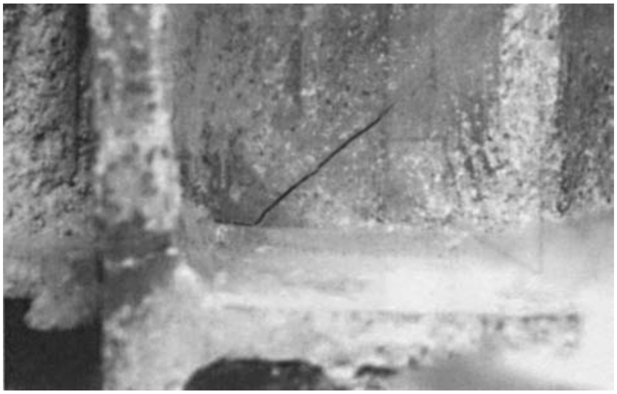


FIGURE 8.10 Photograph of crack through the column flange and into the column web or panel zone of connection.

The reader is referred to "State of the Art Report on Connection Performance," FEMA 355D, Federal Emergency Management Agency, Washington, D.C., September 2000, for a summary of the connection analytical and experimental research. In addition, "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings," FEMA 350, Federal Emergency Management Agency, Washington, D.C., July 2000, summarizes the design recommendations for these moment-frame connections. Many of these recommendations are being incorporated into the AISC Seismic Design Provisions, but the work has demonstrated several connections, which provide good seismic performance but are not yet included in the seismic provisions. These reports provide information on these alternate connections. The SAC Steel Project clearly showed that steel moment frames are capable of achieving superior ductility and inelastic seismic performance. However, the work also shows that the engineer must exercise great care in the selection and design of members and connections. The requirements for special moment frames are summarized briefly in Art. 8.7.1.

Concentric braced frames, defined in Art. 8.4, economically provide much larger strength and stiffness than moment-resisting frames with the same amount of steel. There are a wide range of bracing configurations, and considerable variations in structural performance may result from these different configurations. Figure 8.6 shows some concentric bracing configurations. The braces, which provide the bulk of the stiffness in concentrically braced frames, attract very large compressive and tensile forces during an earthquake. As a result, compressive buckling of the braces often dominates the behavior of these frames. The pinched cyclic force-deflection behavior shown in Fig. 8.7*b* commonly results, and failure of braces may be quite dramatic. Therefore, concentrically braced frames are regarded as stiffer, stronger but less ductile than steel moment-resisting frames. In recent years, research has shown that concentrically braced frames can sustain relatively large inelastic deformation without failure if greater care is used in the design and selection of the braces and the brace connections. However, continuing research work is in progress in establishing the design criteria for the brace and the connections. The work shows that concentrically braced frames that are designed and detailed to the higher ductility standards can be designed for smaller seismic design forces. The AISC Seismic Design Provisions define "special concentrically braced frames" with requirements for details aimed at achieving the higher ductility. Current detailing provisions are summarized in Art. 8.7.2.

Eccentric braced frames, defined in Art. 8.4, can combine the strength and stiffness of concentrically braced frames with the good ductility of moment-resisting frames. Eccentric braced frames incorporate a deliberately controlled eccentricity in the brace connections (Fig. 8.5). The eccentricity and the link beams are carefully chosen to prevent buckling of the brace, and provide a ductile mechanism for energy dissipation. If they are properly designed, eccentric braced frames lead to good inelastic performance as depicted in Fig. 8.7*c*, but they require yet another set of design provisions, which are summarized in Art. 8.7.3.

Buckling-restrained concentrically braced frames are a new option for concentrically braced frames that are not yet fully incorporated in the design specifications. However, because this system offers the potential for superior seismic performance, buckling-restrained braces are expected be included in future seismic design specifications. Buckling-restrained braces employ patented braces in which the axial member yields in tension and compression without brace buckling, as depicted in Fig. 8.11. This is accomplished by encasing the brace bar so as to prevent lateral deformation and buckling without bonding the slender bar to the encasing element. This assures that the slender bar yields in both axial tension and compression, and deterioration in stiffness and resistance due to buckling is avoided. It increases the inelastic energy dissipation, improves axial yield performance, and permits development of large inelastic axial deformations. The seismic performance of buckling-restrained braces depends on both the brace and the connection design. Research is now developing improved design procedures for the brace and the connection. However, a recent guideline proposes design criteria and testing and acceptance criteria that can be used to verify that the buckling-restrained brace is appropriate for the proposed seismic design application. The reader is referred to "Recommended Provisions for Buckling Restrained Braced Frames," Structural Engineering Association of Northern California, Seismology and Structural Standards Committee, San Francisco, Calif., October 2001, for current thinking and design information on this system. The buckling-restrained brace is included in 2005 AISC Seismic Design Provisions (see Art. 8.7.4).

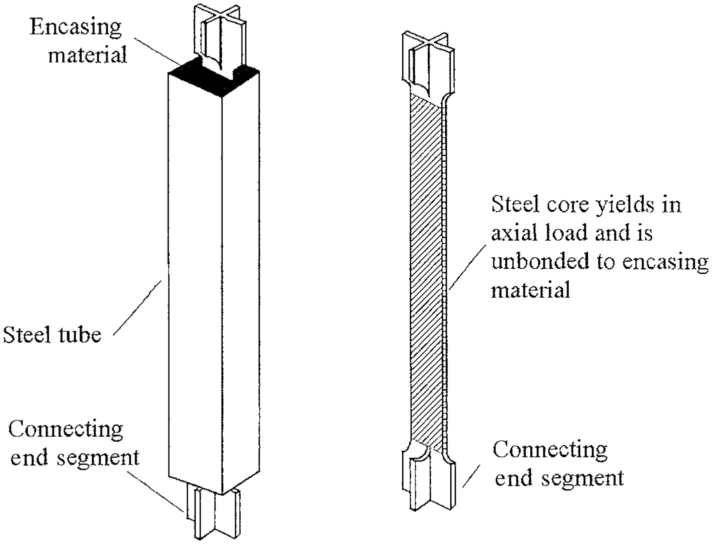


FIGURE 8.11 Conceptual illustration of the buckling-restrained brace.

Dual systems, defined in Art. 8.4, may combine the strength and stiffness of a braced frame and shear wall with the good inelastic performance of special steel moment-resisting frames. Dual systems are frequently assigned an *R* value and seismic design force that are intermediate to those required for either system acting alone. Design provisions provide limits and recommendations regarding the relative stiffness and distribution of resistance of the two components. Dual systems have led to a wide range of structural combinations for seismic design. Many of these are composite or hybrid structural systems. However, steel frames with composite concrete floor slabs are not commonly used for developing seismic resistance, even though composite floors are commonly used for gravity-load design throughout the United States.

8.7 SEISMIC-DESIGN LIMITATIONS ON STEEL FRAMES

A wide range of special seismic design requirements are specified for steel frames to ensure that they achieve the ductility and behavior required for the structural system and the design forces used for the system. Use of systems with poor or uncertain seismic performance is restricted or prohibited for some applications. Most of these requirements are specified in the “Seismic Provisions for Structural Steel Buildings” of the AISC. These provisions are either adopted by reference or they are directly incorporated into the IBC provisions. This article will provide a summary of the provisions for moment-resisting frames, concentrically braced frames, eccentrically braced frames, and buckling-restrained braces for seismic applications, based primarily on the latest draft (2005) of the AISC seismic provisions. Latest versions of this and other applicable codes should be checked for updates.

While the discussion that follows is subdivided by the primary structural system used to achieve ductility and lateral resistance, a few concepts have broad impact on all structural systems. First, seismic design requires that the structural system have a ductile element that is capable of achieving the ductility required from the given seismic-design concept. In most systems, this requires a balance check in which the plastic resistance of the ductile element is compared to the resistance of surrounding, less ductile structural elements. When making this balance check, it is important to base the plastic resistance of the ductile element on the mean or expected resistance rather than the nominal resistance, because expected resistance is normally larger than the nominal capacity. If the expected resistance is larger than anticipated, the ductile element may not achieve its full ductility before a peripheral, less ductile element fails. The AISC provisions address this issue by multiplying the nominal plastic resistance of a ductile element by an expected strength factor R_y. Modern structural steels often vary widely from the nominal yield stress, and thus R_y is defined by

$$R_y = \frac{F_{\text{ye}}}{F_y} \tag{8.18a}$$

where F_{ye} and F_y are the expected and the nominal minimum specified yield stress, respectively. This R_y value can be established through testing or, in the absence of test data, specification-defined values of between 1.1 and 1.6 are provided in the provisions, depending on the grade of steel. R_y is used to evaluate the uncertainty in material properties and how this affects the seismic performance of the building. Similar balance checks are sometimes required using the ultimate tensile stress of the steel. The ratio, R_t, of the expected ultimate tensile stress to the nominal tensile stress is defined as

$$R_t = \frac{F_{\text{te}}}{F_t} \tag{8.18b}$$

where F_{te} and F_t are the expected and the nominal tensile stresses, respectively. The AISC provides values between 1.1 and 1.3 for the steels commonly used in seismic design.

Second, many steel structural systems achieve their ductility by plastic deformation in the steel near welded joints. Welds and the heat-affected zone immediately adjacent to welds may have different properties than steel members, and there is a greater probability of local or internal flaws in welded joints than in steel sections. The FEMA recommendations for special moment-resisting frames resulting require that the welds be a matching metal and require a minimum toughness of welds in regions where large inelastic strain demands occur. This requirement assures adequate inelastic strain capacity of the welded joint to achieve ductile performance. AISC provisions add these requirements to demand-critical welds for other structural systems. These demand-critical welds must use a filler metal capable of providing a minimum Charpy V-notch (CVN) toughness of 20 ft·lb at −20°F and 40 ft·lb at 70°F.

Third, columns are extremely critical elements in all structural systems, since the columns must support gravity load regardless of the earthquake excitation. As a result, the forces and moments in columns are very uncertain when complex inelastic deformations of the frames occur. When the factored axial load on the column exceeds 40% of the nominal capacity, the columns must have adequate resistance to satisfy additional load-factor combinations provided in the IBC. These additional load combinations assure that columns are designed with adequate resistance to support all combinations of earthquake loads and dead loads on the structure. The reader is referred to the IBC provisions for the specific load combinations.

8.7.1 Limitations on Moment-Resisting Frames

Structural tests have shown that steel moment-resisting frames may provide excellent ductility and inelastic behavior under severe seismic loading. Because these frames are frequently quite flexible, drift limits often control the design. The NEHRP seismic provisions recognize this ductility and assigns $R = 8.0$ to special moment-resisting frames (Art. 8.4).

Slenderness Requirements. Special steel moment-resisting frames must satisfy a range of slenderness requirements to control buckling during the plastic deformation in a severe earthquake. The unsupported length, L_b, of bending members must satisfy

$$L_b \leq \frac{0.086\, r_y E}{F_y} \tag{8.19}$$

where r_y is the radius of gyration about the weak axis of the member and F_y is the specified minimum yield stress, ksi, of the steel. The objective of this limit is to control lateral torsional buckling during plastic deformation under cyclic loading. The lateral bracing adjacent to plastic hinges must be applied to both the top and bottom flanges, and the lateral bracing must have adequate lateral resistance to develop 6% of the nominal force in the beam flange at the expected plastic-moment capacity ($M_p = R_y F_y Z$). The flanges of beams and columns must satisfy

$$\frac{b_f}{2t_f} \le 0.30\sqrt{\frac{E}{F_y}} \tag{8.20}$$

where b_f and t_f are the flange width and thickness, respectively. This requirement is to control flange buckling during the plastic deformation expected in a severe earthquake. The webs of members must satisfy

$$\frac{d}{t_w} \le 3.14\sqrt{\frac{E}{F_y}}(1-1.54C_a) \qquad \text{for } C_a < 0.125 \tag{8.21a}$$

$$\frac{d}{t_w} \le 1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \qquad \text{for } C_a > 0.125 \tag{8.21b}$$

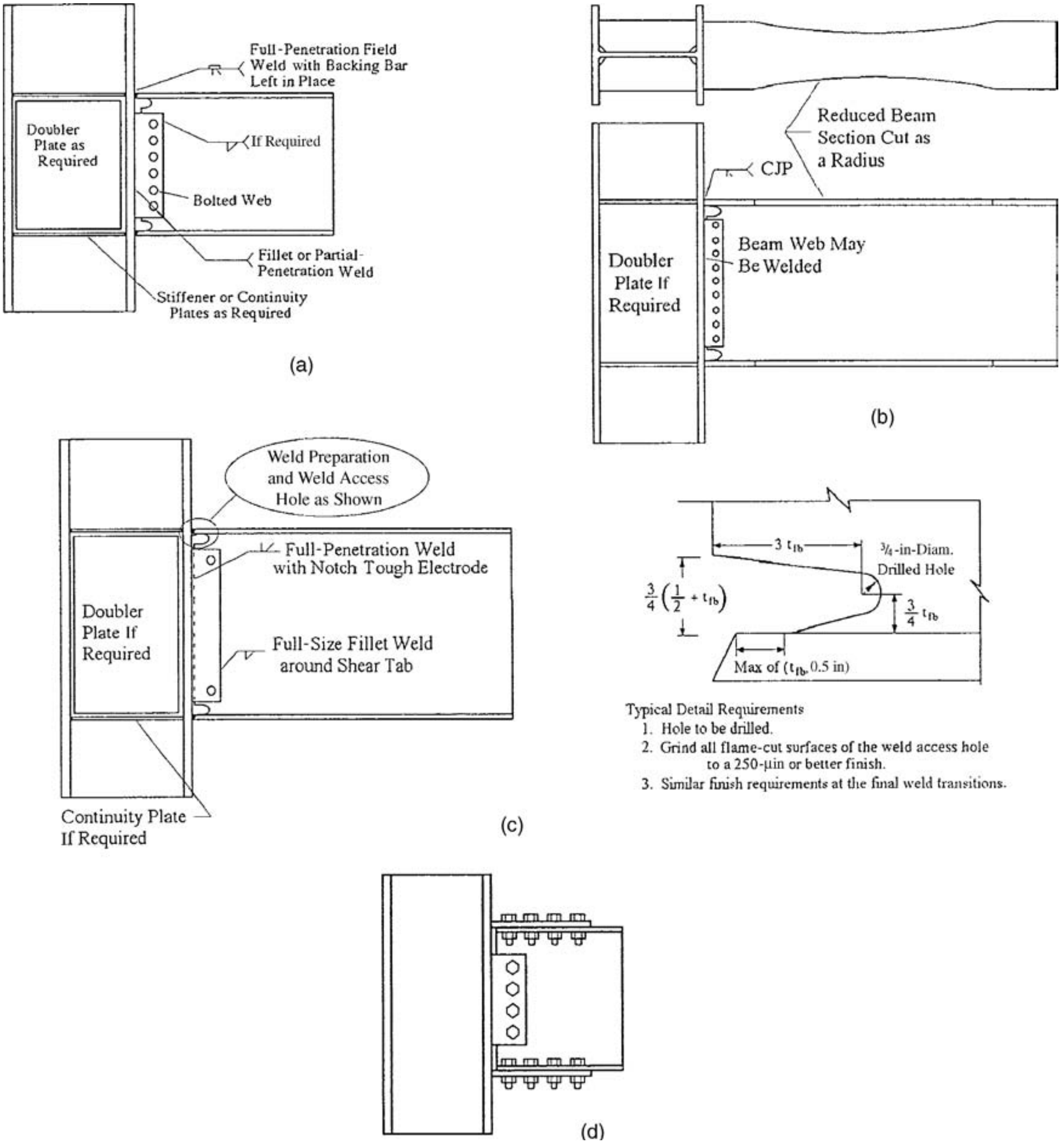
except that

$$\frac{d}{t_w} \le 1.49\sqrt{\frac{E}{F_y}} \tag{8.21c}$$

provides a lower limit beyond which Eq. (8.21b) need not be applied. For these equations, C_a is the ratio of the required axial strength to the available strength, and d and t_w are the depth and web thickness of the member, respectively. These latter equations are required to control web buckling during the plastic deformation expected during severe earthquake excitations. These limits are somewhat more conservative than the normal compactness requirements for steel design because of the great ductility demand of seismic loading.

Beam-to-Column Connections. In special moment-resisting frames, beam-to-column connections have historically been designed as prequalified, welded-flange, bolted-web connections as depicted in Fig. 8.12*a*. The connections were used because experiments performed 25 to 35 years ago indicated that good ductility was achieved with that connection. As noted in Art. 8.6, cracking occurred in a number of these connections during the 1994 Northridge earthquake. There was no building collapse or loss of life in these damaged buildings, but the economic cost of the damage was severe. The cracking was more frequently noted in new buildings and in buildings with relatively heavy members. Further, the damage was more common in buildings in which the lateral resistance was concentrated in limited portions of the structure, since this concentration produces larger member sizes.

A comprehensive research program was completed to address this damage. There were clearly many contributing factors to the observed damage. More comprehensive summaries of the findings and recommendations are available in FEMA Reports 350 and 355D regarding the design and behavior of moment-frame connections. Many have been directly incorporated into the AISC seismic provisions, and further adoptions may be expected in the future. In particular, the pre-Northridge welded-flange, bolted-web connection shown in Fig. 8.12*a* is no longer regarded as a suitable connection for special-moment frames. This connection was typically constructed with E70T-4 welds, and backing bars and runoff tabs for these welds were left in place. These weld practices were shown to result in large flaws in the welded joints, and provided joints without adequate dynamic toughness to avoid joint fracture. Today, tougher weld metals are required for these welded joints, as described earlier. Runoff tabs are removed, and bottom flange backing bars are removed, back gouged, and



Typical Detail Requirements
1. Hole to be drilled.
2. Grind all flame-cut surfaces of the weld access hole to a 250-μin or better finish.
3. Similar finish requirements at the final weld transitions.

FIGURE 8.12 Typical connections used for moment-resisting frames. (*a*) Pre-Northridge welded-flange, bolted-web connection. (*b*) Reduced-beam-section connection. (*c*) Welded-flange, welded-web connection. (*d*) Bolted flange plate connection.

reinforced with a fillet weld. Top flange welds are reinforced at the backing bar. The research showed that greater connection ductility is achieved when the connection yields in flexural yielding of the beam and panel-zone yielding. Connections with inadequate shear resistance were shown to provide reduced connection ductility, and current design procedures place greater emphasis on the shear connection between the beam web and the column. The work showed that deeper, heavier steel sections have inherently less ductility than shallower sections. The research clearly demonstrated the importance of understanding the yield mechanisms and failure modes of the connection, and the balancing of these behaviors to achieve optimal performance

Figure 8.13*a* shows a typical moment–rotation curve for a welded-flange, bolted-web connection with the tough welds, removed runoff tabs, removed bottom-flange backing bar, and reinforced flange welds as described above and currently required in the AISC seismic provisions. The resistance provided by this connection is substantial. The ductility is limited, but the connection has some flexural integrity even after initial flange fracture. However, the performance of this connection is not adequate for demanding seismic design applications for special-moment-resisting frames. As a result, this connection is no longer prequalified for special-moment-resisting frame application. However, the welded-flange, bolted-web connection may still be suitable for ordinary-moment frames.

While the research showed significant flaws in the welded-flange, bolted-web connection, a number of other connections were shown to provide superior performance. In particular, the reduced-beam-section connection, the welded-flange, welded-web connection, and the bolted-flange plate connection (illustrated in Fig. 8.12*b*, *c*, and *d*, respectively) all provided superior performance, with large inelastic deformation capacity as shown by the typical moment–rotation curves of Fig. 8.13*b*, *c*, and *d*, respectively. The AISC seismic provisions recognize that great ductility is required from beam–column connections of steel moment-resisting frames, and that there are a number of connections

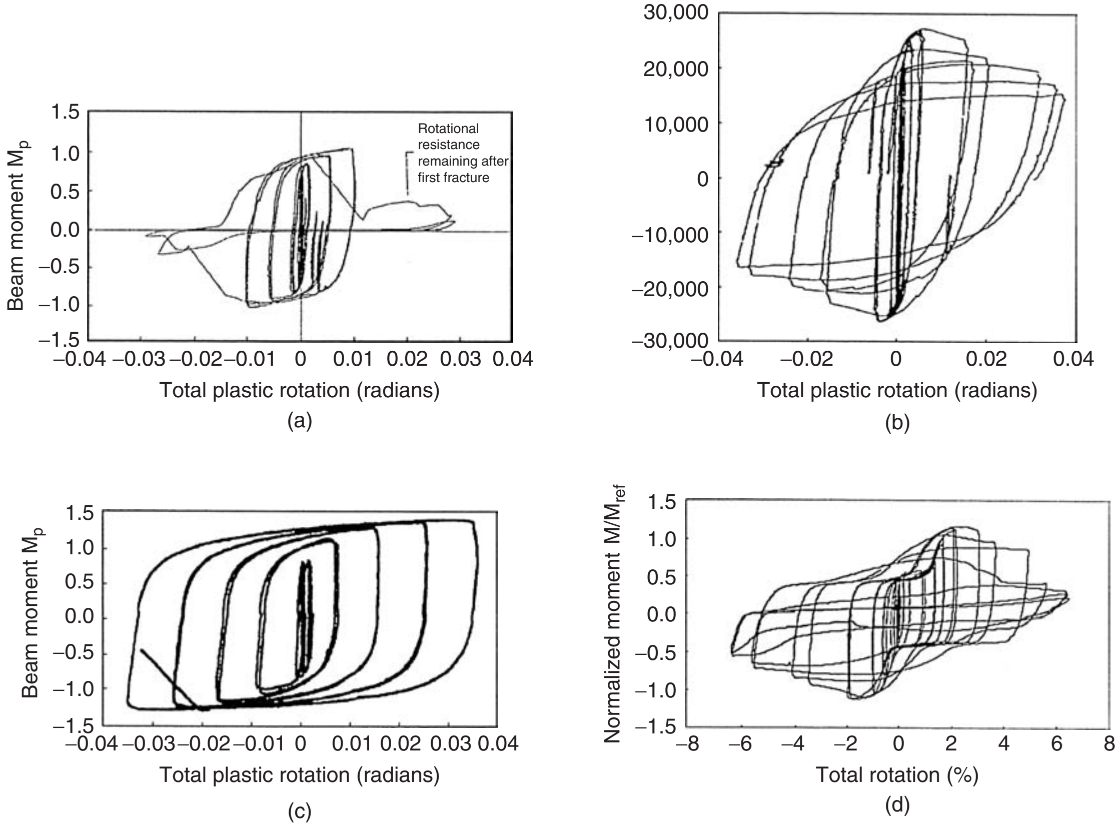


FIGURE 8.13 Typical performance observed with various moment-resisting frame connections. (*a*) Pre-Northridge welded-flange, bolted-web connection. (*b*) Reduced-beam-section connection. (*c*) Welded-flange, welded-web connection. (*d*) Bolted-flange plate connection.

that are available to provide that performance. However, the AISC also requires that the performance of the connection be clearly demonstrated before it is used in practice. To ensure this, a connection test procedure for verifying connection performance is defined in Appendix S of the AISC seismic provisions. With this test procedure, the connection must sustain a total rotation of 0.04 rad without failure, or without deterioration of resistance below 80% of the nominal plastic-moment capacity of the connection, for approval in special-moment-resisting frames. A number of connections are capable of providing this performance, as shown in Fig. 8.13. In many cases, this performance is achieved only within a given range of member sizes. Verification or documentation has not yet been provided for the full range of applicability for most connection types.

The reduced-beam-section connection (see Figs. 8.12*b* and 8.13*b*) has the most widely documented performance of the connection types available to date. This connection achieved its ductility by careful removal of a portion of the beam flange to assure that yielding occurs in the reduced flange area before yielding occurs at the beam flange weld. The reduced section must be carefully radiused and finished to avoid flaws and rough edges, and FEMA 350 provides guidance on these requirements. The reduced beam section provides significant inelastic rotational capacity, but the strain hardening of the connection is somewhat limited by the reduced flange width and the reduced lateral stability of the yielded beam when yielding occurs several feet from the face of the column.

The welded-flange, welded-web connection (see Figs. 8.12*c* and 8.13*c*) requires considerable care in cutting and finishing the weld access hole, and the web must be securely welded to the column with both complete-joint-penetration welds and fillet welds to the shear tab as shown in Fig. 8.12*c*. The weld access-hole preparation shown in this figure is now required by AISC provisions for demand-critical flange welds in special-moment-frame connections. This connection has developed large inelastic deformation capacity combined with significant strain hardening. Such strain hardening is very beneficial, because it provides reserve strength and stiffness during large seismic events, and the postyield stiffness may reduce the maximum inelastic demands on the structure. This may leave the structure more serviceable after a seismic event.

The bolted-flange plate connection (see Figs. 8.12*d* and 8.13*d*) permits complete field bolting of the connection. It is clearly more difficult to design, because peak performance is achieved when the connection design is balanced to achieve flexural yielding of the beam with subsequent shear yielding of the connection panel zone and tensile yielding of the flange plate. This connection also has large inelastic deformation capacity with large strain hardening, but the hysteretic behavior is somewhat pinched by the slip occurring at the bolts at larger seismic loads. The resistance at which bolt slip occurs may represent a serviceability limit state that requires some attention for seismic design, because frame deflections may be excessive if slip occurs at too small a seismic event. This connection is less well documented than those noted earlier, but it may provide the greatest inelastic ductility of those described. It is clearly limited to connections with modest-sized members (W30 beams or shallower), but it is a connection with considerable potential for seismic design. It is used infrequently but offers the potential of being an extremely ductile field-bolted connection. The reader is referred to FEMA 355D for additional recommendations regarding the design and performance of this connection.

Structural engineers today have a wide range of options for connection design, but the increased options place greater demands on the designer with regard to documenting and verifying the seismic performance of the connection.

Panel-Zone Yielding. Seismic bending moments in the beam cause large shear stresses in the column web in the panel zone of the connection (Fig. 8.14). Panel-zone yielding has been increasingly important in recent years, because FEMA-sponsored research showed that shear yielding may reduce connection peformance if it occurs before flexural yielding of the beam occurs. As a result, there has been a slight reduction in the shear capacity of the panel zone in recent editions of the AISC seismic provisions. Today the panel zone-shear force must be limited to

$$V_n = 0.6F_{\mathrm{yc}}d_c t_{\mathrm{wc}} \tag{8.22}$$

where F_{yc} is the nominal yield stress of the column web, and d_c and t_{wc} are the depth and web thickness of the column section, respectively. This shear capacity must be at least greater than the panel-zone

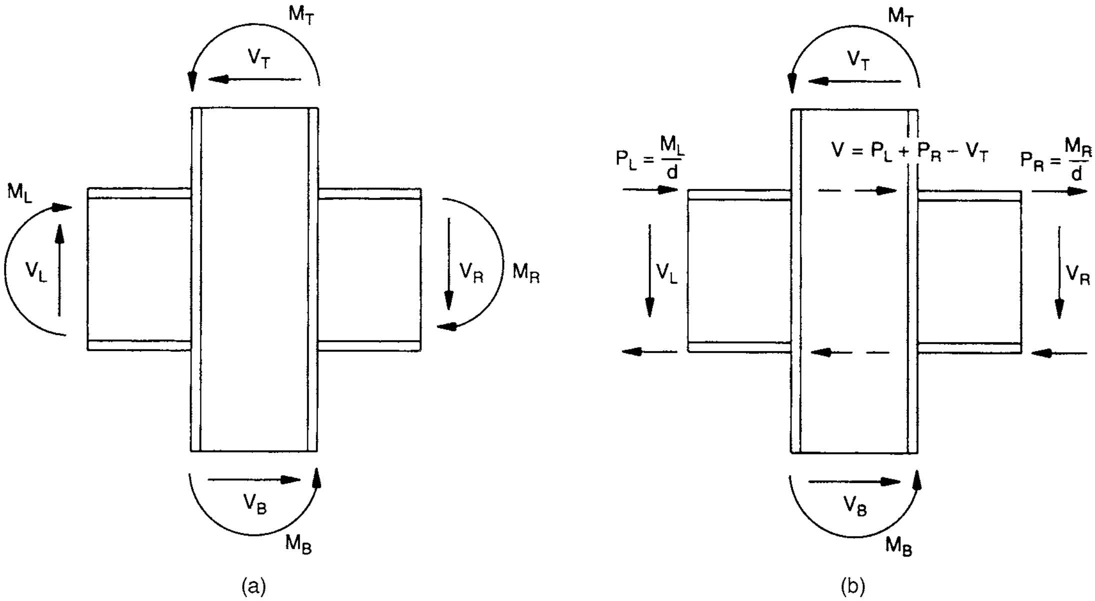


FIGURE 8.14 Forces acting on a column and beam in the panel zone in a typical moment-resisting connection during seismic loading. Forces in (*a*) are equivalent to those in (*b*).

shear force caused by the sum of the expected plastic-moment capacities at the face of the column as illustrated in the figure. If the column web is not thick enough to meet this requirement, doubler plates are frequently added to the column web to achieve the required thickness. The doubler plate is welded around its entire perimeter to attach it to the column web, flange, and continuity plates. The AISC requires additional plug welds when the thickness of the doubler plate, t_{dp}, is insufficient:

$$t_{\mathrm{dp}} = \frac{d_z + w_z}{90} \tag{8.23}$$

where d_z and w_z are the depth and width of the panel zone, respectively.

Shear Resistance of Moment-Frame Connections. The FEMA-sponsored research program showed that the shear resistance (the beam web connection) of moment-frame connections is very important to the inelastic seismic performance of the connection. The minimum shear resistance of this connection must exceed E:

$$E = \frac{2.2R_y M_p}{L_h} \tag{8.24}$$

where L_h is the distance between the plastic hinge locations on the beam.

Other Issues for Special-Moment Frames. Research has also clearly shown that flexural yielding of the beam is the preferred yield mechanism in moment-resisting frames subject to seismic loading. This preference is commonly noted as strong-column, weak-beam behavior. Flexural yielding of the column concentrates inelastic deformation in a single story of a structure, as noted earlier and as illustrated in Fig. 8.9. As a result, the AISC seismic provisions require a balance of the relative plastic capacity of the beam and the column considering the full expected plastic moments ($M_p = R_y F_y Z$) in the beam and the nominal moments in the column.

Column splices also require some special consideration with moment-resisting frames. Inelastic analysis shows that significant bending moments may develop in the columns despite the balancing requirements to assure strong-column, weak-beam behavior. The distribution of plastic deformation varies widely in moment frames during severe earthquake shaking. The consequences of this are that the columns may sustain limited plastic deformation and may temporarily be in single curvature rather than the double curvature assumed in design. Column splices are usually made near midheight of a story, where the bending moment is relatively small. However, the AISC seismic provisions recognize that plastic strains may occur in this region, and the column splice is required to have a minimum resistance in flexure and shear. If groove welds are employed at this splice, the welds must be complete-joint-penetration welds unless a smaller splice-resistance requirement can be shown by inelastic analysis. If bolts or other splice-connection methods are used, the splice-connection flexural resistance must exceed the expected plastic-moment capacity of the column.

Ordinary and Intermediate-Moment Frames. Some steel moment-resisting frames are not designed to satisfy the preceding conditions. In many cases, these frames are used in less seismically active zones. Sometimes, however, they are used in seismically active zones with larger seismic design forces; that is, they are designed with $R = 3.5$. As a result, the design forces would be more than twice as large as required for special-moment frames. The seismic ductility demands will be significantly smaller, but the detailing requirements are also reduced. These are known as ordinary moment frames. Ordinary moment-resisting frames must satisfy some of the requirements noted above but not all, depending on the seismic zone and the design forces in the structure.

Intermediate-moment frames are intermediate alternatives to ordinary and special-moment-resisting frames. They have intermediate ductility demands and detailing requirements, and permit intermediate seismic design-force levels.

8.7.2 Limitations on Concentric Braced Frames

Concentric braced steel frames are much stiffer and stronger than moment-resisting frames, and they frequently lead to economical structures. However, their inelastic behavior is usually inferior to that of special moment-resisting steel frames (Art. 8.6). One reason is that the behavior of concentric braced frames under large seismic forces is dominated by buckling. Furthermore, the columns must be designed for tensile loads and foundation uplift as well as for compression. As with moment-resisting frames, concentric braced frames may be designed to different seismic-design standards. Special concentrically braced frames are designed for the largest R values and the smallest seismic-design forces. Special concentrically braced frames also have more detailed design requirements because of the necessity of achieving greater ductility from the braced frame system. Ordinary concentrically braced frames may also be designed. These latter braced frames use larger seismic-design forces and have less reliance on inelastic deformation capacity and buckling from the braced frame system. As a result, design requirements are somewhat more liberal. Ordinary concentrically braced frames are less commonly used today for demanding seismic applications. Unless otherwise noted, the discussion in this section will focus primarily on the special concentrically braced frame system.

Figure 8.6 shows some of the common bracing configurations for concentric braced frames. Seismic design requirements vary with bracing configurations.

X-bracing has historically used very slender braces designed as tension-only bracing or bracing with only limited compressive buckling capacity. The resulting braces had high slenderness ratios, KL/r. This historic practice lead to economical designs but poor seismic performance. As a result, many past seismic provisions discouraged or disallowed X-bracing. Today a very different practice has evolved with X-braced frames. In many cases the X-bracing extends over multiple stories to effectively combine V- and inverted V-bracing. Second, design requirements for special concentrically braced frames require a balance of the shear resistance provided by braces in tension with braces in compression, and this effectively prevents the very slender braces noted in historic practice. As a result, X-bracing is used more frequently today, since the seismic performance is regarded as improved with the changes in design practice.

V-bracing or inverted V-bracing has the bracing connection at midspan of the beam. Under lateral load, one brace acts in compression while the other acts in tension. The capacity of the tensile brace is significantly larger than the compressive capacity of the other brace, and this unbalanced force at the brace–beam intersection causes beam yielding during severe seismic excitation. Beam flexural yield with this bracing may provide significant increases in energy dissipation. As a consequence, these bracing configurations were more favorably regarded in years past. However, the flexural yielding causes floor damage after an earthquake, which may be quite severe, and the economic consequences of this damage are significant. Further, the concentration of damage to a single floor, which is possible when brace buckling or fracture occurs within a given story level, has resulted in increased concern with the design of these bracing systems. Today, a special concentrically braced frame with V- or inverted V-bracing must be designed so that the beam has adequate bending resistance to withstand the unbalanced forces after brace buckling has occurred. This increased beam resistance results in less damage to the floor system during severe earthquakes, and it also aids in distributing inelastic deformations and demands to other parts of the structure.

Multistory X-bracing may be thought of as another special combination of the V-brace and the inverted V-brace systems. Multistory X-bracing prevents unbalanced brace force after brace buckling, as noted with the V-braced systems. This prevents extensive flexural yielding in the floor beam and reduces the potential for concentration of damage within a given story of the structural system.

Zipper columns are sometimes used with V- and inverted V-bracing as an alternative to the strong beam required for special concentrically braced frames. After brace buckling occurs, the zipper column transfers the unbalanced force at the brace–beam connection to other bracing levels. This procedure negates the need for the heavy beam required to resist the unbalanced force by flexure. It distributes the inelastic deformation to other levels of the steel frame, and prevents the extreme floor damage noted with V-bracing system.

K-bracing (and knee bracing) has an intersection of a tensile and compressive brace at midheight of the column. This application has the same unbalanced force problem as described with the V-bracing systems. However, the inelastic deformation resulting from this unbalanced force occurs within the column rather than the beam. The column is needed to support the gravity loads of the structure. Because this inelastic deformation cannot be tolerated in a ductile system, K-bracing is expressly prohibited for special concentrically braced frames.

Diagonal bracing acts in tension for lateral loads in one direction and in compression for lateral loads in the other direction. The tensile capacity of the brace is significantly larger than the compressive capacity of the brace. As with other bracing systems, the AISC seismic provisions for concentrically braced frames require that the direction of inclination of bracing be balanced to assure appropriate resistance in both directions at all times. Thus, the braces are used in pairs.

Buckling of Bracing. In general, the energy dissipation of concentric braced frames is strongly influenced by postbuckling brace behavior. This behavior is quite different for slender braces than for stocky braces. For example, the compressive strength of a slender brace is much smaller in later cycles of loading than it is in the first cycle. In addition, very slender braces offer less energy dissipation, but are able to sustain more cycles and larger inelastic deformation than stocky braces. In view of this, the slenderness ratio of bracing in special concentric braced frames is limited to

$$\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}} \tag{8.25}$$

where K is the effective length coefficient of the brace, L is the brace length, and r is the radius of gyration. An exception to this limit is permitted for braces with slenderness ratios less than 200, if the column can be shown to have adequate resistance to support the full expected tensile load ($R_yF_y\,A_g$, where A_g is the gross area of the brace) of the brace elements in the building.

Tensile yielding also contributes a significant amount to the inelastic deformation and energy distribution of the concentric braced frame system. This tensile yielding is controlled by the yield stress and the gross area of the brace. The inelastic deformation capacity in tension may be limited by net section fracture, which is controlled by the tensile strength of the steel and the effective net area of the brace. The effective net area, A_e, of a brace is typically smaller than the gross area, A_g, and so therefore there is rational concern that tensile fracture may occur before the brace develops its full elongation in tension. Therefore, the 2005 AISC seismic provisions require that the required tensile capacity of the brace should be greater than the expected yield strength of the brace. That is, this requirement implies that

$$R_y A_g F_y < \phi A_e F_t \tag{8.26a}$$

While the goal of this check is very rational, the actual application of this equation is not totally logical, because the gross area and effective net area occur within the same steel section, and extreme variations in material properties are not expected. Further, this requirement leads to extreme conservatism in the design of the net section. As a result, a special exemption is permitted when the gross area and net section occur within the same member. For these cases,

$$R_y A_g F_y < R_t A_e F_t \tag{8.26b}$$

Bracing contributes most of the lateral strength and stiffness to concentrically braced frames. As noted earlier, bracing also dissipates energy through postbuckling compressive and tensile yielding. Bracing systems that resist too large a portion of the seismic shear force of the frame through either tension or compression sustain greater pinching of their hysteretic behavior and greater deterioration of resistance. As a result, all bracing systems must be designed so that at least 30%, but no more than 70%, of the base shear is carried by bracing acting in tension, while the balance is carried by bracing acting in compression.

Local buckling is also a major concern. Brace elements that are too slender may sustain local buckling, which results in deterioration in resistance or early fracture of the brace. As a result, the local slenderness of angle bracing elements is restricted for special concentrically braced frames to

$$\frac{b}{t} \le 0.30\sqrt{\frac{E}{F_y}} \tag{8.27}$$

where b and t are the leg width and thickness of the angle, respectively. The slenderness of bracing for hollow rectangular and circular tubes of high-strength steel are likewise limited such that

$$\frac{b}{t} \text{ and } \frac{h}{t} \le 0.64\sqrt{\frac{E}{F_y}} \qquad \text{(for hollow rectangular tubes)} \tag{8.28}$$

$$\frac{D}{t} \le 0.94\sqrt{\frac{E}{F_y}} \qquad \text{(for hollow circular tubes)} \tag{8.29}$$

where t is the wall thickness of the tube, D is the diameter of a circular tube, and b and h are the width and depth in compression for a rectangular tube. Beyond the restrictions noted above, the bracing may be compact or noncompact but must not exceed the limit for slender members in the AISC specification.

Connection Strength. The strength of the connections should be stronger than the members themselves, because connection behavior is more complex and less predictable than member behavior, and premature failure of the connection may result in significant reduction in structural ductility. Therefore, the nominal connection resistance (ϕR_n) for all expected behaviors must exceed the lesser of the expected tensile resistance of the brace ($R_y A_g F_y$) or the maximum load effect that can be transferred to the brace. This requirement assures that the energy dissipation occurs in the members rather than the connections.

Selection of R. Once concentric bracing is selected for seismic design, the force reduction factor, R, must be chosen. The discussion to this point has focused on special braced frames which have $R = 6$. This R value is somewhat smaller than that permitted for special steel moment-resisting frames, because concentrically braced frames are known to be dominated by brace buckling. As a result, their resistance may deteriorate and the brace may fracture under seismic loading. Further improvements to the behavior of concentrically braced frames can be achieved if the special concentrically braced frame is combined with a special-moment frame to form a dual system. This dual system will permit smaller seismic design forces, with an $R = 8$. With this system, the moment frame must be able to resist loads which are at least 25% of the total seismic-design base shear. In addition, both the braced frame and the moment frame must be able to resist their appropriate portion of the loading in accordance with their relative stiffness. The braced frame is usually much stiffer than the moment frame, and so this requirement effectively means that the dual system has a greater total resistance than required by the basic design equations.

Beams in V- or Inverted V-Brace Systems. As noted earlier, brace buckling in the V-braced system results in an unbalanced force on the beam. The consequence of this is significant inelastic deformation of the beam during severe earthquakes and concentration of inelastic damage to a single story of the structural system. The plastic deformation of the beam contributes some energy dissipation, but the negative consequences are quite severe. As a consequence, the beams of special concentrically braced frames with V-bracing or inverted V-bracing must be designed so that the beam has adequate bending resistance to resist the unbalanced brace forces after buckling occurs. This requires a significant increase in the beam size, and options such as the zipper column illustrated in Fig. 8.6 are used to overcome this restriction.

Ordinary Concentrically Braced Frames. Special concentrically braced frames require that the braced frame satisfy the requirements summarized previously. Historically, designers have had the option of designing ordinary concentrically braced frames for larger seismic design forces (smaller R values) and reduced ductility and detailing requirements. The option of ordinary concentrically braced frames is still available to the structural engineer. This bracing system can be designed for $R = 5$. However, the benefits in reduced detailing requirements are modest. Detailing requirements for ordinary concentrically braced frames have become increasingly closer to the requirements for special concentrically braced frames. As a result, the potential economic benefit of ordinary concentrically braced frames has been reduced, and no further discussion of this option will be provided.

8.7.3 Eccentric Braced Frames

Eccentric braced frames combine the strength and stiffness of a concentric braced frame with the inelastic performance of a special moment-resisting frame (Fig. 8.7*c*). An R value of 8 is permitted for an eccentric braced frame. This results in seismic-design forces comparable to those required for special moment-resisting frames if the fundamental period of vibration is the same. However, braced frames are invariably stiffer than moment-resisting frames of similar geometry and have a shorter period. This results in a somewhat larger design load than for special moment-resisting frames under comparable conditions. (C. W. Roeder and E. P. Popov, "Eccentrically Braced Steel Frames for Earthquakes," *Journal of Structural Division*, March 1978, American Society of Civil Engineers.)

General Requirements for Ductility. There are a number of special design provisions that must be satisfied by eccentric braced frames. As defined in Art 8.4, a link must be provided at least at one end of each brace. The link beam should be designed so that it is the weak link of the structure under severe seismic loading. This is done by selecting the size of the steel section and the length of the link beam to match seismic-load design requirements. The weak link is assured by the requirement that the brace be designed for a force at least 1.25 times the brace force necessary to yield the link beam considering the expected yield strength (R_yF_y) of the link beam. Yielding or buckling of the columns must also be avoided. Therefore, the column must be designed for the combined axial force of 1.1 times the sum of the expected nominal shear strength of all link beams above the level under consideration. These brace and column design forces are needed to ensure that the brace and column do not buckle as the link-beam strain hardens during inelastic deformation.

Link Beam. Eccentrically braced frames develop good inelastic behavior because yielding in the link beam occurs well before brace buckling or inelastic deformation of the columns, and this yielding permits large inelastic deformations and great energy dissipation during severe earthquakes. The link beam may yield in shear, flexure or a combination of the two depending on the size of the beam and the length of the link. The normal yield shear of the link beam is the lesser of V_p or $2M_p/e$. In this expression, M_p is the normal link beam plastic moment ($M_p = ZF_y$) and e is the clear span eccentricity of the link beam. The plastic shear capacity V_p of the link beam is

$$V_p = 0.60F_y(d - 2t_f)t_w \tag{8.30}$$

where d, t_f, and t_w are the depth, flange thickness, and web thickness of the link beam. The nominal yield shear and moment of the link beam may require further reduction if the axial force in the link beam exceeds 15% of the yield axial force. These reduced capacities are

$$V_{pa} = V_p\sqrt{1-\left(\frac{P_u}{P_y}\right)^2} \tag{8.31a}$$

and

$$M_{pa} = M_p\left(1-\frac{P_u}{P_y}\right) \tag{8.31b}$$

However, it is not very common to design eccentrically braced frames with large axial forces in the link beams.

In general, link beams yielding in shear are preferred, because they have significantly larger inelastic deformation capacity. Link beams for which

$$e \le \frac{1.6M_p}{V_p} \tag{8.32a}$$

are controlled by shear yield behavior, and they have a maximum plastic link rotational angle of 0.08 rad. Link beams for which

$$e \ge \frac{2.6M_p}{V_p} \tag{8.32b}$$

are controlled by flexural yield behavior, and they have a maximum plastic link rotational angle of 0.02 rad. Link beams with lengths between these two limits are intermediate links, and their rotational limit is determined by interpolation. The rotational limit must be compared to the maximum rotation predicted for the link in the analysis of the system under seismic loading.

Stiffeners and Lateral Support of the Link Beam. The link beam is subject to high bending stress, high shear stress, and significant inelastic deformation. As a result, it must have lateral support to both the top and bottom flanges at both ends of the link beam. The lateral supports must have adequate resistance to develop 6% of the expected flange force ($R_y F_y Z/h$). The beam must also satisfy all of the web and flange slenderness requirements previously noted for special moment-resisting

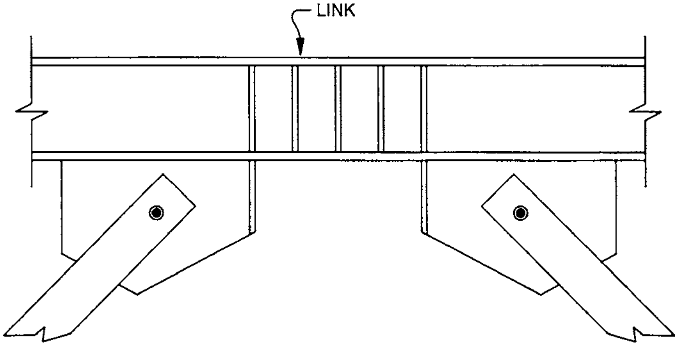


FIGURE 8.15 Typical connection details and stiffener arrangement for an eccentric braced frame.

frames. Full-depth web stiffeners are also required at each end of the link beam, as illustrated in Fig. 8.15. The high shear stress in the web of the link beam results in the potential for web buckling during large inelastic cycles, and intermediate web stiffeners are also likely to be required as shown in Fig. 8.15. Spacing *s* for intermediate stiffeners for link beams with link rotation angle of 0.08 rad is

$$s = 30t_w - \frac{d}{5} \tag{8.33a}$$

When the link rotation angle is 0.02 rad or less,

$$s = 52t_w - \frac{d}{5} \tag{8.33b}$$

For link beams with link rotation angle between 0.02 and 0.08 rad, the spacing must be determined by interpolation. The web stiffeners must all be full depth; however, the intermediate stiffeners may sometimes be lighter than the end stiffeners.

Beam Outside the Link. Special considerations are also required for the beam outside the link. The beam outside the link must be designed for forces which are 1.1 times those caused by the brace force necessary to yield the link beam considering the expected yield stress (R_yF_y) of the link beam. Lateral support and slenderness requirements are required for the beam outside the link.

Beam–Column Connections. Eccentrically braced frames with beam–column connections at one end of the link beam must satisfy all moment-frame connection requirements as discussed in Art. 8.7.1.

Eccentrically Braced Frames in Dual Systems. Eccentrically braced frames may also be designed as part of dual systems with special moment-resisting frames. With dual systems, the special moment-resisting frame must be able to resist loads which are at least 25% of the total seismic-design base shear. In addition, both the eccentrically braced frame and the moment frame must be able to resist their appropriate portion of the loading in accordance with their relative stiffness.

General Comments. Doubler plates and holes or penetrations are not permitted in the link beams. The connections must be strong enough to develop fully the plastic capacity of the link beams. Link beams that are connected directly to columns require the same experimental verification as is presently required for special steel moment-resisting frames.

Eccentrically braced frames are a rational attempt to design steel structures that fully develop the ductility of the steel without loss of strength and stiffness due to buckling. The design of these frames is somewhat more complicated than that of some other steel frames, but eccentric braced frames offer advantages in economical use of steel and seismic performance that cannot be duplicated by other systems.

8.7.4 Limitations on Buckling-Restrained Braced Frames

Buckling-restrained braced frames, illustrated in Fig. 8.11, are a relatively new seismic design concept. These items are patented products provided by a number of different suppliers. The axial force-deformation behavior of these braces is very good, as illustrated in Fig. 8.7*d*. However, the performance of the system depends on many factors in addition to the buckling-restrained brace itself. Buckling-restrained braces are being used with increasing frequency, and are treated in the 2005 AISC seismic provisions. It is likely that more substantial recommendations will be included in future editions of these provisions, as the current provisions require extensive testing to document the performance of the buckling-restrained brace, and the structural system.

Buckling-restrained braces are typically a flat bar or cruciform section encased within a steel shell, as depicted in Fig. 8.11. The flat bar is not bonded to the encasing element, but the encasing element is stiff enough to prevent buckling of the bar in compression. Because the axial bar yields in both axial tension and compression, significant strain hardening is expected. These strain-hardening values vary from system to system, but the strain hardening is commonly between 30% and 50% of the yield resistance. Because the strain hardening places additional demands on columns and connections, the AISC seismic provisions require that the strain hardening be accurately estimated for the buckling-restrained bracing system to obtain an adjusted brace strength. The columns and gusset plate connections must then be designed for the expected maximum brace strength including the strain hardening. Testing and verification of the performance of the buckling-restrained brace is currently required, but in many cases the manufacturers may have performed tests that provide adequate verification.

While most testing of buckling-restrained braces has been performed on individual brace elements, very recent tests have shown that system behavior may be different from that implied by tests performed on the brace only. This occurs primarily because of the stiff gusset plate connections that result with this bracing system, and the frame restraint that is produced by this connection stiffness. Research is in progress to improve connection design criteria, and engineers should be continually aware of new developments in this area.

8.8 FORCES IN FRAMES SUBJECTED TO LATERAL LOADS

The design loads for wind and seismic effects are applied to structures in accordance with the guidelines in Arts. 8.2 to 8.5. Next, the structure must be analyzed to determine forces and moments for design of the members and connections. Member and connection design proceeds quite normally for wind-load design after these internal forces are determined, but seismic design is also subject to the detailed ductility considerations described in Arts. 8.6 and 8.7. Today, steel frames are nearly always designed with the aid of a computer analysis to consider the frame stiffness, deformation, and distribution of forces. However, approximate analysis methods are desirable for preliminary analysis and initial member sizing needed prior to development of computer models. Two such methods for frames subject to lateral loads are the portal and cantilever methods.

The portal method is used for buildings of intermediate or shorter height. In this method, a bent is treated as if it were composed of a series of two-column rigid frames, or portals. Each portal shares one column with an adjoining portal. Thus, an interior column serves as both the windward column of one portal and the leeward column of the adjoining portal. Horizontal shear in each story is distributed in equal amounts to interior columns, while each exterior column is assigned half the shear for an interior column, since exterior columns do not share the loads of adjacent portals. If the bays are unequal, shear may be apportioned to each column in proportion to the lengths of the girders it supports. When bays are equal, the axial load in interior columns due to lateral load is zero.

Inflection points (points of zero moment) are placed at midheight of the columns and midspan of beams. This approximates the deflected shapes and moment diagrams of those members under lateral loads. The location of the inflection points may be adjusted for special cases, such as fixed or pinned base columns, or roof beams and top-story columns, or other special situations. On the basis of the preceding assumptions, member forces and bending moments can be determined entirely from the equations of equilibrium. As an example, Fig. 8.16 indicates the geometry and loading of an eight-story moment-resisting frame, and Fig. 8.17 illustrates the use of the portal method on the upper stories of

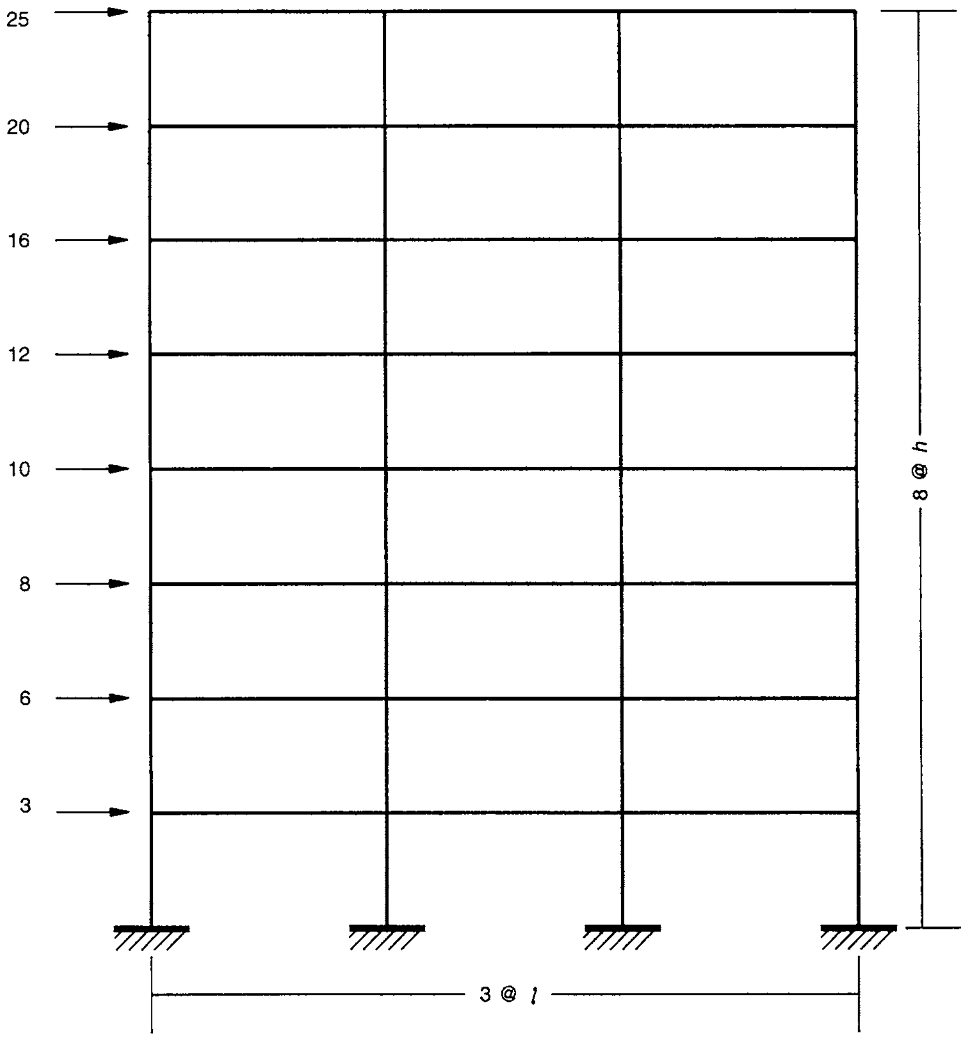


FIGURE 8.16 Eight-story moment-resisting frame subjected to static lateral loading.

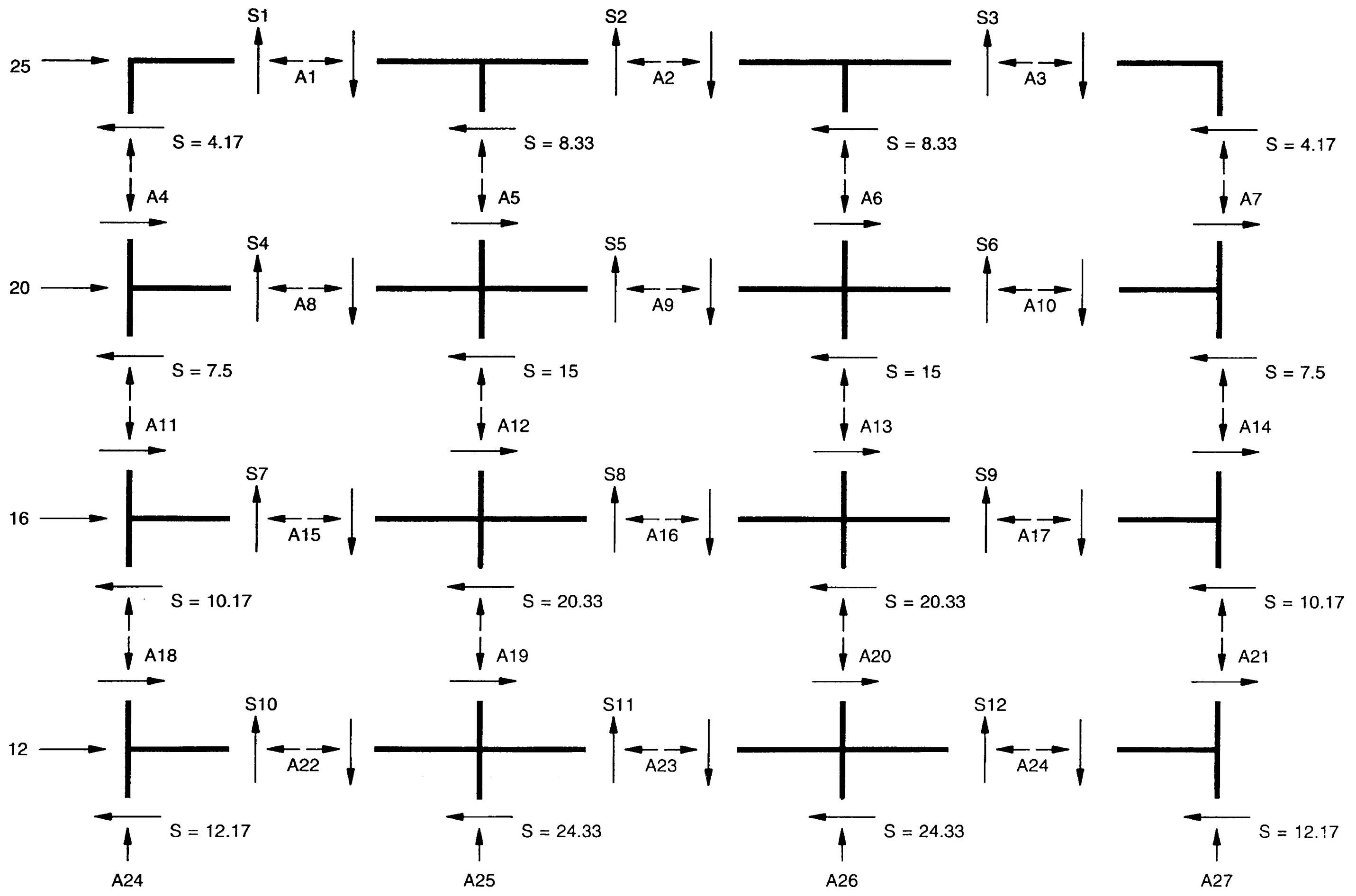


FIGURE 8.17 Forces at midspan of beams and midheight of columns in the frame of Fig. 8.16 as determined by the portal method.

the frame. The frame has two interior columns. So one-third of the shear in each story is distributed to the interior columns and half of this, or one-sixth, is distributed to the exterior columns (Fig. 8.17). The other member forces are computed by equations of equilibrium on each subassemblage. For example, for the subassemblage at the top of the frame in Fig. 8.17, setting the sum of the moments equal to zero yields

$$\frac{l}{2}S_1 = 4.17\frac{h}{2} \qquad \text{or} \qquad S_1 = 4.17\frac{h}{l} \tag{8.34}$$

Setting the sum of the vertical forces equal to zero gives

$$A_4 = -S_1 = -4.17\frac{h}{l} \tag{8.35}$$

Setting the sum of the horizontal forces equal to zero results in

$$A_1 = 25 - 4.17 = 20.83 \tag{8.36}$$

For the central top subassemblage:

$$\frac{l}{2}(S_1 + S_2) = 8.33\frac{h}{2} \qquad \text{or} \qquad S_2 = \frac{h}{l}(8.33 - 4.17) = 4.16\frac{h}{l} \tag{8.37}$$

The remaining axial and shear forces can be determined by this procedure, and bending moments can be determined directly from these forces from equilibrium equations.

The cantilever method is used for tall buildings. It is based on the recognition that axial shortening of the columns contributes to much of the lateral deflections of such buildings (Fig. 8.18). In this method, the floors are assumed to remain plane, and the axial force in each column is assumed to be proportional to the distance of the column from the centroid of the columns. Inflection points are assumed to occur at midheight of the columns and at midspan of the beams. The internal moments and forces are determined from equations of equilibrium, as with the portal method. The determination of the forces and moments in the members at the top floors of the frame in Fig. 8.16 is illustrated in Fig. 8.19. The lateral forces cause overturning moments, which induce axial tensile and compressive forces in the columns. Therefore,

$$A_4 = -A_7 \qquad \text{and} \qquad A_5 = -A_6 \tag{8.38}$$

Since the exterior columns are three times as far from the centroid of the columns as the interior columns,

$$A_4 = 3A_5 \tag{8.39}$$

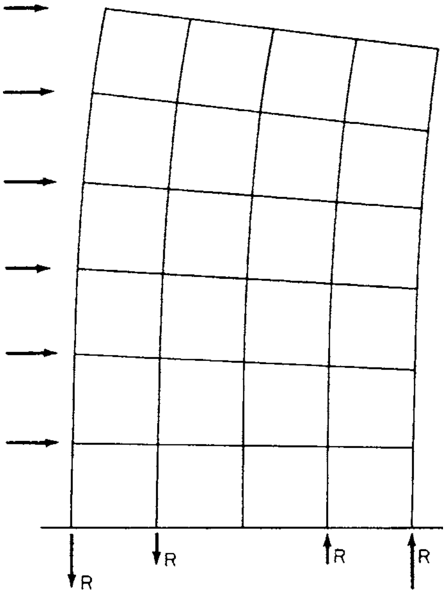


FIGURE 8.18 Drift of a moment-resisting frame assumed for analysis by the cantilever method.

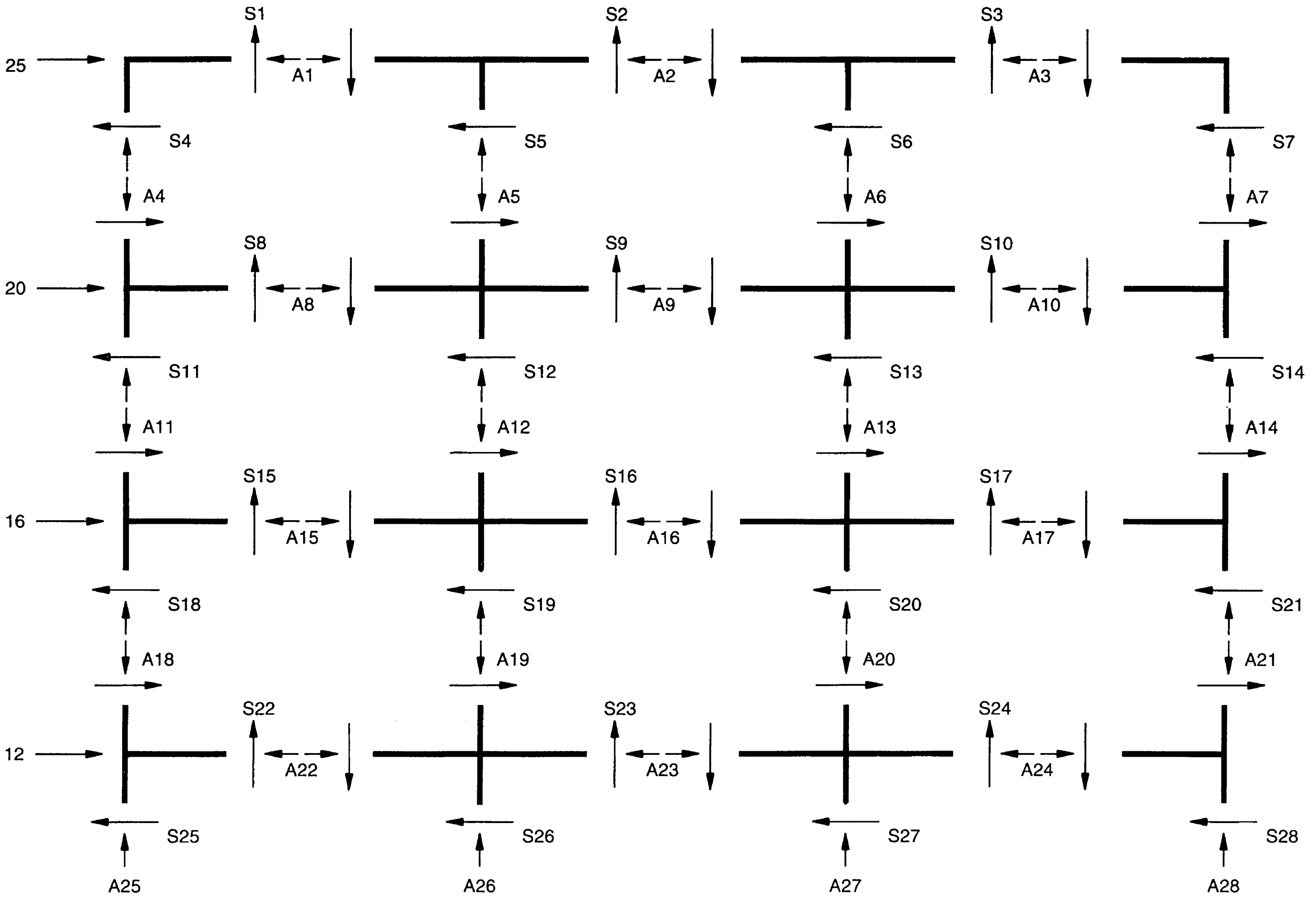


FIGURE 8.19 Cantilever method of determining the forces at midspan of beams and midheight of columns in the frame of Fig. 8.16.

Setting the sum of the moments equal to zero gives

$$3lA_4 + lA_5 = 25\frac{h}{2} \qquad \text{and} \qquad A_5 = 1.25\frac{h}{l} \tag{8.40}$$

Axial forces in the columns can be determined at other levels by the same procedure. Other shear forces and bending moments can be determined by application of the equations of equilibrium for individual subassemblages, as for the portal method.

Analysis of Dual Systems. Approximate analysis of braced frames can be performed as if the bracing were a truss. However, many braced structures are dual systems that combine moment-resisting-frame behavior with braced-frame behavior. Under these conditions, an approximate analysis can be performed by first distributing the lateral forces between the braced-frame and moment-resisting-frame portions of the structure in proportion to the relative stiffness of the components. Braced frames are commonly very stiff and normally would carry the largest portion of the lateral loads. Once this distribution is completed, the moment-resisting frame can be approximately analyzed by the portal or cantilever method and the braced frame can be analyzed as a truss.

8.8.1 Linear Elastic Computer Analysis

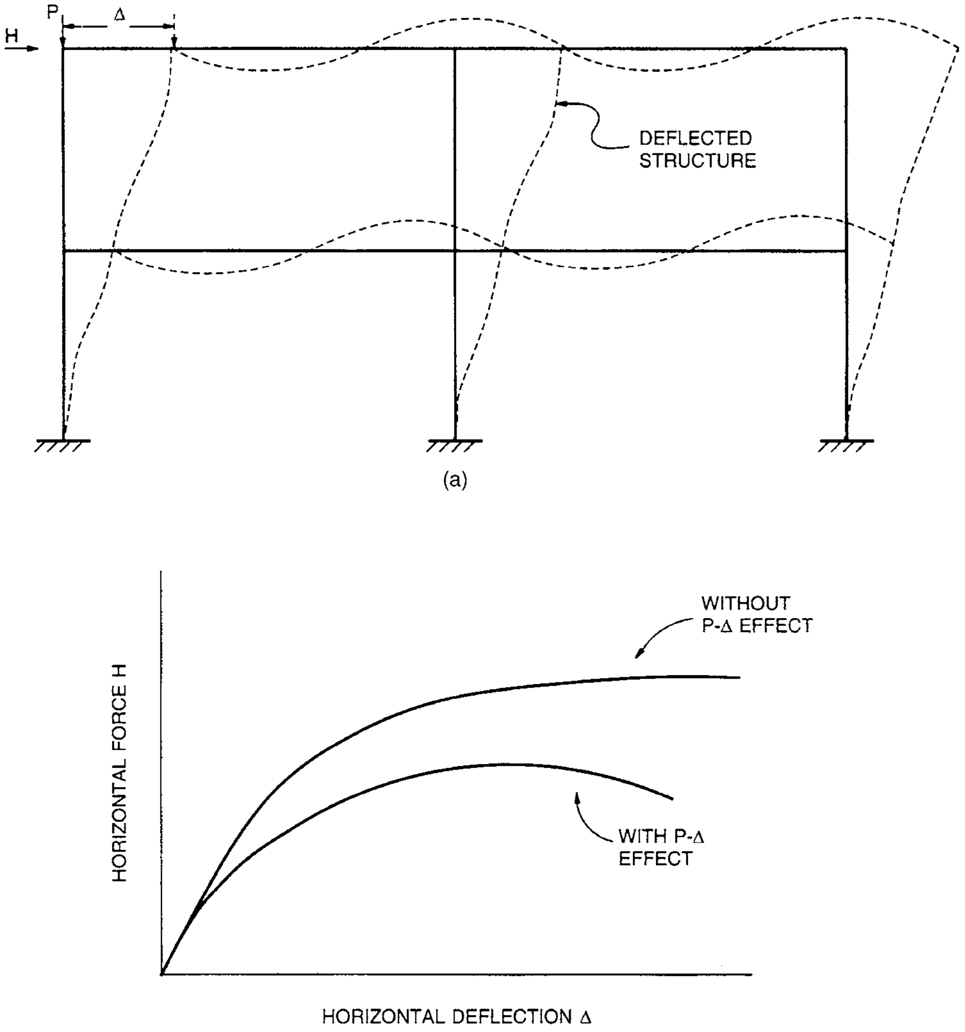
Initial estimates of member and connection forces can be used to complete a preliminary design. At this time, it is possible to reanalyze the structure by any of a number of linear-elastic, finite-element computer programs. While many major, existing structures were designed without benefit of computer analysis techniques, it is not advantageous to design modern buildings for wind and earthquake loading without this capability. It is needed to predict realistic structural response to wind loading and to evaluate occupant comfort, as well as for dynamic design for seismic loading, especially for buildings of unusual geometry. Both the seismic and wind-load provisions in ASCE 7 standards result in local anomalies in the distribution of design forces due to the distribution of mass, stiffness, or local wind pressure, and many elements such as slabs and diaphragms may distribute large forces from one load element to another. Connections in braced frames are not true pins, and many moment-resisting frame connections are not truly rigid, fully restrained, connections. The consequences of these variations may have a significant impact on structure performance, and finite-element analysis provides a much more realistic indication of system performance. The combination of these factors results in the need for finite-element analysis.

8.8.2 Nonlinear Analysis of Structural Frames

Although nonlinear analysis is not commonly used for structural design, it is important for seismic design for several reasons. First, while the seismic-design provisions of various building codes rely on linear-elastic concepts, they are based on inelastic response. Seismic behavior of structures during major earthquakes depends on nonlinear material behavior caused by yielding of steel and cracking of concrete. The reduced stiffness due to yielding makes the stability of structures of great concern, and ensuring stability requires consideration of geometric nonlinearities. Nonlinear analysis permits treatment of these stability effects with P–Δ moments (Fig. 8.20).

Second, design methods such as load-and-resistance-factor design encourage use of flexible, partially restrained (PR) connections. Such connections are inherently nonlinear in their response. Hence, it is necessary to analyze structures with attention to the contribution of connection flexibility. Further nonlinearity may occur due to the effects of connection flexibility on frame stability and P–Δ moments. These nonlinear effects are not commonly considered in design at present. However, computer programs are available to model nonlinear frame behavior and their use is growing.

Third, current seismic-design criteria are based on life safety and collapse prevention, but structural engineers are increasingly concerned with performance-based design and the evaluation of economic damage at various earthquake excitation levels. FEMA 356 (“NEHRP Guidelines for Seismic



(a)

(b)

FIGURE 8.20 Sidesway of a two-story frame subjected to horizontal and vertical loads. (*a*) Position of deflected structure for drift Δ. (*b*) Curves show relationship of horizontal force and drift with and without the *P*–Δ effects.

Rehabilitation of Buildings," Federal Emergency Management Agency, Washington, D.C., 2002) focuses on this design concept, and recognizes two nonlinear analysis methods for evaluation of seismic performance of buildings. These are the nonlinear static procedure (NSP) and the nonlinear dynamic procedure (NDP). The NSP is commonly known as a pushover analysis, in which a given load pattern is applied to the structure, and the sequence of yielding and deterioration is simulated by increasing the structural deflections and changing the stiffness and resistance, while retaining the shape of the lateral load distribution. Linear elastic and nonlinear analysis computer programs may be employed for the NSP method. Local deformation demands are determined from the local pushover-analysis deformation associated with a global deflection level, which is correlated to the predicted deformation demand from a simplified response-spectrum analysis. Multiple-load patterns are usually employed because a single-load pattern may shield a critical element from inelastic deformation.

The NDP uses direct inelastic dynamic analysis of the structure due to specific ground acceleration records, and corrects the major limitations of the NSP. The NDP component deformation demands are true inelastic dynamic response for the structure under the given excitation. The method requires accurate and reliable nonlinear models of structural behavior, and these models are not available for all structural systems. The local component deformation demands obtained by NSP and NDP are compared to the deformation capacity limits obtained from past experiments on structural components. Many engineers are faced with the use of nonlinear analysis methods for seismic design and evaluation, and these procedures will become increasingly common in the future.

8.9 MEMBER AND CONNECTION DESIGN FOR LATERAL LOADS

Wind loads on steel structures are determined by first establishing the pressure distributions on structures after considering the appropriate design wind velocity, the exposure condition, and the local variation of wind pressure on the structure (Art. 8.2). Then, the wind loads on frames and structural elements are determined by distributing the wind pressure in accordance with the tributary areas and relative stiffness of the various components.

Connections used for wind loading run the full range of unrestrained (pinned), fully restrained (FR), and partially restrained (PR) connections. PR connections are frequently used for wind-loading design, because they are economical and are easily fabricated and constructed.

Designs for wind and seismic loading often use floor slabs and other elements to distribute loads from one part of a structure to another (Fig. 8.21). Under these conditions, the slabs and other elements act as diaphragms. These may be considered deep beams and are subject to loadings and behavior quite different from that encountered in gravity-load design. It is important that this behavior be considered, and it is particularly important that the connections between the diaphragms and the structural elements be carefully designed. These connections often involve a composite connection between a steel structural member and a concrete slab, wall, or other component. The design rules for these composite connections are not as well defined as for most steel connections. However, there is general agreement that the connections should be designed for the largest forces to be transferred at the interface, and the design should recognize that large groups of shear connectors or other transfer elements do not necessarily behave as the sum of the individual elements.

Wind-loading design is based on elastic behavior of structures, and strength considerations are adequate for design of wind connections. Seismic design, in contrast, utilizes inelastic behavior and ductility of structures, and many design factors must be taken into account, besides the strength of members and connections. These requirements are intended to assure that inelastic deformations occur in members capable of developing significant inelastic deformations rather than connections (Art. 8.7).

Seismic-design loads are determined by the equivalent static-force and dynamic methods. With the static-force method, the total base shear is determined by Eq. (8.6). It is distributed to bents and structural elements by simple rules combined with consideration of the distribution of mass and stiffness (Art. 8.4). In the dynamic method, the total range of dynamic modes of vibration are considered in determination of the base shear. This base shear is then distributed in accordance with the mode shapes. For both wind and seismic loading, forces and moments in members and connections can be first estimated by approximate analysis techniques (Art. 8.8.1). They are then usually computed to greater accuracy by linear finite-element analysis methods.

Once member and connection forces and moments are determined, design for wind loading is similar to the design methods used for other loading. However, earthquake loading requires greater concern with establishing a ductile element which controls the inelastic resistance and ductility of the structural system, and balancing the resistance of this ductile element with that of all other elements in the structural system. The AISC seismic provisions and the discussion in Art. 8.7 are focused primarily on this concern. The balancing effect typically enforces a form of capacity-based design, in which the capacity of the ductile structural element establishes the minimum resistance for connections and other, less ductile elements. The consequence of these requirements are that seismic design

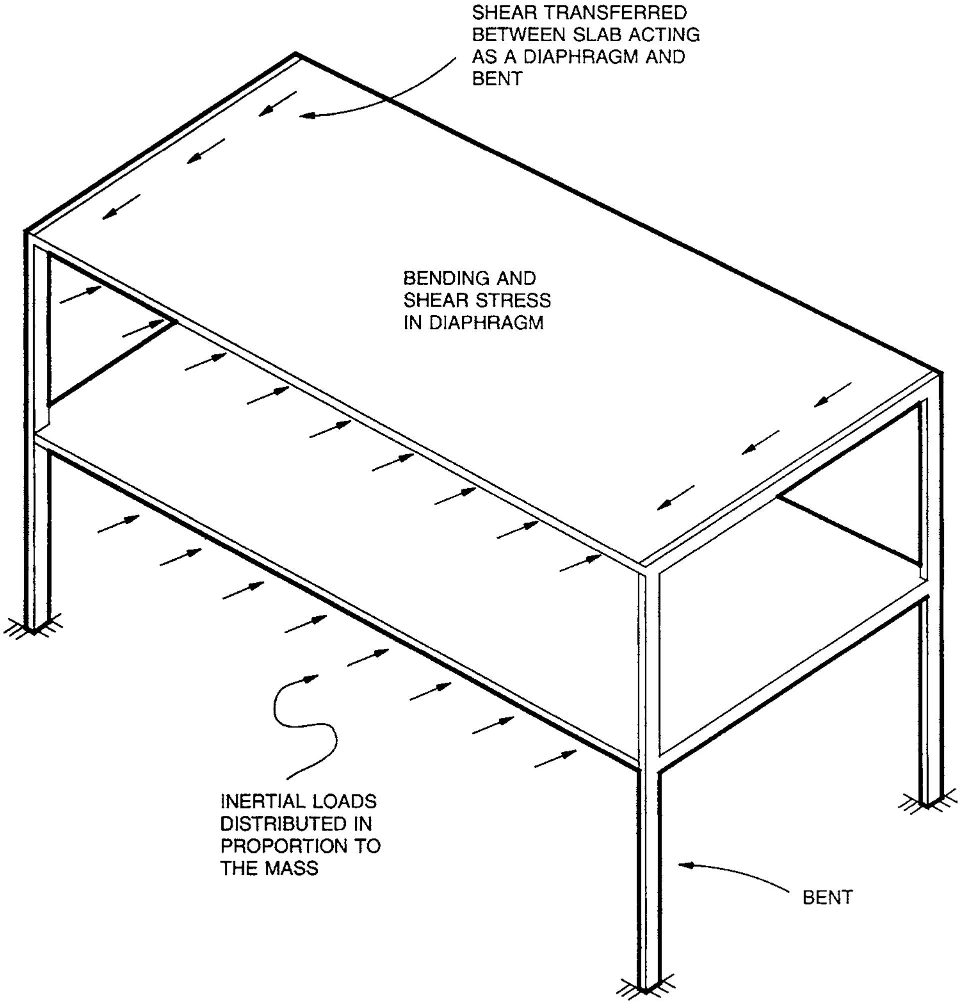


FIGURE 8.21 Slab acting as a diaphragm distributes seismic loads to bents. Bending and shear stresses occur in the diaphragm.

may be somewhat more constrained with respect to the connections and details of the structural design than is wind-loading design.

Connections used in seismic design are normally unrestrained (pinned) or FR connections. PR connections have less seismic resistance than the members that they are connecting, and therefore inelastic deformations during severe earthquakes are concentrated in the connections if PR connections are employed. PR connections have limited energy-dissipation capacity. As a consequence, the total ductility and deformation capacity of a structural frame with PR connections under cyclic loading is uncertain. This uncertainty has limited the use of PR connections in seismic design. Nevertheless, PR connections offer many advantages and may be economical for use in less seismically active regions, rehabilitation projects, and perhaps in the future, major seismic regions. PR connections were investigated in the recent SAC Steel Project funded by FEMA, and FEMA 355D provides an initial overview of their behavior and partial guidelines as to how they may be used in

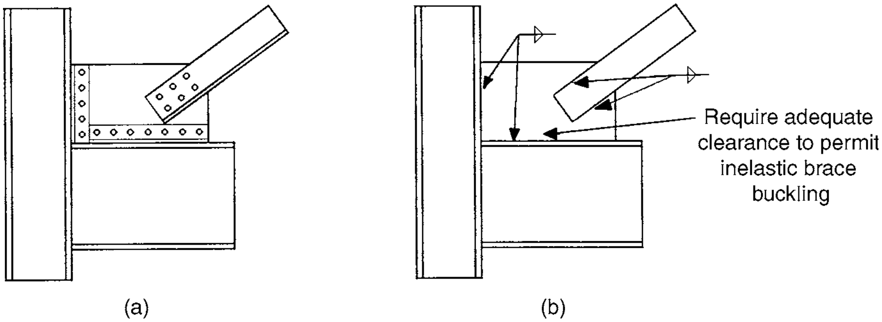


FIGURE 8.22 Typical gusset plate connections. (*a*) Entirely bolted connection. (*b*) Welded gusset plate connection with HSS tube brace such as may be used for seismic design.

engineering practice. It is likely that increased use of PR connections will be possible in future seismic design, because of the improved understanding of the inelastic behavior of these connections.

Connection design for wind loading is primarily an issue of assuring adequate strength and stiffness in the connection. Models used for design of connections should satisfy the equations of equilibrium, and must assure an adequate path for all forces and moments to pass through the connection. Seismic design of connections requires additional concerns related to system ductility and inelastic performance. Seismic-resistant connections for moment frames are discussed in some detail in Arts. 8.6 and 8.7.1. The AISC seismic design provisions, FEMA 350 and FEMA 355D, provide considerable guidance for designing and evaluating these connections, as noted earlier. Braced frames normally employ gusset plate connections such as are illustrated in Fig. 8.22, and seismic design requirements for these connections are not so well established. These connections are designed by variations of the uniform-force method defined in the AISC "Manual of Steel Construction." In general, bolted variations of these connections are commonly used for wind load and regions where seismic-design loads are not very large. However, fully welded variations with tubular bracing are common in regions with large seismic demands. The uniform-force method was not developed for seismic design with the goal of system ductility. The method assures that the connection has adequate strength to resist the factored loads in the brace, but it does not consider the balanced behavior needed to assure ductility of the structural system under severe earthquake loading. Practicing engineers therefore employ wide variations in their adaptation of the uniform-force method to seismic design. Unfortunately these differences in practice may produce very different connection performance. Research work is currently in progress to better understand and improve the seismic performance of gusset plate connections. Hence, it is likely that there will be continuing changes in the design models for these connections for lateral load design.

CHAPTER 9

COLD-FORMED STEEL DESIGN

Roger L. Brockenbrough, P.E.
President
R. L. Brockenbrough & Associates, Inc.
Pittsburgh, Pennsylvania

R. A. LaBoube, Ph.D., P.E.
Distinguished Teaching Professor
Department of Civil Engineering
University of Missouri
Rolla, Missouri

This chapter presents information on the design of structural members that are cold-formed to cross-section shape from sheet steels. Cold-formed steel members include such products as purlins and girts for the construction of metal buildings, studs and joists for light commercial and residential construction, supports for curtain wall systems, formed deck for the construction of floors and roofs, standing seam roof systems, and a myriad of other products. These products have enjoyed significant growth in recent years and are frequently utilized in some shape or form in many projects today. Attributes such as strength, light weight, versatility, noncombustibility, and ease of production make them cost-effective in many applications. Figure 9.1 shows cross sections of typical products.

9.1 DESIGN SPECIFICATIONS AND MATERIALS

Cold-formed members for most applications are designed in accordance with the *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2001, a consensus document used throughout Canada, Mexico, and the United States. In the United States it is published by the American Iron and Steel Institute, Washington, D.C., and referred to herein as the *AISI North American Specification* (*AISI NAS*). (See also *Supplement No. 1*, 2004.) In Canada the document is published by the Canadian Standards Association as *Standard S136*. The technical design provisions for the three countries are similar, except for a few cases covered in Apps. A, B, and C of the document. The *AISI NAS* applies to members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar, not more than 1 in thick, used for load-carrying purposes in buildings. With appropriate allowances, it can be used for other applications as well. The vast majority of applications are in a thickness range from about 0.014 to 0.25 in.

The design information presented in this chapter is based on the *AISI NAS* and its *Commentary.* The design equations are written in dimensionless form, except as noted, so that any consistent system of units can be used. A synopsis of key design provisions is given in this chapter, but reference should be made to the complete Specification and commentary for a more complete understanding.

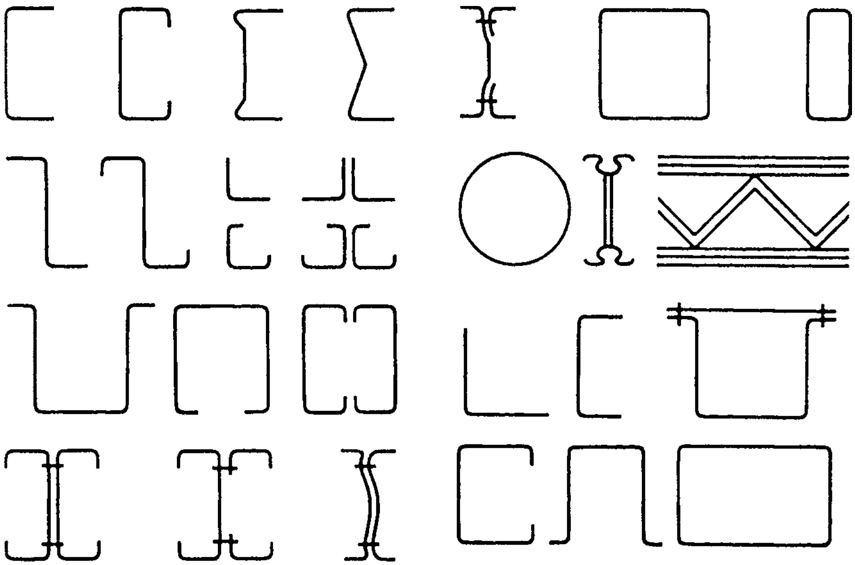


FIGURE 9.1 Typical cold-formed steel members.

The *AISI NAS* lists all of the sheet and strip materials included in Table 1.6 (Art. 1.4) as applicable steels, as well as several of the plate steels included in Table 1.1 (A36, A242, A588, and A572). A283 and A529 plate steels are also included, as well as A500 structural tubing (Table 1.7). Other steels can be used for structural members if they meet ductility and other requirements. The basic requirement is a ratio of tensile strength to yield stress not less than 1.08 and a total elongation of at least 10% in 2 in. If these requirements cannot be met, alternative criteria related to local elongation may be applicable. In addition, certain steels that do not meet the criteria, such as SS Grade 80 (under ASTM A653, A1008, A792, or A875), can be used for multiple-web configurations (roofing, siding, decking, etc.), provided the yield stress is taken as 75% of the specified minimum (or 60 ksi or 414 MPa, if less) and the tensile stress is taken as 75% of the specified minimum (or 62 ksi or 428 MPa if less). Some exceptions apply. Suitability can also be established by structural tests.

9.2 MANUFACTURING METHODS AND EFFECTS

As the name suggests, the cross section of a cold-formed member is achieved by a bending operation at room temperature, rather than the hot-rolling process used for the heavier structural steel shapes. The dominant cold-forming process is known as roll-forming. In this process, a coil of steel is fed through a series of rolls, each of which bends the sheet progressively until the final shape is reached at the last roll stand. The number of roll stands may vary from 6 to 20, depending on the complexity of the shape. Because the steel is fed in coil form, with successive coils weld-spliced as needed, the process can achieve speeds up to about 300 ft/min and is well suited for quantity production. Small quantities may be produced on a press-brake, particularly if the shape is simple, such as an angle or channel cross section. In its simplest form, a press brake consists of a male die which presses the steel sheet into a matching female die.

In general, the cold-forming operation is beneficial in that it increases the yield stress of the material in the region of the bend. The flat material between bends may also show an increase due to squeezing or stretching during roll forming. This increase in strength is attributable to cold working and strain aging effects as discussed in Art. 1.10. The strength increase, which may be small for sections with few bends, can be conservatively neglected. Alternatively, subject to certain limitations, the *AISI NAS* includes provisions for using a section-average *design yield stress* that includes the strength increase from cold-forming. Either full-section tension tests, full-section stub column tests, or an analytical method can be employed. Important parameters include the tensile-strength-to-yield-stress ratio of the virgin steel and the radius-to-thickness ratio of the bends. The forming operation may also induce residual stresses in the member, but these effects are accounted for in the equations for member design.

9.3 NOMINAL LOADS

The nominal loads for design should be according to the applicable code or specification under which the structure is designed or as dictated by the conditions involved. In the absence of a code or specification, the nominal loads in the United States and Mexico should be those stipulated in the American Society of Civil Engineers Standard *Minimum Design Loads for Buildings and Other Structures,* ASCE 7. In Canada, the loads are specified in the *National Building Code of Canada*. The following loads are typically considered.

D = dead load, which consists of the weight of the member itself, the weight of all materials of construction incorporated into the building which are supported by the member, including built-in partitions, and the weight of permanent equipment

E = earthquake load

L = live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance (L includes any permissible load reductions; if resistance to impact loads is taken into account in the design, such effects should be included with the live load)

L_r = roof live load

S = snow load

R_r = rain load, except for ponding

W = wind load

The effects of other loads, such as those due to ponding, should be considered when significant. Also, unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system should be investigated to assure stability under ponding conditions.

9.4 DESIGN METHODS

The *AISI NAS* is structured such that nominal strength equations are given for various types of structural members such as beams and columns. For **allowable strength design** (ASD), the nominal strength is divided by a safety factor and compared to the allowable strength based on nominal loads. For **load and resistance factor design** (LRFD), the nominal strength is multiplied by a resistance factor and compared to the required strength based on factored loads. In Canada, this latter method is known as limit states design (LSD). Both ASD and LRFD are used in the United States and Mexico. LSD is the mandatory method in Canada. Procedures and pertinent load combinations to consider are set forth in the specification as follows.

TABLE 9.1 Safety Factors and Resistance Factors Adopted by the *AISI NAS*

Category	ASD safety factor Ω	LRFD resistance factor ϕ	LSD resistance factor ϕ
Tension members	1.67	0.95	
Flexural members			
(a) Bending strength			
Sections with stiffened or partially stiffened compression flanges	1.67	0.95	0.90
Sections with unstiffened compression flanges	1.67	0.90	0.90
Laterally unbraced beams	1.67	0.90	0.90
Beams having one flange through-fastened to deck or sheathing (*C* or *Z* sections)	1.67	0.90	0.90
Beams having one flange fastened to a standing seam roof system	1.67	0.90	—
(b) Web design			
Shear strength	1.60	0.95	0.80
Web crippling	Varies	Varies	Varies
Transverse stiffeners	2.00	0.85	0.80
Concentrically loaded compression members	1.80	0.85	0.80
Combined axial load and bending			
(a) Tension component	1.67	0.95	0.90
(b) Compression component	1.80	0.85	0.80
(c) Bending component	1.67	0.90/0.95	0.90
Cylindrical tubular members			
(a) Bending	1.67	0.95	0.90
(b) Axial compression	1.80	0.85	0.80
Wall studs			
(a) Compression	1.80	0.85	0.80
(b) Bending	1.67	0.90/0.95	0.90
Diaphragm construction	2.00/3.00	0.50/0.65	0.50
Welded connections			
(a) Groove welds			
Tension or compression	1.70	0.90	0.80
Shear, welds	1.90	0.80	0.70
Shear, base metal	1.70	0.90	0.80
(b) Arc spot welds			
Shear, welds	2.55	0.60	0.50
Shear, connected part	2.20/3.05	0.50/0.70	0.40/0.60
Shear, minimum edge distance	2.20/2.55	0.60/0.70	0.50/0.60
Tension	2.50/3.00	0.50/0.60	0.40/0.50
(c) Arc seam welds			
Shear, welds	2.55	0.60	0.50
Shear, connected part	2.55	0.60	0.50
(d) Fillet welds			
Welds	2.55	0.60	0.50
Connected part, longitudinal loading			
Weld length/sheet thickness <25	2.55	0.60	0.50
Weld length/sheet thickness ≥25	3.05	0.55	0.40
Connected part, transverse loading	2.35	0.60	0.50
(e) Flare groove welds			
Welds	2.55	0.60	0.50
Connected part, longitudinal loading	2.80	0.55	0.45
Connected part, transverse loading	2.80	0.55	0.45

(*Continued*)

TABLE 9.1 Safety Factors and Resistance Factors Adopted by the *AISI NAS* (*Continued*)

Category	ASD safety factor Ω	LRFD resistance factor ϕ	LSD resistance factor ϕ
(f) Resistance welds	2.35	0.65	0.55
Bolted connections			
(a) Minimum spacing and edge distance*			
When $F_u/F_{sy} \geq 1.08$	2.00	0.70	0.75
When $F_u/F_{sy} < 1.08$	2.22	0.60	0.75
(b) Tension strength on net section			
With washers, double shear connection	2.00	0.65	0.55
With washers, single shear connection	2.22	0.55	0.55
Without washers, double or single shear	2.22	0.65	0.55
(c) Bearing strength	2.22/2.50	0.60/0.65	0.50/0.55
(d) Tensile strength of bolts	2.40	0.65	0.65
(e) Shear strength of bolts	2.00/2.25	0.75	0.55
Screw connections	3.00	0.50	0.40

*F_u is tensile strength and F_{sy} is yield stress.

9.4.1 ASD Strength Requirements

A design satisfies the ASD requirements when the allowable design strength of each structural component equals or exceeds the allowable strength, determined on the basis of the nominal loads, for all applicable load combinations. This is expressed as

$$R \leq R_n/\Omega \tag{9.1}$$

where R = allowable strength
R_n = nominal strength (specified in Chaps. B through G of the *AISI NAS*)
Ω = safety factor (see Table 9.1)
R_n/Ω = allowable design strength

9.4.2 LRFD Strength Requirements

A design satisfies the LRFD requirements when the design strength of each structural component equals or exceeds the required strength determined on the basis of the nominal loads, multiplied by the appropriate load factors, for all applicable load combinations. This is expressed as

$$R_u < \phi R_n \tag{9.2}$$

where R_u = required strength
R_n = nominal strength (specified in Chaps. B through G of the *AISI NAS*)
ϕ = resistance factor (see Table 9.1)
ϕR_n = design strength

9.4.3 LSD Strength Requirements

A design satisfies the LSD requirements when the factored resistance of each structural component equals or exceeds the effect of factored loads, for all applicable load combinations. This is expressed as

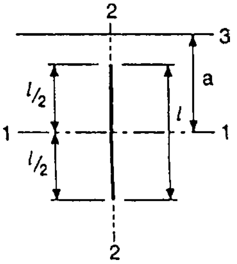
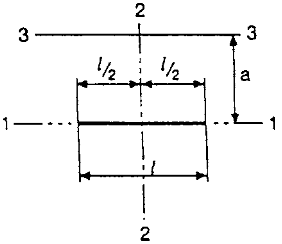
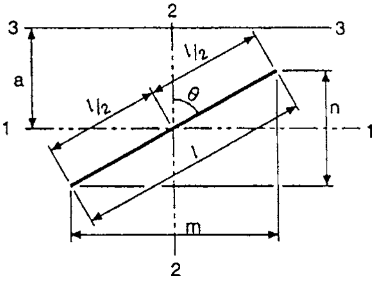
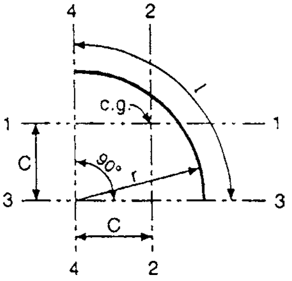
$$\phi R_n > R_f \tag{9.3}$$

where R_f = effect of factored loads
R_n = nominal strength (specified in Chaps. B through G of the *AISI NAS*)
ϕ = resistance factor (see Table 9.1)
ϕR_n = factored resistance

9.5 SECTION PROPERTY CALCULATIONS

Because of the flexibility of the manufacturing method and the variety of shapes that can be manufactured, properties of cold-formed sections often must be calculated for a particular configuration of interest rather than relying on tables of standard values. However, properties of representative or typical sections are listed in the *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, 2002, Washington, D.C. (*AISI Manual*).

TABLE 9.2 Moment of Inertia for Line Elements

$I_1 = \frac{l^3}{12} \qquad I_2 = 0$ $I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$	
$I_1 = 0 \qquad I_2 = \frac{l^3}{12}$ $I_3 = la^2$	
$I_1 = \left(\frac{\cos^2\theta}{12}\right)l^3 = \frac{ln^2}{12}$ $I_2 = \left(\frac{\sin^2\theta}{12}\right)l^3 = \frac{lm^2}{12}$ $I_{12} = \left(\frac{\sin\theta\cos\theta}{12}\right)l^3 = \frac{lmn}{12}$ $I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$	
$l = \frac{\pi r}{2} = 1.57r$ $C = 0.637r$ $I_1 = I_2 = 0.149r^3$ $I_{12} = -0.137r^3$ $I_3 = I_4 = 0.785r^3$ $I_{34} = 0.5r^3$	

Source: Adapted from *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, 2002, Washington, D.C.

Because the cross section of a cold-formed section is generally of a single thickness of steel, computation of section properties may be simplified by using the **linear method.** In this method, the material is considered to be concentrated along the centerline of the steel sheet and **area elements** are replaced by straight or curved **line elements.** Section properties are calculated for the assembly of line elements and then multiplied by the thickness, t. Thus, the cross-section area is given by $A = L \times t$, where L is the total length of all line elements; the moment of inertia of the section is given by $I = I' \times t$, where I' is the moment of inertia determined for the line elements; and the section modulus is calculated by dividing I by the distance from the neutral axis to the extreme fiber, not to the centerline of the extreme element. As discussed later, it is sometimes necessary to use a reduced or **effective width** rather than the full width of an element.

Most sections can be divided into straight lines and circular arcs. The moments of inertia and centroid location of such elements are defined by equations from fundamental theory as presented in Table 9.2.

9.6 EFFECTIVE WIDTH CONCEPT

The design of cold-formed steel differs from heavier construction in that elements of members typically have large width-to-thickness (w/t) ratios and are thus subject to local buckling. Figure 9.2 illustrates local buckling in beams and columns. Flat elements in compression that have both edges parallel to the direction of stress stiffened by a web, flange, lip, or stiffener are referred to as **stiffened elements.** Examples in Fig. 9.2 include the top flange of the channel and the flanges of the *I*-cross-section column.

To account for the effect of local buckling in design, the concept of effective width is employed for elements in compression. The background for this concept can be explained as follows.

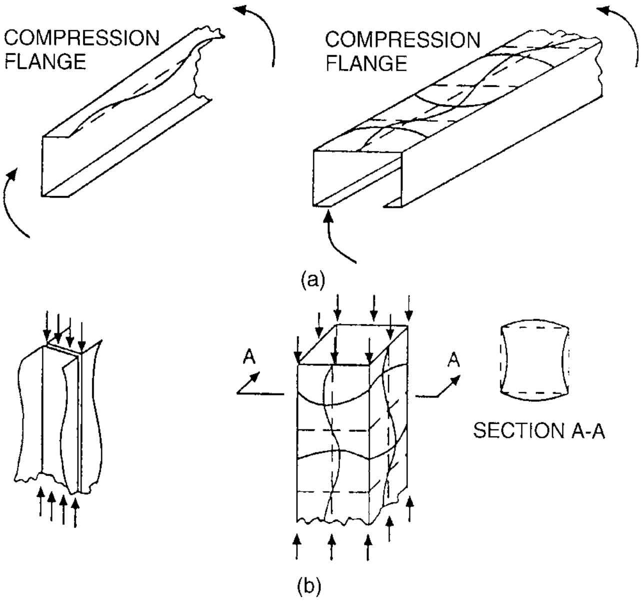


FIGURE 9.2 Local buckling of compression elements. (*a*) In beams. (*b*) In columns. (*Source:* Commentary on the North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

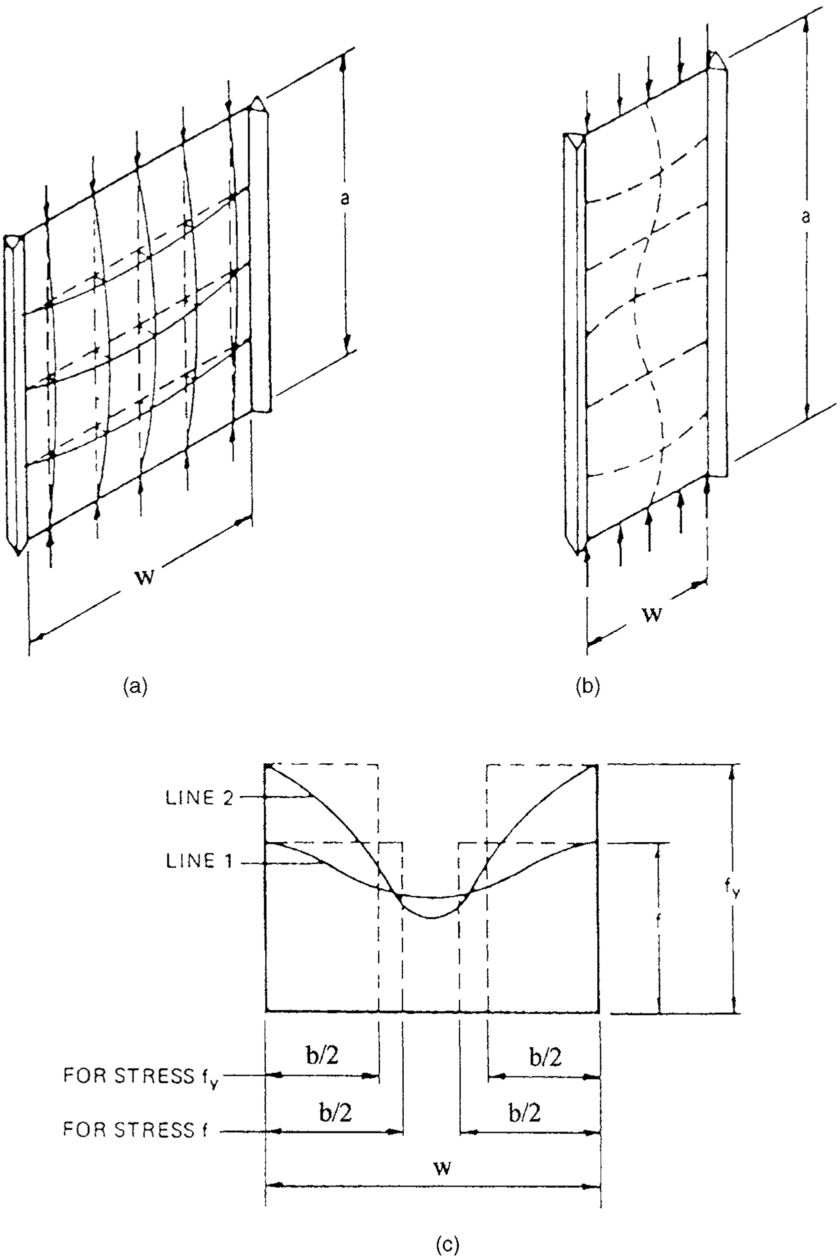


FIGURE 9.3 Effective width concept. (*a*) Buckling of grid model. (*b*) Buckling of plate. (*c*) Stress distributions.

Unlike a column, a plate does not usually attain its maximum load-carrying capacity at the buckling load, but usually shows significant postbuckling strength. This behavior is illustrated in Fig. 9.3, where longitudinal and transverse bars represent a plate that is simply supported along all edges. As the uniformly distributed end load is gradually increased, the longitudinal bars are equally stressed and reach their buckling load simultaneously. However, as the longitudinal bars buckle, the transverse bars develop tension in restraining the lateral deflection of the longitudinal bars. Thus, the longitudinal bars do not collapse when they reach their buckling load but are able to carry additional load because of the transverse restraint. The longitudinal bars nearest the center can deflect more than the bars near the edge, and therefore, the edge bars carry higher loads after buckling than do the center bars.

The postbuckling behavior of a simply supported plate is similar to that of the grid model. However, the ability of a plate to resist shear strains that develop during buckling also contributes to its postbuckling strength. Although the grid shown in Fig. 9.3*a* buckled into only one longitudinal half-wave, a longer plate may buckle into several waves, as illustrated in Figs. 9.2 and 9.3*b*. For long plates, the half-wavelength approaches the width *b*.

After a simply supported plate buckles, the compressive stress will vary from a maximum near the supported edges to a minimum at the mid-width of the plate, as shown by line 1 of Fig. 9.3*c*. As the load is increased the edge stresses will increase, but the stress in the mid-width of the plate may decrease slightly. The maximum load is reached and collapse is initiated when the edge stress reaches the yield stress—a condition indicated by line 2 of Fig. 9.3*c*.

The postbuckling strength of a plate element can be considered by assuming that, after buckling, the total load is carried by strips adjacent to the supported edges which are at a uniform stress equal to the actual maximum edge stress. These strips are indicated by the dashed lines in Fig. 9.3*c*. The total width of the strips, which represents the effective width of the element *b*, is defined so that the product of *b* and the maximum edge stress equals the actual stresses integrated over the entire width. The effective width decreases as the applied stress *f* increases. At maximum load, the stress on the effective width is the yield stress f_y.

Thus, an element with a small enough *w*/*t* will be able to reach the yield point and will be fully effective. Elements with larger ratios will have an effective width that is less than the full width, and that reduced width will be used in section property calculations.

The behavior of elements with other edge-support conditions is generally similar to that discussed above. However, an element supported along only one edge will develop only one effective strip.

Equations for calculating effective widths of elements are given in subsequent articles based on the *AISI NAS*. These equations are based on theoretical elastic buckling theory but modified to reflect the results of extensive physical testing.

9.7 MAXIMUM WIDTH-TO-THICKNESS RATIOS

The *AISI NAS* gives certain maximum width-to-thickness ratios that must be adhered to.

For **flange elements,** such as in flexural members or columns, the maximum flat width-to-thickness ratio, *w*/*t*, disregarding any intermediate stiffeners, is as follows:

Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by

- A simple lip, 60
- Other stiffener with $I_S < I_a$, 90
- Other stiffener with $I_S \geq I_a$, 90

Stiffened compression element with both longitudinal edges connected to other stiffened elements, 500

Unstiffened compression element, 60

In the above, I_S is the moment of inertia of the stiffener about its centroidal axis, parallel to the element to be stiffened, and I_a is the moment of inertia of a stiffener adequate for the element to behave as a stiffened element. Note that, although greater ratios are permitted, stiffened compression elements with $w/t > 250$, and unstiffened compression elements with $w/t > 30$, are likely to develop noticeable deformations at full design strength, but ability to develop required strength will be unaffected.

For **web elements** of flexural members, the maximum web depth-to-thickness ratio, h/t, disregarding any intermediate stiffeners, is as follows:

Unreinforced webs, 200

Webs with qualified transverse stiffeners that include (*a*) bearing stiffeners only, 260; (*b*) bearing and intermediate stiffeners, 300

9.8 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

9.8.1 Uniformly Compressed Stiffened Elements

The effective width for load-capacity determination depends on a slenderness factor λ, defined in terms of the plate elastic buckling stress, F_{cr}, as

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \tag{9.4a}$$

where

$$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \tag{9.4b}$$

With $\mu = 0.30$, this can be restated as

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \tag{9.4c}$$

where k = plate buckling coefficient (4.0 for stiffened elements supported by a web along each longitudinal edge; values for other conditions are given subsequently)
f = maximum compressive stress (with no safety factor applied)
E = modulus of elasticity (29,500 ksi or 203,000 MPa)

For flexural members, when initial yielding is in compression, $f = F_y$, where F_y is the yield stress; when the initial yielding is in tension, f = the compressive stress determined on the basis of effective section. For compression members, f = column buckling stress.

The effective width is as follows:

When $\lambda \le 0.673$, $\quad b = w \quad$ (9.5)

When $\lambda > 0.673$, $\quad b = \rho w \quad$ (9.6)

where the reduction factor ρ is defined as

$$\rho = \frac{1 - 0.22/\lambda}{\lambda} \tag{9.7}$$

Figure 9.4 shows the location of the effective width on the cross section, with one-half located adjacent to each edge.

Effective widths determined in this manner, based on maximum stresses (no safety factor), define the cross section used to calculate section properties for strength determination. However, at service load levels, the effective widths will be greater because the stresses are smaller, and another set of section properties should be calculated. Therefore, to calculate effective width for deflection determination, use the above equations but substitute the compressive stress at design loads, f_d, in Eq. (9.4).

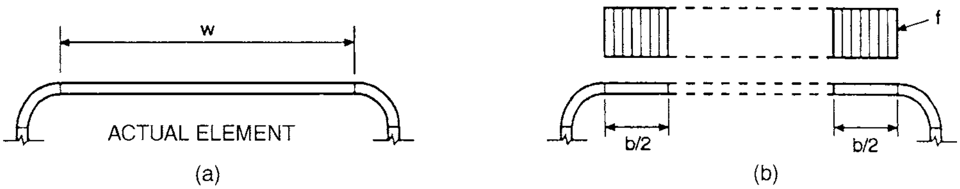


FIGURE 9.4 Illustration of uniformly compressed stiffened element. (*a*) Actual element. (*b*) Stress on effective element. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

9.8.2 Stiffened Elements with Stress Gradient

Elements with stress gradients include webs subjected to compression from bending alone or from a combination of bending and uniform compression. For load-capacity determination, the effective widths b_1 and b_2 illustrated in Fig. 9.5 must be determined. First, calculate absolute value of the ratio of stresses,

$$\psi = \left|\frac{f_2}{f_1}\right| \tag{9.8}$$

where f_1 and f_2 are the stresses as shown, calculated on the basis of effective section, with no safety factor applied. In this case f_1 is compression and f_2 can be either tension or compression. Next, calculate the effective width, b_e, as if the element was in uniform compression (Art. 9.8.1), using f_1 for f and with k determined according to this article.

For beam webs (Fig. 9.5*b*), k is given by

$$k = 4 + 2(1+\psi^3) + 2(1+\psi) \tag{9.9}$$

Effective widths for beam webs are determined from the following equations.

For beam webs with $h_0/b_0 \le 4$,

$$b_1 = \frac{b_e}{3+\psi} \tag{9.10a}$$

$$b_2 = \frac{b_e}{2} \qquad \text{when } \psi > 0.236 \tag{9.10b}$$

$$b_2 = b_e - b_1 \qquad \text{when } \psi \le 0.236 \tag{9.10c}$$

For beam webs with $h_0/b_0 > 4$,

$$b_1 = \frac{b_e}{3+\psi} \tag{9.11a}$$

$$b_2 = \frac{b_e}{1+\psi} - b_1 \tag{9.11b}$$

For other elements with stress gradients (Fig. 9.5*c*), the following apply:

$$k = 4 + 2(1 - \psi^3) + 2(1 - \psi) \tag{9.12a}$$

$$b_1 = \frac{b_e}{3-\psi} \tag{9.12b}$$

$$b_2 = b_e - b_1 \tag{9.12c}$$

The sum of b_1 and b_2 must not exceed the width of the compression portion of the web calculated on the basis of effective section.

Effective width for deflection determination is calculated in the same manner except that stresses are calculated at service-load levels based on the effective section at that load.

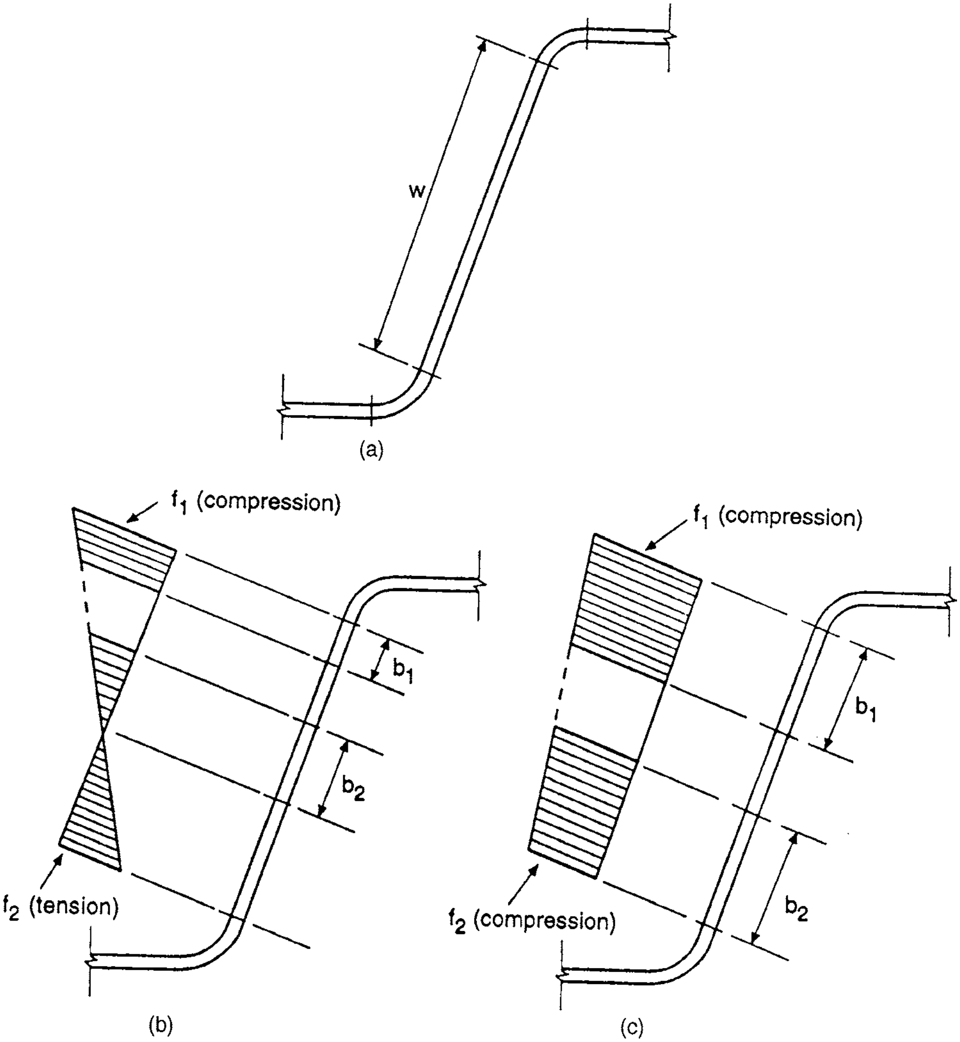


FIGURE 9.5 Illustration of stiffened element with stress gradient. (*a*) Actual element. (*b*) Stress on effective element varying from compression to tension. (*c*) Stress on effective element with nonuniform compression. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

9.9 EFFECTIVE WIDTHS OF UNSTIFFENED ELEMENTS

9.9.1 Uniformly Compressed Unstiffened Elements

The effective widths for uniformly compressed unstiffened elements are calculated in the same manner as for stiffened elements (Art. 9.8.1), except that *k* in Eq. (9.4) is taken as 0.43. Figure 9.6 illustrates the location of the effective width on the cross section.

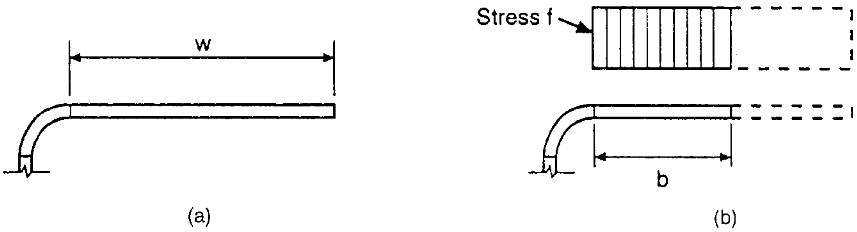


FIGURE 9.6 Illustration of uniformly compressed unstiffened element. (*a*) Actual element. (*b*) Stress on effective element. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

9.9.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

The effective width for an unstiffened element with a stress gradient is calculated as follows:

1. When both f_1 and f_2 are in compression (Fig. 9.7):

- If the stress decreases toward the unsupported edge (Fig. 9.7*a*),

$$k = \frac{0.578}{\psi + 0.34} \tag{9.13a}$$

- If the stress increases toward the unsupported edge (Fig. 9.7*b*),

$$k = 0.57 - 0.21\psi + 0.07\psi^2 \tag{9.13b}$$

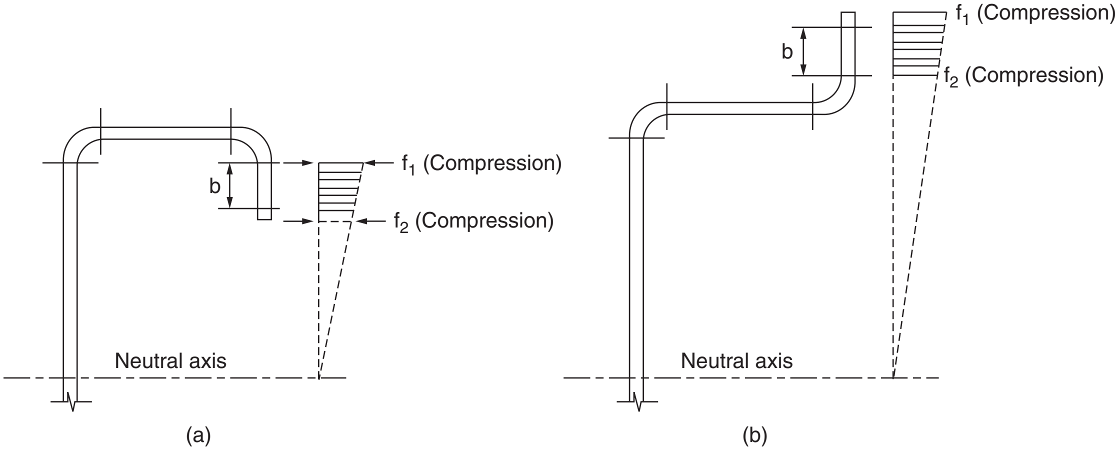


FIGURE 9.7 Illustration of unstiffened compression element with a stress gradient. (*a*) Inward-facing lip. (*b*) Outward-facing lip. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, and 2004 Supplement, with permission.*)

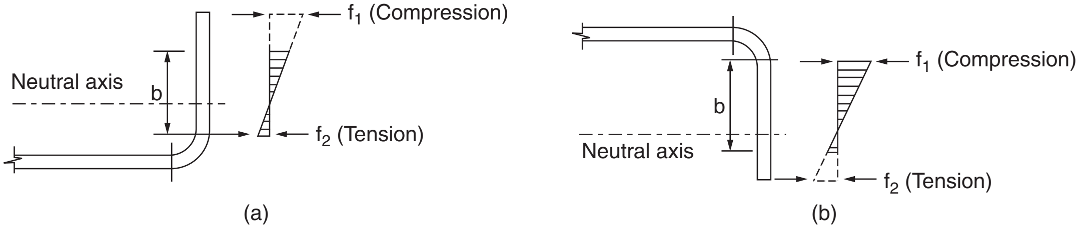


FIGURE 9.8 Illustration of unstiffened compression element with a stress gradient. (*a*) Unsupported edge in compression. (*b*) Supported edge in compression. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, and 2004 Supplement, with permission*.)

2. When f_1 is in compression and f_2 is in tension (Fig. 9.8):

- If the unsupported edge is in compression (Fig. 9.8*a*),

$$\rho = 1 \qquad \text{when } \lambda \le 0.673(1+\psi)$$

$$\rho = (1+\psi)\frac{\left(1-\frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \qquad \text{when } \lambda > 0.673(1+\psi) \tag{9.14}$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \tag{9.15}$$

- If the supported edge is in compression (Fig. 9.8*b*),

For $\psi < 1$

$$\rho = 1 \qquad \text{when } \lambda \le 0.673$$

$$\rho = (1-\psi)\frac{(1-0.22/\lambda)}{\lambda} + \psi \qquad \text{when } \lambda \le 0.673 \tag{9.16}$$

$$k = 1.70 + 5\psi + 17.1\psi^2 \tag{9.17}$$

For $\psi \ge 1$, $\rho = 1$

The effective width for deflection determination is calculated in the same manner except that the stresses are calculated at service-load levels based on the gross section at the load for which serviceability is determined.

9.10 EFFECTIVE WIDTHS OF UNIFORMLY COMPRESSED ELEMENTS WITH EDGE STIFFENER

A commonly encountered condition is a flange with one edge stiffened by a web, the other by an edge stiffener (Fig. 9.9). To determine its effective width for load-capacity determination, one of three cases must be considered. The case selection depends on the relation between the flange flat width-to-thickness ratio, *w*/*t*, and the parameter *S*, defined as

$$S = 1.28\sqrt{\frac{E}{f}} \tag{9.18}$$

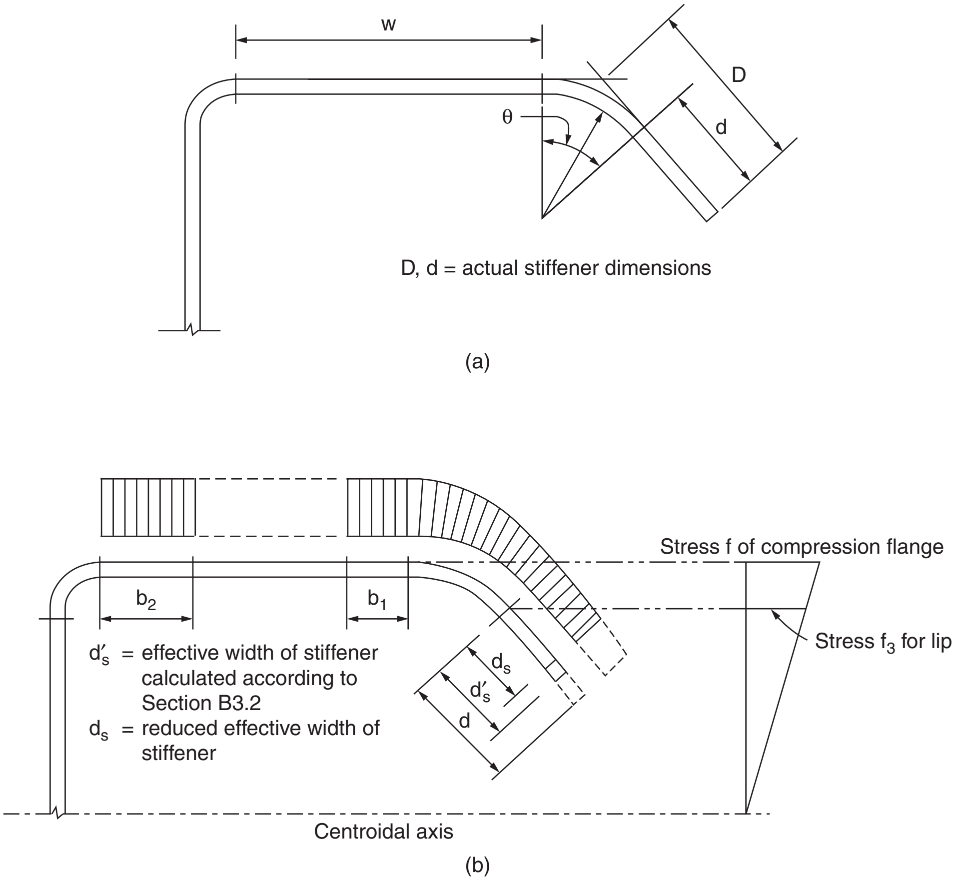


FIGURE 9.9 Illustration of element with edge stiffener. (*a*) Actual element. (*b*) Stress on effective element and stiffener. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

For each case, an equation will be given for determining I_a, the moment of inertia required for a stiffener adequate so that the flange element behaves as a stiffened element, and I_S is the moment of inertia of the full section of the stiffener about its centroidal axis, parallel to the element to be stiffened. A'_S is the effective area of a stiffener of any shape, calculated by methods discussed previously. The reduced area of the stiffener to be used in section property calculations is termed A_S, and its relation to A'_S is given for each case. Note that for edge stiffeners, the rounded corner between the stiffener and the flange is not considered part of the stiffener in calculations. The following additional definitions for a simple lip stiffener illustrated in Fig. 9.9 apply. The effective width d'_S is that of the stiffener calculated according to Arts. 9.9.1 and 9.9.2. The reduced effective width to be used in section property calculations is termed d_S, and its relation to d'_S is given for each case. For the inclined stiffener of flat depth d at an angle θ as shown in Fig. 9.9,

$$I_S = \frac{(d^3 t \sin^2 \theta)}{12} \tag{9.19}$$

$$A'_S = d'_S t \tag{9.20}$$

Limit d/t to 14.

Case I: $w/t \le 0.328S$. For this condition, the flange element is fully effective without an edge stiffener, so $b = w$, $I_a = 0$, $d_S = d'_S$, $A_S = A'_S$.

Case II: $w/t > 0.328S$. For this condition, $b_1 = b/2(R_I)$, $b_2 = b - b_1$, $d_S = d'_S\,(R_I)$, $A_S = A'_S\,(R_I)$, and $(R_I) = I_S/I_a \le 1$.

$$I_a = 399t^4\left[\left(\frac{w/t}{S}\right) - 0.328S\right]^3 \le t^4\left\{\left[\frac{115(w/t)}{S}\right] + 5\right\} \tag{9.21}$$

The effective width b is calculated according to Art. 9.8.1 using the following values for k.

Simple lip edge stiffener (140° ≥ θ ≥ 40°):

$$\text{For } \frac{D}{w} \le 0.25, \qquad k = 3.57(R_I)^n + 0.43 \le 4 \tag{9.22a}$$

$$\text{For } 0.25 < \frac{D}{w} \le 0.80, \qquad k = \left[4.82 - \left(\frac{5D}{w}\right)\right](R_I)^n + 0.43 \le 4 \tag{9.22b}$$

Other edge stiffener shapes:

$$k = 3.57(R_I)^n + 0.43 \le 4 \tag{9.22c}$$

where

$$n = \left[0.582 - \left(\frac{w/t}{4S}\right)\right] \ge \frac{1}{3} \tag{9.22d}$$

For all cases, effective width for deflection determination is calculated in the same manner except that stresses are calculated at service-load levels based on the effective section at that load.

9.11 TENSION MEMBERS

The nominal tensile strength, T_n, of an axial-loaded tension member is the smallest of three limit states: (1) yielding in the gross section, Eq. (9.23); (2) fracture in the net section away from the connections, Eq. (9.24); or (3) fracture in the net section at connections (Art. 9.18.2):

$$T_n = A_g F_y \tag{9.23}$$

$$T_n = A_n F_u \tag{9.24}$$

where A_g is the gross cross-section area, A_n is the net cross-section area, F_y is the design yield stress, and F_u is the tensile strength.

As with all of the member design provisions, these nominal strengths must be divided by a safety factor, Ω, for ASD (Art. 9.4.1) or multiplied by a resistance factor, ϕ, for LRFD (Art. 9.4.2). See Table 9.1 for Ω and ϕ values for the appropriate member or connection category.

9.12 FLEXURAL MEMBERS

In the design of flexural members, consideration must be given to bending strength, shear strength, and web crippling, as well as combinations thereof, as discussed in subsequent articles. Bending strength must consider both yielding and lateral stability. In some applications, deflections are also an important consideration.

9.12.1 Nominal Strength Based on Initiation of Yielding

For a fully braced member, the nominal strength M_n is the effective yield moment based on section strength:

$$M_n = S_e F_y \tag{9.25}$$

where S_e is the elastic section modulus of the effective section calculated with the extreme fiber at the design yield stress, F_y. The stress in the extreme fiber can be compression or tension, depending on which is farthest from the neutral axis of the effective section. If the extreme fiber stress is compression, the effective width (Arts. 9.8–9.10) and the effective section can be calculated directly based on the stress F_y in that compression element. However, if the extreme fiber stress is tension, the stress in the compression element depends on the effective section, and therefore, a trial-and-error solution is required (Art. 9.22).

9.12.2 Nominal Strength Based on Lateral-Torsional Buckling

For this condition, the nominal strength M_n of laterally unbraced segments of singly, doubly, and point-symmetric sections is given by Eq. (9.26). These provisions apply to *I*, *Z*, *C*, and other singly symmetric sections, but not to multiple-web decks, *U*, and box sections. Also, beams with one flange fastened to deck, sheathing, or standing-seam roof systems are treated separately. The nominal strength is

$$M_n = S_c F_c \tag{9.26}$$

where S_c = elastic section modulus of the effective section calculated relative to the extreme compression fiber at stress F_c.

For $F_e \geq 2.78\ F_y$,

$$F_c = F_y \tag{9.27}$$

For $2.78\ F_y > F_e > 0.56 F_y$,

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10 F_y}{36 F_e}\right) \tag{9.28}$$

For $F_e \leq 0.56 F_y$,

$$F_c = F_e \tag{9.29}$$

where F_e = elastic critical lateral-torsional buckling stress calculated according to (*a*) or (*b*) below.

(a) For Singly, Doubly, and Point-Symmetric Sections. For bending about the symmetry axis,

$$F_e = (C_b r_o A \sqrt{\sigma_{ey} \sigma_t})/S_f \tag{9.30}$$

For singly symmetric sections, the *x* axis is the axis of symmetry oriented such that the shear center has a negative *x* coordinate. For point-symmetric sections use $0.5F_e$. Alternatively, F_e can be calculated using the equation for doubly symmetric *I* sections, singly symmetric *C* sections, or point-symmetric *Z* sections given in (*b*).

$$F_e = C_s A \sigma_{ex} \frac{[j + C_s \sqrt{j^2 + r_o^2 (\sigma_t/\sigma_{ex})}]}{(C_{TF}/\sigma_{ex})} \tag{9.31}$$

for bending about the centroidal axis perpendicular to the symmetry axis for singly symmetric sections only.

$C_s = +1$ for moment-causing compression on the shear center side of the centroid

$C_s = -1$ for moment-causing tension on the shear center side of the centroid

A = full cross-sectional area

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \tag{9.32}$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \tag{9.33}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \tag{9.34}$$

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \tag{9.35}$$

$M_{\max}$ = absolute value of maximum moment in the unbraced segment

M_A = absolute value of moment at quarter point of unbraced segment

M_B = absolute value of moment at centerline of unbraced segment

M_C = absolute value of moment at three-quarter point of unbraced segment

C_b is permitted to be conservatively taken as unity for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$. For members subject to combined compressive axial load and bending moment (Art. 9.15), $C_b = 1.0$.

E = modulus of elasticity

S_f = elastic section modulus of full, unreduced cross section, relative to extreme compression fiber

$$C_{TF} = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) \tag{9.36}$$

where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length in the plane of bending, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse-curvature bending) and negative when they are of opposite sign (single-curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, $C_{TF} = 1.0$.

r_o = polar radius of gyration of the cross section about the shear center

$$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} \tag{9.37}$$

r_x, r_y = radii of gyration of the cross section about the centroidal principal axes

G = shear modulus (11,000 ksi or 78,000 MPa)

K_x, K_y, K_t = effective length factors for bending about the x and y axes, and for twisting

L_x, L_y, L_t = unbraced length of compression member for bending about the x and y axes, and for twisting

x_o = distance from the shear center to the centroid along the principal x axis, taken as negative

J = St. Venant torsion constant of the cross section

C_w = torsional warping constant of the cross section

$$j = \frac{1}{2I_y}[\textstyle\int_A x^3\, dA + \int_A xy^2\, dA] - x_o \tag{9.38}$$

(b) For* I *or* Z *Sections Bent about the Centroidal Axis Perpendicular to the Web (*x *Axis). In lieu of (*a*), the following equations may be used to evaluate F_e:

$$F_e = \frac{\pi^2 EC_b dI_{yc}}{S_f(K_y L_y)^2} \quad \text{for doubly symmetric } I \text{ sections and singly symmetric } C \text{ sections} \tag{9.39}$$

$$= \frac{\pi^2 EC_b dI_{yc}}{2S_f(K_y L_y)^2} \quad \text{for point-symmetric } Z \text{ sections} \tag{9.40}$$

d = depth of section

L = unbraced length of the member

I_{yc} = moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full, unreduced section

Other terms are defined in (*a*).

9.12.3 Beams (*C* or *Z* Section) Having One Flange Through-Fastened to Deck or Sheathing

If the tension flange of a beam is screwed to deck or sheathing and the compression flange is unbraced, such as a roof purlin or wall girt subjected to wind suction, the bending strength lies between that for a fully braced member and that for an unbraced member. This is due to the rotational restraint provided by the spaced connections. Therefore, based on numerous tests, the *AISI NAS* gives the nominal strength in terms of a reduction factor R applied to the nominal strength for the fully braced condition (Art. 9.12.1):

$$M_n = RS_e F_y \tag{9.41}$$

For continuous spans, $R = 0.60$ for C sections, and 0.70 for Z sections. For simple spans, R is given in Table 9.3. F_y is limited to 60 ksi.

The provisions do not apply for the region adjacent to a support between inflection points in a continuous beam. Numerous physical conditions are imposed, including member and panel characteristics, span length (33 ft maximum), fasteners, and fastener spacing (12 in maximum).

TABLE 9.3 *R* Values for Simple Spans [See Eq. (9.41)]

Section depth d, in	Profile	R*
$d \le 6.5$	C or Z	0.70
$6.5 < d \le 8.5$	C or Z	0.65
$8.5 < d \le 11.5$	Z	0.50
$8.5 < d \le 11.5$	C	0.40

*For simple spans, multiply R by the correction factor r to account for the effects of compressed insulation between the sheeting and the member: For uncompressed batt insulation of thickness t_i, the factor is $r = 1.00 - 0.01t_i$ (in), or $r = 1.00 - 0.004t_i$ (mm).

9.12.4 Beams (*C* or *Z* Section) Having One Flange Fastened to a Standing Seam Roof System

If the flange of a supporting beam is fastened to a standing seam roof system, the bending strength generally lies between that for a fully braced member and that for an unbraced member, but may equal that for a fully braced member. The strength depends on the details of the system, as well as whether the loading is gravity or uplift, and cannot be readily calculated. Therefore, the *AISI NAS* allows the nominal strength to be calculated by Eq. (9.41), but with the reduction factor R determined by representative tests of the system. Test specimens and procedures are detailed in the "Base Test Method" given in the *AISI Manual.* Alternatively, the rules for discrete point bracing (Art. 9.12.2) can be used.

9.12.5 Nominal Shear Strength

The *AISI NAS* gives three equations for nominal shear strength of beam webs for three categories or conditions of increasing web slenderness. Condition (*a*) is based on yielding, condition (*b*) is based on inelastic buckling, and condition (*c*) is based on elastic buckling.

(*a*) For $h/t \le \sqrt{Ek_v/F_y}$,

$$V_n = 0.60F_y ht \tag{9.42}$$

(*b*) For $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$,

$$V_n = 0.64t^2\sqrt{k_vF_yE} \tag{9.43}$$

(*c*) For $h/t > 1.51\sqrt{Ek_v/F_y}$,

$$V_n = \frac{\pi^2 Ek_v t^3}{12(1-\mu^2)h} = 0.904\frac{Ek_v t^3}{h} \tag{9.44}$$

where v_n = nominal shear strength of beam; t = web thickness; h = depth of the flat portion of the web measured along the plane of the web; and k_v = shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$.
2. For beam webs with transverse stiffeners satisfying AISI requirements:

   When $a/h \le 1.0$,

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \tag{9.45}$$

When $a/h > 1.0$,

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \tag{9.46}$$

where a = shear panel length for unreinforced web element
= clear distance between transverse stiffeners for reinforced web elements

For a web consisting of two or more sheets, each sheet is considered as a separate element carrying its share of the shear force.

9.12.6 Combined Bending and Shear

Combinations of bending and shear may be critical at locations such as near interior supports of continuous beams. To guard against this condition, the *AISI NAS* provides traditional interaction equations, which depend on whether the beam web is unreinforced or transversely stiffened. Although similar in concept, for clarity, separate equations are given for ASD and LRFD. Symbols have common definitions except as noted.

ASD Method. For beams with unreinforced webs, the required flexural strength, M, and required shear strength, V, must satisfy the following:

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0 \tag{9.47}$$

For beams with transverse web stiffeners, the required flexural strength, M, and required shear strength, V, shall not exceed M_n/Ω_b and V_n/Ω_v, respectively. When $\Omega_b M/M_{nxo} > 0.5$ and $\Omega_v V/V_n > 0.7$, then M and V must satisfy the following interaction equation:

$$0.6\left(\frac{\Omega_b M}{M_{nxo}}\right) + \left(\frac{\Omega_v V}{V_n}\right) \leq 1.3 \tag{9.48}$$

where Ω_b = factor of safety for bending (Table 9.1)
Ω_v = factor of safety for shear (Table 9.1)
M_n = nominal flexural strength when bending alone exists
M_{nxo} = nominal flexural strength about the centroidal x axis determined in accordance with AISI, excluding lateral buckling
V_n = nominal shear force when shear alone exists

LRFD Method. For beams with unreinforced webs, the required flexural strength, M_u, and required shear strength, V_u, must satisfy the following:

$$\left(\frac{M_u}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \leq 1.0 \tag{9.49}$$

For beams with transverse web stiffeners the required flexural strength, M_u, and the required shear strength, V_u, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively. When $M_u/(\phi_b M_{nxo}) > 0.5$ and $V_u/(\phi_v V_n) > 0.7$, then M_u and V_u must satisfy the following interaction equation:

$$0.6\left(\frac{M_u}{\phi_b M_{nxo}}\right) + \left(\frac{V_u}{\phi_v V_n}\right) \leq 1.3 \tag{9.50}$$

where ϕ_b = resistance factor for bending (Table 9.1)
ϕ_v = resistance factor for shear (Table 9.1)
M_n = nominal flexural strength when bending alone exists

9.12.7 Web Crippling

At points of concentrated loads or reactions, the webs of cold-formed members are susceptible to web crippling. If the web depth-to-thickness ratio h/t is greater than 200, stiffeners must be used to transmit the loads directly into the webs. For unstiffened webs, the *AISI NAS* gives an equation with multiple coefficients to calculate the nominal bearing strength to resist the concentrated load. The coefficients (Tables 9.4–9.8) are based on the results of numerous tests and provide for several different conditions for load placement and type of section.

The nominal web crippling strength P_n is determined as follows:

$$P_n = Ct^2 F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{9.51}$$

TABLE 9.4 Web Crippling Coefficients for Built-up Sections

Support and flange conditions		Load cases		C	C_R	C_N	C_h	USA and Mexico		Canada	Limits
								ASD Ω_W	LRFD ϕ_W	LSD ϕ_W	
Fastened to support	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20	0.15	0.05	0.003	1.65	0.90	0.80	$R/t \leq 5$
Unfastened	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \leq 3$
		Two-flange loading or reaction	End	15.5	0.09	0.08	0.04	2.00	0.75	0.65	$R/t \leq 3$
			Interior	36	0.14	0.08	0.04	2.00	0.75	0.65	
	Unstiffened flanges	One-flange loading or reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \leq 3$

Notes:
(1) This table applies to I-beams made from two channels connected back to back.
(2) The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$, and $\theta = 90°$.
Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.5 Web Crippling Coefficients for Single-Web Channel and *C* Sections

Support and flange conditions		Load cases		C	C_R	C_N	C_h	USA and Mexico		Canada	Limits
								ASD Ω_W	LRFD ϕ_W	LSD ϕ_W	
Fastened to support	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \leq 5$
		Two-flange loading or reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	0.75	$R/t \leq 12$
			Interior	20	0.10	0.08	0.031	1.75	0.85	0.75	$R/t \leq 12$
Unfastened	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	4	0.14	0.35	0.02	1.85	0.80	0.70	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-flange loading or reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened flanges	One-flange loading or reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R/t \leq 1$
		Two-flange loading or reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Notes:
(1) The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$, and $\theta = 90°$.
(2) For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member should be extended at least $2.5h$. For unfastened cases, the distance from the edge of bearing to the end of the member should be extended at least $1.5h$.
Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.6 Web Crippling Coefficients for Single-Web *Z* Sections

Support and flange conditions		Load cases		C	C_R	C_N	C_h	USA and Mexico		Canada	Limits
								ASD Ω_W	LRFD ϕ_W	LSD ϕ_W	
Fastened to support	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \leq 5$
		Two-flange loading or reaction	End	9	0.05	0.16	0.052	1.75	0.85	0.75	$R/t \leq 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	0.70	$R/t \leq 12$
Unfastened	Stiffened or partially stiffened flanges	One-flange loading or reaction	End	5	0.09	0.02	0.001	1.80	0.85	0.75	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-flange loading or reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened flanges	One-flange loading or reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R/t \leq 1$
		Two-flange loading or reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Notes:

(1) The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$, and $\theta = 90°$.

(2) For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member should be extended at least $2.5h$. For unfastened cases, the distance from the edge of bearing to the end of the member should be extended at least $1.5h$.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.7 Web Crippling Coefficients for Single Hat Sections

Support conditions	Load cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_W	USA and Mexico LRFD ϕ_W	Canada LSD ϕ_W	Limits
Fastened to support	One-flange loading or reaction	End	4	0.25	0.68	0.04	2.00	0.75	0.65	$R/t \le 5$
		Interior	17	0.13	0.13	0.04	1.90	0.80	0.70	$R/t \le 10$
	Two-flange loading or reaction	End	9	0.10	0.07	0.03	1.75	0.85	0.75	$R/t \le 10$
		Interior	10	0.14	0.22	0.02	1.80	0.85	0.75	
Unfastened	One-flange loading or reaction	End	4	0.25	0.68	0.04	2.00	0.75	0.65	$R/t \le 4$
		Interior	17	0.13	0.13	0.04	1.70	0.90	0.75	$R/t \le 4$

Note: The above coefficients apply when $h/t \le 200$, $N/t \le 200$, $N/h \le 2$, and $\theta = 90°$.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

where P_n = nominal web crippling strength
C = coefficient
C_h = web slenderness coefficient
C_N = bearing length coefficient
C_R = inside bend radius coefficient
F_y = design yield point
h = flat dimension of web measured in plane of web
N = bearing length [$^3/_4$ in (19 mm) minimum]
R = inside bend radius
t = web thickness
θ = angle between plane of web and plane of bearing surface, $45° \le \theta \le 90°$

P_n represents the nominal strength for load or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, P_n should be calculated for each individual sheet and the results added to obtain the nominal strength for the full section.

TABLE 9.8 Web Crippling Coefficients for Multiweb Deck Sections

Support conditions	Load cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_W	USA and Mexico LRFD ϕ_W	Canada LSD ϕ_W	Limits
Fastened to support	One-flange loading or reaction	End	4	0.04	0.25	0.025	1.70	0.90	0.80	$R/t \le 7$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	$R/t \le 10$
	Two-flange loading or reaction	End	9	0.12	0.14	0.040	1.80	0.85	0.70	$R/t \le 10$
		Interior	10	0.11	0.21	0.020	1.75	0.85	0.75	
Unfastened	One-flange loading or reaction	End	3	0.04	0.29	0.028	2.95	0.60	0.50	$R/t \le 7$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-flange loading or reaction	End	6	0.16	0.15	0.050	1.65	0.90	0.80	$R/t \le 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	0.80	

Notes:
(1) The above coefficients apply when $h/t \le 200$, $N/t \le 210$, $N/h \le 3$.
(2) $45° \le \theta \le 90°$.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, and 2004 Supplement, with permission.

One-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is greater than 1.5*h*.

Two-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or less than 1.5*h*.

End loading or reaction occurs when the distance from the edge of the bearing to the end of the member is equal to or less than 1.5*h*.

Interior loading or reaction occurs when the distance from the edge of the bearing to the end of the member is greater than 1.5*h*, except as otherwise noted.

9.12.8 Combined Bending and Web Crippling Strength

For beams with unreinforced flat webs, combinations of bending and web crippling near concentrated loads or reactions must satisfy interaction equations given in the *AISI NAS*. Equations are given for two types of webs and for nested *Z* sections, with separate equations for ASD and LRFD. See the *AISI NAS* for various exceptions and limitations that may apply. Symbols have common definitions except as noted.

ASD Method

(***a***) For shapes having single unreinforced webs,

$$\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\leq\frac{1.33}{\Omega} \qquad (9.52)$$

(***b***) For shapes having multiple unreinforced webs such as *I* sections made of two *C* sections connected back to back, or similar sections which provide a high degree of restraint against rotation of the web (such as *I* sections made by welding two angles to a *C* section),

$$0.88\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\leq\frac{1.65}{\Omega} \qquad (9.53)$$

(***c***) For the support point of two nested *Z* sections,

$$\frac{M}{M_{no}}+0.86\left(\frac{P}{P_n}\right)\leq\frac{1.65}{\Omega} \qquad (9.54)$$

In Eqs. (9.52), (9.53), and (9.54),

P = required allowable strength for the concentrated load or reaction in the presence of bending moment
P_n = nominal strength for concentrated load or reaction in the absence of bending moment determined in accordance with Art. 9.12.7
M = required allowable flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P
M_{nxo} = nominal flexural strength about the centroidal *x*-axis determined in accordance with Art. 9.12.1
w = flat width of the beam flange which contacts the bearing plate
t = thickness of the web or flange
Ω = safety factor for combined bending and web crippling = 1.70

LRFD Method

(***a***) For shapes having single unreinforced webs,

$$0.91\left(\frac{P_u}{P_n}\right)+\left(\frac{M_u}{M_{nxo}}\right)\le 1.33\phi \tag{9.55}$$

(***b***) For shapes having multiple unreinforced webs such as *I* sections made of two *C* sections connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as *I* sections made by welding two angles to a *C* section),

$$0.88\left(\frac{P_u}{P_n}\right)+\left(\frac{M_u}{M_{nxo}}\right)\le 1.46\phi \tag{9.56}$$

(***c***) For two nested *Z* sections,

$$\frac{M_u}{M_{no}}+0.85\frac{P_u}{P_n}\le 1.65\phi \tag{9.57}$$

In Eqs. (9.55), (9.56), and (9.57),

ϕ = resistance factor = 0.90
P_u = required strength for the concentrated load or reaction in the presence of bending moment
M_u = required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P_u

9.13 CONCENTRICALLY LOADED COMPRESSION MEMBERS

These provisions are for members in which the resultant of all loads is an axial load passing through the effective section calculated at the stress F_n as subsequently defined. Concentrically loaded angle sections should be designed for an additional moment in certain cases according to the *AISI NAS*.

The nominal axial strength, P_n, is

$$P_n = A_e F_n \tag{9.58}$$

where A_e is the effective area at the stress F_n, which is determined as follows:

For $\lambda_c = \le 1.5$, $$F_n = \left(0.658\lambda_c^2\right)F_y \tag{9.59a}$$

For $\lambda_c = > 1.5$, $$F_n = \left(\frac{0.877}{\lambda_c^2}\right)F_y \tag{9.59b}$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \tag{9.60}$$

F_e is the least of the elastic flexural, torsional, and torsional-flexural buckling stresses (Arts. 9.13.1–9.13.3). Equation (9.59a) is based on elastic buckling while Eq. (9.59b) represents inelastic buckling, providing a transition to the yield point stress as the column length decreases.

9.13.1 Elastic Flexural Buckling

For doubly symmetric, closed, or any other sections that are not subject to torsional or torsional-flexural buckling, the elastic buckling stress is

$$F_e = \frac{\pi^2}{(KL/r)^2} \tag{9.61}$$

where E = modulus of elasticity (29,500 ksi or 203,000 MPa)
K = effective length factor (see Fig. 5.1)
L = unbraced length of member
r = radius of gyration of full, unreduced cross section about axis of buckling

9.13.2 Symmetric Sections Subject to Torsional or Torsional-Flexural Buckling

Singly Symmetric Sections. For singly symmetric sections, such as C sections subject to torsional-flexural buckling, F_e is the smaller of Eq. (9.61) and that given by Eq. (9.62) or (9.63):

$$F_e \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 2\beta\sigma_{ex}\sigma_t}\,\right] \tag{9.62}$$

As an alternative to Eq. (9.62), a conservative estimate can be calculated from the following:

$$F_e = \frac{\sigma_{ex}\sigma_t}{\sigma_{ex} + \sigma_t} \tag{9.63}$$

In the above, σ_{ex} and σ_t are given by Eqs. (9.32) and (9.34), and

$$\beta = 1 - \left(\frac{x_o}{r_o}\right)^2 \tag{9.64}$$

where r_o is given by Eq. (9.37) and x_o = distance from shear center to centroid along principal x axis taken as negative. For singly symmetric sections, the x axis is assumed to be the axis of symmetry.

Doubly Symmetric Sections. For doubly symmetric sections, such as back-to-back C sections subject to torsional buckling, F_e is taken as the smaller of Eq. (9.61) and the torsional buckling stress, σ_t, given by Eq. (9.33).

9.13.3 Point-Symmetric and Nonsymmetric Sections

For point-symmetric sections such as Z sections, F_e should be taken as the lesser of σ_t given by Eq. (9.34) and F_e as given by Eq. (9.61) using the minor principal axis of the section.

For shapes with cross sections that do not have any symmetry, either about an axis or about a point, F_e should be determined by rational analysis or by tests.

9.13.4 Beams (*C* or *Z* Section) Having One Flange Through-Fastened to Deck or Sheathing

For such sections loaded axially in compression, refer to the *AISI NAS*, which give a special set of provisions. Based on the results of structural tests, they account for the partial restraint to weak-axis buckling provided by the deck or sheathing. Strong-axis buckling should be considered using the same equations as for members not attached to sheathing.

9.14 COMBINED TENSILE AXIAL LOAD AND BENDING

Members under combined axial tensile load and bending must satisfy the interaction equations given by the *AISI NAS* to prevent yielding. Separate equations are given for ASD and LRFD, but symbols have common definitions except as noted.

ASD Method. To check the tension flange,

$$\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} + \frac{\Omega_t T}{T_n} \leq 1.0 \tag{9.65}$$

To check the compression flange,

$$\frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} + \frac{\Omega_t T}{T_n} \leq 1.0 \tag{9.66}$$

where T = required allowable tensile axial strength
M_x, M_y = required allowable flexural strengths with respect to the centroidal axes of the section
T_n = nominal tensile axial strength determined in accordance with Art. 9.11
M_{nx}, M_{ny} = nominal flexural strengths about the centroidal axes determined in accordance with Art. 9.12
M_{nxt}, M_{nyt} = $S_{\text{ft}}F_y$
S_{ft} = section modulus of the full section for the extreme tension fiber about the appropriate axis
Ω_b = safety factor for bending, 1.67
Ω_t = safety factor for tension member, 1.67

LRFD Method. To check the tension flange,

$$\frac{M_{ux}}{\phi_b M_{nxt}} + \frac{M_{uy}}{\phi_b M_{nyt}} + \frac{T_u}{\phi_t T_n} \leq 1.0 \tag{9.67}$$

To check the compression flange,

$$\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} - \frac{T_u}{\phi_t T_n} \leq 1.0 \tag{9.68}$$

where T_u = required tensile axial strength
M_{ux}, M_{uy} = required flexural strengths with respect to the centroidal axes
ϕ_b = 0.90 or 0.95 for bending strength, or 0.90 for laterally unbraced beams
ϕ_t = 0.95

9.15 COMBINED COMPRESSIVE AXIAL LOAD AND BENDING

Members under combined compression axial load and bending are generally referred to as beam-columns. Bending in such members may be caused by eccentric loading, lateral loads, or end moments, and the compression load can amplify the bending. These members must satisfy the interaction equations given by the *AISI NAS* to prevent both buckling and yielding. Separate equations are given for ASD and LRFD, but symbols have common definitions except as noted.

ASD Method

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx} \alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny} \alpha_y} \le 1.0 \tag{9.69}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \tag{9.70}$$

When $\Omega_c P/P_n \le 0.15$, the following equation may be used in lieu of the above two equations:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \tag{9.71}$$

where P = required allowable compressive axial strength

M_x, M_y = required allowable flexural strengths with respect to the centroidal axes of the effective section determined for the required compressive axial strength alone. For singly symmetric unstiffened angle sections with unreduced effective area, M_y should be taken as the required flexural strength. For other angle sections, M_y should be taken either as the required flexural strength or the required flexural strength plus $PL/1000$, whichever results in a lower permissible value of P.

P_n = nominal axial strength determined in accordance with Arts. 9.13 and 9.16

P_{no} = nominal axis strength determined in accordance with Arts. 9.13 and 9.16, with $F_n = F_y$

M_{nx}, M_{ny} = nominal flexural strengths about the centroidal axes determined in accordance with Art. 9.12

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \tag{9.72}$$

$$\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} \tag{9.73}$$

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \tag{9.74}$$

$$P_{Ey} = \frac{\pi^2 E I_y}{(K_y L_y)^2} \tag{9.75}$$

Ω_b = safety factor for bending, 1.67

Ω_c = safety factor for compression, 1.80

I_x = moment of inertia of the full, unreduced cross section about the x axis

I_y = moment of inertia of the full, unreduced cross section about the y axis

L_x = unbraced length for bending about the x axis

L_y = unbraced length for bending about the y axis
K_x = effective length factor for buckling about the x axis
K_y = effective length factor for buckling about the y axis
C_{mx}, C_{my} = coefficients whose value is as follows:

1. For compression members in frames subject to joint translation (sidesway),

$$C_m = 0.85$$

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) \tag{9.76}$$

where M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:
   (***a***) For members whose ends are restrained, $C_m = 0.85$
   (***b***) For members whose ends are unrestrained, $C_m = 1.0$

LRFD Method

$$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx} \alpha_x} + \frac{C_{my} M_{uy}}{\phi_b M_{ny} \alpha_y} \le 1.0 \tag{9.77}$$

$$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \le 1.0 \tag{9.78}$$

When $P_u/\phi_c P_n \le 0.15$, the following equation may be used in lieu of the above two equations:

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \le 1.0 \tag{9.79}$$

where P_u = required compressive axial strength
M_{ux}, M_{uy} = required flexural strengths with respect to the centroidal axes of the effective section determined for the required compressive axial strength alone. For singly symmetric unstiffened angle sections with unreduced effective area, M_{uy} should be taken as the required flexural strength. For other angle sections, M_{uy} should be taken either as the required flexural strength or the required flexural strength plus $P_u L/1000$, whichever results in a lower permissible value of P_u.

$$\alpha_x = 1 - \frac{P_u}{P_{Ex}} \tag{9.80}$$

$$\alpha_y = 1 - \frac{P_u}{P_{Ey}} \tag{9.81}$$

ϕ_b = 0.90 or 0.95 for bending strength, or 0.90 for laterally unbraced beams
ϕ_c = 0.85

9.16 CYLINDRICAL TUBULAR MEMBERS

The *AISI NAS* gives separate provisions for cylindrical members, but they apply only if the ratio of outside diameter to wall thickness, D/t, is not greater than $0.441E/F_y$.

9.16.1 Flexure

Cylinders with small D/t ratios can develop their plastic moment strength while those with greater D/t ratios will develop smaller capacities. The nominal flexural strength, M_n, is calculated from the following equations, the selection of which depends on the D/t ratio:

For $D/t \leq 0.0714E/F_y$,

$$M_n = 1.25F_yS_f \tag{9.82}$$

For $0.0714E/F_y < D/t \leq 0.318E/F_y$,

$$M_n = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_yS_f \tag{9.83}$$

For $0.318E/F_y < D/t \leq 0.441E/F_y$,

$$M_n = [0.328E/(D/t)]S_f \tag{9.84}$$

In the above, S_f = elastic section modulus of the full, unreduced cross section.

9.16.2 Axial Compression

The nominal axial strength of round tubes is calculated from the same column equations as for other closed members, Eqs. (9.58) to (9.61) of Art. 9.13, with the following exception. The effective area, A_e, is

$$A_e = A_o + R(A - A_o) \tag{9.85}$$

where

$$R = \frac{F_y}{2F_e} \leq 1.0 \tag{9.86}$$

$$A_o = \left(\frac{0.037}{DF_y/tE} + 0.667\right)A \leq A \tag{9.87}$$

A = area of the unreduced cross section

Torsional or flexural-torsional buckling need not be checked for cylindrical members. However, combined axial load and bending must be checked as for other members (Art. 9.15).

9.17 WELDED CONNECTIONS

Various types of welds may be used to join cold-formed steel members such as groove welds in butt joints, fillet welds, flare groove welds, arc spot welds, arc seam welds, and resistance welds. The nominal strength, P_n, for several of these weld types is given in this article. More complete information may be found in the *AISI NAS*. The provisions given are applicable where the thickness of the thinnest part connected is $^3/_{16}$ in (4.76 mm) or less. For thicker parts, refer to the *AISC*

Specifications. Welders and welding procedures should be qualified in accordance with specifications of the American Welding Society (AWS).

9.17.1 Groove Welds in Butt Joints

For **tension or compression,** the nominal strength is

$$P_n = Lt_eF_y \tag{9.88}$$

For **shear,** the nominal strength is the smaller of the limit based on shear in the weld and that based on shear in the base metal.

For shear in welds,

$$P_n = Lt_e0.6F_{xx} \tag{9.89}$$

For shear in base metal,

$$P_n = \frac{Lt_eF_y}{\sqrt{3}} \tag{9.90}$$

where F_{xx} = filler metal strength designation for AWS electrode classification
F_y = yield stress
L = length of weld
t_e = effective throat dimension

9.17.2 Fillet Welds

Fillet welds are considered to transmit longitudinal and transverse loads with shear stresses. For these welds, the nominal strength is the smaller of the limit based on weld strength and that based on the strength of the connected part.

For weld strength (consider only if $t > 0.10$ in or 2.54 mm),

$$P_n = 0.75t_wLF_{xx} \tag{9.91}$$

For strength of connected part, welds longitudinal to the loading,

When $L/t < 25$
$$P_n = \left(1 - \frac{0.01L}{t}\right)tLF_u \tag{9.92}$$

When $L/t \geq 25$
$$P_n = 0.75tLF_u \tag{9.93}$$

For strength of connected part, welds transverse to the loading,

$$P_n = tLF_u \tag{9.94}$$

where t = least of t_1 and t_2 (see Fig. 9.10)
t_w = effective throat of weld
= $0.70w_1$ or $0.707w_2$, whichever is smaller (see Fig. 9.10)
F_u = tensile strength

9.17.3 Arc Spot Welds

Arc spot welds, also know as puddle welds, are made in the flat welding position to join sheets to thicker members. They are made by using the arc to burn a hole in the top sheet (or sheets), then depositing weld metal to fill the hole and fuse it to the underlying member. Thus, no hole need be punched in the sheet. Such welds should not be made where the top sheet (or sheets) is over 0.15 in (3.81 mm) thick. Where the thickness of the sheet is less than 0.028 in (0.711 mm), a washer should be used on top of the sheet and the weld made inside this washer. The washer should have a thickness

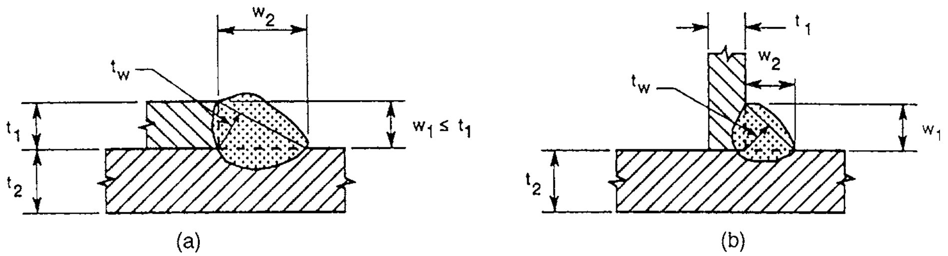


FIGURE 9.10 Cross section of fillet welds. (*a*) At lap joint. (*b*) At tee joint. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

of 0.05 to 0.08 in (1.27 to 2.03 mm), with a prepunched hole of 0.375 in (9.53 mm) diameter. Arc spot welds are specified by minimum effective diameter of fused area, d_e, and the minimum is 0.375 in (9.53 mm). The *AISI NAS* gives provisions for both shear and tension (uplift) loadings.

For **shear,** when sheets are welded to a thicker member, the nominal strength is the smaller of the limit based on the strength of the weld and that based on the strength of the connected part.

For weld strength (use $\phi = 0.60$ or $\Omega = 2.55$),

$$P_n = \frac{\pi d_e^2}{4} 0.75 F_{xx} \tag{9.95}$$

For strength of connected part,

When $(d_a/t) \le 0.815\sqrt{E/F_u}$ (use $\phi = 0.70$ or $\Omega = 2.20$),

$$P_n = 2.20 t d_a F_u \tag{9.96}$$

When $0.815\sqrt{E/F_u} < (d_a/t) < 1.397\sqrt{E/F_u}$ (use $\phi = 0.55$ or $\Omega = 2.80$),

$$P_n = 0.280\left(1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right) t d_a F_u \tag{9.97}$$

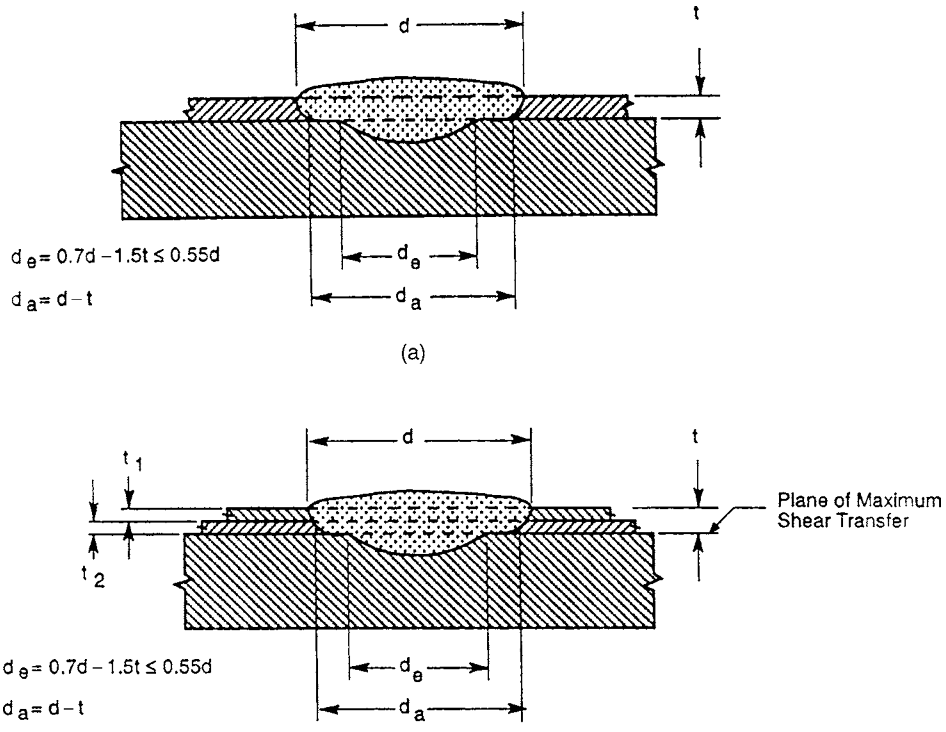
When $(d_a/t) \ge 1.397\sqrt{E/F_u}$ (use $\phi = 0.50$ or $\Omega = 3.05$),

$$P_n = 1.40 t d_a F_u \tag{9.98}$$

where d = visible diameter of outer surface of arc spot weld
d_a = average diameter of the arc spot weld at mid-thickness of t; use $d_a = (d - t)$ for a single sheet, $d_a = (d - 2t)$ for multiple sheets (four sheet maximum)
d_e = effective diameter of fused area at plane of maximum shear transfer = $0.7d - 1.5t$ but $\le 0.55d$
t = total base steel thickness of sheets involved in shear transfer

Spot welds may also be used for sheet-to-sheet connections. In such cases, the nominal strength in shear is

$$P_n = 1.65 t d_a F_u \tag{9.99}$$



(a)

(b)

FIGURE 9.11 Cross section of arc spot weld connecting sheets to underlying member, (*a*) With one sheet connected. (*b*) With two sheets connected. (*Source:* North American Specification for the Design of Cold-Formed Steel Structural Members, *American Iron and Steel Institute, Washington, D.C., 2001, with permission.*)

See Fig. 9.11 for illustration of diameters d, d_a, and d_e.

Also for arc spot welds in shear, the edge distance must be sufficient. The AISI requires that the clear distance from the edge of a weld to the end of a member be not less than 1.0d. Furthermore, the distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld, or to the end of the connected part toward which the force is directed, be not less than 1.5d and also not less than the following:

For ASD,
$$e_{\min} = \frac{P\Omega}{F_u t} \tag{9.100}$$

For LRFD,
$$e_{\min} = \frac{P_u}{\phi F_u t} \tag{9.101}$$

For $F_u/F_{sy} \geq 1.08$, use $\Omega = 2.20$ (ASD) and $\phi = 0.70$ (LRFD). For $F_u/F_{sy} < 1.08$, use $\Omega = 2.55$ (ASD) and $\phi = 0.60$ (LRFD). In the above, P = required strength per weld (nominal force), P_u = required strength per weld (factored force), t = thickness of thinnest connected sheet, and F_{sy} = specified yield stress.

For **tension,** such as caused by uplift, the nominal strength is the smaller of the limit based on the strength of the weld and that based on the strength of the connected part.

For weld strength,

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \tag{9.102}$$

For strength of connected part,

$$P_n = 0.8\left(\frac{F_u}{F_y}\right)^2 t d_a F_u \tag{9.103}$$

Also,

$$t d_a F_u \leq 3 \text{ kips (13.34 kN)} \tag{9.104}$$

In the above, $F_{xx} \geq 60$ ksi (414 MPa), $F_u \leq 82$ ksi (565 MPa) for the connecting sheets, $F_{xx} > F_u$, and $e_{\min} \geq d$. If loading is eccentric, strength is 50% of that calculated. At deck side laps, strength is 70% of that calculated. If connecting multiple sheets, t is the sum of the thicknesses.

9.17.4 Resistance Welds

Resistance welds, often referred to as spot welds, are made by placing two lapped sheets between opposing electrodes that press the sheets together. The weld is created by the heat generated by resistance to current flow. The nominal shear strength is determined as follows, based on the thickness of the thinnest sheet joined, t.

In traditional units:

For $0.01 \leq t < 0.14$ in,

$$P_n \text{ (kips)} = 144t^{1.47} \tag{9.105}$$

For $0.14 \leq t < 0.18$ in,

$$P_n \text{ (kips)} = 43.4t + 1.93 \tag{9.106}$$

In SI units:

For $0.25 \leq t < 3.56$ mm,

$$P_n \text{ (kips)} = 5.51t^{1.47} \tag{9.107}$$

For $3.56 \leq t < 4.57$ mm,

$$P_n \text{ (kips)} = 7.6t + 8.57 \tag{9.108}$$

9.18 BOLTED CONNECTIONS

Bolted connections of cold-formed steel members are designed as bearing-type connections. Bolt pretensioning is not required and installation should be to the snug-tight condition. The *AISI NAS* gives applicable provisions when the thickness, t, of the thinnest connected part is less than $^3/_{16}$ in (4.76 mm). For thicker members, the *AISC Specification* applies. The most commonly used grades are A307 carbon steel bolts and A325 high-strength bolts, but other types can also be used. Standard hole diameter is $d + {}^1/_{32}$ in for $d < {}^1/_2$ in (d + 0.8 mm for $d < 12.7$ mm), where d is bolt diameter. Standard hole diameter is $d + {}^1/_{16}$ in for $d \geq {}^1/_2$ in (d + 1.6 mm for $d \geq 12.7$ mm). See the *AISI NAS* for information on slotted holes.

Several conditions must be checked for a bolted connection, including shearing strength of sheet (edge distance and spacing effects), tension strength in each connected part, bearing strength, bolt shear strength, and bolt tension strength. Each of these is treated in the following articles.

9.18.1 Sheet Shearing (Spacing and Edge Distance)

If bolts are too close to the ends of members, or if the bolts are spaced too closely, the connection may be limited in strength by the shear strength along a line parallel to the member force. Minimum center-to-center spacing of bolts is $3d$ and minimum center-to-edge distance is $1.5d$. Additionally, the nominal strength, P_n, is limited to

$$P_n = teF_u \tag{9.109}$$

where t = thickness of thinnest part and e = distance in line of force from center of hole to nearest edge of adjacent hole or end of connected part.

9.18.2 Fracture in Net Section

The nominal tension strength of the member should be determined as discussed in Art. 9.11. The nominal strength in the connection itself for the limit state of fracture, including shear lag effects where appropriate, should be determined as described in this article.

For flat sheet connections, the nominal tension strength, P_n, on the net area of the section, A_n, of each connected part is

$$P_n = A_n F_t \tag{9.110}$$

Where washers are provided under both the bolt head and the nut, two conditions may apply. For multiple bolts in the line parallel to the force, $F_t = F_u$. For a single bolt, or a single row of bolts perpendicular to the line of force,

$$F_t = \left(0.1 + \frac{3d}{s}\right) F_u \le F_u \tag{9.111}$$

Where only one washer or no washers are provided, consider two conditions. For multiple bolts in the line parallel to the force, $F_t = F_u$. For a single bolt, or a single row of bolts perpendicular to the line of force,

$$F_t = \left(\frac{2.5d}{s}\right) F_u \le F_u \tag{9.112}$$

In the above, F_u = tensile strength of sheet, s = sheet width divided by number of bolt holes in cross section being analyzed, and d = nominal bolt diameter. Where holes are staggered, the net area, A_n, is determined from

$$A_n = 0.90\left[A_g - n_b d_h t + \left(\frac{\sum s'^2}{4g}\right) t\right] \tag{9.113}$$

where A_g = gross cross-section area, n_b = number of bolts in cross section, d_h = hole diameter, t = thickness, s' = longitudinal spacing, and g = transverse spacing.

For connected components other than flat sheets, the nominal strength is

$$P_n = A_n U F_u \tag{9.114}$$

where U is a factor that reflects the nonuniform distribution of stresses over the cross section (shear lag) and is defined as follows. For angle members having two or more bolts in the line of force,

$$U = 1.0 - \frac{1.20\bar{x}}{L} < 0.9 \text{ but } \ge 0.4 \tag{9.115}$$

For channel members having two or more bolts in the line of force,

$$U = 1.0 - \frac{0.36\bar{x}}{L} < 0.9 \text{ but } \ge 0.5 \tag{9.116}$$

In the above, $\bar{x}$ = distance from shear plane to centroid of cross section and L = length of connected part.

TABLE 9.9 Bearing Factor C for Bolted Connections

Thickness of connected part, t, in (mm)	Ratio of fastener diameter to member thickness, d/t	C
$0.024 \le t < 0.1875$	$d/t < 10$	3.0
$(0.61 \le t < 4.76)$	$10 \le d/t \le 22$	$4 - 0.1(d/t)$
	$d/t > 22$	1.8

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

9.18.3 Bearing

The nominal bearing strength depends on whether deformation around the bolt holes can be tolerated. When such deformation is not a design consideration, the nominal bearing strength, P_n, of the sheet for each bolt is

$$P_n = m_f C d t F_u \tag{9.117}$$

where C is a bearing factor (Table 9.9), d = nominal bolt diameter, t = thickness, F_u = tensile strength of sheet, m_f = modification factor for type of connection (Table 9.10).

When deformation around the bolt holes is a design consideration, the nominal bearing strength of the sheet for each bolt is as follows.

For U.S. units (in, ksi),

$$P_n = (4.64t + 1.53)dtF_u \text{ (kips)} \tag{9.118}$$

For SI units (mm, MPa),

$$P_n = (0.183t + 1.53)dtF_u \text{ (N)} \tag{9.119}$$

9.18.4 Shear and Tension in Bolts

The nominal bolt strength resulting from shear, tension, or a combination thereof is calculated as follows:

$$P_n = A_b F \tag{9.120}$$

where A_b = gross cross-sectional area of bolt. For bolts in shear, $F = F_{nv}$ from Table 9.11. For bolts in tension, $F = F_{\text{nt}}$ from Table 9.11. For bolts subject to a combination of shear and tension, $F = F'_{\text{nt}}$ from Tables 9.12–9.15, depending on the design method (ASD or LRFD) and the system of units.

TABLE 9.10 Modification Factor m_f for Bolted Connections

Type of bearing connection	m_f
Single-shear and outside sheets of double-shear connection with washers under both bolt head and nut	1.00
Single-shear and outside sheets of double-shear connection without washers under both bolt head and nut, or with only one washer	0.75
Inside sheet of double-shear connection with or without washers	1.33

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.11 Nominal Tensile and Shear Strength for Bolts

Description of bolts	Tensile strength: Factor of safety Ω (ASD)	Tensile strength: Resistance factor ϕ (LRFD)	Tensile strength: Nominal stress F_{nt}, ksi (MPa)	Shear strength: Factor of safety Ω (ASD)	Shear strength: Resistance factor ϕ (LRFD)	Shear strength: Nominal stress F_{nv}, ksi (MPa)
A307 bolts, Grade A, ¼ in (6.4 mm) ≤ d < ½ in (12.7 mm)	2.25	0.75	40.5 (279)	2.4	0.65	24.0 (165)
A307 bolts, Grade A, d ≥ ½ in	2.25	0.75	45.0 (310)	2.4	0.65	27.0 (186)
A325 bolts, when threads are not excluded from shear planes	2.0	0.75	90.0 (621)	2.4	0.65	54.0 (372)
A325 bolts, when threads are excluded from shear planes	2.0	0.75	90.0 (621)	2.4	0.65	72.0 (496)

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.12 Nominal Tension Stress, F'_{nt} (ksi), for Bolts Subject to Combination of Shear and Tension—ASD Method*

Description of bolts	Threads not excluded from shear planes	Threads excluded from shear planes	Factor of safety Ω
A325 bolts	$110 - 3.6f_v \leq 90$	$110 - 2.8f_v \leq 90$	2.0
A354 Grade BD bolts	$122 - 3.6f_v \leq 101$	$122 - 2.8f_v \leq 101$	2.0
A449 bolts	$100 - 3.6f_v \leq 81$	$100 - 2.8f_v \leq 81$	2.0
A490 bolts	$136 - 3.6f_v \leq 112.5$	$136 - 2.8f_v \leq 112.5$	2.0
A307 bolts, Grade A			2.25
When ¼ in ≤ d < ½ in	$52 - 4f_v \leq 40.5$		
When d ≥ ½ in	$58.5 - 4f_v \leq 45$		

*The shear stress f_v must also satisfy Table 9.11.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.13 Nominal Tension Stress, F'_{nt} (ksi), for Bolts Subject to Combination of Shear and Tension—LRFD Method*

Description of bolts	Threads not excluded from shear planes	Threads excluded from shear planes	Resistance factor ϕ
A325 bolts	$113 - 2.4f_v \leq 90$	$113 - 1.9f_v \leq 90$	0.75
A354 Grade BD bolts	$127 - 2.4f_v \leq 101$	$127 - 1.9f_v \leq 101$	0.75
A449 bolts	$101 - 2.4f_v \leq 81$	$101 - 1.9f_v \leq 81$	0.75
A490 bolts	$141 - 2.4f_v \leq 112.5$	$141 - 1.9f_v \leq 112.5$	0.75
A307 bolts, Grade A			0.75
When ¼ in ≤ d < ½ in	$47 - 2.4f_v \leq 40.5$		
When d ≥ ½ in	$52 - 2.4f_v \leq 45$		

*The shear stress f_v must also satisfy Table 9.11.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

TABLE 9.14 Nominal Tension Stress, F'_{nt} (MPa), for Bolts Subject to Combination of Shear and Tension—ASD Method*

Description of bolts	Threads not excluded from shear planes	Threads excluded from shear planes	Factor of safety Ω
A325 bolts	$758 - 25f_v \leq 607$	$758 - 19f_v \leq 607$	2.0
A354 Grade BD bolts	$841 - 25f_v \leq 676$	$841 - 19f_v \leq 676$	2.0
A449 bolts	$690 - 25f_v \leq 552$	$690 - 19f_v \leq 552$	2.0
A490 bolts	$938 - 25f_v \leq 745$	$938 - 19f_v \leq 745$	2.0
A307 bolts, Grade A			2.25
When 6.4 mm $\leq d <$ 12.7 mm	$359 - 28f_v \leq 276$		
When $d \geq$ 12.7 mm	$403 - 28f_v \leq 310$		

*The shear stress f_v must also satisfy Table 9.11.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

9.19 SCREW CONNECTIONS

Screws are frequently used for connections in cold-formed steel because they can be driven with a hand-held drill, usually without punching a hole. The *AISI NAS* gives provisions for calculating nominal strength for self-tapping screws with $0.08 \leq d \leq 0.25$ in ($2.03 \leq d \leq 6.35$ mm) where d is the nominal screw diameter. The screws can be of the thread-forming or thread-cutting type, with or without a self-drilling point.

The distance between the centers of fasteners, and the distance from the center of a fastener to the edge of any part, should not be less than $3d$. However, if the connection is subjected to shear force in one direction only, the minimum edge distance in the direction perpendicular to the force is $1.5d$.

Nominal strength equations are given for shear and for tension using the following notation:

P_{ns} = nominal shear strength per screw
P_{ss} = nominal shear strength per screw as reported by manufacturer or as tested
P_{nt} = nominal tension strength per screw
P_{not} = nominal pull-out strength per screw
P_{nov} = nominal pull-over strength per screw
P_{ts} = nominal shear strength per screw as reported by manufacturer or as tested

TABLE 9.15 Nominal Tension Stress, F'_{nt} (MPa), for Bolts Subject to Combination of Shear and Tension—LRFD Method*

Description of bolts	Threads not excluded from shear planes	Threads excluded from shear planes	Resistance factor ϕ
A325 bolts	$779 - 17f_v \leq 621$	$779 - 13f_v \leq 621$	0.75
A354 Grade BD bolts	$876 - 17f_v \leq 696$	$876 - 13f_v \leq 696$	0.75
A449 bolts	$696 - 17f_v \leq 558$	$696 - 13f_v \leq 558$	0.75
A490 bolts	$972 - 17f_v \leq 776$	$972 - 13f_v \leq 776$	0.75
A307 bolts, Grade A			0.75
When 6.4 mm $\leq d <$ 12.7 mm	$324 - 25f_v \leq 279$		
When $d \geq$ 12.7 mm	$359 - 25f_v \leq 310$		

*The shear stress f_v must also satisfy Table 9.11.

Source: *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C., 2001, with permission.

t_1 = thickness of member in contact with the screw head
t_2 = thickness of member not in contact with the screw head
F_{u1} = tensile strength of member in contact with the screw head
F_{u2} = tensile strength of member not in contact with the screw head

For screw connections, $\Omega = 3.0$ and $\phi = 0.50$.

9.19.1 Shear

The nominal shear strength per screw, P_{ns}, should be determined as follows:

For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of

$$P_{\text{ns}} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2} \tag{9.121}$$

$$P_{\text{ns}} = 2.7 t_1 d F_{u1} \tag{9.122}$$

$$P_{\text{ns}} = 2.7 t_2 d F_{u2} \tag{9.123}$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{\text{ns}} = 2.7 t_1 d F_{u1} \tag{9.124}$$

$$P_{\text{ns}} = 2.7 t_2 d F_{u2} \tag{9.125}$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} should be determined by linear interpolation between the above two cases.

Table 9.16 gives values of P_{ns} for #8, #10, and #12 screws calculated from the preceding equations. Additionally, the following limit based on the strength of the screw itself applies:

$$P_{\text{ns}} = 0.8 P_{\text{ss}} \tag{9.126}$$

9.19.2 Tension

For screws that carry tension, the diameter of the head of the screw, or of the washer if one is used, must be at least $^5/_{16}$ in (7.94 mm). Washers must be at least 0.05 in (1.27 mm) thick. Two conditions must be checked: (1) pull-out of the screw and (2) pull-over of the sheet. In addition, the nominal tensile strength must not exceed the following limit based on the strength of the screw itself:

$$P_{\text{nt}} = 0.8 P_{\text{ts}} \tag{9.127}$$

Pull-Out. The nominal pull-out strength, P_{not}, is calculated as

$$P_{\text{not}} = 0.85 t_c d F_{u2} \tag{9.128}$$

where t_c is the lesser of the depth of screw penetration and the thickness t_2.

Pull-Over. The nominal pull-over strength, P_{nov}, is calculated as

$$P_{\text{nov}} = 1.5 t_1 d_w F_{u1} \tag{9.129}$$

where d_w is the larger of the screw head diameter or the washer diameter, and must be taken no larger than $^1/_2$ in (12.7 mm).

9.20 OTHER LIMIT STATES AT CONNECTIONS

The *AISI NAS* gives procedures for checking certain other important limit states at member end connections. Included are shear-lag effects in bolted, screwed, and welded connections where not all elements of the cross section are connected, shear strength along a plane through fasteners in beam webs where one or both flanges are coped, and block shear rupture where a connecting element can fail through a combination of shear on one plane and tension on a perpendicular plane. Many of these requirements are similar to those of the AISC.

TABLE 9.16 Nominal Shear Strength of Screws, P_{ns}, kips (kips × 4.448 = kN)

Screw designation	Diameter, in	Thickness of member in contact with screw head, in	Thickness of member not in contact with the screw head, in						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
(a) Screws in sheet with $F_u = 45$ ksi									
#8	0.1640	0.036	0.52	0.72	0.72	0.72	0.72	0.72	0.72
		0.048	0.52	0.80	0.96	0.96	0.96	0.96	0.96
		0.060	0.52	0.80	1.12	1.20	1.20	1.20	1.20
		0.075	0.52	0.80	1.12	1.49	1.49	1.49	1.49
		0.090	0.52	0.80	1.12	1.49	1.79	1.79	1.79
		0.105	0.52	0.80	1.12	1.49	1.79	2.09	2.09
		0.135	0.52	0.80	1.12	1.49	1.79	2.09	2.69
#10	0.1900	0.036	0.56	0.83	0.83	0.83	0.83	0.83	0.83
		0.048	0.56	0.87	1.11	1.11	1.11	1.11	1.11
		0.060	0.56	0.87	1.21	1.39	1.39	1.39	1.39
		0.075	0.56	0.87	1.21	1.69	1.73	1.73	1.73
		0.090	0.56	0.87	1.21	1.69	2.08	2.08	2.08
		0.105	0.56	0.87	1.21	1.69	2.08	2.42	2.42
		0.135	0.56	0.87	1.21	1.69	2.08	2.42	3.12
#12	0.2160	0.036	0.60	0.93	0.94	0.94	0.94	0.94	0.94
		0.048	0.60	0.92	1.26	1.26	1.26	1.26	1.26
		0.060	0.60	0.92	1.29	1.57	1.57	1.57	1.57
		0.075	0.60	0.92	1.29	1.80	1.97	1.97	1.97
		0.090	0.60	0.92	1.29	1.80	2.36	2.36	2.36
		0.105	0.60	0.92	1.29	1.80	2.36	2.76	2.76
		0.135	0.60	0.92	1.29	1.80	2.36	2.76	3.54
(b) Screws in sheet with $F_u = 65$ ksi									
#8	0.1640	0.036	0.76	1.04	1.04	1.04	1.04	1.04	1.04
		0.048	0.76	1.16	1.38	1.38	1.38	1.38	1.38
		0.060	0.76	1.16	1.62	1.73	1.73	1.73	1.73
		0.075	0.76	1.16	1.62	2.16	2.16	2.16	2.16
		0.090	0.76	1.16	1.62	2.16	2.59	2.59	2.59
		0.105	0.76	1.16	1.62	2.16	2.59	3.02	3.02
		0.135	0.76	1.16	1.62	2.16	2.59	3.02	3.89
#10	0.1900	0.036	0.81	1.20	1.20	1.20	1.20	1.20	1.20
		0.048	0.81	1.25	1.60	1.60	1.60	1.60	1.60
		0.060	0.81	1.25	1.75	2.00	2.00	2.00	2.00
		0.075	0.81	1.25	1.75	2.44	2.50	2.50	2.50
		0.090	0.81	1.25	1.75	2.44	3.00	3.00	3.00
		0.105	0.81	1.25	1.75	2.44	3.00	3.50	3.50
		0.135	0.81	1.25	1.75	2.44	3.00	3.50	4.50
#12	0.2160	0.036	0.87	1.34	1.36	1.36	1.36	1.36	1.36
		0.048	0.87	1.33	1.82	1.82	1.82	1.82	1.82
		0.060	0.87	1.33	1.86	2.27	2.27	2.27	2.27
		0.075	0.87	1.33	1.86	2.61	2.84	2.84	2.84
		0.090	0.87	1.33	1.86	2.61	3.41	3.41	3.41
		0.105	0.87	1.33	1.86	2.61	3.41	3.98	3.98
		0.135	0.87	1.33	1.86	2.61	3.41	3.98	5.12

Source: Adapted from *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, D.C., 2002.

9.21 WALL STUD ASSEMBLIES

Steel studs are being used increasingly for the construction of interior and exterior walls in residential and light commercial applications. Typical studs for load-bearing walls are *C* sections of 33 ksi yield-point steel, 3.5 in deep by 1.625 in wide, 0.033 or 0.043 in thick, with a $^1/_2$-in stiffener lip on the edge of the flange. The stud depth is sometimes increased to 6 in so that thicker insulation can be used in the cavity, or for high walls or severe loadings. A steel bracing member (blocking) usually runs between the studs at mid-height. Often the exterior surface of the wall is sheathed with CDX or plywood, and the interior with gypsum board. In some cases, both surfaces may be sheathed with gypsum. Similar *C* sections, 6 to 12 in deep, are used for floor joists. Roof construction may be with steel rafters or with steel trusses, available fabricated from *C* sections or from proprietary shapes. Figure 9.12 depicts the various components in a typical steel-framed dwelling.

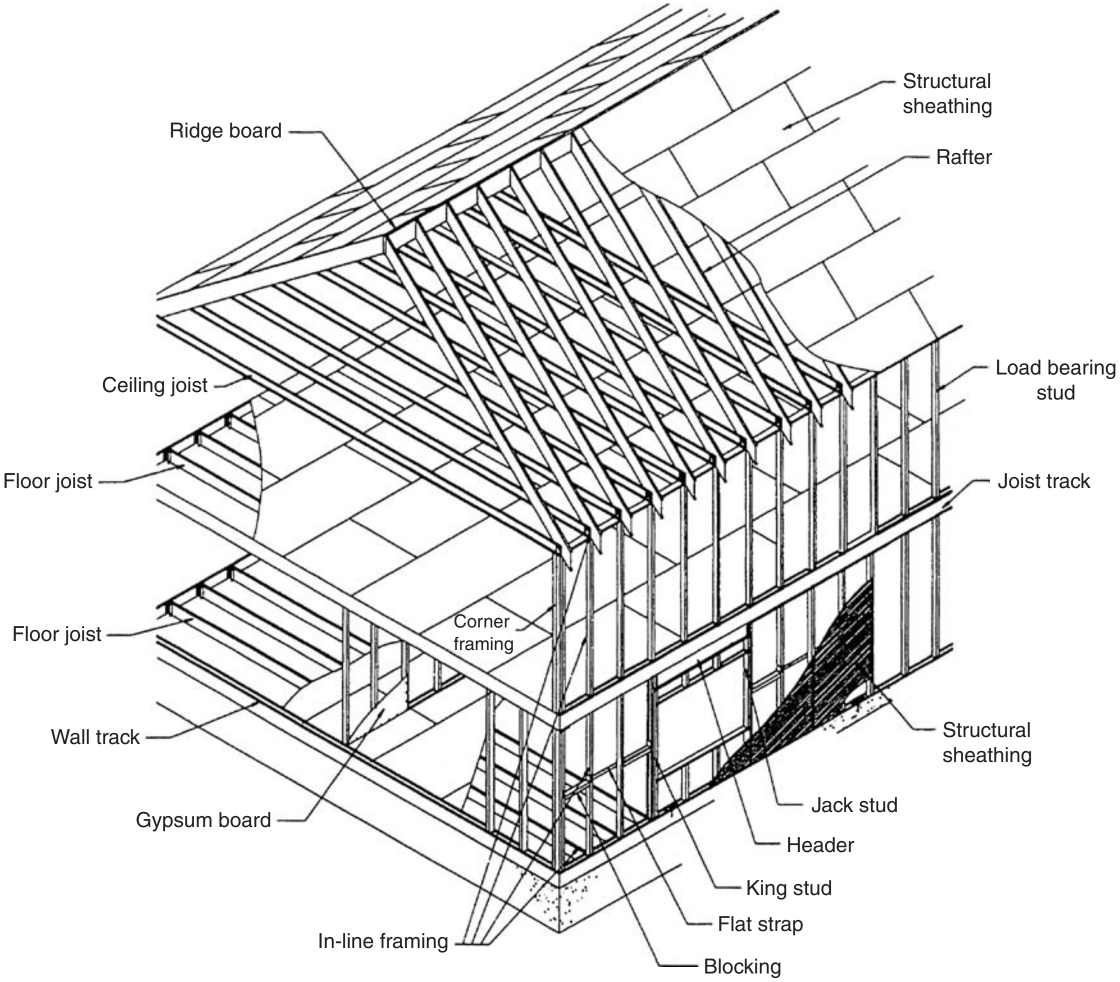


FIGURE 9.12 Steel framing in residential construction. (*Source:* Prescriptive Method for Residential Cold-Formed Steel Framing, *2d ed., NAHB Research Center, Upper Marlboro, Md., 1997, with permission.*)

Prior to 2004, the *AISI Specification* contained requirements for sheathing braced design. In 2004, these provisions were removed. The *AISI NAS* now permits sheathing braced design in accordance with an appropriate theory, tests, or rational engineering analysis.

The AISI *Standard for Cold-Formed Steel Framing—Wall Design* (*AISI Wall Stud Standard*) permits the design of wall studs to be based on either an all-steel design in which discrete braces are provided along the member's length, or based on a sheathing-braced design. The AISI *Wall Stud Standard* stipulates that when sheathing-braced design is used, the wall stud must be evaluated without the sheathing bracing for the dead loads and loads that may occur during construction. This provides for the possibility that the sheathing has been removed or has accidentally become ineffective. Also, sheathing-braced design for wall stud assemblies assumes that identical sheathing is attached to both sides of the wall stud. This limit recognizes that identical sheathing will aid in minimizing the twisting of the section. If only single-sided sheathing is used, additional twisting of the section will occur, thus placing a greater demand on the sheathing; therefore, the stud must be designed and braced as an all-steel assembly.

Steel stud wall assemblies with sheathing also serve as in-plane diaphragms and shear walls to brace the structure and resist racking from wind or seismic loads. The AISI *Standard for Cold-Formed Steel Framing—Lateral Design* addresses the levels of performance of shear walls for both wind and seismic loads.

(For further information on wall stud assemblies and steel framing, see *Standard for Cold-Formed Steel Framing—Prescriptive Method*, 2001 ed., American Iron and Steel Institute, Washington, D.C.; *Standard for Cold-Formed Steel Framing—Lateral Design*, American Iron and Steel Institute, Washington, D.C.; *Tech Notes*; Light Gauge Steel Engineers Association, Washington, D.C.)

9.22 EXAMPLE OF EFFECTIVE SECTION CALCULATION

A 5.5-in-deep by 1.25-in-wide by 0.057-in-thick *C* section without lips is shown in Fig. 9.13*a*. The specified minimum yield stress for the material is 33 ksi. It is required to determine the effective section modulus, S_e, for a maximum bending stress equal to the yield stress.

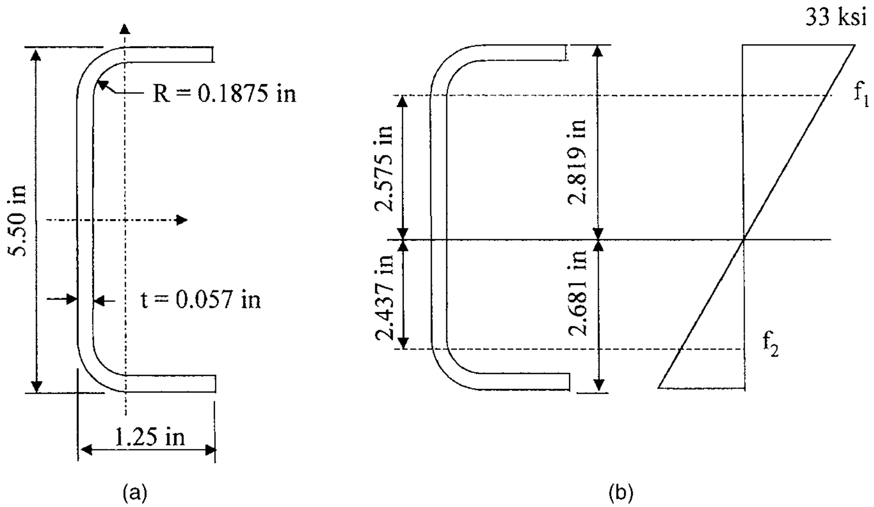


FIGURE 9.13 Unstiffened *C* section for example problem (Art. 9.22). (*a*) Cross section. (*b*) Stress distribution on effective section.

First, determine the effective width of the compression (top) flange (Arts. 9.8.1 and 9.9.1). The radius to mid-thickness of the bend is

$$\begin{aligned} r &= R + \frac{t}{2} \\ &= 0.1875 + \frac{0.057}{2} \\ &= 0.216 \text{ in} \end{aligned}$$

The flat width of the flange is

$$\begin{aligned} w &= 1.25 - 0.216 - \frac{0.057}{2} \\ &= 1.006 \text{ in} \end{aligned}$$

$$\frac{w}{t} = \frac{1.006}{0.057} = 17.65$$

The plate buckling coefficient (Art. 9.9.1) is $k = 0.43$.

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}}$$

$$\lambda = \frac{1.052}{\sqrt{0.43}}(17.65)\sqrt{\frac{33}{29{,}500}} \qquad \text{[Eq. (9.4c)]}$$

$$= 0.947 > 0.673$$

$$\rho = \frac{1 - 0.22/\lambda}{\lambda}$$

$$= \frac{1 - 0.22/0.947}{0.947} \qquad \text{[Eq. (9.7)]}$$

$$= 0.811$$

The effective width of the top flange is

$$\begin{aligned} b &= \rho w \\ &= (0.811)(1.006) \qquad \text{[Eq. (9.6)]} \\ &= 0.816 \text{ in} \end{aligned}$$

The next step is to determine whether the web is fully effective. To do this, first determine the location of the neutral axis. Because the top flange is not fully effective, the neutral axis will be located below the centroidal axis of the gross cross section. Table 9.17 shows the calculations to determine the distance of the neutral axis from the top fiber, $\bar{y}$, and the moment of inertia of the effective section, I_x. The web is treated as a stiffened element with a stress gradient (Art. 9.8.2). With a stress of 33 ksi in the top flange, the stresses at the edges of the flat web, f_1 and f_2 (Fig. 9.5*b*), can be readily determined from similar triangles. The other calculations follow from Art. 9.8.2.

$$f_1 = \frac{2.819 - 0.216 - 0.057/2}{2.819}(33) = 30.14 \text{ ksi}$$

$$f_2 = -\frac{5.500 - 2.819 - 0.216 - 0.057/2}{2.819}(33) = -28.52 \text{ ksi}$$

$$\psi = \left|\frac{f_2}{f_1}\right| = \frac{28.52}{30.14} = 0.946 \qquad \text{[Eq. (9.8)]}$$

TABLE 9.17 Example of Effective Section Property Calculations

Element	L (in)	y from top fiber (in)	Ly (in^2)	Ly^2 (in^3)	I_x about own axis (in^3)
Top flange	0.816	0.0285	0.023	0.001	—
Top radius	0.339	0.1069	0.036	0.004	0.002
Web	5.011	2.7500	13.780	37.896	10.486
Bottom radius	0.339	5.3931	1.828	9.860	0.002
Bottom flange	1.006	5.4715	5.504	30.117	—
Sum Σ	7.511		21.171	77.878	10.490

$\bar{y} = \Sigma Ly/\Sigma L$
$= 21.171/7.511 = 2.819$ in below top fiber
$I_x = [\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$
$= [10.490 + 77.878 - (2.819)^2(7.511)](0.057)$
$= 1.635$ in^4

Source: Adapted from *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, D.C., 1996.

$$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$$

$$= 4 + 2(1+0.946)^3 + 2(1+0.946) \qquad \text{[Eq. (9.9)]}$$

$$= 22.63$$

$$w = 5.500 - 2(0.216) - 0.057 = 5.011 \text{ in}$$

$$\frac{w}{t} = \frac{5.011}{0.057} = 87.91$$

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}}$$

$$= \frac{1.052}{\sqrt{22.63}}(87.91)\sqrt{\frac{30.14}{29500}} \qquad \text{[Eq. (9.4c)]}$$

$$= 0.621 < 0.673, \text{ therefore, } b_e = w = 5.011 \text{ in}$$

$$b_1 = b_e/(3+\psi)$$

$$= 5.011/(3+0.946) = 1.270 \text{ in} \qquad \text{[Eq. (9.10a)]}$$

$$\frac{h_0}{b_0} = \frac{5.50}{1.25} = 4.40 > 4$$

$$b_2 = \frac{b_e}{(1+\psi)} - b_1 \qquad \text{[Eq. (9.11b)]}$$

$$= \frac{5.011}{1+0.946} - 1.270 = 1.305 \text{ in}$$

$$b_1 + b_2 = 1.270 + 1.305 = 2.575 \text{ in}$$

Based on the assumption of a fully effective web, the width that is in compression, Fig. 9.13*b*, is $2.819 - 0.057/2 - 0.216 = 2.575$ in. Because $b_1 + b_2$ does not exceed 2.575 in, the web is fully effective and no further iteration is required. If the web had not been fully effective, additional iterations in section property calculations would be required until the final effective section was determined for the stresses acting on that effective section. In the present case, the effective section modulus is $S_e = I_x/\bar{y} = 1.635/2.819 = 0.580$ in^3.

9.23 EXAMPLE OF BENDING STRENGTH CALCULATION

For the unstiffened C section in Art. 9.22, determine the moment strength based on initiation of yielding for a fully braced section. Then determine the allowable moment based on ASD and the maximum factored moment based on LRFD.

From Art. 9.12.1, the nominal strength is

$$\begin{aligned} M_n &= S_e F_y \\ &= 0.580 \times 33 && \text{[Eq. (9.25)]} \\ &= 19.1 \text{ in}\cdot\text{kip} \end{aligned}$$

From Table 9.1, $\Omega = 1.67$ and $\phi = 0.90$. Therefore, for ASD, the allowable moment based on nominal loads is

$$\begin{aligned} M &= \frac{M_n}{\Omega} \\ &= \frac{19.1}{1.67} \\ &= 11.4 \text{ in}\cdot\text{kip} \end{aligned}$$

For LRFD, the maximum moment based on factored loads is

$$\begin{aligned} M_u &= \phi M_n \\ &= 0.90 \times 19.1 \\ &= 17.2 \text{ in}\cdot\text{kip} \end{aligned}$$

CHAPTER 10

HIGHWAY BRIDGE DESIGN CRITERIA*

Dennis R. Mertz, Ph.D., P.E.
Professor of Civil Engineering
University of Delaware
Newark, Delaware

This chapter provides guidance to highway bridge designers in the application of standard design specifications to the more common types of bridges. In addition, it provides rules of thumb to result in cost-effective and safe structures. Because of the complexity of modern bridge design and construction, this chapter does not provide comprehensive treatment of all types of bridges. Because specifications are continually being revised, readers are cautioned to use the latest edition of the applicable specification, including interims, in practical applications.

10.1 SPECIFICATIONS

Traditionally, design of most highway bridges in the United States was in accord with the "Standard Specifications for Highway Bridges" (Standard Specifications) published by the American Association of State Highway and Transportation Officials (AASHTO), 444 N. Capitol St., NW, Washington, DC 20001. However, in the early 1990s, AASHTO introduced a new, more modern and comprehensive highway bridge design specification, the "LRFD Bridge Design Specifications" (LRFD Specifications) as an equal alternative to the Standard Specifications. AASHTO publishes new editions of these specifications periodically, and annual revisions to each are published as "Interim Specifications."

The Federal Highway Administration (FHWA) and AASHTO established a goal to design all new highway bridges after October 2007 in accord with the LRFD Specifications. Since 1994, the states have been slowly moving toward full adoption of the LRFD Specifications. Interim changes to the Standard Specifications have not been made since 2000, and a final complete version as frozen in 2000 was issued as the 17th edition in 2002.

The design criteria for highway bridges in this chapter are based on the 17th (2002) edition of the Standard Specifications, and the 3d (2004) edition of the LRFD Specifications, with 2005 Interims. Many of the provisions of the specifications are common to both. However, where appropriate, the differences between the LRFD Specifications and the Standard Specifications are discussed.

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*Revised and updated from "Application of Criteria for Cost-Effective Highway Bridge Design" by Robert L. Nickerson, P.E., President, NBE, Ltd., and Dennis R. Mertz, Ph.D., P.E., University of Delaware, Sec. 11, Part 1, in the Third Edition.

TABLE 10.1 Maximum Central Angle for Neglecting Curvature in Determining Primary Bending Moments

Number of beams	Angle for one span	Angle for two or more spans
2	2°	3°
3 or 4	3°	4°
5 or more	4°	5°

For complex design-related items or modifications involving new technology, AASHTO issues tentative "Guide Specifications," to allow further assessment and refinement of the new criteria. AASHTO may adopt a Guide Specification, after a trial period of use, as part of its specifications.

States usually adopt the AASHTO bridge specifications as minimum standards for highway bridge design. Because conditions vary from state to state, however, many bridge owners modify the standard specifications to meet specific needs in their own design manuals. For example, California has specific requirements for earthquake resistance that may not be appropriate for less active seismic regions.

To ensure safe, cost-effective, and durable structures, designers should meet the requirements of the latest specifications and guides available. For unusual types of structures, including long-span bridges, designers should make a more detailed application of theory and performance than is possible with standard criteria or the practices described in this chapter. While the Standard Specifications are specifically limited to bridges less than 500 ft long, the LRFD Specifications include no limit on span length. Use of much of the specifications, however, is appropriate for unusual structures, inasmuch as these generally are composed of components to which the specifications are applicable.

Horizontally curved steel girders are fully covered by the LRFD Specifications, but are not a part of the Standard Specifications. The LRFD Specifications, as a result of the 2005 Interim Revisions, incorporate horizontally curved steel I-girders and box girders as a part of a unified treatment of straight and curved girders. For the design of bridges with horizontally curved steel girders using the Standard Specifications, refer to the AASHTO "Guide Specifications for Horizontally Curved Steel Girder Highway Bridges," 2002, as well as any Interim Specifications. Where the maximum central angle is less than that listed in Table 10.1, curvature has usually been neglected in determining primary bending moments.

For a complete catalog of AASHTO publications that may be useful in the design, fabrication, and erection of steel highway bridges, see www.transportation.org or contact AASHTO at 444 N. Capitol St., NW, Washington, DC 20001.

10.2 GENERAL DESIGN CONSIDERATIONS

10.2.1 Geometric and Traffic Design

The primary purpose of a highway bridge is to safely carry (geometrically and structurally) the necessary traffic volumes and loads. Normally, traffic volumes, present and future, determine the number and width of traffic lanes, establish the need for, and width of, shoulders. The Standard Specifications provide a range of design truck load models so that the minimum design truck weight can also be a function of traffic volumes. The LRFD Specifications eliminate this option within the concept that heavy trucks can occur throughout the system. The geometric and traffic design requirements are usually established by the owner's planning and highway design section using the roadway design criteria contained in "A Policy on Geometric Design of Highways and Streets," AASHTO. Where lane widths, shoulders, and other pertinent dimensions are not established by the owner, this AASHTO policy should be used for guidance. Ideally, bridge designers will be part of the highway design team to ensure that unduly complex bridge geometric requirements, or excessive bridge lengths, are not generated during the highway-location approval process.

Traffic considerations for bridges are not necessarily limited to overland vehicles. In many cases, ships, rail traffic, and construction equipment must be considered. Requirements for safe passage of extraordinary traffic over *and* under the structure may impose additional restrictions on the design that could be quite severe.

10.2.2 Service Life

The LRFD Specifications address service life by requiring design and material considerations that will achieve a specific 75-year design life. The Standard Specifications have historically not included requirements for a specified design service life for bridges. It is assumed that if the design provisions are followed, proper materials are specified, a quality assurance procedure is in place during construction, and adequate maintenance is performed, an acceptable service life will be achieved. An examination of the existing inventory of steel bridges throughout the United States indicates this to be generally true, although there are examples where service life is not acceptable. The predominant causes for reduced service life are geometric deficiencies because of increases in traffic that exceed the original design-traffic capacity.

10.2.3 Deflection Limitations

The Standard Specifications impose deflection limitations. Highway bridges consisting of simple or continuous spans should be designed so that deflection due to live load plus impact does not exceed 1/800th of the span. For bridges available to pedestrians in urban areas, this deflection should be limited to 1/1000th of the span. For cantilevers, the deflection should generally not exceed 1/300th of the cantilever arm, or 1/375th where pedestrian traffic may be carried.

Live-load deflection computations for beams and girders should be based on gross moment of inertia of the cross section, or of the transformed section for composite girders. For a truss, deflection computations should be based on the gross area of each member, except for sections with perforated cover plates. For such sections, the effective area (net volume divided by length center to center of perforations) should be used.

Deflection of steel bridges has always been important in design, becoming even more significant with the trend toward high-performance steels (HPS) of yield strengths greater than 50 ksi. If a bridge is too flexible, the public often complains about bridge vibrations, especially if sidewalks are present that provide access to the public. There is also a concern that bridge vibrations may cause premature deck deterioration. In an attempt to satisfy all these concerns, the above deflection limitations, as well as minimum depth–span ratios, have been imposed as a means of ensuring sufficient stiffness of bridge members. However, there is some doubt about the need for these limitations, especially relative to the potential for increased deck cracking. Many studies indicate that flexing of the superstructure is not a cause of increased deck cracking. Most European countries do not have live-load deflection limits. Nonetheless, the states desire control of gross bridge stiffness.

In the LRFD Specifications, these same limits are optional. If applied, the LRFD Specifications require that deflections be checked as part of the service limit state and include in the "Commentary" the statement: "Service limit states are intended to allow the bridge to perform acceptably for its service life. . . . Bridges should be designed to avoid undesirable structural or psychological effects from their deflection and vibrations. While no specific deflection, depth, or frequency limitations are specified herein, except for orthotropic decks, any large deviation from past successful practice regarding slenderness and deflections should be cause for review of the design to determine that it will perform adequately." The optional criteria for deflections apply to all structure types, not just steel. The LRFD Specifications also require checking compact I-section members for permanent deflections.

10.2.4 Stringers and Floorbeams

Stringers are beams generally placed parallel to the longitudinal axis of the bridge, or direction of traffic, in highway bridges, such as truss bridges. Usually they should be framed into floorbeams. However, if they are supported on the top flanges of the floorbeams, it is desirable that the stringers he continuous over two or more panels. In bridges with wood floors, intermediate cross frames or diaphragms should be placed between stringers more than 20 ft long.

In skew bridges without end floorbeams, the stringers, at the end bearings, should be held in correct position by end struts also connected to the main trusses or girders. Lateral bracing in the end panels should be connected to the end struts and main trusses or girders.

Floorbeams preferably should be perpendicular to main trusses or girders. Also, connections to those members should be positioned to permit attachment of lateral bracing, if required, to both floorbeam and main truss or girder.

Main material of floorbeam hangers should not be coped or notched. Built-up hangers should have solid or perforated web plates or lacing.

10.2.5 Stringer and Girder Spacing

One of the major factors affecting the economy of highway bridges with a concrete deck on stringers or longitudinal girders is spacing of the main members. Older bridges typically had spacing of 8 ft or less. Now, however, longer concrete-deck spans (up to 15 ft) are practicable through the use of such devices as stay-in-place metal or precast-concrete forms. This allows the designer to use fewer girders. (To eliminate the fracture-critical designation when I-shape girders are used, at least three girders should be provided. To facilitate future redecking, a minimum of four girders should be considered.) Although the steel weight per square foot of bridge may be higher with fewer girders, more substantial overall savings result from the reduced costs of fabrication, handling, transportation, erecting, and painting, if required. For economy, girder spacing should generally be at least 10 ft.

10.2.6 Span Lengths

Another important factor that affects economy is span length. Where there is an opportunity to use different span lengths, site-specific studies should be made, including costs of both superstructure and substructure. Many designers believe that steel girders, because of their lower weight per foot, should have longer spans than concrete beams for a bridge at the same location, but this is not necessarily the case. Some studies, including of the cost of substructure units, have shown substantial economies for the steel alternative when the spans are kept the same. However, as with any preliminary study, site-specific considerations may indicate otherwise. For example, where the foundation or substructure costs, or both, are extremely high, it is probable that longer steel girders, with fewer substructure units, will be more cost-effective than shorter spans.

10.2.7 Constructability

Sometimes, unnecessary problems develop during construction of a bridge that could easily have been prevented with an appropriate design. Also, the construction procedures used by a contractor may lock in stresses unaccounted for in design that will adversely influence the performance of the bridge. Two specific areas in which difficulties have occurred have been in construction of horizontally curved girder bridges and in deck-concrete placing sequences, especially when the bridge has a large skew.

As part of bridge design, the designers should assume an erection and concrete placing sequence and check for construction stresses. The assumed methods should be included on the contract plans for the contractor's information, with the understanding that deviations will be accepted subject to the ability of the contractor to demonstrate that no adverse stresses will result from the proposed method.

The LRFD Specifications, to ensure that designers properly consider constructability, specify that bridges be designed so that fabrication and erection can be performed without undue difficulty or distress and that the effects of locked-in construction forces are within tolerable limits. When the method of construction of a bridge is not self-evident, or could induce unacceptable locked-in stresses, the designer should propose at least one feasible method on the plans. If the design requires some strengthening or temporary bracing or support during erection by the selected method, the plans should indicate the need thereof.

To provide for the above, designers should check for what is essentially a construction limit state, using the following factored load combination:

$$1.25D + 1.5L + 1.25W + 1.0\ \Sigma(\text{other forces as appropriate})$$

where D is the weight of the structure and appurtenances, L is the construction equipment (including dynamic effects), and W is the wind load. While the Standard Specifications are silent on the issue, this concept should be applied to all designs, regardless of which specification is used.

10.2.8 Inspectability

Inspectability of all bridge members and connections is an essential design-stage consideration. This is especially apparent when the structure includes enclosed sections, such as box girders. Bridge service life has been impaired in the past when designers, concerned with stress distribution, either did not include access holes or made them so small it was impossible for an inspector to perform an adequate inspection. To ensure inspectability, experienced bridge inspectors should review the bridge design at an early stage of development.

Another consideration is safety of inspectors and traffic using the bridge during the inspection. A preferred method of inspection has been the use of a type of crane that allows easy access to underbridge members. However, on routes with very high traffic volumes, the presence of an inspection vehicle on the bridge creates a safety hazard to both inspection personnel and the traveling public. Other means of inspection should be provided in these instances, such as inspection ladders, walkways, catwalks, covered access holes, and provision for lighting, if necessary.

10.3 DESIGN METHODS

The Standard Specifications present two design methods for steel bridges: service-load, or allowable stress, design (ASD) and strength, or load-factor, design (LFD). The LRFD Specifications present the load and resistance factor design (LRFD) method. Although procedures for ASD are presented in many of the articles in this chapter, because it is still widely used, LFD or LRFD may often yield more economical results. As indicated previously, AASHTO is replacing the ASD and LFD methods with the LRFD method.

10.3.1 ASD Method

Allowable stress design is a method of proportioning structural members using design loads and forces (nominal values), allowable stresses that include a safety factor, and other design limitations as appropriate for service conditions. For example, fatigue and deflection under design loadings must be considered. Based on elastic behavior, ASD is the traditional method used by designers, preceding the earliest specifications.

10.3.2 LFD Method

Load factor design is a method of proportioning structural members for multiples of the design loads, that is, **factored loads.** Serviceability and durability must be addressed, by controlling permanent deflections under overload, and considering fatigue and deflection under design loadings. Elastic behavior is assumed in calculating moments, shears, and other forces. In contrast to the ASD method, the nominal loads are increased by the specified factors instead of applying a safety factor to the stresses. For example, under specified conditions, the maximum bending strength of a compact flexural member in LFD may be equal to its plastic moment (yield point times plastic section modulus). However, the moment that must be resisted is calculated for a factored load that is greater than the service load.

10.3.3 LRFD Method

Load and resistance factor design is a method of proportioning structural members by applying factors to both the design loads and the nominal strength of the member. The specified factors were established after an extensive study, based on the theory of reliability and statistical knowledge of load and material characteristics. (See also Chap. 5.) A structure designed by LRFD should be better proportioned, with all parts of the structure theoretically designed for the same degree of reliability. The LRFD Specifications identify methods of modeling and analysis, and incorporate many of the existing AASHTO Guide Specifications. Also included are more up-to-date features that are equally applicable to ASD and LFD, which are simply not included in the Standard Specifications. For example, the LRFD specifications include serviceability requirements for durability of bridge materials, inspectability of bridge components, maintenance that includes deck-replacement considerations in adverse environments, constructability, ridability, economy, and esthetics.

The LRFD Specifications require bridges "to be designed for specified **limit states** to achieve the objectives of constructability, safety and serviceability, with due regard to issues of inspectability, economy and aesthetics." These limit states may be considered as simple groupings of the traditional design criteria. Each component and connection must satisfy Eq. (10.1) for each limit state. All limit states are considered of equal importance. The basic relationship requires that the effect of the sum of the factored loads, Q, must be less than or equal to the factored resistance, R_r, of the bridge component being evaluated for *each* limit state. This is expressed as

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \tag{10.1}$$

where η_i = factor combining the effects of ductility η_D, redundancy η_R, and importance η_i; for a non-fracture-critical member on a typical steel bridge, η_i will be 1.0

γ_i = statistically based factor accounting for uncertainty of the load to be applied to the various load effects

Q_i = effect of each individual load as included in Art. 10.3.4; this could be a moment, shear, stress, deformation, etc.

ϕ = statistically based resistance factor accounting for the uncertainty of the resistance to be applied to the nominal resistance, as discussed in Art. 10.3.5

R_n = nominal resistance of the member (or connection) being evaluated

R_r = factored resistance, $R_n \times \phi$

There are four limit states to be satisfied: Service; Fatigue and Fracture; Strength; and Extreme Event. The Service Limit State has four different combinations of load factors, which place restrictions on stress, deformation, and crack width under regular service conditions. One service limit state, Service II, relates to steel superstructures. The Service II load combination controls permanent deformation due to yielding of compact steel members and slip of slip-critical connections. This service limit state corresponds to the "overload" check of the Standard Specifications.

The Fatigue and Fracture Limit State checks the dynamic effect on the bridge components of a single truck known as the **fatigue truck.** Restrictions are placed on the range of stress induced by passage of trucks on the bridge. This limit is intended to prevent initiation of fatigue cracking during the design life of the bridge. Article 10.9 provides additional discussion of the Fatigue Limit State.

Fracture is controlled by the requirement for minimum material toughness values included in the LRFD Specifications and the AASHTO or ASTM material specifications, and depends on where the bridge is located. (See Art. 1.1.5.) Article 10.8 provides additional discussion of the Fracture Limit State.

The Strength Limit State has five different combinations of load factors to be satisfied. This limit state assures the component and/or connection has sufficient strength to withstand the designated combinations of the different permanent and transient loadings that could statistically happen during

TABLE 10.2 Partial Load Combinations and Load Factors for LRFD

	Factors for indicated load combinations*				
Limit state	*DC*, *DD*, *DW*, *EH*, *EV*, *ES*	*LL*, *IM*, *CE*, *BR*, *PL*, *LS*	*WA*	*WS*	*WL*
Strength I	γ_p	1.75	1.00	—	—
Strength II	γ_p	1.35	1.00	—	—
Strength V	γ_p	1.35	1.00	0.40	1.00
Service II	1.00	1.30	1.00	—	—
Fatigue (*LL*, *IM*, and *CE* only)	—	0.75	—	—	—

*See Table 10.3 for γ_p values. See Art. 10.5 for load descriptions.

the life of the structure. This is the most important limit state, since it checks the basic strength requirements. Strength I is the basic check for normal usage of the bridge. Strength II is the check for owner-specified permit vehicles. Strength III checks for the effects of high winds (>55 mi/h) with no live load on the bridge, since trucks would not be able to travel safely under this condition. Strength IV checks strength under a possible high dead-to-live load force–effect ratio, such as for very long spans. This condition governs when the ratio exceeds 7.0. Strength V checks the strength when live load is on the bridge and a 55 mi/h wind is blowing.

Extreme Event Limit State is intended "to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle or ice flow possibly under a scoured condition." This design requirement recognizes that structural damage is acceptable under extreme events, but collapse should be prevented.

Performance ratios, defined as the ratio of a calculated value to the corresponding allowable value, are useful for determining the relative importance of the various LRFD requirements in a particular design. It is recommended that designers develop performance ratios for all designs to aid in evaluating and optimizing the design.

10.3.4 LRFD Load Combinations

The effects of each of the loads discussed in Art. 10.5, appropriately factored, must be evaluated in various combinations for LRFD as indicated in Tables 10.2 and 10.3. These combinations are statistically based determinations for structure design. Only those applicable to steel bridge superstructure designs are listed. See the LRFD Specifications for a complete listing.

10.3.5 LRFD Resistance Factors

The nominal resistance of the various bridge components, such as flexural members, webs in shear, and fasteners (bolts or welds), is given by equations in the LRFD Specifications. Each nominal resistance must be multiplied by a resistance factor, ϕ, which is a statistically based number that accounts for uncertainties between calculated strength according to the specifications and the possible actual

TABLE 10.3 LRFD Load Factors for Permanent Load, γ_p

	Load factor	
Type of load	Maximum	Minimum
DC: component and attachments	1.25	0.90
DW: wearing surface and utilities	1.50	0.65

TABLE 10.4 Resistance Factors, ϕ, for Strength Limit State for LRFD

Flexure	$\phi_f = 1.00$
Shear	$\phi_v = 1.00$
Axial compression, steel only	$\phi_c = 0.90$
Axial compression, composite	$\phi_c = 0.90$
Tension, fracture in net section	$\phi_u = 0.80$
Tension, yielding in gross section	$\phi_y = 0.95$
Bearing on pins, in reamed, drilled or bolted holes and milled surfaces	$\phi_b = 1.00$
Bolts bearing on material	$\phi_{bb} = 0.80$
Shear connectors	$\phi_{sc} = 0.85$
A325 and A490 bolts in tension	$\phi_t = 0.80$
A307 bolts in tension	$\phi_t = 0.80$
A307 bolts in shear	$\phi_s = 0.65$
A325 and A490 bolts in shear	$\phi_s = 0.80$
Block shear	$\phi_{bs} = 0.80$
Weld metal in complete penetration welds:	
Shear on effective area	$\phi_{c1} = 0.85$
Tension or compression normal to effective area	ϕ = base metal ϕ
Tension or compression parallel to axis of weld	ϕ = base metal ϕ
Weld metal in partial penetration welds:	
Shear parallel to axis of weld	$\phi_{c2} = 0.80$
Tension or compression parallel to axis of weld	ϕ = base metal ϕ
Compression normal to the effective area	ϕ = base metal ϕ
Tension normal to the effective area	$\phi_{c1} = 0.80$
Weld metal in fillet welds:	
Tension or compression parallel to axis of the weld	ϕ = base metal
Shear in throat of weld metal	$\phi_{c2} = 0.80$

Note: All resistance factors for the extreme event limit state, except for bolts, are taken as 1.0.

strength of the member. The ϕ factor, Table 10.4, provides for inaccuracies in theory, variations in material properties and dimensions, and the consequences of failure. Expressions for the nominal resistance of many types of members are given in other chapters of this Handbook. The nominal strength of slip-critical bolts, which follows in Art. 10.3.6, illustrates the approach.

10.3.6 Nominal Resistance of Slip-Critical Bolts

The LRFD Specifications specify the nominal resistance of slip-critical bolts as follows. Field connections in beams and girders are almost always made using high-strength bolts. Bolts conforming

TABLE 10.5 Minimum Required Bolt Tension

	Required tension, P_t, kips	
Bolt diameter, in	M164 (A325)	M253 (A490)
$^5/_8$	19	27
$^3/_4$	28	40
$^7/_8$	39	55
1	51	73
$1^1/_8$	56	92
$1^1/_4$	72	116
$1^3/_8$	85	139
$1^1/_2$	104	169

TABLE 10.6 Values of K_h

Standard-size holes	1.0
Oversize and short-slotted holes	0.85
Long-slotted holes with slot perpendicular to direction of force	0.70
Long-slotted holes with slot parallel to direction of force	0.60

TABLE 10.7 Values of K_s

Class A surface conditions	0.33
Class B surface conditions	0.50
Class C surface conditions	0.33

Notes:

Class A surfaces are with unpainted clean mill scale, or blast cleaned surfaces with a Class A coating.

Class B surfaces are unpainted and blast cleaned, or painted with a Class B coating.

Class C surfaces are hot-dipped galvanized, and roughened by wire brushing.

to AASHTO M164 (ASTM A325) are the most-used types. AASHTO M253 (ASTM A490) are another type, but are rarely used. The LRFD Specifications require that bolted connections "subject to stress reversal, heavy impact loads, severe vibration or where stress and strain due to joint slippage would be detrimental to the serviceability of the structure" be designed as slip-critical. Slip-critical connections must be proportioned at Service II Limit State load combinations as specified in Table 10.2. The nominal slip resistance R_n of each bolt is

$$R_n = K_h K_s N_s P_t \tag{10.2}$$

where N_s = number of slip planes per bolt
P_t = minimum required bolt tension (see Table 10.5)
K_h = hole size factor (see Table 10.6)
K_s = surface condition factor (see Table 10.7)

10.4 SIMPLIFIED COMPARISON OF DESIGN METHODS

10.4.1 Loads and Load Factors

Every component of substructure and superstructure should be proportioned to resist all combinations of loads applicable to the type of bridge and its site. The AASHTO specifications designate certain combinations of loads that must be considered. Table 10.8 shows the **loading groups** according to the Standard Specifications, and Table 10.2 shows the **load combinations** according to the LRFD Specifications. Each member should be selected based on the most severe limit state and load combination. In the case of the Standard Specifications where fatigue is not represented in the loading groups, members must also be checked to make sure that allowable fatigue stresses are not exceeded.

For ASD, the allowable unit stresses given in AASHTO Standard Specifications depend on the loading group. (See Table 10.8.) No increase in allowable stress is permitted for members that carry only wind loads.

For LFD, the loading combinations are multiplied by a load factor. The forces, moments, and shears calculated for each of these loading combinations must not exceed the applicable member (or connection) strength given in the Standard Specifications.

For LRFD, the loading combinations are multiplied by a load factor, and in addition, resistance factors (1.0 or less) are applied to the nominal strength of members to compensate for various uncertainties in behavior.

To compare the effects of the design philosophies of ASD, LFD, and LRFD, the group loading requirements of the three methods will be examined. For simplification, only *D*, *L*, and *I* (dead load, live load, and impact) of Group I loading will be considered. Although not usually stated as such,

TABLE 10.8 Loading Combinations for Allowable Stress Design

Group	Loading combination	Percentage of basic unit stress
I	$D + L + I + CF + E + B + SF$	100
IA	$D + 2(L + I)$	150
IB	$D + (L + I)^* + CF + E + B + SF$	†
II	$D + E + B + SF + W$	125
III	$D + L + I + CF + E + B + SF + 0.3W + WL + LF$	125
IV	$D + L + I + E + B + SF + T$	125
V	$D + E + B + SF + W + T$	140
VI	$D + I + CF + E + B + SF + 0.3W + WL + LF + T$	140
VII	$D + E + B + SF + EQ$	133
VIII	$D + L + I + CF + E + B + SF + ICE$	140
IX	$D + E + B + SF + W + ICE$	150
X‡	$D + L + I + E$	100

where D = dead load
L = live load
I = live-load impact
E = earth pressure (factored for some types of loadings)
B = buoyancy
W = wind load on structure
WL = wind load on live load of 0.10 kip/lin ft
LF = longitudinal force from live load
CF = centrifugal force
T = temperature
EQ = earthquake
SF = stream-flow pressure
ICE = ice pressure

*For overload live load plus impact as specified by the operating agency.

†Percentage = $\dfrac{\text{maximum unit stress (operating rating)}}{\text{allowable basic unit stress}} \times 100$

‡For culverts.

the three methods can be compared using the same general equation for determining the effects of the combination of loads:

$$N\sum(F \times \text{load}) \leq RF \times \text{nominal resistance} \tag{10.3}$$

where N = design factor used in LRFD for ductility, redundancy, and operational importance of the bridge
= 1.0 for ASD and LFD
$\sum(F \times \text{load})$ = sum of the factored loads for a combination of loads
F = load factor that is applied to a specific load
= 1.0 for ASD for D, L, and I
load = one or more service loads that must be considered in the design
RF = resistance factor that is applied to the nominal resistance
= reciprocal of safety factor for ASD
nominal resistance = strength of a member based on the type of loading, e.g., tension, compression, or shear

For a noncompact flexural member subjected to bending by dead load, live load, and impact forces, let D, L, and I represent the maximum tensile stress in the extreme surface due to dead load, live load, and impact, respectively. Then, for each of the design methods, the following must be satisfied:

ASD:
$$D + L + I \leq 0.55F_y \tag{10.4a}$$

which can be rewritten as

$$1.82D + 1.82(L + I) \leq F_y \tag{10.4b}$$

LFD:

$$1.30D + 2.17(L + I) \leq F_y \tag{10.5}$$

Clearly, the LFD method acknowledges the uncertainty of load through greater load factors for live load than dead load. However, the load factors, while different, are not calibrated to produce uniform reliability for all bridges.

For strength limit state *I*, assuming *D* is for components and attachments,

LRFD:

$$1.25D + 1.75(L + I) \leq 1.0F_y \tag{10.6}$$

For LFD and LRFD, if the section is compact and adequately braced, the flexural strength can be as great as the full plastic moment. For this example, the flexural strength is conservatively assumed to be limited to the yield moment.

The effect of the applied loads appears to be less for LRFD in this example, but there are many other considerations for LRFD designs. For instance, the design live-load model produces greater force effects for LRFD. For steel superstructures, LRFD also requires checking five different **strength** limit states, one **service** limit state, a **fatigue-and-fracture** limit state, and two **extreme-event** limit states. Although each structure may not have to be checked for all these limit states, the basic philosophy of the LRFD specifications is to assure serviceability over the design service life, safety of the bridge through redundancy and ductility of all components and connections, and survival (prevention of collapse) of the bridge when subjected to an extreme event, e.g., a 500-year flood. (See Art. 10.3.3.)

10.4.2 Member Design

To compare the results of a design by ASD, LFD, and LRFD, a 100-ft, simple-span girder bridge is selected as a simple example. It has an 8-in-thick, noncomposite concrete deck, and longitudinal girders, made of Grade 50 steel, spaced 12 ft center to center. It will carry HS20 live load. The section modulus *S*, in^3, will be determined for a laterally braced interior girder with a live-load distribution factor of 1.0. The bending moment due to dead loads is estimated to be about 2200 ft·kips. The maximum moment due to the HS20 truck loading is 1524 ft·kips (Table 10.9).

$$\text{LRFD lane-load live-load moment} = \frac{wL^2}{8} = \frac{0.64(100)^2}{8} = 800 \text{ ft·kips}$$

For both ASD and LFD, the impact factor [see Art. 10.5.2, Eq. (10.7)] is

$$I = \frac{50}{100 + 125} = 0.22$$

For LRFD, $IM = 0.33$ (Table 10.10).

Allowable Stress Design. The required section modulus *S* for the girder for allowable stress design is computed as follows. The design moment is

$$M = M_D + (1 + I)ML = 2200 + 1.22 \times 1524 = 4059 \text{ ft·kips}$$

For $F_y = 50$ ksi, the allowable stress is $F_b = 0.55 \times 50 = 27$ ksi. The section modulus required is then

$$S = \frac{M}{F_b} = \frac{4059 \times 12}{27} = 1804 \text{ in}^3$$

TABLE 10.9 Maximum Moments, Shears, and Reactions for Truck or Lane Loads on One Lane, Simple Spans[*]

	H15		H20		HS15		HS20	
Span, ft	Moment[†]	End shear and end reaction[‡]	Moment[†]	End shear and end reaction[‡]	Moment[†]	End shear and end reaction[‡]	Moment[†]	End shear and end reaction[‡]
10	60.0[§]	24.0[§]	80.0[§]	32.0[§]	60.0[§]	24.0[§]	80.0[§]	32.0[§]
20	120.0[§]	25.8[§]	160.0[§]	34.4[§]	120.0[§]	31.2[§]	160.0[§]	41.6[§]
30	185.0[§]	27.2[§]	246.6[§]	36.3[§]	211.6[§]	37.2[§]	282.1[§]	49.6[§]
40	259.5[§]	29.1	346.0[§]	38.8	337.4[§]	41.4[§]	449.8[§]	55.2[§]
50	334.2[§]	31.5	445.6[§]	42.0	470.9[§]	43.9[§]	627.9[§]	58.5[§]
60	418.5	33.9	558.0	45.2	604.9[§]	45.6[§]	806.5[§]	60.8[§]
70	530.3	36.3	707.0	48.4	739.2[§]	46.8[§]	985.6[§]	62.4[§]
80	654.0	38.7	872.0	51.6	873.7[§]	47.7[§]	1,164.9[§]	63.6[§]
90	789.8	41.1	1,053.0	54.8	1,008.3[§]	48.4[§]	1,344.4[§]	64.5[§]
100	937.5	43.5	1,250.0	58.0	1,143.0[§]	49.0[§]	1,524.0[§]	65.3[§]
110	1,097.3	45.9	1,463.0	61.2	1,277.7[§]	49.4[§]	1,703.6[§]	65.9[§]
120	1,269.0	48.3	1,692.0	64.4	1,412.5[§]	49.8[§]	1,883.3[§]	66.4[§]
130	1,452.8	50.7	1,937.0	67.6	1,547.3[§]	50.7	2,063.1[§]	67.6
140	1,648.5	53.1	2,198.0	70.8	1,682.1[§]	53.1	2,242.8[§]	70.8
150	1,856.3	55.5	2,475.0	74.0	1,856.3	55.5	2,475.1	74.0
160	2,075.0	57.9	2,768.0	77.2	2,076.0	57.9	2,768.0	77.2
170	2,307.8	60.3	3,077.0	80.4	2,307.8	60.3	3,077.1	80.4
180	2,551.5	62.7	3,402.0	83.6	2,551.5	62.7	3,402.1	83.6
190	2,807.3	65.1	3,743.0	86.8	2,807.3	65.1	3,743.1	86.8
200	3,075.0	67.5	4,100.0	90.0	3,075.0	67.5	4,100.0	90.0
220	3,646.5	72.3	4,862.0	96.4	3,646.5	72.3	4,862.0	96.4
240	4,266.0	77.1	5,688.0	102.8	4,266.0	77.1	5,688.0	102.8
260	4,933.5	81.9	6,578.0	109.2	4,933.5	81.9	6,578.0	109.2
280	5,649.0	86.7	7,532.0	115.6	5,649.0	86.7	7,532.0	115.6
300	6,412.5	91.5	8,550.0	122.0	6,412.5	91.5	8,550.0	122.0

[*] Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials. Impact not included.
[†] Moments is thousands of ft·lb (ft·kips).
[‡] Shear and reaction in kips. Concentrated load is considered placed at the support. Loads used are those stipulated for shear.
[§] Maximum value determined by standard truck loading. Otherwise, standard lane loading governs.

The section in Fig. 10.1, weighing 280.5 lb/ft, supplies a section modulus within 1% of the required *S* and is acceptable.

Load Factor Design. The design moment for LFD is

$$\begin{aligned} M &= 1.3M_D + 2.17(1 + I)M_L \\ &= 1.3 \times 2200 + 2.17 \times 1.22 \times 1524 = 6895 \text{ ft·kips} \end{aligned}$$

For $F_y = 50$ ksi, the section modulus required for LFD is

$$S = \frac{M_u}{F_y} = \frac{6895 \times 12}{50} = 1655 \text{ in}^3$$

TABLE 10.10 Dynamic Load Allowance, *IM*, for Highway Bridges for LRFD

Component	Limit state	Dynamic load allowance, %
Deck joints	All	75
All other components	Fatigue and fracture	15
	All	33

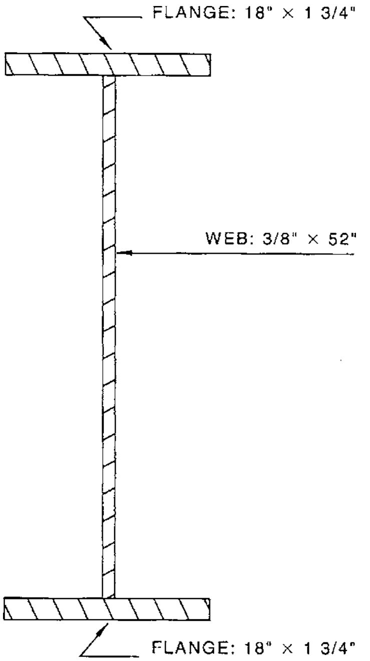

FIGURE 10.1 Girder with transverse stiffeners determined by ASD and LRFD for a 100-ft span: S = 1799 in^3; w = 280.5 lb/ft.

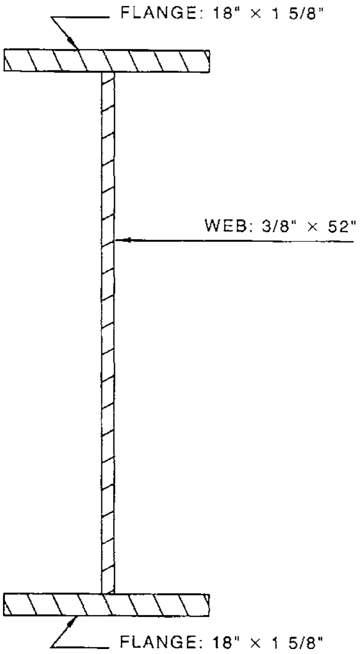

FIGURE 10.2 Girder with transverse stiffeners determined by LFD for a 100-ft span: S = 1681 in^3; w = 265 lb/ft.

If a noncompact section is chosen, this value of S is the required elastic section modulus. For a compact section, it is the plastic section modulus Z. Figure 10.2 shows a noncompact section supplying the required section modulus, with a $^3/_8$-in-thick web and $1^5/_8$-in-thick flanges. For a compact section, a $^5/_8$-in-thick web is required and $1^1/_4$-in-thick flanges are satisfactory. In this case, the noncompact girder is selected and will weigh 265 lb/ft.

Load and Resistance Factor Design. The live-load moment M_L is produced by a combination of truck and lane loads, with impact applied only to the truck moment:

$$M_L = 1.33 \times 1524 + 800 = 2827 \text{ ft·kips}$$

The load factor N is a combination of factors applied to the loadings. Assuming that the bridge is a typical steel girder bridge, N = 1.0. The design moment for limit state I is

$$\begin{aligned} M_u &= N(F_D M_D + F_L M_L) \\ &= 1.0(1.25 \times 2200 + 1.75 \times 2827) \\ &= 7697 \text{ ft·kips} \end{aligned}$$

Hence, since the resistance factor for flexure is 1.0, the section modulus required for LRFD is

$$S = \frac{7697 \times 12}{50} = 1847 \text{ in}^3$$

The section selected for ASD (Fig. 10.1) is satisfactory for LRFD if a 2% overstress is deemed acceptable.

For this example, the weight of the girder for LFD is 94% of that required for ASD and LRFD. The heavier girder required for LRFD is due primarily to the larger live load specified. For both LFD and LRFD, a compact section would be more advantageous, because it reduces the need for transverse stiffeners for the same basic weight of girder.

10.5 HIGHWAY DESIGN LOADINGS

10.5.1 Load Classifications

The Standard Specifications require bridges to be designed to carry dead loads and live loads with impact, the dynamic effect of the moving live load. Structures should also be capable of sustaining other loads to which they may be subjected, such as longitudinal, centrifugal, thermal, seismic, and erection forces. Various combinations of these loads must be considered as designated in **loading groups** I through X. (See Table 10.8.)

The LRFD Specifications separate loads into two categories, permanent and transient, and considers them in certain **load combinations**. (See Table 10.2.) Following are the loads to be considered and their designations.

Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = accumulated locked-in force effects resulting from construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CY = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance (traditionally termed impact)
LL = vehicular live load
LS = live-load surcharge
PL = pedestrian live load
SE = settlement
SH = shrinkage
TG = temperature gradient
TU = uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

10.5.2 Loads for Design

Certain loads applicable to the design of superstructures of steel beam (girder) and composite bridges are discussed in detail below.

Dead Loads. Designers should use the actual dead weights of materials specified for the structure. For the more commonly used materials, AASHTO specifications provide the weights to be used. For other materials, designers must determine the proper design loads. It is important that the dead loads used in design be noted on the contract plans for analysis purposes during possible future rehabilitations.

Live Loads. Four standard classes of highway vehicle loadings are included in the Standard Specifications: H15, H20, HSI5, and HS20. The AASHTO Geometric Guide states that the minimum design loading for new bridges should be HS20 (Fig. 10.3) for all functional classes (local roads through freeways) of highways. Therefore, most bridge owners require design for HS20 truck loadings or greater. AASHTO also specifies an alternative tandem loading of two 25-kip axles spaced 4 ft center to center.

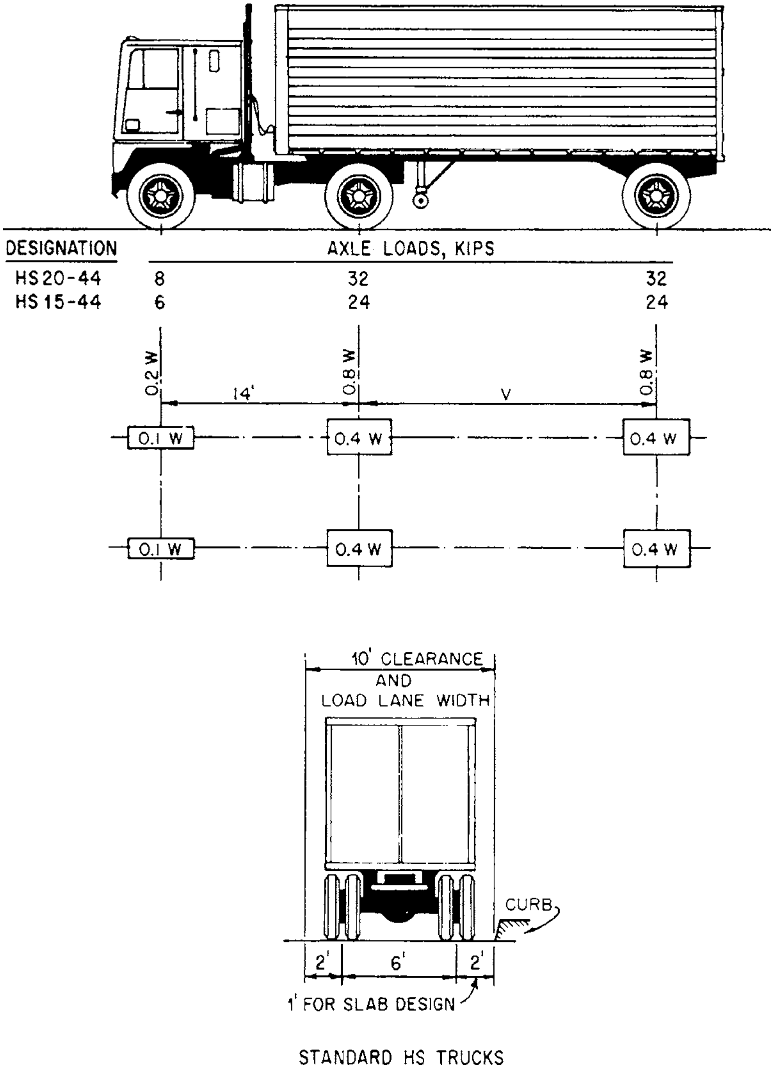

FIGURE 10.3 Standard HS loadings for design of highway bridges. Truck loading for ASD and LFD. *W* is the combined weight of the first two axles. *V* is the spacing of the axles, between 14 and 30 ft, inclusive, that produces maximum stresses.

TABLE 10.11 Multiple-Presence Factors

Number of loaded lanes	Multiple-presence factor m
1	1.20
2	1.00
3	0.85
>3	0.65

The difference in truck gross weights is a direct ratio of the HS number, e.g., HSI5 is 75% of HS20. (The difference between the H and HS trucks is the use of a third axle on an HS truck.) Many bridge owners, recognizing the trucking industry's use of heavier vehicles, are specifying design loadings greater than HS20.

For longer-span bridges, lane loadings are used to simulate multiple vehicles in a given lane. For example, for HS20 loading on a simple span, the lane load is 0.64 kips/ft plus an 18-kip concentrated load for moment or a 26-kip load for shear. A simple-span girder bridge with a span longer than about 140 ft would be subjected to a greater live-load design moment for the lane loading than for the truck loading (Table 10.9). (For end shear and reaction, the breakpoint is about 120 ft.) Truck and lane loadings are not applied concurrently for ASD or LFD.

In ASD and LFD, if maximum stresses are induced in a member by loading of more than two lanes, the live load for three lanes should be reduced by 10%, and for four or more lanes, by 25%.

For LRFD, the design vehicle design load is a combination of truck (or tandem) and lane loads and differs for positive and negative moment. Figure 10.4 shows the governing live loads for LRFD to produce maximum moment in a beam. The vehicular design live loading is one of the major differences in the LRFD Specifications. Through statistical analysis of existing highway loadings, and their effect on highway bridges, a combination of the design truck (the traditional HS20 truck), or design tandem (similar to the alternative military load of the Standard Specifications and intended primarily for short spans), and the design lane load (the traditional HS20 lane load without the concentrated loads), constitutes the HL-93 design live load for LRFD. As in previous specifications, this loading occupies a 10-ft width of a design lane. Depending on the number of design lanes on the bridge, the possibility of more than one truck being on the bridge must be considered. The effects of the HL-93 loading should be factored by the multiple-presence factor (see Table 10.11). However, the multiple-presence factor should not to be applied for fatigue calculations, or when the subsequently discussed approximate live-load distribution factors are used.

Impact or Dynamic Load Allowance. A factor is applied to the statically applied vehicular live loads to represent amplification of loading due to dynamic effects of the moving vehicles. In the Standard Specifications, the impact factor I is a function of span and is determined from

$$I = \frac{50}{L + 125} \leq 0.30 \tag{10.7}$$

In this equation, L, ft, should be taken as follows:

	For moment	For shear
For simple spans	L = design span length for roadway decks, floorbeams, and longitudinal stringers	L = length of loaded portion from point of consideration to reaction
For cantilevers	L = length from point of consideration to farthermost axle	Use I = 0.30
For continuous spans	L = design length of span under consideration for positive moment; average of two adjacent loaded spans for negative moment	L = length as for simple spans

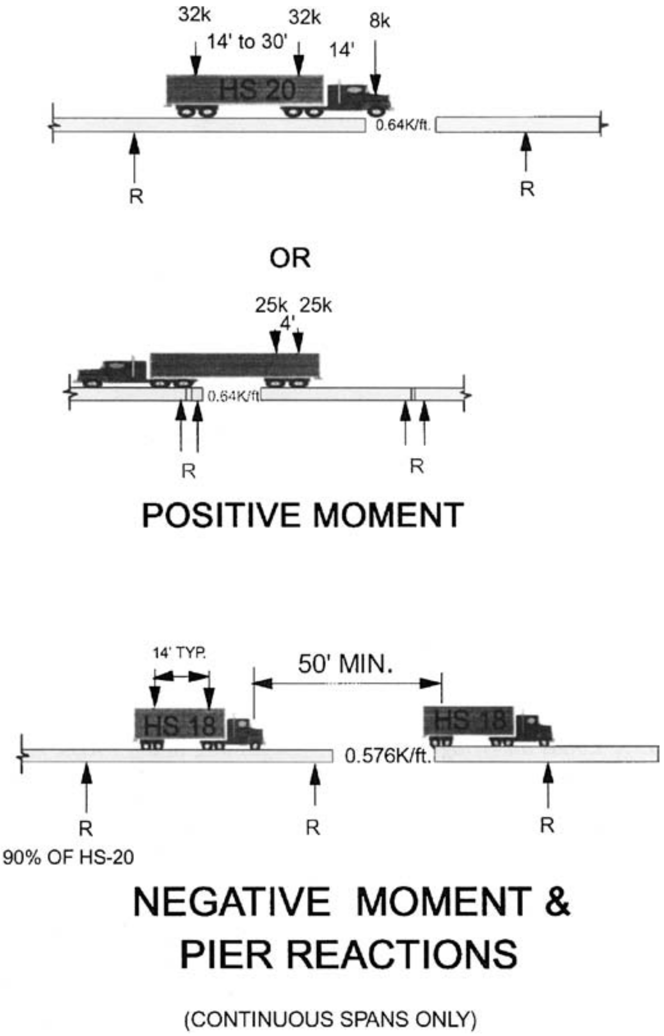

FIGURE 10.4 Loadings for maximum moment and reaction for LRFD design of highway bridges.

For LRFD, the impact factor is modified in recognition of the concept that the factor should be based on the type of bridge component, rather than the span. Termed **dynamic load allowance,** values are given in Table 10.10 with the basic value a constant 33%. It is applied only to the vehicle portion of the live load, not the lane load.

Live Loads on Bridge Railings. Beginning in the 1960s, AASHTO specifications increased minimum design loadings for railings to a 10-kip load applied horizontally, intended to simulate the force of a 4000-lb automobile traveling at 60 mi/hr and impacting the rail at a 25° angle. In 1989, AASHTO published the Guide Specifications for Bridge Railings, with requirements more representative of current vehicle impact loads and dependent on the class of highway. Since the effects of impact-type loadings are difficult to predict, the AASHTO Guide requires that railings be subjected to full-scale impact tests to a performance level *PL* that is a function of the highway type, design speed, percent of trucks in traffic, and bridge-rail offset. Generally, only low-volume, rural roads may utilize a rail tested to the PL-1 level, and high-volume interstate routes require a PL-3 rail. The full-scale tests apply the forces that must be resisted by the rail and its attachment details to the bridge deck.

PL-I represents the forces delivered by an 1800-lb automobile traveling at 50 mi/h, or a 5400-lb pickup truck at 45 mi/h, and impacting the rail system at an angle of 20°. PL-2 represents the forces delivered from an automobile or pickup as in PL-1, but traveling at a speed of 60 mi/h, in addition to an 18,000-lb truck at 50 mi/h at an angle of 15°. PL-3 represents forces from an automobile or pickup as in PL-2, in addition to a 50,000-lb van-type tractor-trailer traveling at 50 mi/h and impacting at an angle of 15°.

The performance criteria require not only resistance to the vehicle loads but also acceptable performance of the vehicle after the impact. The vehicle may not penetrate or hurdle the railing, must remain upright during and after the collision, and be smoothly redirected by the railing. Thus, a rail system that can withstand the impact of a tractor-trailer truck may not be acceptable if redirection of a small automobile is not satisfactory.

The LRFD Specifications have included the above criteria, updated to include strong preference for use of rail systems that have been subjected to full-scale impact testing, because the force effects of impact-type loadings are difficult to predict. Test parameters for rail-system impact testing are included in NCHRP Report 350, "Recommended Procedures for the Safety Performance Evaluation of Highway Features." These full-scale tests provide the forces that the rail-to-bridge deck attachment details must resist.

Because of the time and expense involved in full-scale testing, it is advantageous to specify previously tested and approved rails. State highway departments may provide these designs on request.

Earthquake Loads. Seismic design is governed in a section of the Standard Specifications titled "Division I-A for Seismic Design." In the case of the LRFD Specifications, seismic provisions appear throughout the specifications wherever appropriate. Engineers should be familiar with the total content of these complex specifications to design adequate earthquake-resistant structures. These specifications are also the basis for the earthquake "extreme-event" limit state of the LRFD specifications, where the intent is to allow the structure to suffer damage but have a low probability of collapse during seismically induced ground shaking. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. (See Art. 10.10.)

The purpose of the seismic design provisions is to "establish design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes." Each structure is assigned to a seismic performance category (SPC), which is a function of location relative to anticipated design ground accelerations and to the importance classification of the highway routing. The SPC assigned, in conjunction with factors based on the site soil profile and response modification factor for the type of structure, establishes the minimum design parameters that must be satisfied.

Steel superstructures for beam/girder bridges are rarely governed by earthquake criteria. Also, because a steel superstructure is generally lighter in weight than a concrete superstructure, lower seismic forces are transmitted to the substructure elements.

Vessel Impact Loads. A loading that should be considered by designers for bridges that cross navigable waters is that induced by impact of large ships. Guidance for consideration of vessel impacts on a bridge is included in the AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges. This Guide Specification is based on probabilistic theories, accounting for differences in size and frequency of ships that will be using a waterway. The Guide also forms the basis for the extreme-event limit state for vessel collision incorporated into the LRFD Specifications.

Thermal Loads. Provisions must be included in bridge design for stresses *and* movements resulting from temperature variations to which the structure will be subjected. In the Standard Specifications, for steel structures, anticipated temperature extremes are as follows:

Moderate climate: 0 to 120°F

Cold climate: –30 to +120°F

With a coefficient of expansion of 65×10^{-7} in/in/°F, the resulting change in length of a 100-ft-long bridge member is

Moderate climate: $120 \times 65 \times 10^{-7} \times 100 \times 12 = 0.936$ in
Cold climate: $150 \times 65 \times 10^{-7} \times 100 \times 12 = 1.170$ in

If a bridge is erected at the average of high and low temperatures, the resulting change in length will be one-half of the above.

For complex structures such as trusses and arches, length changes of individual members may induce secondary stresses that must be taken into account.

The LRFD Specifications give two methods for determining temperature range. Procedure A is analogous to the method of the Standard Specifications. Procedure B is based on new research and utilizes temperature contour maps of the United States for both maximum and minimum design temperatures. Further, the LRFD Specifications mandate a load factor of 1.2 in determining movements and 0.5 in determining stresses. The 0.5 load factor acknowledges the inelastic response of the structure in redistributing stresses.

Longitudinal Forces. Roadway decks are subjected to braking forces, which they transmit to supporting members. The Standard Specifications specify a longitudinal design force of 5% of the live load in all lanes carrying traffic in the same direction, without impact. The force should be assumed to act 6 ft above the deck.

For LRFD, braking forces should be taken as 25% of the axle weights of the design truck or tandem per lane, placed in all design lanes that are considered to be loaded and that carry traffic headed in the same direction. These forces are applied 6.0 ft above the deck in either longitudinal direction to cause extreme force effects.

Centrifugal Force on Highway Bridges. Curved structures will be subjected to centrifugal forces by the live load. Such forces should be applied 6 ft above the roadway surface, measured at the centerline of the roadway. The Standard Specifications specify that the force be applied as a percentage (CF) of the live load without impact, as follows:

$$\frac{CF = 6.68S^2}{R} = \frac{0.00117}{S^2 D} \tag{10.8a}$$

where S = design speed, mi/hr
D = degree of curve = 5729.65/R
R = radius of curve, ft

For LRFD, AASHTO specifies a coefficient (C) to be multiplied by the weights of the design truck or tandem:

$$C = \frac{4v^2}{3gR} \tag{10.8b}$$

where v = highway design speed, ft/s (mi/h/0.682)
g = gravitational acceleration, 32.2 ft/s^2
R = radius of curvature, ft

Sidewalk Loadings. In the interest of safety, many highway structures in nonurban areas are designed so that the full shoulder width of the approach roadway is carried across the structure. Thus, the practical necessity for a sidewalk or a refuge walk is eliminated. There is no practical necessity that refuge walks on highway structures exceed 2 ft in width. Consequently, no live load need be applied. Current safety standards eliminate refuge walks on full-shoulder-width structures.

In urban areas, however, structures should conform to the configuration of the approach roadways. Consequently, bridges normally require curbs or sidewalks, or both. In these instances, the Standard Specifications indicate that sidewalks and supporting members should be designed for a live load of 85 lb/ft^2. Girders and trusses should be designed for the following sidewalk live loads, lb/ft^2 of sidewalk area:

Spans 0 to 25 ft: 85
Spans 26 to 100 ft: 60
Spans over 100 ft: $P = [30 + (3000/L)][(55 - W)50] \leq 60$

where L = loaded length, ft, and W = sidewalk width, ft.

For LRFD, a load of 75 lb/ft^2 is applied to all sidewalks wider than 2 ft.

Structures designed for exclusive use of pedestrians should be designed for 85 lb/ft^2 under either AASHTO specification.

Curb Loading. For ASD or LFD, curbs should be designed to resist a lateral force of at least 0.50 kip per lin ft of curb. This force should be applied at the top of the curb or 10 in above the bridge deck if the curb is higher than 10 in. For LRFD, curbs are limited to no more than 8 in high.

Where sidewalk, curb, and traffic rail form an integral system, the traffic railing loading applies. Stresses in curbs should be computed accordingly.

Wind Loading on Highway Bridges. The wind forces prescribed below, based on the Standard Specifications, Group II and Group V loadings, are considered a uniformly distributed, moving live load. They act on the exposed vertical surfaces of all members, including the floor system and railing as seen in elevation, at an angle of 90° to the longitudinal axis of the structure. These forces are presumed for a wind velocity of 100 mi/hr. They may be modified in proportion to the square of the wind velocity if conditions warrant change.

Superstructure. For trusses and arches: 75 lb/ft^2 but not less than 0.30 kip per lin ft in the plane of loaded chord, nor 0.15 kip per lin ft in the plane of unloaded chord.

For girders and beams: 50 lb/ft^2 but not less than 0.30 kip per lin ft on girder spans.

Wind on Live Load. A force of 0.10 kip per lin ft should be applied to the live load, acting 6 ft above the roadway deck.

Substructure. To allow for the effect of varying angles of wind in design of the substructure, the following longitudinal and lateral wind loads for the skew angles indicated should be assumed to be acting on the superstructure at the center of gravity of the exposed area.

When acting in combination with live load, the wind forces given in Table 10.12 may be reduced 70%. However, they should be combined with the wind load on the live load, as given in Table 10.13.

For usual girder and slab bridges with spans not exceeding about 125 ft, the following wind loads on the superstructure may be used for substructure design in lieu of the loading specified in Tables 10.12 and 10.13:

Wind on structure: 50 lb/ft^2 transverse; 12 lb/ft^2 longitudinal
Wind on live load: 100 lb/ft^2 transverse; 40 lb/ft^2 longitudinal

TABLE 10.12 Skewed Superstructure Wind Forces for Substructure Design*

	Trusses		Girders	
Skew angle of wind, deg	Lateral load, lb/ft^2	Longitudinal load, lb/ft^2	Lateral load, lb/ft^2	Longitudinal load, lb/ft^2
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	25	50	17	19

*"Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

TABLE 10.13 Wind Forces on Live Loads for Substructure Design*

Skew angle of wind, deg	Lateral load, lb/lin ft	Longitudinal load, lb/lin ft
0	100	0
15	88	12
30	82	24
45	66	32
60	34	38

*"Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

Transverse and longitudinal loads should be applied simultaneously.

Wind forces applied directly to the substructure should be assumed at 40 lb/ft^2 for 100-mi/hr wind velocity. For wind directions skewed to the substructure, this force may be resolved into components perpendicular to end and side elevations, acting at the center of gravity of the exposed areas. This wind force may be reduced 70% when acting in combination with live load.

Overturning Forces. In conjunction with forces tending to overturn the structure, there should be added an upward wind force, applied at the windward quarter-point of the transverse superstructure width, of 20 lb/ft^2, assumed to be acting on the deck and sidewalk plan area. For this load also, a 70% reduction may be applied when it acts in conjunction with live load.

The LRFD Specifications assume a base design wind velocity of 100 mi/h at 30 ft above ground level or low water, but the veolocity is modified based on the actual height of the bridge and the nature of the surrounding terrain. From this design wind velocity at the design elevation, a base wind pressure is calculated based on the component under consideration and the direction of the wind. For detailed LRFD wind load calculations, see Art. 13.8.2.

Uplift on Highway Bridges. Provision should be made to resist uplift by adequately attaching the superstructure to the substructure. The Standard Specifications recommend engaging a mass of masonry equal to the larger force for the following two conditions:

1. 100% of the calculated uplift caused by any loading or combination of loading in which the live-plus-impact loading is increased 100%.

2. 150% of the calculated uplift at working-load level.

Anchor bolts under the above conditions should be designed at 150% of the basic allowable stress.

The LRFD Specifications require designing for calculated uplift forces due to buoyancy, etc., and specifically require hold-down devices in seismic zones 2, 3, and 4.

Forces of Stream Current, Ice, and Drift on Highway Bridges. All piers and other portions of structures should be designed to resist the maximum stresses induced by the forces of flowing water, floating ice, or drift.

For ASD or LFD, the longitudinal pressure P, lb/ft^2, of flowing water on piers should be calculated from

$$P = KV^2 \tag{10.9}$$

where V = velocity of water, ft/s, and K = constant. In the AASHTO Standard Specifications, $K = 1.4$ for all piers subject to drift buildup and for square-ended piers, 0.7 for circular piers, and 0.5 for angle-ended piers where the angle is 30° or less.

In the AASHTO LRFD Specifications, the pressure P, ksf, is calculated from

$$P = \frac{C_D V^2}{1000} \tag{10.10}$$

where V = velocity of water, ft/s, for design flood and appropriate limit state, and C_D is a drag coefficient (0.7 for semicircular nosed pier, 1.4 for square-ended pier, 1.4 for debris launched against pier, and 0.8 for wedge-nosed pier with nose angle 90° or less).

For ice and drift loads, see AASHTO specifications.

Buoyancy should be taken into account in the design of substructures, including piling, and the design of superstructures, where necessary.

10.6 DISTRIBUTION OF LOADS THROUGH DECKS

Both of the specifications require that the width of a bridge roadway between curbs be divided into design traffic lanes 12 ft wide and loads located to produce maximum stress in supporting members. (Fractional parts of design lanes are not used.) Roadway widths from 20 to 24 ft, however, should have two design lanes, each equal to one-half the roadway width. Truck and lane loadings are assumed to occupy a width of 10 ft placed anywhere within the design lane to produce maximum effect.

If curbs, railings, and wearing surfaces are placed after the concrete deck has gained sufficient strength, their weight may be distributed equally to all stringers or beams. Otherwise, the dead load on the outside stringer or beam is the portion of the slab it carries.

The strength and stiffness of the deck determine, to some extent, the distribution of the live load to the supporting framing.

Shear. For determining end shears and reactions, the deck may be assumed to act as a simple span between beams for lateral distribution of the wheel load. For shear elsewhere, the wheel load should be distributed by the method required for bending moment for the Standard Specifications. The LRFD Specifications provide specific distribution factors for shear.

Moments in Longitudinal Beams. For ASD and LRFD, the fraction of a wheel load listed in Table 10.14 should be applied to each interior longitudinal beam for computation of live-load bending moments.

TABLE 10.14 Fraction of Wheel Load *DF* Distributed to Longitudinal Beams for ASD and LRFD*

Deck	Bridge with one traffic lane	Bridge with two or more traffic lanes
Concrete:		
On I-shaped steel beams	$S/7$, $S \leq 10$†	$S/5.5$, $S \leq 14$†
On steel box girders	$W_L = 0.1 + 1.7R + 0.85/N_w$‡	
Steel grid:		
Less than 4 in thick	$S/4.5$	$S/4$
4 in or more thick	$S/6$, $S \leq 6$†	$S/5$, $S \leq 10.5$†
Timber:		
Plank	$S/4$	$S/3.75$
Strip 4 in thick or multiple-layer floors over 5 in thick	$S/4.5$	$S/4$
Strip 6 in or more thick	$S/5$, $S \leq 5$†	$S/4.25$, $S \leq 6.5$†

*Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

†For larger values of S, average beam spacing, ft, the load on each beam should be the reaction of the wheel loads with the deck assumed to act as a simple span between beams.

‡Provisions for reduction of live load do not apply to design of steel box girders with W_L, fraction of a wheel (both front and rear).

R = number of design traffic lanes N_w divided by number of box girders ($0.5 \leq R \leq 1.5$)

$N_w = W_c/12$, reduced to nearest whole number

W_c = roadway width, ft, between curbs or barriers if curbs are not used.

TABLE 10.15 Fraction of Wheel Load Distributed to Transverse Beams*

Deck	Fraction per beam
Concrete	S/6†
Steel grid:	
Less than 4 in thick	S/4.5
4 in or more thick	S/6†
Timber:	
Plank	S/4
Strip 4 in thick, wood block on 4-in plank subfloor, or multiple-layer floors more than 5 in thick	S/4.5
Strip 6 in or more thick	S/5†

*Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

†When the spacing of beams *S*, ft, exceeds the denominator, the load on the beam should be the reaction of the wheel loads when the deck is assumed to act as a simple span between beams.

For an outer longitudinal beam, the live-load bending moments should be determined with the reaction of the wheel load when the deck is assumed to act as a simple span between beams. When four or more longitudinal beams carry a concrete deck, the fraction of a wheel load carried by an outer beam should be at least $S/5.5$ when the distance between that beam and the adjacent interior beam S, ft, is 6 or less. For $6 < S < 14$, the fraction should be at least $S/(4 + 0.25S)$. For $S > 14$, no minimum need be observed.

Moments in Transverse Beams. When a deck is supported directly on floorbeams without stringers, each beam should receive the fraction of a wheel load listed in Table 10.15, as a concentrated load, for computation of live-load bending moments.

Distribution for LRFD. Research has led to recommendations for changes in the distribution factors *DF* in Tables 10.14 and 10.15. AASHTO has adopted these recommendations as the basis for an approximate method in the LRFD Specifications and Guide Specification for Distribuiton of Loads for Highway Bridges when a bridge meets specified requirements. The Guide Specification may be used in conjunction with the Standard Specifications. In the LRFD Specifications, as an alternative to these distribution factors, a more refined method such as finite-element analysis is permitted.

The LRFD Specifications give the following equations as the approximate method for determining the distribution factor for moment for steel girders. They are in terms of the LRFD design truck load per lane. For one lane loaded,

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{10.11}$$

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12L_s^3}\right)^{0.1} \tag{10.12}$$

where S = beam spacing, ft

L = span, ft

t_s = thickness of concrete slab, in

$K_s = n(I + Ae_g^2)$

n = modular ratio = ratio of steel modulus of elasticity E_s to the modulus of elasticity E_c of the concrete slab

I = moment of inertia, in^4, of the beam

A = area, in^2, of the beam

e_s = distance, in, from neutral axis of beam to center of gravity of concrete slab

Equations (10.11) and (10.12) apply only for spans from 20 to 240 ft with $4^1/_2$- to 12-in-thick concrete decks (or concrete filled, or partially filled, steel grid decks), on four or more steel girders spaced between 3.5 and 16.0 ft. The multiple-presence factors, m, in Table 10.10 are *not* to be used when this approximate method of load distribution is used. For girder spacing outside the above limits, the live load on each beam is determined by the lever rule (summing moments about one support to find the reaction at another support by assuming the supported component is hinged at interior supports). When more refined methods of analysis are used, the LRFD Specifications state that "a table of live load distribution coefficients for extreme force effects in each span shall be provided in the contract documents to aid in permit issuance and rating of the bridge."

10.7 BASIC ALLOWABLE STRESSES FOR BRIDGES—ASD

Where the LRFD Specifications simply specify different resistance factors, ϕ, for each different resistance such as moment, shear, etc., for ASD in the Standard Specifications, different basic allowable stresses are given for each comparable resistance. Table 10.16 lists the basic allowable stresses for highway bridges according to the Standard Specifications. The stresses are related to the minimum yield strength F_y, ksi, or minimum tensile strength F_u, ksi, of the material in all cases except those for which stresses are independent of the grade of steel being used.

The basic stresses may be increased for loading combinations (Art. 10.5.1). They may be superseded by allowable fatigue stresses (Art. 10.9).

TABLE 10.16 Basic Allowable Stresses, ksi, for Allowable Stress Design of Highway Bridges[a]

Loading condition	Allowable stress, ksi
Tension:	
Axial, gross section without bolt holes	$0.55F_y^b$
Axial, net section	$0.50F_u^b$
Bending, extreme fiber of rolled shapes, girders, and built-up sections, gross section[c]	$0.55F_y$
Compression:	
Axial, gross section in:	
Stiffeners of plate girders	$0.55F_y$
Splice material	$0.55F_y$
Compression members;[d]	
$KL/r \le C_c$	$\frac{F_y}{F.S.}\left[1-\frac{(KL/r)^2 F_y}{4\pi^2 E}\right]$
$KL/r \ge C_c$	$\frac{\pi^2 E}{F.S.\,(KL/r)^2}$
Bending, extreme fiber of:	
Rolled shapes, girders, and built-up sections with:	
Compression flange continuously supported	$0.55F_y$
Compression flange intermittently supported[e]	$\frac{50\times10^6 C_b}{S_{xc}}\left(\frac{I_{yc}}{L}\right)$ $\times\sqrt{0.772\frac{J}{I_{yc}}+9.87\left(\frac{d}{L}\right)^2}$
Pins	$0.80F_y$
Shear:	
Webs of rolled beams and plate girders, gross section	$0.33F_y$
Pins	$0.40F_y$

(*Continued*)

TABLE 10.16 Basic Allowable Stresses, ksi, for Allowable Stress Design of Highway Bridges[a] (*Continued*)

Loading condition	Allowable stress, ksi
Bearing:	
Milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80F_y$
Pins:	
Not subject to rotation[f]	$0.80F_y$
Subject to rotation (in rockers and hinges)	$0.40F_y$

[a] F_y = minimum yield strength, ksi, and F_u = minimum tensile strength, ksi. Modulus of elasticity E = 29,000 ksi.

[b] Use $0.46F_u$ for ASTM A709, Grades 100/100W (M270) steels. Use net section if member has holes more than $1^1/_4$ in in diameter.

[c] When the area of holes deducted for high-strength bolts or rivets is more than 15% of the gross area, that area in excess of 15% should be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$ in diameter should be deducted. For ASTM A709 Grades 100/100W (M270) steels, use $0.46F_u$ on net section instead of $0.55F_y$ on gross section. For other steels, limit stress on net section to $0.50F_u$ and stress on gross section to $0.55F_y$.

[d] K = effective length factor. See Art. 5.4.

$C_c = \sqrt{2\pi^2 E/F_y}$

E = modulus of elasticity of steel, ksi
r = governing radius of gyration, in
L = actual unbraced length, in
$F.S.$ = factor of safety = 2.12

[e] Not to exceed $0.55F_y$.

L = length, in, of unsupported flange between lateral connections, knee braces, or other points of support
I_{yc} = moment of inertia of compression flange about the vertical axis is the plane of the web, in^4
d = depth of girder, in
$J = \frac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$, where b and t are the flange width and thickness, in, of the compression and tension flange, respectively, and t_w and D are the web thickness and depth, in, respectively
S_{xc} = section modulus with respect to compression flange, in^3
$C_b = 1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \le 2.3$, where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.
C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments. (For the use of larger C_b values, see Structural Stability Research Council, *Guide to Stability Design Criteria for Metal Structures.* If cover plates are used, the allowable static stress at the point of theoretical cutoff should be determined by the formula.)

[f] Applicable to pins used primarily in axially loaded members, such as truss members and cable adjusting links, and not applicable to pins used in members subject to rotation by expansion or deflection.

Allowable Stresses in Welds. AASHTO specifications require that weld metal used in bridges conform to the Bridge Welding Code, ANSI/AASHTO/AWS DI.5, American Welding Society.

Yield and tensile strengths of weld metal usually are specified to be equal to or greater than the corresponding strengths of the base metal. The allowable stresses for welds in bridges generally are as follows.

Groove welds are permitted the same stress as the base metal joined. When base metals of different yield strengths are groove-welded, the lower yield strength governs.

Fillet welds are allowed a shear stress of $0.27F_u$, where F_u is the tensile strength of the electrode classification or the tensile strength of the connected part, whichever is less. When quenched and tempered steels are joined, an electrode classification with strength less than that of the base metal may be used for fillet welds, but this should be clearly specified in the design drawings.

Plug welds are permitted a shear stress of 12.4 ksi.

These stresses may be superseded by fatigue requirements (Art. 10.9). The basic stresses may be increased for loading combinations as noted in Art. 10.5.

Effective area of groove and fillet welds for computation of stresses equals the effective length times effective throat thickness. The effective shearing area of plug welds equals the nominal cross-sectional area of the hole in the plane of the faying surface.

Effective length of a groove weld is the width of the parts joined, perpendicular to the direction of stress. The effective length of a straight fillet weld is the overall length of the full-sized fillet, including end returns. For a curved fillet weld, the effective length is the length of line generated by the center point of the effective throat thickness. For a fillet weld in a hole or slot, if the weld area computed from this length is greater than the area of the hole in the plane of the faying surface, the latter area should be used as the effective area.

Effective throat thickness of a groove weld is the thickness of the thinner piece of base metal joined. (No increase is permitted for weld reinforcement. It should be removed by grinding to improve fatigue strength.) The effective throat thickness of a fillet weld is the shortest distance from the root to the face, computed as the length of the altitude on the hypotenuse of a right triangle. For a combination partial-penetration groove weld and a fillet weld, the effective throat is the shortest distance from the root to the face minus $^1/_8$ in for any groove with an included angle less than 60° at the root of the groove.

In some cases, strength may not govern the design. Standard specifications set maximum and minimum limits on size and spacing of welds.

Allowable Stresses for Bolts. Bolted shear connections are classified as either bearing-type or slip-critical. The latter are required for connections subject to stress reversal, heavy impact, large vibrations, or where joint slippage would be detrimental to the serviceability of the bridge. These connections are discussed in Art. 3.2.6. Bolted bearing-type connections in bridges are restricted to members in compression and secondary members.

Fasteners for bearing-type connections may be ASTM A307 carbon steel bolts or A325 or A490 high-strength bolts. High-strength bolts are required for slip-critical connections and where fasteners are subjected to tension or combined tension and shear.

Bolts for highway bridges are generally $^3/_4$ or $^7/_8$ inch in diameter. Holes for high-strength bolts may be standard, oversize, short-slotted, or long-slotted. Standard holes may be up to $^1/_{16}$ in larger in diameter than the nominal diameters of the bolts. Oversize holes may have a maximum diameter of $^{15}/_{16}$ in for $^3/_4$-in bolts and $^{11}/_{16}$ in for $^7/_8$-in bolts. Minimum diameter of a slotted hole is the same as that of a standard hole. For $^3/_4$- and $^7/_8$-in bolts, short-slotted holes may be up to 1 and $1^1/_8$ in long, respectively, and long-slotted holes, a maximum of $1^7/_8$ and $2^3/_{16}$ in long, respectively.

In the computation of allowable loads for shear or tension on bolts, the cross-sectional area should be based on the nominal diameter of the bolts. For bearing, the area should be taken as the product of the nominal diameter of the bolt and the thickness of the metal on which it bears.

Allowable stresses for bolts specified in Standard Specifications are summarized in Tables 10.17 and 10.18. The percentages of stress increase specified for load combinations in Art. 10.5 also apply to high-strength bolts in slip-critical joints, but the percentage may not exceed 133%.

In addition to satisfying these allowable-stress requirements, connections with high-strength bolts should also meet the requirements for combined tension and shear and for fatigue resistance.

Furthermore, the load P_s, kips, on a slip-critical connection should be less than

$$P_s = F_s A_b N_b N_s \tag{10.13}$$

where F_s = allowable stress, ksi, given in Table 10.17 for a high-strength bolt in a slip-critical joint
A_b = area, in^2, based on the nominal bolt diameter
N_b = number of bolts in the connection
N_s = number of slip planes in the connection

Surfaces in slip-critical joints should be Class A, B, or C, as described in Table 10.17, but coatings providing a slip coefficient less than 0.33 may be used if the mean slip coefficient is determined by test. In that case, F_s for use in Eq. (10.13) should be taken as for Class A coatings but reduced by the ratio of the actual slip coefficient to 0.33.

Tension on high-strength bolts may result in prying action on the connected parts. See Art. 3.5.

Combined shear and tension on a slip-critical joint with high-strength bolts is limited by the interaction relationships in Eqs. (10.14) to (10.16). The shear f_v, ksi (slip load per unit area of bolt), for A325 bolts may not exceed

$$f_v = F_s\left(1 - \frac{1.88 f_t}{F_u}\right) \tag{10.14}$$

TABLE 10.17 Allowable Shear and Tension Stresses, ksi, on Bolts in Highway Bridges—ASD

		Allowable shear F_v				
		Slip-critical connections				
				Long-slotted holes		
ASTM designation	Allowable tension F_t	Standard-size holes	Oversize and short-slotted holes	Transverse load	Parallel load	Bearing-type joints
A307	18					11
A325	38[a]					19
		15*	13*	11*	9*	
		23†	19†	16†	14†	
		15‡	13‡	11‡	9‡	
A490	47[a]					25
		19*	16*	13*	11*	
		29†	24†	20†	17†	
		19‡	16‡	13‡	11‡	

***Class A:** When contact surfaces have a slip coefficient of 0.33, such as clean mill scale and blast-cleaned surfaces, with Class A coating.

†**Class B:** When contact surfaces have a slip coefficient of 0.50, such as blast-cleaned surfaces and such surfaces with Class B coating.

‡**Class C:** When contact surfaces have a slip coefficient of 0.40, such as hot-dipped galvanized and roughened surfaces.

Class A and B coatings include those with a mean slip coefficient of at least 0.33 or 0.50, respectively. See Appendix A, "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections of the Engineering Foundation.

where f_t = computed tensile stress in the bolt due to applied loads including any stress due to prying action, ksi

F_s = nominal slip resistance per unit of bolt area from Table 10.17

F_u = 120 ksi for A325 bolts up to 1 inch in diameter

= 105 ksi for A325 bolts over 1 inch in diameter

= 150 ksi for A490 bolts

Where high-strength bolts are subject to both shear and tension, the tensile stress may not exceed the value obtained from the following equations:

For $f_v/F_v \le 0.33$,
$$F_t' = F_t \tag{10.15}$$

For $f_v/F_v > 0.33$,
$$F_t' = F_t\sqrt{1-\left(\frac{f_v}{F_v}\right)^2} \tag{10.16}$$

TABLE 10.18 Allowable Bearing Stresses, ksi, on Bolted Joints in Highway Bridges—ASD

Conditions for connection material	A307 bolts	A325 bolts	A490 bolts
Threads permitted in shear planes	20		
Single bolt in line of force in a standard or short-slotted hole		$0.9F_u$*†	$0.9F_u$*†
Two or more bolts in line of force in standard or short-slotted holes		$1.1F_u$*†	$1.1F_u$*
Bolts in long-slotted holes		$0.9F_u$*†	$0.9F_u$*

*F_u = specified minimum tensile strength of connected parts. Connections with bolts in oversize holes or in slotted holes with the load applied less than about 80° or more than about 100° to the axis of the slot should be designed for a slip resistance less than that computed from Eq. (10.13).

†Not applicable when the distance, parallel to the load, from the center of a bolt to the edge of the connected part is less than $1\frac{1}{2}d$, where d is the nominal diameter of the bolt, or the distance to an adjacent bolt is less than $3d$.

TABLE 10.19 Allowable Tensile Fatigue Stresses for Bolts in Highway Bridges*—ASD

Number of cycles	A325 bolts	A490 bolts
20,000 or less	39.5	48.5
20,000 to 500,000	35.5	44.0
More than 500,000	27.5	34.0

*As specified in "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

where f_v = computed bolt shear stress in shear, ksi
F_s = allowable shear stress on bolt from Table 10.17, ksi
F_t = allowable tensile stress or bolt from Table 10.17, ksi
F_t' = reduced allowable tensile stress on bolt due to the applied shear stress, ksi

Combined shear and tension in a bearing-type connection is limited by the interaction equation:

$$f_v^2 + 0.36 f_t^2 = F_v^2 \tag{10.17}$$

where f_v = computed bolt shear stress, ksi, and F_v = allowable bolt shear, ksi (Table 10.17). Equation (10.17) is based on the assumption that bolt threads are excluded from the shear plane.

Fatigue may control design of a bolted connection. To limit fatigue, service-load tensile stress on the area of a bolt based on the nominal diameter, including the effects of prying action, may not exceed the stress in Table 10.19. Also, the prying force may not exceed 60% of the externally applied load.

10.8 FRACTURE CONTROL

Fracture-critical members are treated in the LRFD Specifications and in the AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members. A fracture-critical member (FCM) or member component is a tension member or component whose failure is expected to result in collapse of the bridge or the inability of the bridge to perform its function. Although the definition is limited to tension members, failure of other members or components could result in catastrophic failure in some cases. This concept applies to members of any material.

The Standard Specifications contain provisions for structural integrity. These recommend that, for new bridges, designers specify designs and details that employ continuity and redundancy to provide one or more alternate load paths. Also, external systems should be provided to minimize effects of probable severe loads.

The LRFD Specifications, in particular, require that multiple-load-path structures be used unless "there are compelling reasons to the contrary." Also, main tension members and components whose failure may cause collapse of the bridge must be designated as FCM and the structural system must be designated nonredundant. Furthermore, the LRFD Specifications includes fracture control in the fatigue-and-fracture limit state.

Design of structures can be modified to eliminate the need for special measures to prevent catastrophe from a fracture, and when this is cost-effective, it should be done. Where use of an FCM is unavoidable, for example, the tie girder of a tied arch, as much redundancy as possible should be provided via continuity, internal redundancy through use of multiple plates, and similar measures.

Steels used in FCM must have supplemental impact properties as listed in Table 1.2. FCM should be so designated on the plans with the appropriate temperature zone (Table 1.2) based on the anticipated minimum service temperature. Fabrication requirements for FCM are outlined in ANSI/AASHTO/AWS DI.5.

High-performance steels (HPS), as discussed in Art. 1.1.5, provide an opportunity to significantly increase reliability of steel bridges. With impact properties for this steel usually exceeding 100 ft·lb at −10°F, it easily meets the most severe requirements for fracture critical material. For example, the

HPS70W material requirement for welded, 4-in-thick plates, in FCMs in a temperature Zone 3 application, is 35 ft·lb at –30°F (see Table 1.2).

10.9 REPETITIVE LOADINGS

Most structural damage to steel bridges is the result of repetitive loading from trucks or wind. Often, the damage is caused by secondary effects, for example, when live loads are distributed transversely through cross frames and induce large out-of-plane distortions that were not taken into account in design of the structure. Such strains may initiate small fatigue cracks. Under repetitive loads, the cracks grow. Unless the cracks are discovered early and remedial action taken, they may create instability under a combination of stress, loading rate, and temperature, and brittle fracture could occur. Proper detailing of steel bridges can prevent such fatigue crack initiation.

To reduce the probability of fracture, the structural steels included in the AASHTO specifications for M270 steels, and ASTM A709 steels when "supplemental requirements" are ordered,* are required to have minimum impact properties (Art. 1.1.5). The higher the impact resistance of the steel, the larger a crack has to be before it is susceptible to unstable growth. With the minimum impact properties required for bridge steels, the crack should be large enough to allow discovery during the biannual bridge inspection before fracture occurs. The M270 specification requires average energy in a Charpy V-notch test of 15 ft·lb for Grade 36 steels and ranging up to 35 ft·lb for Grade 100 steels, at specified test temperatures. More conservative values are specified for FCM members (Art. 10.8). Toughness values depend on the lowest ambient service temperature (LAST) to which the structure may be subjected. Test temperatures are 70°F higher than the LAST to take into account the difference between the loading rate as applied by highway trucks and the Charpy V-notch impact tests.

Allowable Fatigue Stresses for ASD and LFD. Members, connections, welds, and fasteners should be designed so that maximum stresses do not exceed the basic allowable stresses (Art. 10.7) and the range in stress due to loads does not exceed the allowable fatigue stress range. Table 10.20 lists allowable fatigue stress ranges in accordance with the number of cycles to which a member or component will be subjected and several stress categories for structural details. The allowable stresses apply to load combinations that include live loads and wind. For dead plus wind loads, use the stress range for 100,000 cycles. Table 10.21 lists the number of cycles to be used for design.

Examples of members and details included in stress categories A through F are as follows. See the AASHTO Standard Specifications for complete descriptions and details.

Stress Category A: Thermal-cut edges of members with ANSI smoothness of 1000 or less.

Stress Category B: Base metal and weld metal in members without attachments connected by continuous full-penetration groove welds (backing bars removed) or by continuous fillet welds, parallel to direction of stress; or at full-penetration groove weld splices that meet AASHTO detail criteria in other than Grade 100 steel. Base metal at gross section of high-strength bolted slip-resistant connections, base metal at net section of high-strength bolted bearing-type connections.

Stress Category B′: Base metal and weld metal in members without attachments connected by continuous full-penetration groove welds (backing bars not removed) or by continuous partial-penetration welds, parallel to direction of stress; or at full-penetration groove weld splices that meet AASHTO detail criteria in Grade 100 steel.

Stress Category C: Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges; at full-penetration groove weld splices that meet AASHTO detail criteria, reinforcement not removed; certain groove-welded attachments and fillet-welded connections, depending on details.

Stress Category D: Groove-welded attachments and fillet-welded connections, with stress concentrations more severe than Category C.

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*ASTM A709 steels thus specified are equivalent to AASHTO material specification M270 steels and grade designations are similar.

TABLE 10.20 Allowable Stress Range, ksi, for Repeated Loads on Highway Bridges[a]—ASD and LFD

	Number of loading cycles			
Stress category	100,000[b]	500,000[c]	2,000,000[d]	More than 2,000,000[d]
	(a) For redundant load-path structures			
A	63 (49)[e]	37 (29)[e]	24 (18)[e]	24 (16)[e]
B	49	29	18	16
B′	39	23	14.5	12
C	35.5	21	13	10
				12[g]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8
	(b) For nonredundant load-path structures			
A	50 (39)[e]	29 (23)[e]	24 (16)[e]	24 (16)[e]
B	39	23	16	16
B′	31	18	11	11
C	28	16	10	9
			12[f]	11[f]
D	22	13	8	5
E[g]	17	10	6	2.3
E′	12	7	4	1.3
F	12	9	7	6

[a] Based on data in the "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

[b] Equivalent to about 10 applications every day for 25 years.

[c] Equivalent to about 50 applications every day for 25 years.

[d] Equivalent to about 200 applications every day for 25 years.

[e] Values in parentheses apply to unpainted weathering steel A709, all grades, when used in conformance with Federal Highway Administration "Technical Advisory on Uncoated Weathering Steel in Structure," Oct. 3, 1989.

[f] For welds of transverse stiffeners to webs or flanges of girders.

[g] AASHTO prohibits use of partial-length welded cover plates on flanges more than 0.8 in thick in nonredundant load-path structures.

Stress Category E: Groove-welded attachments and fillet-welded connections, with stress concentrations more severe than Category D.

Stress Category E′: Groove-welded attachments and fillet-welded connections, with stress concentrations more severe than Category D.

Stress Category F: Shear stress on throat of fillet welds.

TABLE 10.21 Design Stress Cycles for Main Load-Carrying Members for ASD

Type of road	Case	ADTT[a]	Truck loading	Lane loading[b]
Freeways, expressways, major highways, and streets	I	2,500 or more	2,000,000[c]	500,000
Freeways, expressways, major highways, and streets	II	Less than 2,500	500,000	100,000
Other highways and streets not included in Case I or II	III		100,000	100,000

[a] Average daily truck traffic (one direction).

[b] Longitudinal members should also be checked for truck loading.

[c] Members must also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge.

Stress range is the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

Table 10.20*a* is applicable to redundant load-path structures. These provide multiple loads paths so that a single fracture in a member or component cannot cause the bridge to collapse. The Standard Specifications list as examples a simply supported, single-span bridge with several longitudinal beams and a multielement eye bar in a truss. Table 10.20*b* is applicable to nonredundant load-path structures. The Standard Specifications give as examples flange and web plates in bridges with only one or two longitudinal girders, one-element main members in trusses, hanger plates, and caps of single- or two-column bents.

Improved ASD and LFD Provisions for Fatigue Design. AASHTO has published Guide Specifications for Fatigue Design of Steel Bridges. These indicate that the fatigue provisions in the "Standard Specifications for Highway Bridges" do not accurately reflect the actual fatigue conditions in such bridges; instead, they combine an artificially high stress range with an artificially low number of cycles to get a reasonable result. The actual effective stress ranges rarely exceed 5 ksi, whereas the number of truck passages in the design life of a bridge can exceed many million.

For this reason, these Guide Specifications give alternative fatigue-design procedures to those in the standard specifications. They are based on a more realistic loading, equal to 75% of a single HS20 (or HS15) truck with a fixed rear-axle spacing of 30 ft. The procedures accurately reflect the actual conditions in bridges subjected to traffic loadings and provide the following additional advantages: (1) They permit more flexibility in accounting for differing traffic conditions at various sites. (2) They permit design for any desired design life. (3) They provide reasonable and consistent levels of safety over a broad range of design conditions. (4) They are based on extensive research and can be conveniently modified in the future if needed to reflect new research results. (5) They are consistent with fatigue-evaluation procedures for existing bridges.

The Guide Specifications use the same detail categories and corresponding fatigue strength data as the standard specifications. They also use methods of calculating stress ranges that are similar to those used with the Standard Specifications.

Thus, it is important that designers possess both the Standard Specifications and the Guide Specifications to design fatigue-resistant details properly. However, there is a prevailing misconception in the interpretation of the term **fatigue life.** For example, the Guide Specifications state, "The safe fatigue life of each detail shall exceed the desired design life of the bridge." The implication is that the initiation of a fatigue crack is the end of the service life of the structure. In fact, the initiation of a fatigue crack does *not* mean the end of the life of an existing bridge, or even of the particular member, as documented by the many bridges that have experienced fatigue cracking and even full-depth fracture of main load-carrying members. These cracks and fractures have been successfully repaired by welding, drilling a hole at the crack tip, or placing bolted cover plates over a fracture. These bridges continue to function without reduction in load-carrying capacity or remaining service life.

Fatigue Provisions for LRFD. The AASHTO load and resistance factor design specifications can be best understood by considering a schematic log-log fatigue-resistance curve in which stress range is plotted against number of cycles, Fig. 10.5. The curve represents the locus of points of equal fatigue damage. Along the sloping portion, for a given stress range, a corresponding finite life is anticipated. The constant-amplitude fatigue threshold represented by the dashed horizontal line defines the infinite-life fatigue resistance. If all of the stress ranges experienced by a detail are less than the stress range defined by the fatigue threshold, it is anticipated that the detail will not crack.

The LRFD Specifications attempt to combine the best attributes of the Guide Specification, including the special fatigue loading described previously, and those of the Standard Specifications, including the detail category concept. The LRFD Specifications define the nominal fatigue resistance for each fatigue category as

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{\mathrm{TH}} \qquad (10.18)$$

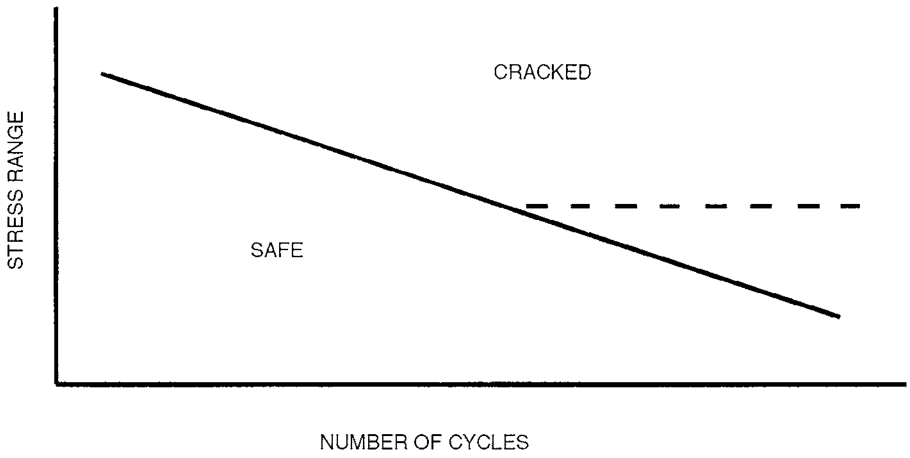


FIGURE 10.5 Schematic fatigue-resistance curve.

TABLE 10.22 Detail Category* Constant *A*

Detail category	Constant *A*
A	250.0×10^{-8}
B	120.0×10^{-8}
B′	61.0×10^{-8}
C	44.0×10^{-8}
C′	44.0×10^{-8}
D	22.0×10^{-8}
E	11.0×10^{-8}
E′	3.9×10^{-8}
M164 (A325) bolts in axial tension	17.1×10^{-8}
M253 (A490) bolts in axial tension	31.5×10^{-8}

*See LRFD Specifications for complete details.

TABLE 10.23 Cycles per Truck Passage, *n*

(a) Longitudinal members

	Span length	
Member type	>40.0 ft	≤40.0 ft
Simple-span girders	1.0	2.0
Continuous girders		
(1) Near interior support	1.5	2.0
(2) Elsewhere	1.0	2.0
Cantilever girders	5.0	
Trusses	1.0	

(b) Transverse members

Spacing	
>20.0 ft	≤20.0 ft
1.0	2.0

TABLE 10.24 Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$

Detail category	Threshold, ksi
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
M164 (A325) bolts in axial tension	31.0
M253 (A490) bolts in axial tension	38.0

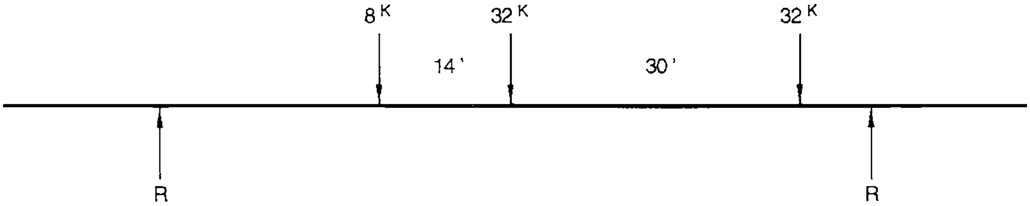


FIGURE 10.6 Design truck for calculation of fatigue stresses. Impact is taken as 15% of live load.

where $N = (365)(75)n(\mathrm{ADTT})_{\mathrm{SL}}$
A = fatigue detail category constant, Table 10.22
n = number of stress range cycles per truck passage, Table 10.23
$(\mathrm{ADTT})_{\mathrm{SL}}$ = single-lane ADTT (average daily truck traffic)
$(\Delta F)_{\mathrm{TH}}$ = constant-amplitude fatigue threshold, ksi, Table 10.24

However, the nominal fatigue resistance range for base metal at details connected with transversely loaded fillet welds, where a discontinuous plate is loaded, is taken as the lesser of $(\Delta F)_n^c$ and

$$(\Delta F)_n = (\Delta F)_n^c \left(\frac{0.06 + 0.79 H/t_p}{1.1 t_p^{1/6}} \right) \tag{10.19}$$

where $(\Delta F)_n^c$ = the nominal fatigue resistance for detail Category C, ksi
H = effective throat of fillet weld, in
t_p = thickness of loaded plate, in

The term $(A/N)^{1/3}$ in Eq. (10.18) represents the sloping line in Fig. 10.5, and $(\Delta F)_{\mathrm{TH}}$ the horizontal line. The multiplier of $^1/_2$ represents the ratio of the factored fatigue load to the maximum load. In other words, if the stress range due to the factored fatigue truck is less than one-half of the constant-amplitude fatigue threshold, the detail should experience infinite life. The load factor for fatigue is 0.75, Table 10.2. The truck loading for fatigue is shown in Fig. 10.6.

The fatigue resistance defined in LRFD is similar to that in earlier specifications, although the format is different. Complete LRFD design examples, including fatigue designs of typical girder details, have demonstrated that design in accord with the LRFD Specifications is basically equivalent to design in accordance with the provisions for redundant structures in the Standard Specifications. In developing the LRFD provisions, it was determined that because of the greater fracture toughness specified for nonredundant structures, a reduction in allowable stress range for such structures was unnecessary.

10.10 DETAILING FOR EARTHQUAKES

Bridges must be designed so that catastrophic collapse cannot occur from seismic forces. Damage to a structure, even to the extent that it becomes unusable, may be acceptable, but collapse is not!

The Standard Specifications and the LRFD Specifications contain standards for seismic design that are comprehensive in nature and embody several important concepts. They are based on observed performance of bridges during past earthquakes and on research. LRFD specifications include seismic design as part of the Extreme-Event Limit State.

Although the specifications establish design seismic-force guidelines, of equal importance is the emphasis placed on proper detailing of bridge components. For instance, one of the leading causes of collapse when bridges are subjected to earthquakes is the displacement that occurs at bridge seats. If beam seats are not properly sized, the superstructure will fall off the substructure during an earthquake. Minimum support lengths to be provided at beam ends, based on seismic performance category, is a part of the specifications. Thus, to ensure earthquake-resistant structures, both displacements and loads must be taken into account in bridge design.

Retrofitting existing structures to provide earthquake resistance is also an important consideration for critical bridges. Guidance is provided in "Seismic Retrofitting Guidelines for Highway Bridges," Federal Highway Administration (FHWA) Report No. RD-83/007; "Seismic Design and Retrofit Manual for Highway Bridges," FHWA Report No. IP-87-6; and M. J. N. Priestley, F. Seible, and G. M. Calvi (1996), *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, New York.

10.11 DETAILING FOR BUCKLING

Prevention of buckling is important in bridge design because of the potential for collapse. Three forms of buckling must be considered in bridge design.

10.11.1 Types of Buckling

The first, and most serious, is primary buckling of an axially loaded compression member. Such column buckling may include Euler-type elastic buckling and inelastic buckling. This is a rare occurrence with highway bridges, attesting to the adequacy of the current design provisions.

A second form of buckling is local plate buckling. This form of buckling usually manifests itself in the form of excessive distortion of plate elements. This may not be acceptable from a visual perspective, even though the member capacity may be sufficient. When very thin plates are specified, in the desire to achieve minimum weight and supposedly minimum cost, distortions due to welding may induce initial out-of-plane deformations that then develop into local buckling when the member is loaded. Proper welding techniques and use of transverse or longitudinal stiffeners, while maintaining recommended width-thickness limitations on plates and stiffeners, minimize the probability of local buckling.

The third, and perhaps the most likely form of buckling to occur in steel bridges, is lateral buckling. It develops when compression causes a flexural member to become unstable. Such buckling can be prevented by use of lateral bracing, members capable of preventing deformation normal to the direction of the compressive stress at the point of attachment.

Usually, lateral buckling is construction-related. For example, it can occur when a member is fabricated with very narrow compression flanges without adequate provision for transportation and erection stresses. It also can occur when adequate bracing is not provided during deck-placing sequences. Consequently, designers should ensure that compression flanges are proportioned to provide stability during all phases of the service life of bridges, including construction stages, when temporary lateral bracing may be required.

10.11.2 Maximum Slenderness Ratios of Bridge Members

Ratios of effective length to least radius of gyration of columns should not exceed the values listed in Table 10.25.

The length of top chords of half-through trusses should be taken as the distance between laterally supported panel points. The length of other truss members should be taken as the distance between panel-point intersections, or centers of braced points, or centers of end connections.

10.11.3 Plate-Buckling Criteria for Compression Elements

The Standard Specifications set a maximum width–thickness ratio b/t or D/t for compression members as given in Table 10.26.

TABLE 10.25 Maximum Slenderness Ratios for Highway Bridge Members for ASD, LFD, and LRFD

Member	Highway
Main compression members	120
Wind and sway bracing in compression	140
Tension members	
Main	200
Main subject to stress reversal	140
Bracing	240

TABLE 10.26 Maximum Width–Thickness Ratios for Compression Elements of Highway Bridge Members for ASD

Components	Limiting stress, ksi[a]	b/t for calculated stress less than the limiting stress[b]	b/t for calculated stress equal to the limiting stress[a]
(a) Plates supported on only one side			
Compression members[c]	$0.44F_y$	$51.4/\sqrt{f_a} \leq 12$	$75/\sqrt{F_y}$
Welded-girder flange[d]	$0.55F_y$	$103/\sqrt{f_b} \leq 24$	$140/\sqrt{F_y}$
Composite girder[d]		$122/\sqrt{f_{\text{dl}}}$	
Bolted-girder flange[e]	$0.55F_y$	$51.4/\sqrt{f_b} \leq 12$	$70/\sqrt{F_y}$
Composite girder[e]		$61/\sqrt{f_{\text{dl}}}$	
(b) Plates supported on two sides			
Girder web without stiffeners[f]	F_v	$270/\sqrt{f_v} \leq 150$	$470/\sqrt{F_y}$
Girder web with transverse stiffeners[f]	F_b	$730/\sqrt{f_b} \leq 170$	$990/\sqrt{F_y}$
Girder web with longitudinal stiffeners[f,h]	F_b	$128/\sqrt{k}/\sqrt{f_b} \leq 340$	
Girder web with transverse stiffeners and one longitudinal stiffener[f]	F_b		$1980/\sqrt{F_y}$
Box-shapes—main plates or web[g]	$0.44F_y$	$126/\sqrt{f_a} \leq 45$	$190/\sqrt{F_y}$
Box or H shapes—solid cover plates or webs between main elements[g]	$0.44F_y$	$158/\sqrt{f_a} \leq 50$	$240/\sqrt{F_y}$
Box shapes—perforated cover plates[g]	$0.44F_y$	$190/\sqrt{f_a} \leq 55$	$285/\sqrt{F_y}$

[a] F_y = specified minimum yield strength of the steel, ksi; F_b = allowable bending stress, ksi; F_v = allowable shear stress, ksi.

[b] f_a = computed compressive stress, ksi; f_b = computed compressive bending stress, ksi; f_v = computed shear stress, ksi; f_{dl} = top flange compressive stress due to noncomposite dead load.

[c] For outstanding plates, outstanding legs of angles, and perforated plates at the perforations. Width b is the distance from the edge of plate or edge of perforation to the point of support, and t is the thickness.

[d] b is the width of the compression flange and t is the thickness.

[e] b is the width of flange angles in compression, except those reinforced by plates, and t is the thickness.

[f] b represents the depth of the web D, clear unsupported distance between flanges.

[g] When used as compression members, b is the distance between points of support for the plate and between roots of flanges for webs of rolled elements; t is the thickness.

[h] Plate buckling coefficient k is defined as follows: For $d_s/D_c \geq 0.4$, $k = 5.17\ (D/d_s)^2$; for $d_s/D_c < 0.4$, $k = 11.64\ [D/(D_c - d_s)]^2$ (d_s is the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component, and D_c is the depth of the web in compression).

10.11.4 Stiffening of Girder Webs (ASD)

Bending of girders tends to buckle thin webs. This buckling may be prevented by making the web sufficiently thick (Table 10.26) or by stiffening the web with plates attached normal to the web. The stiffeners may be set longitudinally or transversely (vertically), or both ways. (See Art. 10.16.)

Bearing stiffeners are required for plate girders at concentrated loads, including all points of support. Rolled beams should have web stiffeners at bearings when the unit shear stress in the web exceeds 75% of the allowable shear. Bearing stiffeners should be placed in pairs, one stiffener on each side of the web. Plate stiffeners or the outstanding legs of angle stiffeners should extend as close as practicable to the outer edges of the flanges. The stiffeners should be ground to fit against the flange through which the concentrated load, or reaction, is transmitted, or they should be attached to that flange with full-penetration groove welds. They should be fillet-welded to both flanges if they also serve as diaphragm connections. They should be designed for bearing over the area actually in contact with the flange. No allowance should be made for the portions of the stiffeners fitted to fillets of flange angles or flange-web welds. A typical practice is to clip plate stiffeners at 45° at upper and lower ends to clear such fillets or welds. Connections of bearing stiffeners to the web should be designed to transmit the concentrated load, or reaction, to the web.

Bearing stiffeners should be designed as columns. For ordinary welded girders, the column section consists of the plate stiffeners and a strip of web. (At interior supports of continuous hybrid girders, however, when the ratio of web yield strength to tension-flange yield strength is less than 0.7, no part of the web should be considered effective.) For stiffeners consisting of two plates, the effective portion of the web is a centrally located strip 18*t* wide, where *t* is the web thickness, in (Fig. 10.7*a*). For stiffeners consisting of four or more plates, the effective portion of the web is a centrally located strip included between the stiffeners and extending beyond them a total distance of 18*t* (Fig. 10.7*b*). The radius of gyration should be computed about the axis through the center of the web. The width–thickness ratio of a stiffener plate or the outstanding leg of a stiffener angle should not exceed

$$\frac{b}{t} = \frac{69}{\sqrt{F_y}} \tag{10.20}$$

where F_y = yield strength, ksi, for stiffener steel.

For highway bridges, no stiffeners, other than bearing stiffeners, are required, in general, if the depth–thickness ratio of the web does not exceed the value for girder webs without stiffeners in Table 10.26. However, stiffeners may be required for attachment of cross frames.

Transverse stiffeners should be used for highway girders where D/t exceeds the aforementioned values, where D is the depth of the web, the clear unsupported distance between flanges. When transverse stiffeners are used, the web depth–thickness ratio should not exceed the values given in Table 10.26 for webs without longitudinal stiffeners and with one longitudinal stiffener. Intermediate stiffeners may be A36 steel, whereas web and flanges may be a higher grade.

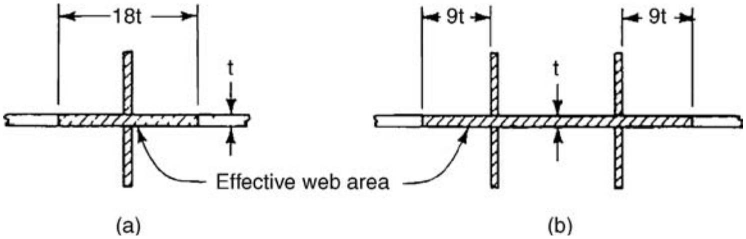


FIGURE 10.7 Effective column areas for design of stiffeners: (*a*) for one pair of stiffeners; (*b*) for two pairs.

Where required, transverse stiffeners may be attached to the highway-girder web singly or in pairs. Where stiffeners are placed on opposite sides of the web, they should be fitted tightly against the compression flange. Where a stiffener is placed on only one side of the web, it must be in bearing against, but need not be attached to, the compression flange. Intermediate stiffeners need not bear against the tension flange. However, the distance between the end of the stiffener weld and the near edge of the web-to-flange fillet welds must not be less than $4t$ or more than $6t$.

Transverse stiffeners may be used, where not otherwise required, to serve as connection plates for diaphragms or cross frames. In such cases, the stiffeners must be rigidly connected to both the tension and compression flanges to prevent web fatigue cracks due to out-of-plane movements. The stiffener may be welded to both flanges, or a special bolted detail may be used to connect to the tension flange. The appropriate fatigue category must be used for the tension flange to reflect the detail used (see Art. 10.9).

Transverse stiffeners should be proportioned so that

$$I \geq d_o t^3 J \tag{10.21}$$

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \geq 0.5 \tag{10.22}$$

where I = moment of inertia, in^4, of transverse intermediate stiffener
J = ratio of rigidity of stiffener to web
d_o = actual distance, in, between transverse stiffeners
t = web thickness, in

For stiffener pairs, I should be taken about the center of the web. For single stiffeners, I should be taken about the web face in contact with the stiffeners. In either case, transverse stiffeners should project a distance, in, from the web of at least $b_f/4$, where b_f is the flange width, in, and at least $D'/30 + 2$, where D' is the girder depth, in. Thickness should be at least $^1/_{16}$ of this width.

Intermediate transverse stiffeners should have a gross cross-sectional area A, in^2, of at least

$$A = Y\left[0.15BDt_w(1-C)\left(\frac{f_v}{F_v}\right) - 18t_w^2\right] \tag{10.23}$$

where Y = ratio of the yield strength of the web steel to the yield strength of the stiffener steel
t_w = web thickness, in
f_v = computed shear stress, ksi, in the web
F_v = allowable shear stress, ksi, in the web
B = 1.0 for pairs of stiffeners
= 1.8 for single angles
= 2.4 for single plates
C = ratio of buckling shear stress to yield shear stress

$$= 1.0 \qquad \text{when } D/t_w < 190\sqrt{k/F_y} \tag{10.24a}$$

$$= \frac{6000}{D/t_w}\sqrt{\frac{k}{F_y}} \qquad \text{when } 190\sqrt{\frac{k}{F_y}} \leq \frac{D}{t_w} \leq 237\sqrt{\frac{k}{F_y}} \tag{10.24b}$$

$$= \frac{45{,}500k}{(D/t_w)^2 F_y} \qquad \text{when } D/t_w > 237\sqrt{\frac{k}{F_y}} \tag{10.24c}$$

$$k = 5\left[1 + \left(\frac{D}{d_o}\right)^2\right] \tag{10.24d}$$

When A computed from Eq. (10.23) is very small or negative, transverse stiffeners need only satisfy Eq. (10.21) and the width–thickness limitations given previously.

Intermediate transverse stiffeners, with or without longitudinal stiffeners, should be spaced close enough that the computed shear stress f_v' does not exceed

$$f_v' = F_v\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \tag{10.25a}$$

where C is defined by Eqs. (10.24a) to (10.24d). Spacing is limited to a maximum of $3D$, or for panels without longitudinal stiffeners, to ensure efficient fabrication, handling, and erection of the girders, to $67{,}600D(t_w/D)^2$. At a simple support, the first intermediate stiffener should be close enough to the support that the shear stress in the end panel does not exceed

$$f_v' = \frac{CF_y}{3} \le \frac{F_y}{3} \tag{10.25b}$$

but not farther than $1.5D$.

If the shear stress is larger than $0.6F_v$ in a girder panel subjected to combined shear and bending moment, the bending stress F_s with live loads positioned for maximum moment at the section should not exceed

$$F_s = \left(0.754 - \frac{0.34f_v}{f_v'}\right)F_y \tag{10.26}$$

Fabricators should be given leeway to vary stiffener spacing and web thickness to optimize costs. Girder webs often compose 40% to 50% of the girder weight but only about 10% of girder bending strength. Hence, least girder weight may be achieved with minimum web thickness and many stiffeners but not necessarily at the lowest cost. Thus, the contract drawings should allow fabricators the option of choosing stiffener spacing. The contract drawings should also note the thickness requirements for a web with a minimum number of stiffeners. (A stiffener is required at every cross frame.) This allows fabricators to choose the most economical fabrication process. If desired, flange thicknesses can be reduced slightly if the thicker-web option is selected. In some cases, the most economical results may be obtained with a stiffened web having a thickness $^1/_{16}$ in less than that of an unstiffened web (Art. 10.16).

Preferably, the drawings should show the details for a range from unstiffened to fully stiffened webs. During the design stage, this is a relatively simple task. In contrast, after a construction contract has been awarded, the contractor cannot be expected to submit alternative girder designs, with or without value engineering, because it is often more trouble than the effort is worth. Contractors generally bid on what is shown on the plans, risking the possibility of losing the contract to a concrete alternative or to another contractor. On the other hand, by providing contract documents with sufficient flexibility, owners can profit from the fact that different fabricators have different methods of cost-effective fabrication that can be utilized on behalf of owners.

Longitudinal stiffeners should be used where D/t exceeds the values given in Table 10.26. They are required, even if the girder has transverse stiffeners, if the values of D/t for a web with transverse stiffeners is exceeded.

The optimum distance d_s of a plate longitudinal stiffener from the inner surface of the compression flange is $D/5$ for a symmetrical girder. The optimum distance for an unsymmetrical composite girder in positive-moment regions may be determined from

$$\frac{d_s}{D_{cs}} = \frac{1}{1+1.5\sqrt{f_{DL+LL}/f_{DL}}} \tag{10.27}$$

where D_{cs} is the depth of the web in compression of the noncomposite steel beam or girder, f_{DL} is the noncomposite dead-load stress in the compression flange, and f_{DL+LL} is the total noncomposite and composite dead-load plus the composite live-load stress in the compression flange at the most highly stressed section of the web. The optimum distance, d_s, of the stiffener in negative-moment regions of composite sections is $2D_c/5$, where D_c is the depth of the web in compression of the composite section at the most highly stressed section of the web. The stiffener should be proportioned so that

$$I \geq Dt^3\left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13\right] \tag{10.28a}$$

where I = moment of inertia, in^4, of longitudinal stiffener about edge in contact with web and d_o = actual distance, in, between transverse stiffeners. Width–thickness ratio of the longitudinal stiffener should not exceed

$$\frac{b_s}{t_s} = \frac{82.22}{\sqrt{F_y}} \tag{10.28b}$$

Bending stress in the stiffener should not exceed the allowable for the stiffener steel. The stiffener may be placed on only one side of the web. Not required to be continuous, it may be interrupted at transverse stiffeners.

Spacing of transverse stiffeners used with longitudinal stiffeners should satisfy Eq. (10.25*a*) but should not exceed 1.5 times the subpanel depth in the panel adjacent to a simple support as well as in interior panels. The limit on stiffener spacing given previously to ensure efficient handling of girders does not apply when longitudinal stiffeners are used. Also, in computation of required moment of inertia and area of transverse stiffeners from Eqs. (10.21) to (10.23), the maximum subpanel depth should be substituted for D.

Longitudinal stiffeners become economical for girder spans over 300 ft. Often, however, they are placed on fascia girders for esthetic reasons and may be used on portions of girders subject to tensile stresses or stress reversals. If this happens, designers should ensure that butt splices used by the fabricators for the longitudinal stiffeners are made with complete-penetration groove welds of top quality. (Plates of the sizes used for stiffeners are called **bar stock** and are available in limited lengths, which almost always make groove-welded splices necessary.) Many adverse in-service conditions have resulted from use of partial-penetration groove welds instead of complete-penetration.

10.11.5 Lateral Bracing

In **highway girder bridges,** AASHTO requires that the need for lateral bracing be investigated. The stresses induced in the flanges by the specified wind pressure must be within specified limits. In many cases lateral bracing will not be required, and a better structure can be achieved by eliminating fatigue-prone details. Flanges attached to concrete decks or other decks of comparable rigidity will not require lateral bracing. When lateral bracing is required, it should be placed in the exterior bays between diaphragms or cross frame, in or near the plane of the flange being braced.

Bracing consists of members capable of preventing rotation or lateral deformation of other members. This function may be served in some cases by main members, such as floorbeams where they frame into girders; in other cases by secondary members especially incorporated in the steel framing for the purpose; and in still other cases by other construction, such as a concrete deck. Preferably, bracing should transmit forces received to foundations or bearings, or to other members that will do so.

AASHTO specifications state that the smallest angle used in bracing should be $3 \times 2^1/_2$ in. Size of bracing often is governed by the maximum permissible slenderness ratio (Table 10.25) or width–thickness ratio of components (Table 10.26). Some designers prefer to design bracing for a percentage, often 2%, of the axial force in the member.

Through-truss, deck-truss, and spandrel-braced-arch highway bridges should have top and bottom lateral bracing (Fig. 10.8). For compression chords, lateral bracing preferably should be as deep as the chords and connected to top and bottom flanges.

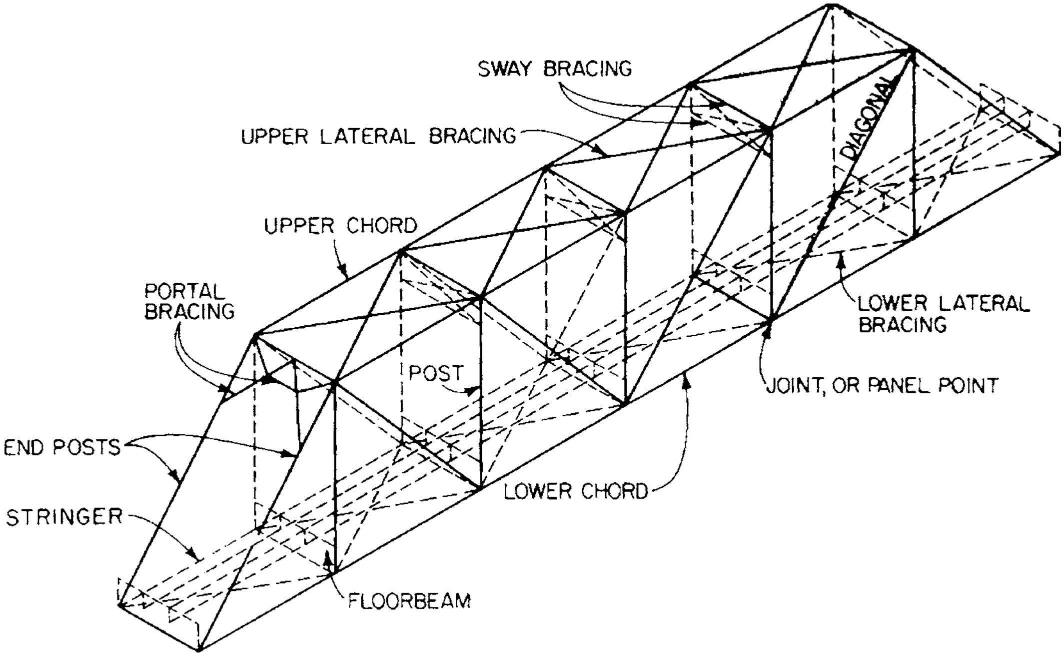


FIGURE 10.8 Components of a through-truss bridge.

If a double system of bracing is used (top and bottom laterals), both systems may be considered effective simultaneously if the members meet the requirements as both tension and compression members. The members should be connected at their intersections.

AASHTO ASD and LFD specifications require that a horizontal wind force of 50 lb/ft^2 on the area of the superstructure exposed in elevation be included in determining the need for, or in designing, bracing. Half of the force should be applied in the plane of each flange. The maximum induced stresses, F, ksi, in the bottom flange from the lateral forces can be computed from

$$F = RF_{cb} \tag{10.29a}$$

where $R = (0.2272L - 11)/S_d^{3/2}$ without bottom lateral bracing
$= (0.059L - 0.640)/\sqrt{S_d}$ with bottom lateral bracing
L = span, ft
S_d = diaphragm or cross-frame spacing, ft
$F_{cb} = 72M_{cb}/t_f b_f^2$
$M_{cb} = 0.08WS_d^2$
W = wind loading, kips/ft, along exterior flange
t_f = flange thickness, in
b_f = flange width, in

10.11.6 Cross Frames and Diaphragms for Deck Spans

In highway bridges, Standard rolled beams and plate girders should be braced with cross frames or diaphragms at each end. Standard Specifications for ASD and LFD require that intermediate cross frames or diaphragms be spaced at intervals of 25 ft or less. They should be placed in all bays. Cross frames should be as deep as practicable. Diaphragms should be at least one-third and preferably one-half the girder depth. Cross frames and diaphragms should be designed for wind forces as described above for lateral bracing. The maximum horizontal force in the cross frames or diaphragms may be computed from

$$F_c = 1.14WS_d \tag{10.29b}$$

End cross frames or diaphragms should be designed to transmit all lateral forces to the bearings. Cross frames between horizontally curved girders should be designed as main members capable of transferring lateral forces from the girder flanges.

Although AASHTO Standard Specifications require cross frames or diaphragms at intervals of 25 ft or less, it is questionable whether spacing that close is necessary for bridges in service. Often, a three-dimensional finite-element analysis will show that few, if any, cross frames or diaphragms are necessary. Inasmuch as most fatigue-related damage to steel bridge is a direct result of out-of-plane forces induced through cross frames, the possibility of eliminating them should be investigated for all new bridges. However, although cross frames may not be needed for service loads, they may be necessary to ensure stability during girder erection and deck placement.

The LRFD Specifications do not require cross frames or diaphragms but specify that the need for diaphragms or cross frames should be investigated for all stages of assumed construction procedures and the final condition. Diaphragms or cross frames required for conditions other than the final condition may be specified to be temporary bracing. If permanent cross frames or diaphragms are included in the structural model used to determine force effects, they should be designed for all applicable limit states for the calculated member loads.

For plate girders, stiffeners used as cross-frame connection stiffeners should be connected to both flanges to prevent distortion-induced fatigue cracking. Although many designers believe welding stiffeners to the tension flange is worse than leaving the connection stiffener unattached, experience has proven otherwise. Virtually no cracks result from the attachment weld, but a proliferation of cracks develop when connection stiffeners are not connected to the tension flange. The LRFD Specifications also recommend that, where cross frames are used, the attachment be designed for a transverse force of 20 kips (Fig. 10.9). This applies to straight, nonskewed bridges when better information is not available.

10.11.7 Portal and Sway Bracing

End panels of simply supported, through-truss bridges have compression chords that slope to meet the bottom chords just above the bearings. Bracing between corresponding sloping chords of a pair of main trusses is called **portal bracing** (Fig. 10.8). Bracing between corresponding vertical posts of a pair of main trusses is called **sway bracing** (Fig. 10.8).

All through-truss bridges should have portal bracing, made as deep as clearance permits. Portal bracing preferably should be of the two-plane or box type, rigidly connected to the flanges of the end posts (sloping chords). If single-plane portal bracing is used, it should be set in the central transverse plane of the end posts. Diaphragms then should be placed between the webs of the end posts, to distribute the portal stresses.

Portal bracing should be designed to carry the end reaction of the top lateral system. End posts should be designed to transfer this reaction to the truss bearings.

Through trusses should have sway bracing at least 5 ft deep in highway bridges at each intermediate panel point. Top lateral struts should be at least as deep as the top chord.

Deck trusses should have sway bracing between all corresponding panel points. This bracing should extend the full depth of the trusses below the floor system. End sway bracing should be designed to carry the top lateral forces to the supports through the truss end posts.

10.11.8 Bracing of Towers

Towers should be braced with double systems of diagonals and with horizontal struts at caps, bases, and intermediate panel points. Sections of members of longitudinal bracing in each panel should not be less than those of members in corresponding panels of the transverse bracing.

Column splices should be at or just above panel points. Bracing of a long column should fix the column about both axes at or near the same point.

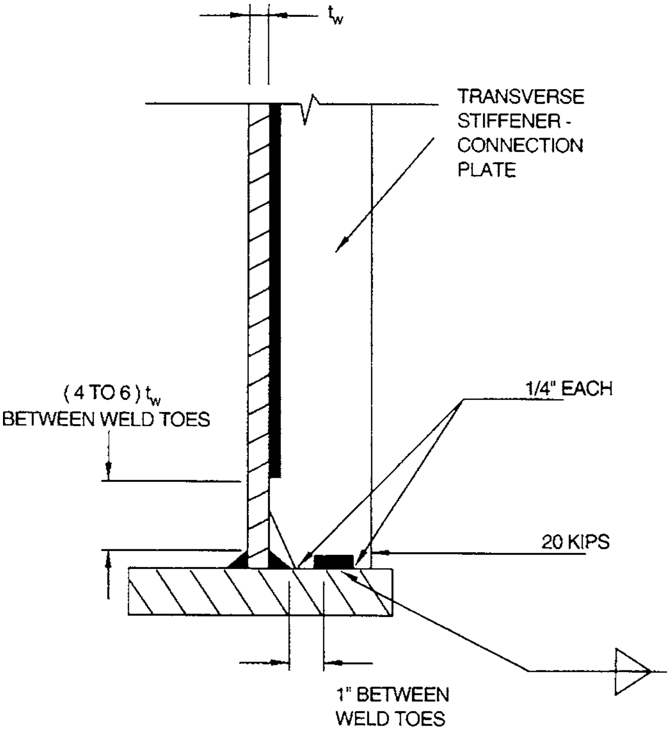


FIGURE 10.9 Girder connects to a cross frame through a transverse stiffener.

Horizontal diagonal bracing should be placed, at alternate intermediate panel points, in all towers with more than two vertical panels. In double-track towers, horizontal bracing should be installed at the top to transmit horizontal forces.

Bottom struts of towers should be strong enough to slide the movable shoes with the structure unloaded, when the coefficient of friction is 0.25. Column bearings should be designed for expansion and contraction of the tower bracing.

10.12 CRITERIA FOR BUILT-UP TENSION MEMBERS

A tension member and all its components must be proportioned to meet the requirements for maximum slenderness ratio given in Table 10.25. The member also must be designed to ensure that the allowable tensile stress on the net section is not exceeded.

The **net section** of a high-strength-bolted tension member is the sum of the net sections of its components. The net section of a component is the product of its thickness and net width.

Net width is the minimum width normal to the stress minus an allowance for holes. The diameter of a hole for a fastener should be taken as $^1/_8$ in greater than the nominal fastener diameter. The chain of holes that is critical is the one that requires the largest deduction for holes and may lie on a straight line or in a zigzag pattern. The deduction for any chain of holes equals the sum of the diameters of all the holes in the chain less, for each gage space in the chain, $s^2/4g$, where s is the pitch, in, of any two successive holes and g is the gage, in, of those holes.

For angles, the gross width should be taken as the sum of the widths of the legs less the thickness. The gage for holes in opposite legs is the sum of the gages from back of angle less the thickness. If a double angle or tee is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net section may be considered effective. However, if double angles, or a single angle or tee, are connected on the same side of a gusset plate, the effective area should be taken as the net section of the connected leg or flange plus one-half the area of the outstanding leg. When angles connect to separate gusset plates, as in a double-webbed truss, and the angles are interconnected close to the gussets, for example, with stay plates, the full net area may be considered effective. Without such interconnection, only 80% of the net area may be taken as effective.

For built-up tension members with perforated plates, the net section of the plate through the perforation may be considered the effective area.

In pin-connected tension members other than eyebars, the net section across the pinhole should be at least 140%, and the net section back of the pinhole at least 100% of the required net section of the body of the member. The ratio of the net width, through the pinhole normal to the axis of the member, to thickness should be 8 or less. Flanges not bearing on the pin should not be considered in the net section across the pin.

To meet stress requirements, the section at pinholes may have to be reinforced with plates. These should be arranged to keep eccentricity to a minimum. One plate on each side should be as wide as the outstanding flanges will allow. At least one full-width plate on each segment should extend to the far side of the stay plate and the others at least 6 in beyond the near edge. These plates should be connected with fasteners or welds arranged to distribute the bearing pressure uniformly over the full section.

Eyebars should have constant thickness, no reinforcement at pinholes. Thickness should be between $^1/_2$ and 2 in, but not less than $^1/_8$ the width. The section across the center of the pinhole should be at least 135%, and the net section back of the pinhole at least 75% of the required net section of the body of the bar. The width of the body should not exceed the pin diameter divided by $^3/_4 + F_y/400$, where F_y is the steel yield strength, ksi. The radius of transition between head and body of eyebar should be equal to or greater than the width of the head through the center of the pinhole.

Eyebars of a set should be symmetrical about the central plane of the truss and as nearly parallel and close together as practicable. But adjacent bars in the same panel should be at least $^1/_2$ in apart. The bars should be held against lateral movement.

Stitching. In built-up members, welds connecting plates in contact should be continuous. Spacing of fasteners should be the smaller of that required for sealing, to prevent penetration of moisture, or stitching, to ensure uniform action. The pitch of stitch fasteners on any single line in the direction of stress should not exceed $24t$, where t = thickness, in, of the thinner outside plate or shape. If there are two or more lines of fasteners with staggered pattern, and the gage g, in, between the line under consideration and the farther adjacent line is less than $24t$, the staggered pitch in the two lines, considered together, should not exceed $24t$ or $30t - 3g/4$. The gage between adjacent lines of stitch fasteners should not exceed $24t$.

Cover Plates. When main components of a tension member are tied together with cover plates, the shear normal to the member in the planes of the plates should be assumed to be equally divided between the parallel plates. The shearing force should include that due to the weight of the member plus other external forces.

When perforated cover plates are used, the openings should be ovaloid or elliptical (minimum radius of periphery $1^1/_2$ in). Length of perforation should not exceed twice its width. Clear distance between perforations in the direction of stress should not be less than the distance l between the nearer lines of connections of the plate to the member. The clear distance between the end perforation and end of the cover plate should be at least $1.25l$. For plates groove-welded to the flange edge of rolled components, l may be taken as the distance between welds when the width–thickness ratio of the flange projection is less than 7; otherwise, the distance l should be taken between the roots of the flanges. Thickness of a perforated plate should be at least $^1/_{50}$ of the distance between nearer line of connection.

When stay plates are used to tie components together, the clear distance between them should be 3 ft or less. Length of end stay plates between end fasteners should be at least $1.25l$, and length of intermediate stay plates at least $0.563l$. Thickness of stay plates should not be less than $l/50$ in main members and $l/60$ in bracing. They should be connected by at least three fasteners on each side to the other components. If a continuous fillet weld is used, it should be at least $^5/_{16}$ in.

Tension-member components also may be tied together with end stay plates and lacing bars like compression members. The last fastener in the stay plates preferably should also pass through the end of the adjacent bar.

10.13 CRITERIA FOR BUILT-UP COMPRESSION MEMBERS

Compression members should be designed so that main components are connected directly to gusset plates, pins, or other members. Stresses should not exceed the allowable for the gross section. The radius of gyration and the effective area of a member with perforated cover plates should be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates, the effective area should be considered the same as for a section with perforations in the same transverse plane.

Solid-Rib Arches. A compression member and all its components must be proportioned to meet the requirements for maximum slenderness ratio in Table 10.25. The member also must satisfy width–thickness requirements (Table 10.26). In addition, for solid-rib arches, longitudinal stiffeners are required when the depth–thickness ratio of each web exceeds

$$\frac{D}{t} = \frac{158}{\sqrt{f_a}} \leq 60 \tag{10.30}$$

where D = unsupported distance, in, between flange components
t = web thickness, in
f_a = maximum compressive stress in web, ksi

If one longitudinal stiffener is used, it should have a moment of inertia I_s, in^4, of at least

$$I_s = 0.75Dt_w^3 \tag{10.31}$$

where D = clear unsupported depth of web, in, and t_w = web thickness, in. If the stiffener is placed at mid-depth of the web, the width–thickness ratio should not exceed

$$\frac{D}{t_w} = \frac{237}{\sqrt{f_a}} \tag{10.32}$$

If two longitudinal stiffeners are used, each should have a moment of inertia of at least

$$I_s = 2.2Dt_w^3 \tag{10.33}$$

If the stiffeners are placed at the third points of the web depth, the width–thickness ratio should not exceed

$$\frac{D}{t_w} = \frac{316}{\sqrt{f_a}} \tag{10.34}$$

Maximum width–thickness ratio for an outstanding element of a stiffener is given by

$$\frac{b'}{t_s} = \frac{51.4}{\sqrt{f_a + f_b / 3}} \leq 12 \tag{10.35}$$

where b' = width of outstanding element, in
t_s = thickness of the element, in
f_b = maximum compressive bending stress, ksi

The preceding relationships for webs applies when

$$0.2 \le \frac{f_b}{f_b + f_a} \le 0.7 \tag{10.36}$$

For flange plates between the webs of a solid-rib arch, the width–thickness ratio should not exceed

$$\frac{b_f}{t_f} = \frac{134}{\sqrt{f_a + f_b}} \le 47 \tag{10.37}$$

Maximum width–thickness ratio for the overhang of flange plates is given by

$$\frac{b'_f}{t_f} = \frac{51.4}{\sqrt{f_a + f_b}} \le 12 \tag{10.38}$$

Stitching. In built-up members, welds connecting plates in contact should be continuous. Spacing of fasteners should be the smaller of that required for sealing, to prevent penetration of moisture, or stitching, to ensure uniform action and prevent local buckling. The pitch of stitch fasteners on any single line in the direction of stress should not exceed $12t$, where t = thickness, in, of the thinner outside plate or shape. If there are two or more lines of fasteners with staggered pattern, and the gage g, in, between the line under consideration and the farther adjacent line is less than $24t$, the staggered pitch in the two lines, considered together, should not exceed $12t$ or $15t - 3g/8$. The gage between adjacent lines of stitch fasteners should not exceed $24t$.

Fastener Pitch at Ends. Pitch of fasteners connecting components of a compression member over a length equal to 1.5 times the maximum width of member should not exceed 4 times the fastener diameter. The pitch should be increased gradually over an equal distance farther from the end.

Shear. On the open sides of compression members, components should be connected with perforated plates or by lacing bars and end stay plates. The shear normal to the member in the planes of the plates or bars should be assumed equally divided between the parallel planes. The shearing force should include that due to the weight of the member, other external forces, and a normal shearing force, kips, given by

$$V = \frac{P}{100}\left(\frac{100}{L/r + 10} + \frac{L/r}{3300/F_y}\right) \tag{10.39}$$

where P = allowable compressive axial load on member, kips
L = length of member, in
r = radius of gyration, in, of section about axis normal to plane of lacing or perforated plate

Perforated Plates. When perforated cover plates are used, the openings should be ovaloid or elliptical (minimum radius of periphery $1^1/_2$ in). Length of perforation should not exceed twice its width. Clear distance between perforations in the direction of stress should not be less than the distance l between the nearer lines of connections of the plate to the member. The clear distance between the end perforation and end of the cover plate should be at least $1.25l$. For plates groove-welded to the flange edge of rolled components, l may be taken as the distance between welds when the width–thickness ratio of the flange projection is less than 7; otherwise, the distance l should be taken between the roots of the flanges. Thickness should meet the requirements for perforated plates given in Table 10.26.

10.14 PLATE GIRDERS AND COVER-PLATED ROLLED BEAMS

Where longitudinal beams or girders support through bridges, the spans preferably should have two main members. They should be placed sufficiently far apart to prevent overturning by lateral forces.

Spans. For calculation of stresses, span is the distance between center of bearings or other points of support. For computing span–depth ratio for continuous beams, span should be taken as the distance between dead-load points of inflection.

Allowable Stress Design. Beams and plate girders should be proportioned by the moment-of-inertia method; that is, for pure bending, to satisfy the flexure formula,

$$\frac{I}{c} \geq \frac{M}{F_b} \tag{10.40}$$

where I = moment of inertia, in^4, of gross section for compressive stress and of net section for tensile stress
c = distance, in, from neutral axis to outermost surface
M = bending moment at section, in kips
F_b = allowable bending stress, ksi

The neutral axis should be taken along the center of gravity of the gross section. For computing the moment of inertia of the net section, the area of holes for high-strength bolts in excess of 15% of the flange area should be deducted from the gross area.

Span–Depth Ratio. Depth of steel beams or girders for highway bridges should preferably be at least $^1/_{25}$ of the span.

For bracing requirements, see Art. 10.11.5.

Cover-Plated Rolled Beams. Welds connecting a cover plate to a flange should be continuous and capable of transmitting the horizontal shear at any point. When the unit shear in the web of a rolled beam at a bearing exceeds 75% of the allowable shear for girder webs, bearing stiffeners should be provided to reinforce the web. They should be designed to satisfy the same requirements as bearing stiffeners for girders in Art. 10.11.4.

The theoretical end of a cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations. **Terminal distance,** or extension of cover plate beyond the theoretical end, is twice the nominal cover-plate width for plates not welded across their ends and 1.5 times the width for plates welded across their ends. Length of a cover plate should be at least twice the beam depth plus 3 ft. Thickness should not exceed twice the flange thickness.

Partial-length welded cover plates should extend beyond the theoretical end at least the terminal distance or a sufficient distance so that the stress range in the flange equals the allowable fatigue stress range for base metal at fillet welds, whichever is greater. Ends of tapered cover plates should be at least 3 in wide. Welds connecting a cover plate to a flange within the terminal distance should be of sufficient size to develop the computed stress in the cover plate at its theoretical end.

Because of their low fatigue strength, cover-plated beams are seldom cost-effective.

Girder Flanges. Width–thickness ratios of compression flanges of plate girders should meet the requirements given in Art. 10.11. For other girders, see Arts. 10.15, 10.17, and 10.18.

Each flange of a welded plate girder should consist of only one plate. To change size, plates of different thicknesses and widths may be joined end to end with complete-penetration groove welds and appropriate transitions.

Plate girders composed of flange angles, web plate, and cover plates attached with bolts or rivets are no longer used. In existing bolted girders, flange angles formed as large a part of the flange area as practicable. Side plates were used only where flange angles more than $^7/_8$ in thick would otherwise be required. Except in composite design, the gross area of the compression flange could not be less than the gross area of the tension flange.

When cover plates were needed, at least one cover plate of the top flange extended the full length of the girder unless the flange was covered with concrete. If more than one cover plate was desirable, the plates on each flange were made about the same thickness. When of unequal thickness, they were arranged so that they decreased in thickness from flange angles outward. No plate could be thicker than the flange angles. Fasteners connecting cover plates and flange were required to be adequate to transmit the horizontal shear at any point. Cover plates over 14 in wide should have four lines of fasteners.

Partial-length cover plates extended beyond the theoretical end far enough to develop the plate capacity or to reach a section where the stress in the remainder of the flange and cover plates equals the allowable fatigue stress range, whichever distance is greater.

Flange-to-Web Connections. Welds or fasteners for connecting the flange of a plate girder to the web should be adequate to transmit the horizontal shear at any point plus any load applied directly to the flange. AASHTO permits the web to be connected to each flange with a pair of fillet welds.

Girder Web and Stiffeners. The web should be proportioned so that the average shear stress over the gross section does not exceed the allowable. In addition, depth–thickness ratio should meet the requirements of Art. 10.13. Also, stiffeners should be provided, where needed, in accordance with those requirements.

Camber. Girders should be cambered to compensate for dead-load deflection. Also, on vertical curves, camber preferably should be increased or decreased to keep the flanges parallel to the profile grade line.

See also Art. 10.16.

10.15 COMPOSITE CONSTRUCTION WITH I GIRDERS

With shear connectors welded to the top flange of a beam or girder, a concrete slab may be made to work with that member in carrying bending stresses. In effect, a portion of the slab, called the **effective width,** functions much like a steel cover plate. In fact, the effective slab area may be transformed into an equivalent steel area for computation of composite-girder stresses and deflection. This is done by dividing the effective concrete area by the modular ratio n, the ratio of modulus of elasticity of steel, 29,000 ksi, to modulus of elasticity of the concrete. The equivalent area is assumed to act at the center of gravity of the effective slab. The equivalent steel section is called the **transformed section.**

Allowable Stress Design. Composite girders, in general, should meet the requirements of plate girders (Art. 10.14). Bending stresses in the steel girder alone and in the transformed section may be computed by the moment-of-inertia method, as indicated in Art. 10.14, or by load factor design, and should not exceed the allowable for the material. The stress range at the shear connector must not exceed the allowable for a Category C detail.

The allowable concrete stress may be taken as $0.4f'_c$, where f'_c = unit ultimate compressive strength of concrete, psi, as determined by tests of 28-day-old cylinders. The allowable tensile stress of steel reinforcement for concrete should be taken as 20 ksi for A615 Grade 40 steel bars and 24 ksi for A615 Grade 60 steel bars. The modular ratio n may be assumed as follows:

f'_c	n
2000–2300	11
2400–2800	10
2900–3500	9
3600–4500	8
4600–5900	7
6000 or more	6

To account for creep of the concrete under dead load, design of the composite section should include the larger of the dead-load stresses when the transformed section is determined with n or $3n$.

The neutral axis of the composite section preferably should lie below the top flange of the steel section. Concrete on the tension side should be ignored in stress computations.

Effective Slab Width. The assumed effective width of slab should be equal to or less than one-quarter the span, distance center to center of girders, and 12 times the least slab thickness (Fig. 10.10). For exterior girders, the effective width on the exterior side should not exceed the actual overhang. When an exterior girder has a slab on one side only, the assumed effective width should be equal to or less than one-twelfth the span, half the distance to the next girder, and 6 times the least slab thickness (Fig. 10.10).

Span–Depth Ratios. For composite highway girders, depth of steel girder alone should preferably be at least $^1/_{30}$ of the span. Depth from top of concrete slab to bottom of bottom flange should preferably be at least $^1/_{25}$ of the span. For continuous girders, spans for this purpose should be taken as the distance between dead-load inflection points.

Girder Web and Stiffeners. The steel web should be proportioned so that the average shear stress over the gross section does not exceed the allowable. The effects of the steel flanges and concrete slab should be ignored. In addition, depth–thickness ratio should meet the requirements of Art. 10.11. Also, stiffeners should be provided, where needed, in accordance with those requirements.

Bending Stresses. If, during erection, the steel girder is supported at intermediate points until the concrete slab has attained 75% of its required 28-day strength, the composite section may be assumed to carry the full dead load and all subsequent loads. When such shoring is not used, the steel girder alone must carry the steel and concrete dead loads. The composite section will support all loads subsequently applied. Thus, maximum bending stress in the steel of an unshored girder equals the sum of the dead-load stress in the girder alone plus stresses produced by loads on the composite section. Maximum bending stress in the concrete equals the stresses produced by those loads on the composite section at its top surface.

The positive-moment portion of continuous composite-girder spans should be designed in the same way as for simple spans. The negative-moment region need not be designed for composite action, in which case shear connectors need not be installed there. But additional connectors should be placed in the region of the dead-load inflection point as indicated later. If composite action is desired in the negative-moment portion, shear connectors should be installed. Then, longitudinal

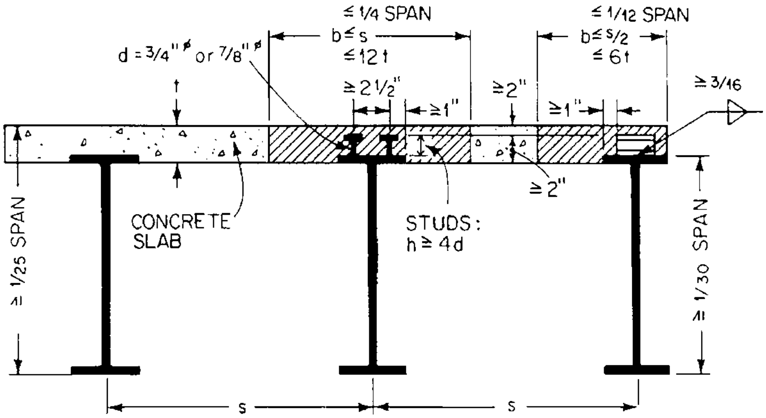


FIGURE 10.10 Effective width of concrete slab for composite construction.

steel reinforcement in the concrete should be provided to carry the full tensile force. The concrete should be assumed to carry no tension.

Shear Connectors. To ensure composite action, shear connectors must be capable of resisting both horizontal and vertical movements between concrete and steel. They should permit thorough compaction of the concrete so that their entire surfaces are in contact with the concrete. Usually, headed steel studs or channels, welded to the top flange of the girder, are used.

Channels should be attached transverse to the girder axis, with fillet welds at least along heel and toe. Minimum weld size permitted for this purpose is $^3/_{16}$ in.

Studs should be $^3/_4$- or $^7/_8$-in nominal diameter. Overall length after welding should be at least 4 times the diameter. Steel should be A108, Grade 1015, 1018, or 1020, either fully or semikilled. The studs should be end-welded to the flange with automatically timed equipment. If a 360° weld is not obtained, the interrupted area may be repaired with a $^3/_{16}$-in fillet weld made by low-hydrogen electrodes in the shielded metal-arc process. Usually, two or more studs are installed at specific sections of a composite girder, at least four stud diameters center to center.

Clear depth of concrete cover over the tops of shear connectors should be at least 2 in. In addition, connectors should penetrate at least 2 in above the bottom of the slab. Clear distance between a flange edge and a shear-connector edge should not be less than 1 in in highway bridges, $1^1/_2$ in in railroad bridges.

Pitch of Shear Connectors. In general, shear connectors should not be spaced more than 24 in center to center along the span. Over interior supports of continuous beams, however, wider spacing may be used to avoid installation of connectors at points of high tensile stress.

Pitch may be determined by fatigue shear stresses due to change in horizontal shear or by ultimate-strength requirements for resisting total horizontal shear, whichever requires the smaller spacing. (Also, see the following method for stress design.)

Fatigue. As live loads move across a bridge, the vertical shear at any point in a girder changes. For some position of the loading, vertical shear at the point due to live load plus impact reaches a maximum. For another position, shear there due to live load plus impact becomes a minimum, which may be opposite in sign to the maximum. The algebraic difference between maximum and minimum shear, kips, is the range of shear V_r.

The range of horizontal shear, kips/lin in, at the junction of a slab and girder at the point may be computed from

$$S_r = \frac{V_r Q}{I} \tag{10.41}$$

where Q = statical moment, in^3, about the neutral axis of the composite section, of the transformed compressive concrete area, or for negative bending moment, of the area of steel reinforcement in the concrete

I = moment of inertia, in^4, about the natural axis, of the transformed composite girder in positive-moment regions, and in negative-moment regions, the moment of inertia, in^4, about the neutral axis, of the girder and concrete reinforcement if the girder is designed for composite action there, or without the reinforcement if the girder is non-composite there

The allowable range of shear, kips per connector, is

For channels:
$$Z_r = Bw \tag{10.42}$$

For welded studs:
$$Z_r = \alpha d^2 \qquad \left(\frac{h}{d} \geq 4\right) \tag{10.43}$$

where w = transverse length of channel, in
d = stud diameter, in
h = overall stud height, in
B = 4 for 100,000 cycles of maximum stress
= 3 for 500,000 cycles
= 2.4 for 2,000,000 cycles
= 2.1 for more than 2,000,000 cycles
α = 13 for 100,000 cycles of maximum stress
= 10.6 for 500,000 cycles
= 7.85 for 2,000,000 cycles
= 5.50 for more than 2,000,000 cycles

The required pitch p_r, in, of shear connectors for fatigue is obtained from

$$p_r = \frac{\sum Z_r}{S_r} \tag{10.44}$$

where $\sum Z_r$ is the allowable range of horizontal shear of all connectors at a cross section. Over interior supports of continuous beams, the pitch may be modified to avoid installation of connectors at points of high tensile stress. But the total number of connectors should not be decreased.

Ultimate Strength. The total number of connectors provided for fatigue, in accordance with Eq. (10.44), should be checked for adequacy at ultimate strength under dead load plus live load and impact. The connectors must be capable of resisting the horizontal forces H, kips, in positive-moment regions and in negative-moment regions. Thus, at points of maximum moment, H may be taken as the smaller of the values given by Eqs. (10.45) and (10.46):

$$H_1 = A_s F_y \tag{10.45}$$

$$H_2 = 0.85 f'_c bt \tag{10.46}$$

where A_s = cross-sectional area of steel girder, in^2
F_y = steel yield strength, ksi
f'_c = 28-day compressive strength of concrete, ksi
b = effective width of concrete slab, in
t = slab thickness, in

At points of maximum negative moment, H should be taken as

$$H_3 = A_{\mathrm{rs}} F_{\mathrm{ry}} \tag{10.47}$$

where A_{rs} = total area of longitudinal reinforcing steel at interior support within effective slab width, in^2, and F_{ry} = yield strength, ksi, of reinforcing steel. The total number of shear connectors required in any region then is

$$N = \frac{1{,}000H}{\phi Q_u} \tag{10.48}$$

where Q_u = ultimate strength of shear connector, lb, and ϕ = reduction factor, 0.85. In Eq. (10.48), the smaller of H_1 or H_2 should be used for H for determining the number of connectors required between a point of maximum positive moment and an end support in simple beams, and between a point of maximum positive moment and a dead-load inflection point in continuous beams. H_3 should be used for H for determining the total number of shear connectors required between a point of maximum negative moment and a dead-load inflection point in continuous beams. $H_3 = 0$ if slab reinforcement is not used in the computation of section properties for negative moment.

For channels:

$$Q_u = 550\left(t_f + \frac{t_w}{2}\right) l \sqrt{f'_c} \tag{10.49}$$

For welded studs:

$$Q_u = 0.4 d^2 \sqrt{f'_c E_c} \qquad \left(\frac{h}{d} > 4\right) \tag{10.50}$$

where E_c = modulus of the concrete, psi $= 33w^{3/2}\sqrt{f'_c}$
t_f = average thickness of channel flange, in
t_w = thickness of channel web, in
l = length of channel, in
f'_c = 28-day strength of concrete, psi
w = weight of the concrete, lb/ft^3
d = stud diameter, in
h = stud height, in

Additional Connectors at Inflection Points. In continuous beams, the positive-moment region under live loads may extend beyond the dead-load inflection points, and additional shear connectors are required in the vicinity of those points when longitudinal reinforcing steel in the concrete slab is not used in computing section properties. The number needed is given by

$$N_c = A_{\mathrm{rs}}\frac{f_r}{Z_r} \tag{10.51}$$

where A_{rs} = total area, in^2, of longitudinal reinforcement at interior support within effective slab width
f_r = range of stress, ksi, due to live load plus impact in slab reinforcement over support (10 ksi may be used in the absence of accurate computations)
Z_r = allowable range, kips, of shear per connector, as given by Eqs. (10.42) and (10.43)

This number should be placed on either side of or centered about the inflection point for which it is computed, within a distance of one-third the effective slab width.

10.16 COST-EFFECTIVE PLATE-GIRDER DESIGNS

To get cost-effective results from the many different designs of fabricated girders that can satisfy the requirements of specifications, designers should obtain advice from fabricators and contractors whenever possible. Also useful are steel-industry-developed rules of thumb intended to help designers. The following recommendations, modified to reflect current trends, should be considered for all designs.

1. Load and resistance factor design (LRFD) is the preferred design procedure. Load factor design (LFD) yields more economical girder designs than does allowable stress design (ASD).
2. Properly designed for their environment, unpainted weathering-steel bridges are more economical in the long run than those requiring painting. Consider the following grades of weathering steels: ASTM A709 Grade 50W, 70W, HPS70W, or 100W. Grade 50W is the most often used.
3. The most economical painted design is that for hybrid girders, using 36-ksi and 50-ksi steels. Painted homogenous girders of 50-ksi steel are a close second. The most economical design with high-performance steel (HPS) will also be hybrid, utilizing Grade 50W steel for all stiffeners, diaphragm members, webs, and flanges, where Grade 70W strength is not required. Rolled sections (angles, channels, etc.) are not available in HPS grades.
4. The fewer the girders, the greater the economy. Girder spacing must be compatible with deck design, but sometimes other factors govern selection of girder spacing. For economy, girder spacing should be 10 ft or more.
5. Transverse web stiffeners, except those serving as diaphragm or cross-frame connections, should be placed on only one side of a web.
6. Web depth may be several inches larger or small than the optimum without significant cost penalty.
7. A plate girder with a nominally stiffened web—$^1/_{16}$ in thinner than an unstiffened web—will be the least costly or very close to it. (Unstiffened webs are generally the most cost-effective for

web depths less than 52 in. Nominally stiffened webs are most economical in the 52- to 72-in range. For greater depths, fully stiffened webs may be the most cost-effective.)

8. Web thickness should be changed only where splices occur. (Use standard-plate-thickness increments of $^1/_{16}$ in for plates up to 2 in thick and $^1/_8$-in increments for plates over 2 in thick.)
9. Longitudinal stiffeners should be considered for plate girders only for spans over 300 ft.
10. Not more than three plates should be butt-spliced to form the flanges of field sections up to 130 ft long. In some cases, it is advisable to extend a single flange-plate size the full length of a field section.
11. To justify a welded flange splice, about 700 lb of flange steel would have to be eliminated. However, quenched-and-tempered plates are limited to 50-ft lengths.
12. A constant flange width should be used between flange field splices. (Flange widths should be selected in 1-in increments.)
13. For most conventional cross sections, haunched girders are not advantageous for spans under 400 ft.
14. Bottom lateral bracing should be omitted where permitted by AASHTO specifications. Omit intermediate cross frames where permitted by AASHTO (see LRFD Specification, Art. 6.7.4), but indicate on the plans where temporary bracing will be required for girder stability during erection and deck placement. Space permanent intermediate cross frames, if required, at the maximum spacing consistent with final loading conditions.
15. Elastomeric bearings are preferable to custom-fabricated steel bearings.
16. Composite construction may be advantageous in negative-moment regions of composite girders.

Designers should bear in mind that such techniques as finite-element analysis, use of high-strength steels, and load and resistance factor design often lead to better designs.

Consideration should be given to use of 40-in-deep and 42-in-deep rolled sections. These may be cost-effective alternatives to welded girders for spans up to 100 ft or longer. Economy with these beams may be improved with end-bolted cover-plate details that allow use of Category B stress ranges (Art. 10.9). Contract documents that allow either rolled beams or welded girders ensure cost-effective alternatives for owners.

With fabricated girders, designers should ensure that flanges are wide enough to provide lateral stability for the girders during fabrication and erection. Flange width should be at least 12 in, but possibly even greater for deeper girders. The AISC recommends that, for shipping, handling, and erection, the ratio of length to width of compression flanges should be about 85 or less.

Designers also should avoid specifying thin flanges that make fabrication difficult. A thin flange is subject to excessive warping during welding of a web to the flange. To reduce warping, a flange should be at least $^3/_4$ in thick.

To minimize fabrication and deck forming costs when changes in the area of the top flange are required, the width should be held constant and required changes made by thickness transitions.

10.17 BOX GIRDERS (ASD)

Closed-section members, such as box girders, often are used in highway bridges because of their rigidity, economy, appearance, and resistance to corrosion. Box girders have high torsional rigidity. With their wide bottom flanges (Fig. 10.11), relatively shallow depths can be used economically. And for continuous box girders, intermediate support often can be individual, slender columns simply connected to concealed cross frames.

While box girders may be multicell (with three or more webs), single-cell girders, as illustrated in Fig. 10.11, are generally preferred. For short spans, such girders can be entirely shop-fabricated, permitting assembly by welding under closely controlled and economical conditions. Longer spans often can be prefabricated to the extent that only one field splice is necessary. One single-cell girder can be used to support bridges with one or two traffic lanes. Usually, however, multiple boxes are used to carry two or more lanes to keep box width small enough to meet shipping-clearance requirements.

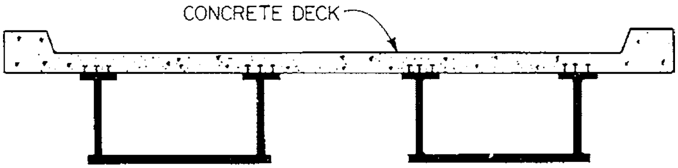


FIGURE 10.11 Composite construction with box girders.

Through the use of shear connectors welded to the top flanges, a concrete deck can be made to work with the box girders in carrying bending stresses. In such cases the concrete may be considered part of the top flange, and the steel top flange need be only wide enough for erection and handling stability, load distribution to the web, and placement of required shear connectors (Fig. 10.11).

Composite box girders are designed much like plate girders (Arts. 10.14 and 10.15). Criteria that are different are summarized in the following. For distribution of live loads to box girders, see Art. 10.7.

Additional criteria apply to curved box-girder bridges.

Girder Spacing. The criteria are applicable to bridges with multiple single-cell box girders. Width center to center of top steel flanges in each girder should nearly equal the distance center to center between adjacent top steel flanges of adjacent boxes. (Width of boxes should nearly equal distance between boxes.) Cantilever overhang of deck, including curbs and parapets, should not exceed 6 ft or 60% of the distance between centers of adjacent top steel flanges of adjacent box girders.

Bracing. Diaphragms, cross frames, or other bracing should be provided within box girders at each support to resist transverse rotation, displacement, and distortion. Intermediate internal bracing for these purposes is not required if stability during concrete placement and curing has been otherwise ensured.

Lateral systems generally are not required between composite box girders. Need for a lateral system should be determined as follows: A horizontal load of 25 psf on the area of the girder exposed in elevation should be applied in the plane of the bottom flange. The resisting section should comprise the bottom flange serving as web, while portions of the box-girder webs, with width equal to 12 times their thickness, serve as flanges. A lateral system should be provided between bottom flanges if the combined stresses due to the 25-lb/ft^2 load and dead load of steel and deck exceed 150% of the allowable stress.

Access and Drainage. Manholes or other openings to the box interior should be provided for form removal, inspection, maintenance, drainage, or access to utilities.

Box-Girder Webs. Web plates may be vertical or inclined. A trapezoidal box generally requires a heavier bottom plate, and sometimes also a heavier concrete slab, but it may reduce the number of girders needed to support a deck. Design shear for an inclined web, kips, may be calculated from

$$V_w = \frac{V_v}{\cos\theta} \tag{10.52}$$

where V_v = vertical shear, kips, on web and θ = angle web makes with the vertical.

Transverse bending stresses due to distortion of the cross section and bottom-flange vibrations need not be considered if the web slope relative to the plane of the bottom flange is 4:1 or more and the bottom-flange width does not exceed 20% of the span. Furthermore, transverse bending stresses due to supplementary loadings, such as utilities, should not exceed 5 ksi. When any of the preceding limits are exceeded, transverse bending stresses due to all causes should be restricted to a maximum stress or stress range of 20 ksi.

Bottom Flange in Tension. Bending stress cannot be assumed uniformly distributed horizontally over very wide flanges. To simplify design, only a portion of such a flange should be considered effective, and the horizontal distribution of the bending stresses may be assumed uniform over that portion.

For simply supported girders, and between inflection points of continuous spans, the bottom flange may be considered completely effective if its width does not exceed one-fifth the span. For wider flanges, effective width equals one-fifth the span.

Unstiffened Compression Flanges. Compression flanges designed for the basic allowable stress of $0.55F_y$ need not be stiffened if the width–thickness ratio does not exceed

$$\frac{b}{t} = \frac{194}{\sqrt{F_y}} \tag{10.53}$$

where b = flange width between webs, in
t = flange thickness, in
F_y = steel yield strength for flange, ksi

When $194/\sqrt{F_y} < b/t \le 420/\sqrt{F_y}$, but not more than 60, the stress in an unstiffened bottom flange, ksi, should not exceed

$$F_b = F_y\left(0.326 + 0.244\sin\frac{c\pi}{2}\right) \tag{10.54}$$

$$c = \frac{420 - (b/t)\sqrt{F_y}}{226} \tag{10.55}$$

When $b/t\ 420/\sqrt{F_y}$, the stress, ksi, in the flange should not exceed

$$F_b = \frac{57{,}600}{(b/t)^2} \tag{10.56}$$

b/t preferably should not exceed 60, except in areas of low stress near inflection points.

Longitudinally Stiffened Compression Flanges. When $b/t > 45$, use of longitudinal stiffeners should be considered. When used, they should be equally spaced across the compression flange. The number required depends heavily on the ratio of spacing to flange thickness.

For the flange, including the longitudinal stiffeners, to be designed for the basic allowable stress $0.55F_y$, this ratio should not exceed

$$\frac{w}{t} = \frac{97}{\sqrt{F_y/k}} \tag{10.57}$$

where w = width of flange, in, between longitudinal stiffeners or distance, in, from a web to nearest stiffener and k = buckling coefficient, which may be assumed to be between 2 and 4. For larger values of w/t, but not more than 60 or $210/\sqrt{F_y/k}$, the stress, ksi, in the flange should not exceed

$$F_b = F_y\left(0.326 + 0.224\sin\frac{c'\pi}{2}\right) \tag{10.58}$$

$$c' = \frac{210 - (w/t)\sqrt{F_y/k}}{113} \tag{10.59}$$

When $210/\sqrt{F_y/k} < w/t \le 60,$ the stress, ksi, should not exceed

$$F_b = \frac{14{,}400k}{(w/t)^2} \tag{10.60}$$

Stiffeners should be proportioned so that the depth–thickness ratio of any outstanding element does not exceed

$$\frac{d_s}{t_s} = \frac{82.2}{\sqrt{F_y}} \tag{10.61}$$

where d_s = depth, in, of outstanding element and t_s = thickness, in, of element. The moment of inertia, in^4, of each longitudinal stiffener about an axis through the base of the stiffener and parallel to the flange should be least

$$I_s = \phi w t^3 \tag{10.62}$$

where $\phi = 0.07k^3n^4$ for $n > 1$
$= 0.125k^3$ for $n = 1$
n = number of longitudinal stiffeners

Longitudinal stiffeners should be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds. At least one transverse stiffener should be installed near dead-load inflection points. It should be the same size as the longitudinal stiffeners.

Compression Flanges Stiffened Longitudinally and Transversely. When $w/t > 97/\sqrt{F_y/k}$ and the number of longitudinal stiffeners exceeds two, addition of transverse stiffeners should be considered. They are not necessary, however, if the ratio of their spacing to flange width b exceeds 3. For the flange, including stiffeners, to be designed for the basic allowable stress $0.55F_y$, w/t for the longitudinal stiffeners should not exceed

$$\frac{w}{t} = \frac{97}{\sqrt{F_y/k_1}} \tag{10.63}$$

$$k_t = \frac{[1+(a/b)^2]^2 + 87.3}{(n+1)(a/b)^2[1+0.1(n+1)]} \le 4 \tag{10.64}$$

where a = spacing, in, of transverse stiffeners. For larger values of w/t but not more than 60 or $210\sqrt{F_y/k_1}$, the stress, ksi, in the flange should not exceed

$$F_b = F_y\left(0.326 + 0.224\sin\frac{c''\pi}{2}\right) \tag{10.65}$$

$$c'' = \frac{210 - (w/t)\sqrt{F_y/k_1}}{113} \tag{10.66}$$

When $210\sqrt{F_y/k_1} < w/t \le 60$, the stress, ksi, should not exceed

$$F_b = \frac{14{,}400k_1}{(w/t)^2} \tag{10.67}$$

Spacing of transverse stiffeners should not exceed $4w$ when k_1 has its maximum value of 4.

When transverse stiffeners are used, each longitudinal stiffener should have a moment of inertia I_s as given by Eq. (10.62) with $\phi = 8$. Each transverse stiffener should have a moment of inertia, in^4, about an axis through its centroid parallel to its bottom edge of at least

$$I_t = \frac{0.10(n+1)^3 w^3 f_b A_f}{Ea} \tag{10.68}$$

where f_b = maximum longitudinal bending stress, ksi, in flange in panels on either side of transverse stiffener
A_f = area, in^2, of bottom flange, including stiffeners
E = modulus of elasticity of flange steel, ksi

Depth–thickness ratio of outstanding elements should not exceed the value determined by Eq. (10.61).

Transverse stiffeners need not be connected to the flange, but they should be attached to the girder webs and longitudinal stiffeners. Each of these web connections should be capable of resisting a vertical force, kips,

$$R_w = \frac{F_y S_t}{2b} \tag{10.69}$$

where S_t = section modulus, in^3, of transverse stiffener and F_y = yield strength, ksi, of stiffener. Each connection of a transverse and longitudinal stiffener should be capable of resisting a vertical force, kips,

$$R_s = \frac{F_y S_t}{nb} \tag{10.70}$$

Flange-to-Web Welds. Total effective thickness of welds connecting a flange to a web should at least equal the web thickness, except that when two or more diaphragms per span are provided, minimum size fillet welds may be used. If fillet welds are used, they should be placed on both sides of the flange or web.

10.18 HYBRID GIRDERS (ASD)

When plate girders are to be used for a bridge, costs generally can be cut by using flanges with higher yield strength than that of the web. Such construction is permitted for highway bridges under AASHTO specifications if the girders qualify as hybrid girders. Such girders are cost-effective because the web of a plate girder contributes relatively little to the girder bending strength and the web shear strength depends on the depth–thickness ratio.

Hybrid girders, in general, may be designed for fatigue as if they were homogeneous plate girders of the flange steel. Composite and noncomposite I-shaped girders may qualify as hybrid.

Noncomposite girders must have both flanges of steel with the same yield strength. Yield strength of web steel should be lower, but not more than 35% less. Different areas may be used at the same cross section for top and bottom flanges. If, however, the bending stress in either flange exceeds $0.55F_{yw}$, where F_{yw} is the specified minimum yield stress of the web, ksi, the tension-flange area should be larger than the compression-flange area. In composite construction, the transformed area of the effective concrete slab or reinforcing steel should be included in the top-flange area.

Composite girders, in contrast, may have a compression flange of steel with yield strength less than that of the tension flange but not less than that of the web. Yield strength of web steel should be lower, but not by more than 35%, than the yield strength of the tension flange.

Criteria governing design of hybrid girders generally are the same as for homogeneous plate girders (Arts. 10.14 and 10.15). Those that differ follow.

Web. Average shear stress in the web should not exceed the allowable for the web steel.

The bending stress in the web may exceed the allowable for the web steel if the stress in each flange does not exceed the allowable for the flange steel multiplied by a reduction factor R.

$$R = 1 - \frac{\beta\psi(1-\alpha)^2(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)} \tag{10.71}$$

where $\alpha = F_{yw}/F_{yf}$
F_{yw} = minimum specified yield strength of web, ksi
F_{yf} = minimum specified yield strength of flange, ksi
β = ratio of web area to tension-flange area
ψ = ratio of distance, in, between outer edge of tension flange and neutral axis (of the transformed section for composite girders) to depth, in, of steel section

In computation of maximum permissible depth–thickness ratios for a web, f_b should be taken as the calculated bending stress, ksi, in the compression flange divided by R.

In design of bearing stiffeners at interior supports of continuous hybrid girders for which $\alpha < 0.7$, no part of the web should be assumed to act in bearing.

Flanges. In composite girders, the bending stress in the concrete slab should not exceed the allowable stress for the concrete multiplied by R.

In computation of maximum permissible width–thickness ratios of a compression flange, f_b should be taken as the calculated bending stress, ksi, in the flange divided by R.

10.19 ORTHOTROPIC-DECK BRIDGES

In orthotropic-deck construction, the deck is a steel plate overlaid with a wearing surface and stiffened and supported by a rectangular grid. The steel deck assists its supports in carrying bending stresses. Main components usually are the steel deck plate, longitudinal girders, transverse floorbeams, and longitudinal ribs. Ribs may be open-type (Fig. 10.12*a*) or closed (Fig. 10.12*b*).

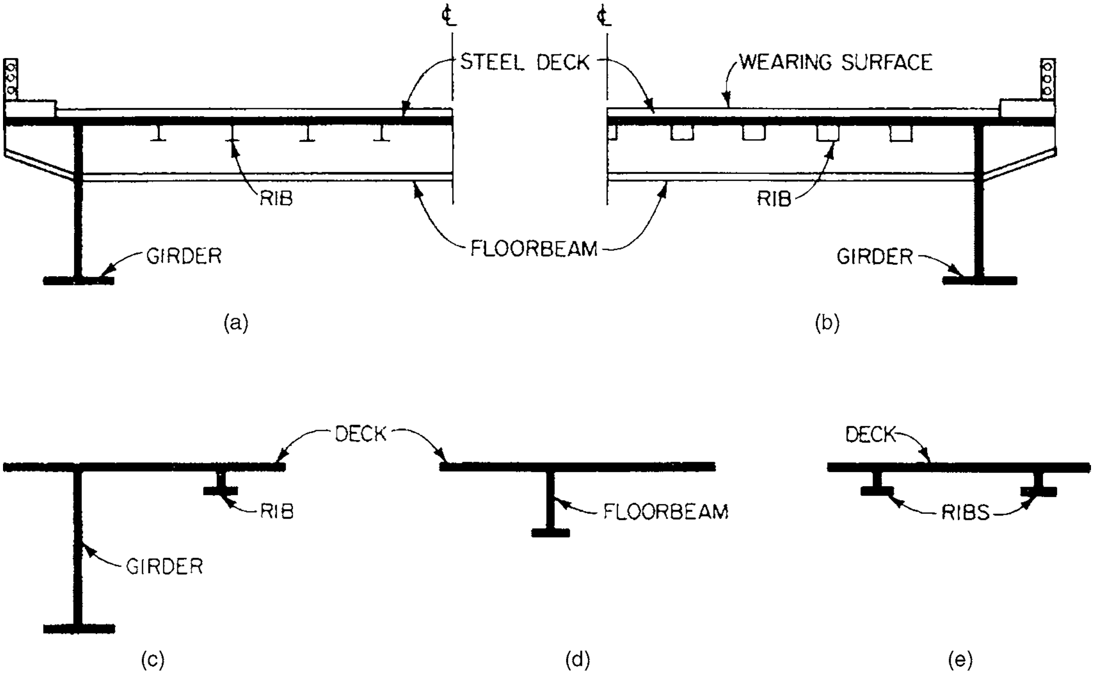


FIGURE 10.12 Orthotropic-plate construction. (*a*) With open ribs. (*b*) With closed ribs. (*c*) Deck and ribs act as the top flange of the main girder. (*d*) Deck acts as the top flange of the floorbeam. (*e*) Deck distributes loads to the ribs.

The steel deck acts as the top flange of the girders (system I, Fig. 10.12*c*). Also, the steel deck serves as the top flange of the ribs (Fig. 10.12*e*) and floorbeams (system II, Fig. 10.12*d*). In addition, the deck serves as an independent structural member that transmits loads to the ribs (system III, Fig. 10.12*e*).

Load Distribution. In determining direct effects of wheel loads on the deck plate, in design of system III for H20 or HS20 loadings, single-axle loads of 24 kips, or double-axle loads of 16 kips each spaced 4 ft apart, should be used. The contact area of one 12- or 8-kip wheel may be taken as 20 in wide (perpendicular to traffic) and 8 in long at the roadway surface. The loaded area of the deck may be taken larger by the thickness of the wearing surface on all sides, by assuming a 45° distribution of load through the pavement.

Deck Thickness. Usually, the deck plate is made of low-alloy steel with a yield point of 50 ksi. Thickness should be at least $^3/_8$ in and is determined by allowable deflection under a wheel, unless greater thickness is required by design of System I or II. Deflection due to wheel load plus 30% impact should not exceed $^1/_{300}$ of spacing of deck supports. Deflection computations should not include the stiffness of the wearing surface. When support spacing is 24 in or less, the deck thickness, in, that meets the deflection limitation is

$$t = 0.07ap^{1/3} \tag{10.72}$$

where a = spacing, in, of open ribs, or maximum spacing, in, of walls of closed ribs and p = pressure at top of steel deck under 12-kip wheel, ksi.

Allowable Stresses (ASD). Stresses in ribs and deck acting as the top flange of the girders and in the ribs due to local bending under wheel loads should be within the basic allowable tensile stress. However, when the girder-flange stresses and local bending stresses are combined, they may total up to 125% of the basic allowable tensile stress. Local bending stresses are those in the deck plate due to distribution of wheel loads to ribs and beams. AASHTO Standard Specifications limit local transverse bending stresses for the wheel load plus 30% impact to a maximum of 30 ksi unless fatigue analysis or tests justify a higher allowable stress. If the spacing of transverse beams is at least three times that of the webs of the longitudinal ribs, local longitudinal and transverse bending stresses need not be combined with other bending stresses, as indicated in the following.

Elements of the longitudinal ribs and the portion of the deck plate between rib webs should meet the minimum thickness requirements given in Table 10.26. The stress f_a may be taken as the compressive bending stress due to bending of the rib, bending of the girder, or 75% of the sum of those stresses, whichever is largest. Unless analysis shows that compressive stresses in the deck induced by bending of the girders will not cause overall buckling of the deck, the slenderness ratio L/r of any rib should not exceed

$$\frac{L}{r} = 1000\sqrt{\frac{1.5}{F_y} - 2.7\frac{F}{F_y^2}} \tag{10.73}$$

where L = distance, in, between transverse beams
r = radius of gyration, in^3, about the horizontal centroidal axis of the rib plus effective area of deck plate
F_y = yield strength, ksi, of rib steel
F = maximum compressive stress, ksi (taken positive) of the deck plate acting as the top flange of the girders

The effective width, and hence the effective area, of the deck plate acting as the top flange of a longitudinal rib or a transverse beam should be determined by analysis of the orthotropic-plate system. Approximate methods may be used. (See, for example, "Design Manual for Orthotropic Steel Plate Deck Bridges," American Institute of Steel Construction.) For the girders, the full width of the deck plate may be considered effective as the top flange if the girder span is at least 5 times the maximum girder spacing and 10 times the maximum distance from the web to the nearest edge of the deck. (For continuous beams, the span should be taken as the distance between inflection points.) If these conditions are not met, the effective width should be determined by analysis.

The elements of the girders and beams should meet requirements for width–thickness and depth–thickness ratios given in Table 10.26 and for stiffeners (Art. 10.11.4).

When connections between ribs and webs of beams, or holes in beam webs for passage of the ribs, or rib splices occur in tensile regions, they may affect the fatigue life of the bridge adversely. Consequently, these details should be designed to resist fatigue as described in Art. 10.9. Similarly, connections between the ribs and the deck plate should be designed for fatigue stresses in the webs due to transverse bending induced by wheel loads.

At the supports, some provision, such as diaphragms or cross frames, should be made to transmit lateral forces to the bearings and to prevent transverse rotation and other deformations.

The same method of analysis used to compute stresses in the orthotropic-plate construction should be used to calculate deflections. Maximum deflections of ribs, beams, and girders due to live load plus impact should not be more than $^1/_{500}$ of the span.

10.20 BEARINGS

Bridges should be designed so that a total movement due to temperature change of $1^1/_4$ in can take place per 100 ft. Also, provisions should be made for changes in length of span resulting from live-load stresses. In spans over 300 ft long, allowance should be made for expansion and contraction in the floor system.

Expansion bearings may be needed to permit such movements. (See also Art. 10.23.) In addition, to control the movements at least one fixed bearing is required in each simple or continuous span. A fixed bearing should be firmly anchored against horizontal and vertical movement, but it may permit the end of the member supported to rotate in a vertical plane. An expansion bearing should permit only end rotation and movement parallel to the longitudinal axis of the supported member, unless provisions for transverse expansion are necessary.

Allowable bearing on granite is 800 psi and on sandstone or limestone, 400 psi, when the masonry projects 3 in or more beyond the edge of the bearing plate. For smaller projections, only 75% of these stresses is allowed. For reinforced concrete, the basic allowable stress f_c is 30% of the 28-day compressive strength. When the supporting surface is wider on all sides than the loaded area A_1, the allowable stress may be multiplied by $\sqrt{A_2 / A_1} \leq 2$, where A_2 is the area of the supporting surface.

Bearings for spans of 50 ft or more should be designed to permit end rotation. For the purpose, curved bearing plates, elastomeric pads, or pin arrangements may be used. Elastomeric bearings are generally preferred. At expansion bearings, such spans may be provided with rollers, rockers, or sliding plates. Shorter spans may slide on metal plates with smooth surfaces.

In all cases, design of supports should ensure against accumulation of dirt, which could obstruct free movement of the span, and against trapping of water, which could accelerate corrosion. Beams, girders, or trusses should be supported so that bottom chords or flanges are above the bridge seat.

Self-lubricating bronze or copper-alloy sliding plates, with a coefficient of friction of 0.10 or less, may be used in expansion bearings instead of elastomeric pads, rollers, or rockers. These plates should be at least $^1/_2$ in thick and chamfered at the ends.

Rockers generally are preferred to rollers because of the smaller probability of becoming frozen by dirt or corrosion. The upper surface of a rocker should have a pin or cylindrical bearing. The lower surface should be cylindrical with center of rotation at the center of rotation of the upper bearing surface. At the nominal centerline of bearing, the lower portion should be at least $1^1/_2$ in thick. The effective length of rocker for computing line bearing stress should not exceed the length of the upper bearing surface plus the distance from lower to upper bearing surface. Adequate web material should be provided and arranged to ensure uniform load distribution over the effective length. The rocker should be doweled to the base plate.

Rollers are the alternative when the pressure on a rocker would require it to have too large a radius to keep bearing stress within the allowable. Rollers may be cylindrical or segmental. They should be at least 6 in in diameter. They should be connected by substantial side bars and guided by gearing or other means to prevent lateral movement, skewing, and creeping. The roller nest should be designed so that the parts may be easily cleaned.

Effective bearing area for rockers and rollers equals effective length times effective width. Effective length of bearing area may be taken equal to effective length of rocker, or to roller length plus twice the thickness of the base plate. Effective width of bearing area may be taken as four times the base-plate thickness for rockers, or the distance between end rollers plus four times the base plate thickness for rollers. The vertical load may be assumed uniformly distributed over the effective bearing area, except for eccentricity from rocker travel.

Sole plates and masonry plates should be at least $^3/_4$ in thick. For bearings with sliding plates but without hinges, the distance from centerline of bearing to edge of masonry plate, measured parallel to the longitudinal axis of the supported member, should not exceed 4 in plus twice the plate thickness. For spans on inclines exceeding 1% without hinged bearings, the bottom of the sole plate should be radially curved or beveled to be level.

Elastomeric pads are bearings made partly or completely of elastomer. They are used to transmit loads from a structural member to a support while allowing movements between the bridge and the support. Pads that are not all elastomer (reinforced pads) generally consist of alternate layers of steel or fabric reinforcement bonded to the elastomer. In addition to the reinforcement, the bearings may have external steel plates bonded to the elastomeric bearings. AASHTO prohibits tapered elastomeric layers in reinforced bearings.

The Standard Specifications contain specifications for the materials, fabrication, and installation of the bearings. The specifications also present two methods for their design, both based on service loads without impact and the shear modulus at 73°F. The grade of elastomer permitted depends on the temperature zone in which the bridge is located. The specifications also require that either (1) a positive-slip apparatus be installed and bridge components be able to withstand forces arising from a bearing force equal to twice the design shear force or (2) bridge components be able to sustain the forces arising from a bearing force equal to four times the design shear force. If the shear force exceeds one-fifth the dead-load compressive force, the bearing should be fixed against horizontal movement.

Design should allow for misalignment of girders because of fabrication or erection tolerances, camber, or other sources. It should also provide for subsequent replacement of bearings, when necessary. Also, it should ensure that bearings are not subjected to uplift when in service.

A beam or girder flange seated on an elastomeric bearing should be stiff enough to avoid damaging it. Stiffening may be achieved with a sole plate or bearing stiffeners. I beams and girders placed symmetrically on a bearing do not require such stiffening if the width–thickness ratio b_f/t_f of the bottom flange does not exceed

$$\frac{b_f}{t_f} = 2\sqrt{\frac{F_y}{3.4 f_c}} \tag{10.74}$$

where b_f = total width, in, of the flange
t_f = thickness, in, of flange or flange plus sole plate
F_y = minimum yield strength, ksi, of girder steel
f_c = average compressive stress P/A, ksi, due to dead plus live load, without impact

PTFE pads are bearings with sliding surfaces made of polytetrafluoroethylene (PTFE), which may consist of filled or unfilled sheet, fabric with PTFE fibers, interlocked bronze and filled PTFE structures, PTFE-perforated metal composites and adhesives, or stainless steel mating surfaces. The Standard Specifications contain specifications for the materials, fabrication, and installation of the bearings.

The sliding surfaces of the pads permit translation or rotation by sliding of the PTFE surfaces over a smooth, hard mating surface. This should preferably be made of stainless steel or other corrosion-resistant material. To prevent local stresses on the sliding surface, an expansion bearing should permit rotation of at least 1° due to live load, changes in camber during construction, and misalignment of the bearing. This may be achieved with such devices as hinges, curved sliding surfaces, elastomeric pads, or preformed fabric pads.

PTFE sliding surfaces should be factory-bonded or mechanically fastened to a rigid backup material capable of resisting bending stresses to which the surfaces may be subjected. The surface mating to the PTFE should be an accurate mate, flat, cylindrical, or spherical, as required, and should cover the PTFE completely in all operating positions of the bearing. Preferably, the mating surface should be oriented so that sliding will cause dirt and dust to fall off it.

Pot bearings are used mainly for long-span bridges. They are available as fixed, guided expansion, and nonguided expansion bearings, designed to provide for thermal expansion and contraction, rotation, camber changes, and creep and shrinkage of structural members. They consist of an elastomeric rotational element, confined and sealed by a steel piston and steel base pot. In effect, a structure supported on a pot bearing floats on a low-profile hydraulic cylinder, or pot, in which the liquid medium is an elastomer.

To facilitate rotation of the elastomeric rotational element, either PTFE sheets are attached to the top and bottom of the elastomeric disk or the element is lubricated with a material compatible with the elastomer. To permit longitudinal or transverse movements, the upper surface of the steel piston is faced with a PTFE sheet and supports a steel sliding-top bearing plate. The mating surface of that plate is faced with polished stainless steel.

Pot bearings have low resistance to bending in their plane. Consequently, a sole plate, beveled if necessary, should be provided on top of the bearing and a masonry plate should be installed on the bottom. A member should not be supported on both a pot bearing and a bearing with different properties.

To ensure contact between the piston and the elastomer, minimum load should be at least 20% of the design vertical load capacity.

Pedestals and shoes, if required, usually are made of cast steel or structural steel. Design should be based on the assumption that the vertical load is uniformly distributed over the entire bearing surface. The difference in width or length between top and bottom bearing surfaces should not exceed twice the vertical distance between them. For hinged bearings, this distance should be measured from the center of the pin.

AASHTO recommends that the web plates and angles connecting built-up pedestals and shoes to the base plate should be at least $^5/_8$ in thick.

If pedestal size permits, webs should be rigidly connected transversely to ensure stability of the components. Webs and pinholes in them should be arranged to keep eccentricity to a minimum. The net section through a pinhole should provide at least 140% of the net area required for the stress transmitted through the pedestal or shoe. All parts of pedestals and shoes should be prevented from lateral movement on the pins.

Nuts with washers should be used to hold pins in place. Length of pins should be adequate for full bearing.

Anchor bolts subject to tension should be designed to engage a mass of masonry that will provide resistance to uplift equal to 150% of the calculated uplift due to service loads or 100% of loading combinations for which live load plus impact is increased 100%, whichever is larger. The bolts, however, may be designed for 150% of the basic allowable stress. Resistance to pullout of anchor bolts may be obtained by use of swage bolts or by placing on each embedded end of a bolt a nut and washer or plate. Minimum requirements for number of bolts for each bearing, diameter, and embedment are given in Table 10.27 for ASD and LRFD. The LRFD Specifications do not set minimums.

TABLE 10.27 Minimum Number of Anchor Bolts per Bearing for ASD and LFD

Span, ft	No. of bolts	Diameter, in	Embedment, in
(a) Trusses and girders			
50 or less	2	1	10
51–100	2	$1^1/_4$	12
101–150	2	$1^1/_2$	15
150 or more	4	$1^1/_2$	15
(b) Rolled beams			
All outer spans	2	1	10

10.21 DETAILING FOR WELDABILITY

Overdetailing of weld sizes and joint configurations can cause unnecessary fabrication and in-service problems and higher costs. Some designers believe “more weld metal is better” and “complete-penetration groove welds are better than fillet welds.” However, oversizing welds or specifying joint figurations that are not practical can cause weld defects that are otherwise avoidable.

Whenever possible, designers should allow fabricators to select the type of joint to be used and the size of weld (Fig. 10.13). Include maximum and minimum sizes for fillet welds as follows.

Limitations on Fillet-Weld Size. The maximum size of a fillet weld is the same as the material thickness, up to $^1/_4$ in. For material $^1/_4$ in thick or more, size is limited to $^1/_{16}$ in less than the material thickness, unless the drawings indicate that the weld should be built up to get full throat thickness.

Minimum size of fillet weld is based on the base-metal thickness of the thinner part joined, and single-pass welds must be used. For material $^3/_4$ in thick or less, weld size should be at least $^1/_4$ in. For thicker material, weld size may not be less than $^5/_{16}$ in. Only if the strength requirement exceeds that provided by the minimum size of fillet weld is it necessary to indicate the size of a fillet weld on the drawings. The Bridge Welding Code, ANSI/AASHTO/AWS D1.5, provides adequate assurance of proper weld strength and quality. Letting fabricators select joint details for efficient utilization of their plant setup ensures the most cost-effective fabrication.

The AASHTO specifications also require that the minimum length of a fillet weld be four times its size but at least $1^1/_2$ in. If a fillet weld is subjected to repeated stress or to a tensile force not parallel to its axis, it should not end at a corner of a part or a member. Instead, it should be turned continuously around the corner for a distance equal to twice the weld size (if the return can be made in the same plane). End returns should not be provided around transverse stiffeners. Seal welds should be continuous.

Welding of Box Girders. Poor detailing of a box girder or other type of enclosed member has been another source of fabrication problems and has contributed to adverse in-service performance when designs have not provided properly for fabrication. For example, designers often specify a complete-penetration groove weld for a corner, and the backing bar needed to ensure integrity of the weld is not always installed properly. Backing bars are sometimes left discontinuous, and this soon causes a fatigue crack to initiate. Also, when internal stiffeners are required for a box girder, which is frequently the case for large sections, assembly problems are encountered where welds or backing bars are interrupted at the stiffeners. Figure 10.14 shows a detail with backing bar that is not recommended for a box girder and a preferred arrangement that eliminates both the need for a backing bar and for welding to be done inside the box for attachment of the web to the top plate.

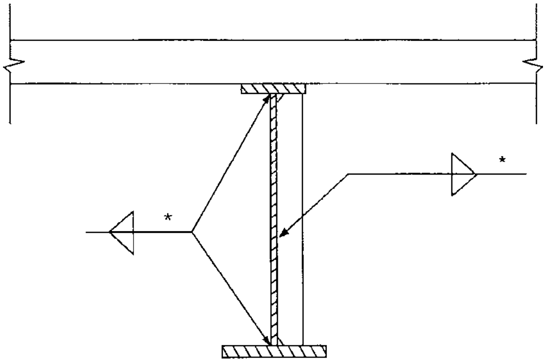


FIGURE 10.13 Symbols indicate welds to be made to a girder. Asterisks indicate that the weld sizes are to be selected by the fabricator. A note should be placed on the drawing to that effect. This does not apply when stress levels control.

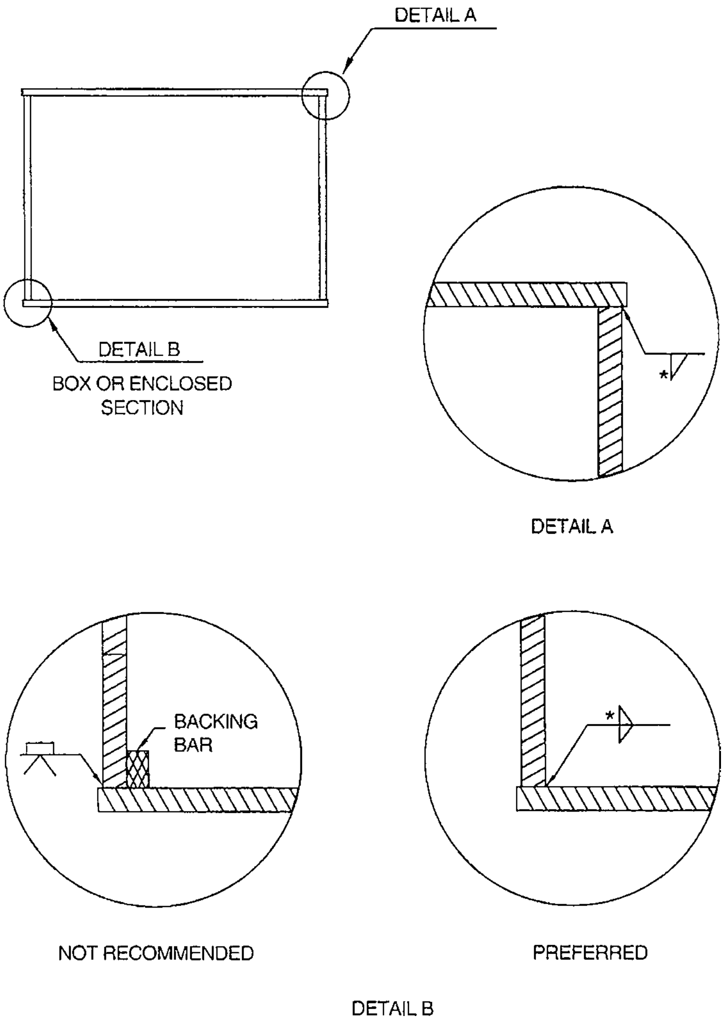


FIGURE 10.14 Corner joints for a box-shape member. Detail A requires a fillet weld between web and top flange. Asterisk indicates that the size of the weld is to be selected by the fabricator. This does not apply when stress levels control. Detail B shows two schemes for welding of the web to the bottom flange, one not recommended and the other preferred.

The assembly procedure requires first welding of the two webs to the bottom flange. For the purpose, continuous fillet welds are placed on one or both sides. Then, the stiffeners are welded to the webs (also to the compression flange if the member will be subjected to bending). Finally, the top flange is connected to the webs with fillet welds. The advantage of this procedure lies in the fact that it is usually practicable to get a fillet weld of better quality, easier to inspect with a nondestructive test, and less expensive than a complete-penetration weld.

Welding of HPS Steels. With the introduction of HPS to the designers inventory of steels, additional weld parameters must be considered. Plates thicker than 2 in are furnished as quenched-and-tempered (Q & T) steel. The LRFD Specification states that the engineer may specify electrode classifications with strengths less than the base metal when detailing fillet welds for Q&T steels. The Bridge

Welding Code, AWS D1.5, also allows use of undermatched fillet welds for all steels where the stress is in tension or compression parallel to the weld axis, and shear on the effective area meets AASHTO design requirements. Although undermatched welds are applicable to any design, it is of particular importance for steels with strengths of 70 ksi and higher.

Rules for Fillet Welds. The following rules are recommended for detailing of fillet welds for all girders, particularly those of HPS.

1. Use only minimum-size fillet welds, except where greater strength is required.
2. Use undermatched fillet welds (consumables for Grade 50 steels) for Grade 70 steels and higher.
3. Use nonweathering consumables for all single-pass fillet welds (AWS D1.5, Art. 4.1.5), even on unpainted structures.
4. For fillet welds joining steels of two different yield points, use consumables applicable to the lower-strength base metal.

10.22 BRIDGE DECKS (ASD AND LFD)

Highway-bridge decks usually are constructed of reinforced concrete. Often, this concrete is made with conventional aggregate and weighs about 150 lb/ft^3. Sometimes, it is made with lightweight aggregate, resulting in 100- to 110-lb/ft^3 concrete. Lightweight aggregate normally consists of slag, expanded shale, or expanded clay.

In some concrete decks, the wearing surface is cast integrally with the structural slab. In others, a separate wearing surface, consisting of asphaltic concrete or conventional concrete, is added after the structural slab has been placed.

In instances where weight saving is important, particularly in movable spans, or in spans where aerodynamic stability is of concern, an open, steel-grid floor is specified. Where compromise is necessary, this grid is partly or completely filled with asphaltic or lightweight concrete to provide protection under the structure or to provide a more suitable riding surface.

For orthotropic-plate structures, it is necessary to provide over the steel deck a wearing surface on which traffic rides. These wearing surfaces are generally of three types: a layered system, stabilized mastic system, or thin combination coatings.

The layered system consists of a steel-deck prime coat, such as zinc metallizing, bituminous-base materials, or epoxy coatings. Over this coat is applied a copper or aluminum foil, or an asphalt mastic, followed by a leveling course of asphalt binder or stabilized mastic, and a surface course of stone-filled mastic asphalt or asphaltic concrete.

The stabilized mastic system consists of a prime coat on the steel, as in the layered system, followed by a layer of mastic, which is choked with rolled-in crushed rock.

Combination coatings contain filled epoxies or alkyd-resin binders in a single coating with silica sand.

A bridge deck serves as a beam on elastic foundations to transfer wheel loads to the supporting structural steel. In orthotropic bridges, the deck also contributes to the load-carrying capacity of longitudinal and transverse structural framing. In composite construction, the concrete deck contributes to the load-carrying capacities of girders. In fulfilling these functions, decks are subject to widely varying stresses and strains, due not only to load but also to temperature changes and strains of the main structure.

In general, bridge decks are designed as flexural members spanning between longitudinal or transverse beams and supporting wheel loads. A wheel usually is considered a concentrated load on the span but uniformly distributed in the direction normal to the span.

Concrete Slabs. The **effective span** S, ft, for a concrete slab supported on steel beams should be taken as the distance between edges of flanges plus half the width of a beam flange.

Allowable Stresses. The allowable compressive stress for concrete in design of slabs is $0.4f_c'$, where f_c' = 28-day compressive strength of concrete, ksi. The allowable tensile stress for reinforcing bars for Grade 40 is 20 ksi and for Grade 60, 24 ksi. Slabs designed for bending moment in accordance with the following provisions may be considered satisfactory for bond and shear.

Bending Moment. Because of the complexity of determining the exact load distribution, AASHTO specifications permit use of a simple empirical method. The method requires use of formulas for maximum bending moment due to live load (impact not included). Two principal cases are treated, depending on the direction in which main reinforcement is placed. The equations are summarized in Table 10.28. In these equations, S is the effective span, ft, of the slab, as previously defined.

For rectangular slabs supported along all edges and reinforced in two directions perpendicular to the edges, the proportion of the load carried by the short span may be assumed for uniformly distributed loads as

$$p = \frac{b^4}{a^4 + b^4} \tag{10.75}$$

For a load concentrated at the center,

$$p = \frac{b^3}{a^3 + b^3} \tag{10.76}$$

where a = length of short span of slab, ft, and b = length of long span of slab, ft. If the length of slab exceeds 1.5 times the width, the entire load should be assumed carried by the reinforcement of the short span. The distribution width E, ft, for the load taken by either span should be determined as provided for other slabs in Table 10.28. Reinforcement determined for bending moments computed with these assumptions should be used in the center half of the short and long spans. Only 50% of this reinforcement need be used in the outer quarters. Supporting beams should be designed taking into account the nonuniform load distribution along their spans.

All slabs with main reinforcement parallel to traffic should be provided with edge beams. They may consist of a slab section with additional reinforcement, a beam integral with but deeper than the slab, or an integral, reinforced section of slab and curb. Simply supported edge beams should be designed for a live-load moment, ft·kips, of $1.6S$ for HS20 loading and $1.2S$ for HS15 loading, where S is the beam span, ft. For positive and negative moments in continuous beams, these values may be reduced 20%.

Distribution reinforcement is required in the bottom of all slabs transverse to the main reinforcement, for distribution of concentrated wheel loads. The minimum amounts to use are the following percentages of the main reinforcement steel required for positive moment:

TABLE 10.28 Live-Load Bending Moments, ft·kips/ft of Width, in Concrete Slabs for ASD and LFD*

Direction of main reinforcement and type of span	Loading	
	HS20	HS15
Perpendicular to traffic ($2 \le S \le 24$):		
Simple spans	$0.5(S + 2)$	$0.37(S + 2)$
Continuous spans	$\pm 0.4(S + 2)$	$\pm 0.3(S + 2)$
Cantilevers, $E = 0.8x + 3.75$†	$16x/E$†	$12x/E$†
Parallel to traffic:		
Simple spans:		
$S \le 50$	$0.900S$	$0.675S$
$50 < S \le 100$	$1.3S - 20$	$0.750(1.3S - 20)$
Continuous spans	By analysis‡	By analysis‡
Cantilevers, $E = 0.35x + 3.2 \le 7$†	$16x/E$†	$12x/E$†

*Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

†x = distance, ft, from load to support.

‡Moments in continuous spans with main reinforcement parallel to traffic should be determined by analysis for the truck or appropriate lane loading. Distribution of wheel loads $E = 4 + 0.06S \le 7$ ft. Lane loads should be distributed over a width of $2E$.

For main reinforcement parallel to traffic, $\dfrac{100}{\sqrt{S}} \leq 50\%$

For main reinforcement perpendicular to traffic, $\dfrac{220}{\sqrt{S}} \leq 67\%$

where S = effective span of slab, ft. When main reinforcing steel is perpendicular to traffic, the distribution reinforcement in the outer quarters of the slab span need be only 50% of the required distribution reinforcement.

Transverse unsupported edges of the slab, such as at ends of a bridge or expansion joints, should be supported by diaphragms, edge beams, or other means, designed to resist moments and shears produced by wheel loads.

The effective length, ft, of slab resisting post loadings may be taken as

$$E = 0.8x + 3.75 \tag{10.77}$$

where no parapet is used, with x = distance, ft, from center of post to point considered. If a parapet is used, $E = 0.8x + 5$.

Steel Grid Floors. For grid floors filled with concrete, the loads distribution and bending moments should be determined as for concrete slabs. The strength of the composite steel and concrete slab should be computed by the transformed-area method (Art 10.15). If necessary to ensure adequate load transference normal to the main grid elements, reinforcement should be welded transverse to the main steel.

For open-grid floors, a wheel load should be distributed normal to the main bars over a distance equal to twice the center-to-center spacing of main bars plus 20 in for H20 loading, or 15 in for H15 loading. The portion of the load assigned to each bar should be uniformly distributed over a length equal to the rear-tire width (20 in for H20 loading and 15 in for H15). The strength of the section should be determined by the moment-of-inertia method (Art. 10.14). Supports should be provided for all edges of open-grid floors.

10.23 ELIMINATION OF EXPANSION JOINTS IN HIGHWAY BRIDGES

At expansion bearings and at other points where necessary, expansion joints should be installed in the floor system to permit it to move when the span deflects or changes length. If apron plates are used, they should be designed to bridge the joint and prevent accumulation of dirt on the bridge seats. Preferably, the apron plates should be connected to the end floorbeam. For amount of movement to provide for, see Art. 10.5.2. However, jointless bridges have many advantages and should be considered where possible.

Short-span bridges usually have expansion joints at one or both abutments. Longer-span structures usually have such joints at pier or off-pier hinges. Although these joints may relieve some forces caused by restraint of thermal movements, the joints have been a major source of bridge deterioration and poor ridability. The LRFD Specifications acknowledge that “Completely effective joint seals have yet to be developed for some situations. . . .” To provide more durable bridges, the goal in design should be to minimize the number of joints. One way to do this for multiple-span bridges is to use continuous beams or girders. Another, more general, alternative is to eliminate joints completely.

Some states permit jointless, or integral, steel-girder bridges with spans up to about 400 ft or longer. With this type of construction, restriction of the change in bridge length due to maximum temperature change induces longitudinal forces at fixed piers and abutments. This must be taken into account in design of substructures. Experience has shown, however, that the effect of these forces on superstructure design is negligible and that, with proper detailing, substructure design is relatively unaffected.

Tennessee is a major user of jointless steel-girder bridges for spans of 400 ft or more. Through experience, they have developed details that are able to resist thermal forces and movements (Fig. 10.15), thus eliminating leaking bridge joints. Tennessee has successfully completed a two-span continuous bridge 473 ft long with integral abutments at each end.

The Standard Specifications specify that movement calculations for integral abutments take into account not only temperature changes but also creep of the concrete deck and pavements. The abutments should be designed to sustain the forces generated by restraint to thermal movements developed by the pressures of fills behind the abutments. (The Specifications prohibit use of integral abutments constructed on spread footings keyed into rock.) Approach slabs should be connected directly to abutments and wingwalls, to prevent intrusion of water behind the abutments. Nevertheless, means should be provided for draining away water that may get entrapped.

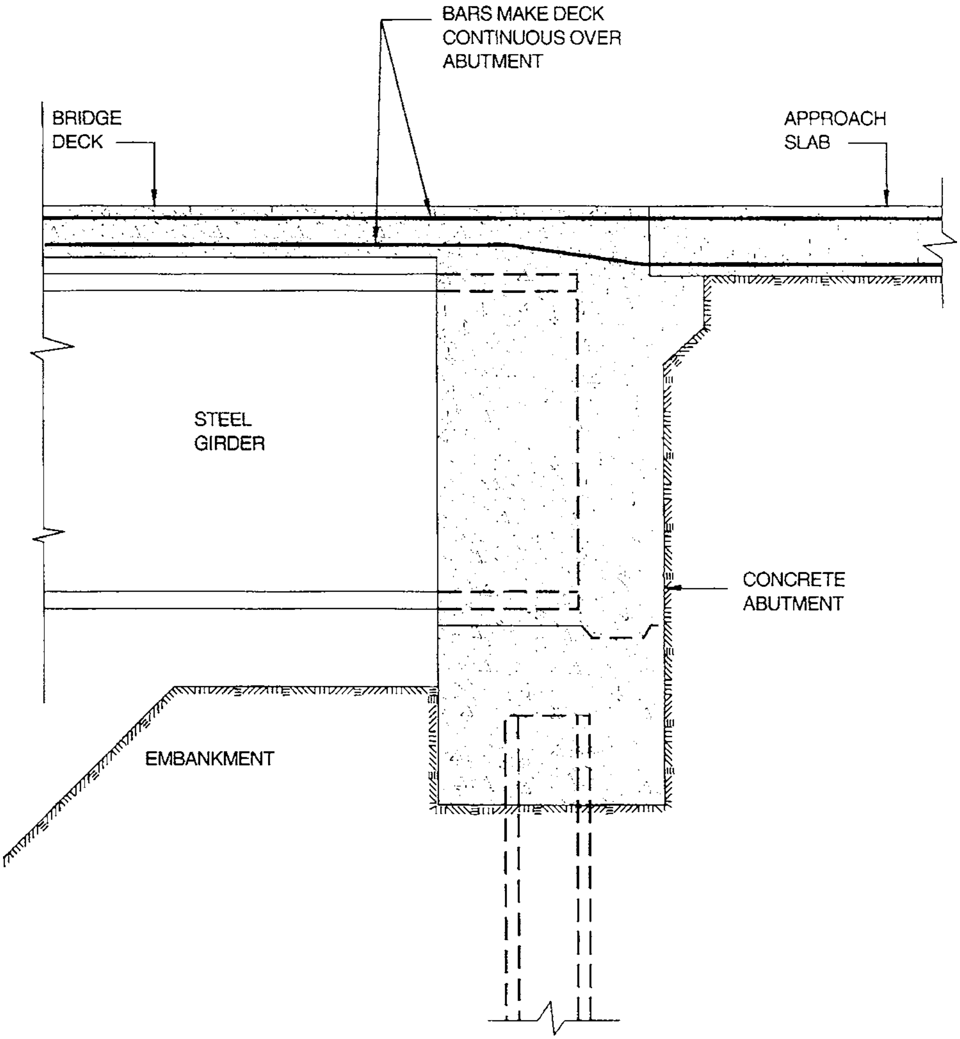


FIGURE 10.15 Details for an integral abutment.

The Standard Specifications also require that details comply with recommendations in Technical Advisory T5140.13, Federal Highway Administration. These recommendations include the following.

Steel bridges with an overall length less than 300 ft should be constructed continuously and, if unrestained, have integral abutments. ("An unrestrained abutment is one that is free to rotate, such as a stub abutment on one row of piles or an abutment hinged at the footing."—"Structure Memorandum," State of Tennessee.) Greater lengths may be used when experience dictates that such designs are satisfactory.

In the area immediately behind integral abutments, traffic will compact the fill where it is partly distributed by abutment movement, if not prevented from doing so. For the purpose, approach slabs should be provided to span this area. The span length should be at least equal to a minimum of 4 ft for bearing on the soil plus the depth of the abutment (based on the assumption of a 1:1 slope from the bottom of the rear face of the abutment.) The Advisory suggests that a practical slab length is 14 ft.

The Advisory recommends that approach slabs be designed for live-load bending movements as indicated for the case of main reinforcement parallel to traffic in Table 10.28, with S = slab length minus 2 ft.

The Advisory also recommends that the slabs be anchored by steel reinforcement to the superstructure. In addition, positive anchorage should be provided between integral abutments and the superstructure. Figure 10.15 is an example of such construction.

The Advisory calls attention to a detail used by North Dakota that it considers desirable. To accommodate pavement growth and bridge movement, the state inserts a roadway expansion joint 50 ft away from the bridge.

Properly detailed and constructed, jointless bridges eliminate the maintenance that would be required if expansion joints were used, especially corrosion and deterioration of substructure and superstructure because of leakage. Also, jointless bridges provide better ridability. As a bonus, the cost of joints is eliminated. The LRFD Specifications encourage the use of jointless bridges to improve "rideability" of the roadway surface, but provide minimal design guidance. However, comprehensive design and detailing provisions for bridges with integral abutments are available from the American Iron and Steel Institute (AISI), as *Integral Abutments for Steel Bridges.* A design procedure for the piles supporting the integral abutment is included.

Where foundation conditions are not considered acceptable for integral abutment bridges, **semi-integral** abutments are acceptable, within the same length limitations. A semi-integral abutment is virtually identical to an integral abutment, except that there is a horizontal joint separating the backwall and beam from the pile footing. Thus, bridges with battered piles or rock foundations are candidates for semi-integral abutments. Semi-integral abutments are also used effectively in bridge rehabilitations to eliminate joints.

10.24 BRIDGE STEELS AND CORROSION PROTECTION

One of the most important decisions designers have to make is selection of the proper grade of steel and corrosion-protection system. These should not only meet structural needs but also provide an economical structure capable of long-term, low-maintenance performance.

Specifications of the AASHTO recognize structural steels designated M270 with a specified grade. These are equivalent to ASTM A709 steels, Table 1.2, except for the manner in which notch toughness is specified (mandatory in M270, as a supplementary requirement in A709). AASHTO M270 steels are prequalified for welded bridges.

Designers should have available AASHTO "Standard Specifications for Transportation Materials and Methods of Sampling and Testing," Part 1, "Specifications," and Part 2, "Tests," to ensure that appropriate material properties are specified for their designs.

High-performance steels (HPS) are the newest additions to the family of bridge steels. They are being used increasingly to improve reliability and reduce cost, with approximately 200 bridges in service in 2005. The initial grade, HPS70W, with a specified minimum yield stress of stress of 70 ksi, has been used most. Introduced later, HPS50W with a specified minimum yield stress of 50 ksi, has also become popular. HPS100W, with a specified minimum yield stress of 100 ksi, is available to reduce thickness where members are highly loaded. To qualify as HPS, the material has to provide improved weathering characteristics and significantly higher impact toughness. HPS has a corrosion index, *I*, of 6.5 and higher, thus providing increased resistance to weathering over earlier grades of steels designated as weathering (*W*). Weathering grades are defined as having a corrosion index *I* of 6.0 and higher as calculated using ASTM Standard G101. In addition, Charpy V-notch impact properties for this steel usually exceed 100 ft·lb at −10°F.

10.24.1 Minimum Steel Thickness

Because structural steel in bridges is exposed to the weather, minimum thickness requirements are imposed on components to obtain a long life despite corrosion. Where steel will be exposed to unusual corrosive influences, the component should be increased in thickness beyond required thickness or specially protected against corrosion.

In highway bridges, structural steel components, except railings, fillers, and webs of certain rolled shapes, should be at least $^5/_{16}$ in thick. Web thickness of rolled beams or channels should be at least 0.23 in (0.25 in for LRFD). Closed ribs in orthotropic-plate decks should be at least $^3/_{16}$ in thick (0.25 in for LRFD). Fillers less than $^1/_4$ in thick should not be extended beyond splicing material. In addition, minimum thickness may be governed by slenderness ratios (Table 10.25) or maximum width–thickness or depth–thickness ratios (Table 10.26).

10.24.2 Weathering Steels

A preferred way to achieve economy for bridges is to use steel of a **weathering** grade when conditions permit. This is a type of steel that has enhanced atmospheric corrosion resistance *when properly used* and does not require painting under most conditions. Although it costs slightly more per pound than other steels of equivalent grade, its initial cost and life-cycle cost is usually less than that of painted structural steel. The weathering grades are available only with yield points of 50 ksi and higher. Before selecting a weathering steel, designers should determine the corrosivity of the environment in which the bridge will be located as a first step. This will determine whether the use of an unpainted steel of grade 50W, 70W, HPS70W, or 100W (Art. 1.1.5) is appropriate. These steels provide the most cost-effective grade that can be used in most situations and have proven to be capable of excellent performance even in areas where deicing salts are used. However, use of good detailing practices, such as jointless bridges, is imperative to assure adequate performance (Art. 10.23)

The Federal Highway Administration "Guidelines for the Use of Unpainted Weathering Steel," to ensure a long-term and adequate performance of unpainted steels, recommends the following:

If the proposed structure is to be located at a site with any of the environmental or location characteristics noted below, use of uncoated weathering-grade steels should be considered with caution. A study of both the macroenvironment and microenvironment by a corrosion consultant may be required. In all environments, designers must pay careful attention to detailing, specifically as noted in the following recommendations for design details. Also, owners should implement, as a minimum, the maintenance actions as noted in the following.

Environments to be treated with caution include marine coastal areas; regions with frequent high rainfall, high humidity, or persistent fog; and industrial areas where concentrated chemical fumes may drift directly onto structures.

Locations to be treated with caution include grade separations in tunnel-like conditions, where concentration of vehicle exhausts may be highly corrosive; also, low-level water crossings, with clearance of 10 ft or less over stagnant, sheltered water or 8 ft or less over moving water.

Design details for uncoated steel in bridges and other highway structures require careful consideration of the following:

1. Elimination of bridge joints where possible.
2. If expansion joints are used, they must be able to control water that comes on the deck. A trough under the deck joint may serve to divert water away from vulnerable elements.

3. Painting all superstructure steel within a distance of $1^1/_2$ times the depth of girder from bridge joints.
4. Avoiding use of welded drip bars where fatigue stresses may be critical.
5. Minimizing the number of bridge-deck scuppers.
6. Eliminating details that serve as water and debris "traps."
7. If box girders are used, they should be hermetically sealed, when possible, or provided with weep holes to allow proper drainage and circulation of air. All openings in boxes that are not sealed should be covered or screened.
8. Protecting pier caps and abutment walls to minimize staining.
9. Sealing overlapping surfaces exposed to water, to prevent capillary penetration of moisture.

Maintenance actions advisable include the following:

1. Implementing procedures designed to detect and minimize corrosion.
2. Controlling roadway drainage by diverting roadway drainage away from the bridge structure, cleaning troughs or resealing deck joints, maintaining deck drainage systems, and periodically cleaning and, when needed, repainting all steel within a minimum distance of $1^1/_2$ times the depth of the girder from bridge joints.
3. Regularly removing all dirt, debris, and other deposits that trap moisture.
4. Regularly removing all vegetation and other matter that can prevent the natural drying of wet steel surfaces.
5. Maintaining covers and screens over access holes.

The preceding recommendations are applicable to all structures, painted or unpainted, to ensure satisfactory performance. Unpainted structures that have been in existence for 30 years or more in environments consistent with these recommendations have provided excellent service, testifying to the adequacy of the weathering grades of steel. ("Performance of Weathering Steel in Highway Bridges—A Third Phase Report," American Iron and Steel Institute, Washington, D.C., 1995.)

10.24.3 Paint Systems

Where weathering grades of steel are not appropriate, only high-performance paint systems should be specified for corrosion protection. Designers should be aware, however, that recommendations for paint systems change periodically, due primarily to the need for consideration of environmental impacts. Lead-based paints, for example, are no longer acceptable due to their health hazard. Also, concern for the effect of volatile organic compounds on the ozone in the atmosphere has caused a change from mineral-based to water-based paints. Consequently, designers should ensure that only current technology is specified in contract documents.

The AASHTO "Guide for Painting Steel Structures" provides state-of-the-art information for the painting of new bridge steels, as well as paint removal and repainting of existing steel bridges.

CHAPTER 11

RAILROAD BRIDGE DESIGN CRITERIA

John F. Unsworth, P.Eng.
Manager, Structural Planning and Design
Canadian Pacific Railway
Calgary, Alberta, Canada

11.1 STANDARD SPECIFICATIONS

The primary purpose of railroad bridges is to safely and reliably carry freight and passenger train traffic within the railroad operating environment. Recommended practices for the design of railroad bridges are developed and maintained by the American Railway Engineering and Maintenance-of-Way Association (AREMA), 8201 Corporate Drive, Suite 1125, Landover, MD, 20785-2230. Recommended practice for the design of fixed railroad bridges is outlined in Part 1, Design, and Part 5, Special Types of Construction, in Chap. 15, Steel Structures, of the AREMA *Manual for Railway Engineering* (MRE). Recommended practice for the design of movable railroad bridges is outlined in Part 6, Movable Bridges, in Chap. 15, Steel Structures, of the AREMA *MRE*. The information in Chap. 15 is prepared and continuously reviewed and updated by AREMA Committee 15. Chapter 15 provides detailed recommendations for the design of steel railway bridges for spans up to 400 ft in length, standard-gage track (56.5 in), and North American freight and passenger equipment at speeds up to 70 and 90 mi/h, respectively. The recommendations may be used for longer span bridges with supplemental requirements.

11.2 DESIGN METHOD

Elastic analysis procedures are usually used for steel railroad bridges, and Chap. 15 of the AREMA *MRE* provides recommendations for allowable stress design. The design service life of railroad bridges is generally considered to be about 80 years. Serviceability criteria, such as fatigue and deflection, often govern the design of modern railway bridges.

11.3 RAILROAD OPERATING ENVIRONMENT

Railroad bridge designers must be cognizant of the railroad operating practices at the location of any new bridge being designed, and of specific issues concerning railway bridge behavior and maintenance.

Most new railroad bridges are constructed on existing routes and on existing alignments. Construction methodologies that minimize the interference to normal rail traffic, enable simple erection, and are cost-effective must be considered during the design process. Often, in order to minimize interruption to railroad traffic, innovative construction techniques such as sliding spans into position on falsework constructed adjacent to existing bridges, erection of spans from river barges, and the use of large cranes must be considered. These methodologies may add cost to the reconstruction project that are acceptable in lieu of the costs associated with extended interruption to railway traffic. The following should also be carefully considered in conjunction with railroad bridge design:

- Expected service life. Railroads have many old structures in the operating inventory and often expect bridges with service lives of 80 or more years.
- Simple span construction is generally preferred by railroads, due to their relative ease of erection in comparison to continuous spans or spans requiring field splicing.
- Serviceability criteria, such as fatigue and deflection, are important in steel railway bridge design. Railroads may limit span deflections based on operating conditions. Welded connections and fatigue-prone details must be avoided in the high-magnitude cyclical live-load stress-range regime that railroad bridges are subjected to. Railway equipment, such as long unit trains (some with up to 150 cars), can create a significant number of stress cycles, particularly on bridge members with relatively small influence lines.
- Dynamic amplification (impact) is very large in railroad structures.
- The performance of railway bridges in seismic events. Steel railway bridges have generally performed well in seismic events because of the type of construction usually employed (i.e., relatively light superstructures, large bridge seat dimensions, and substantial bracing and anchor bolts used to resist longitudinal and lateral live loads).
- Constructability and maintainability are of critical importance in the railroad operating environment.

11.4 DESIGN CONSIDERATIONS

11.4.1 Materials

Structural steel [including notch toughness requirements for both main load-carrying members and fracture-critical members (FCMs)] and fastener material requirements are outlined in Chap. 15 of the AREMA *MRE*. Chapter 15 also contains recommendations for riveted fasteners. However, because riveted construction is generally limited to historical restoration of existing structures, and modern steel railway bridge design uses bolted and welded connections, discussion of riveted connection design is not covered here. Modern railway bridge engineers often use high-strength atmospheric corrosion-resistant steels in their designs (weathering steels). These modern steels have the increased corrosion resistance, high strength, and high toughness characteristics desirable for railroad bridge design, construction, and maintenance.

Steel members should not have any components less than 0.335 in thick (with the exception of fillers), but some railroad companies specify a greater minimum material thickness (often $^3/_8$ or $^1/_2$ in). Where components are subject to corrosive conditions, they should be made thicker than otherwise required or protected against corrosion. Gusset plates used to connect chord and web members in trusses should be proportioned for the force transmitted, but should not be less than 0.50 in thick.

11.4.2 Types of Steel Spans

Rolled or welded-beam deck spans are typically used for spans less than 50 ft in length. Bolted or welded deck or through-plate girder spans are typically used for spans less than 150 ft in length. Bolted or welded deck or through-truss spans are typically used for spans greater than 150 ft in length.

11.4.3 Types of Steel Span Decks

On **open bridge decks,** the deck ties (usually timber) are directly supported on steel structural elements (i.e., stringers, beams, girders). Dead load is relatively small but dynamic amplification can be considerable, as the track modulus is discontinuous (generally, open-deck bridges are more rigid than the approach-track structure). Continuous welded rail (CWR) can create differential movements that cause damage to open bridge decks. Bridge tie sizes can be large for supporting elements spaced far apart, and careful consideration to the deck fastening systems is required. Open bridge decks are often the least costly deck system and are free-draining, but generally require more maintenance during the deck service life. Most railroads have open bridge deck standards based on the design criteria recommended by AREMA *MRE* Chap. 7, Timber Structures.

On **ballasted bridge decks,** track ties are laid in ballast that is supported by steel or concrete decks. The deck design may be composite or noncomposite. Composite steel-and-concrete construction is structurally efficient, but may not be feasible due to site constraints (i.e., need for falsework and site concrete supply). Noncomposite concrete-and-steel deck systems should be considered when site and installation-time constraints exist in the particular railroad operating environment. Dead load can be considerable, but dynamic effects are reduced and train ride quality is improved due to a relatively constant track modulus. Ballasted decks generally require less maintenance and are often used because of curved track geometry, or when the bridge crosses over a roadway or sensitive waterway. The railway can generally utilize existing track maintenance equipment on ballasted deck structures. Ballasted deck structures also allow for easier track elevation changes, but drainage must be carefully considered. Drainage of the deck is often accomplished by sloping the deck surface to scuppers or through drains. In some cases the through drains are connected to conduits to carry water to the ends of spans. In particular, deck drainage at the ends of spans using expansion plates under the ballast between decks must be carefully considered. Most railroads have standards for minimum ballast depth and waterproofing requirements. AREMA *MRE* Chap. 8, Concrete Structures and Foundations, contains information on recommended deck waterproofing systems.

Direct-fixation decks are most often used for passenger rail service, with rails fastened directly to steel or concrete decks. Dead load and structure depth are reduced, but dynamic forces can be large. Direct-fixation decks are generally not used in freight rail bridges and require careful design and detailing to avoid failure in the railroad high-stress regime.

11.4.4 Bridge Stability

Girders and trusses must be spaced to prevent overturning instability. The spacing should be greater than $^1/_{20}$ of the span length for through spans and greater than $^1/_{15}$ of the span length for deck spans. The spacing between the center of pairs of beams, stringers, or girders should not be less than 6.5 ft. The stability of spans and towers should be calculated using a live load, without impact, of 1200 lb/ft. On multiple-track bridges this live load should be placed on the most leeward track on the bridge.

Cross frames and diaphragms (Art. 11.9.8) in beam and girder spans requiring lateral bracing (Art. 11.9.8) should be checked with a single line of wheel loads, including impact, at a 5-ft eccentricity from track centerline. This load represents possible derailment effects, and the applicable allowable stresses (Art. 11.6) may be increased by 50% for this stability check.

11.4.5 Bridge Framing Details

Open-deck through-plate girder, through-truss, and some deck truss spans usually contain floor systems comprised of longitudinal stringers and transverse floorbeams (Fig. 11.1*a*).

Ballasted-deck through-plate girder spans generally have the concrete or steel plate decks supported on closely spaced transverse floorbeams framing into the main girder (Fig. 11.1*b*). In some cases, such as through-truss spans, stringers with less closely spaced transverse floorbeams are used.

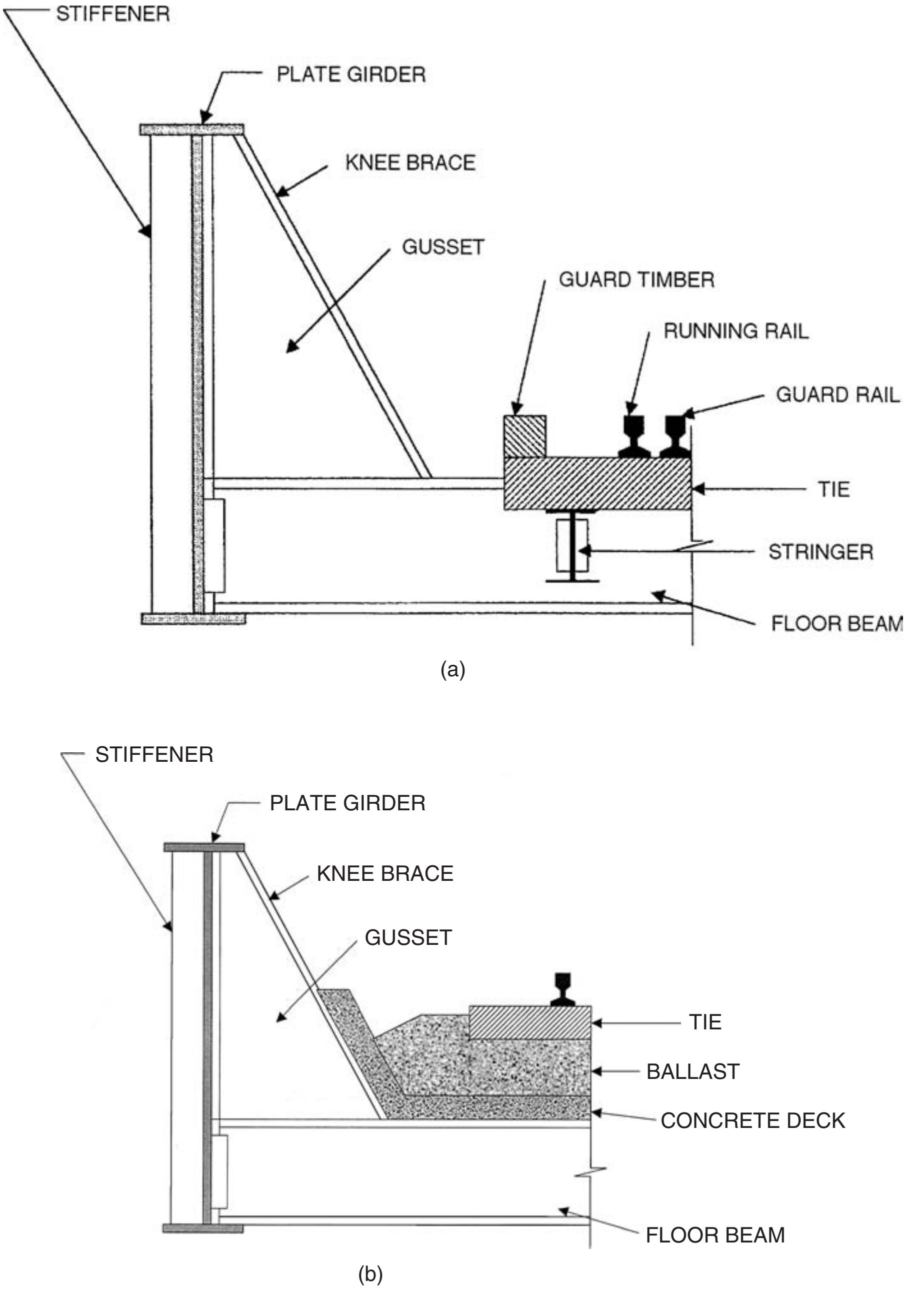


FIGURE 11.1 Part section of through-girder railway bridges. (*a*) Open-deck construction. (*b*) Ballast-deck construction.

Stringers should be placed parallel to the longitudinal axis of the bridge, and transverse floorbeams should be perpendicular to main girders or trusses. Stringers are usually framed into the floorbeams and have intermediate cross frames or diaphragms. The recommendations for cross frames and diaphragms for both open-deck and ballasted-deck construction are given in Art. 11.9. Floorbeams should frame into the main girders or trusses such that lateral bracing may be connected to both the floorbeam and main member.

End connections of stringers and floorbeams should generally be made with two angle elements and designed to ensure flexibility of the connection in accordance with the structural analysis used. The angles should be made as long as permitted by the beam flanges. If bracket or shelf angles are used in the connection during erection, their load-carrying capacity should be ignored when designing the end connection for specified loads. Welded end connections are not permitted on the flexing leg of connections. Connection angles should be not less than $^1/_2$ in thick and the outstanding leg should be 4 in or greater in width. For stringers, in open- and ballast-deck construction, the gage distance, in, from the back of the connection angle to the first line of fasteners, over the top one-third of the depth of the stringer, should be not less than $\sqrt{Lt/8}$, where L is the length of the stringer span, in, and t is the angle thickness, in.

Railway bridge spans should have end floorbeams, or other members, designed to permit lifting of the superstructure without producing stresses in excess of the basic allowable stresses by 50%.

The main material of floorbeam hangers in through-truss construction should not be coped or notched. The webs of built-up hangers should be of solid or perforated plates, or lacing bars. The main material for floorbeam hangers should not be less than 0.50 in thick.

Multiple beams, girders, and stringers should be arranged to distribute live load evenly to all the members.

Typically, the span-to-depth ratio for railway bridges is about 12:1.

11.4.6 Deflections

Simple span deflection should be computed for the live load plus impact that produces the maximum bending moment at midspan. The maximum deflection should not exceed $^1/_{640}$ of the span length, center-to center of supports. The gross moment of inertia may be used for prismatic flexural members. Railroad companies may limit deflections to values less than $^1/_{640}$ of the span length, based on their operating practices.

11.4.7 Clearances

Appropriate clearances must be provided for in the design of all structures. Through-girder and through-truss bridges should provide a minimum of 9.0 ft horizontal side clearance, measured from the centerline of track. A minimum vertical distance of 23.0 ft above the plane of the top of the high rail should be provided in through-truss bridges. The designer should consult AREMA *MRE* Chap. 28, Clearances, and the railroad company concerning clearance requirements for a particular bridge location.

11.4.8 Skewed Bridges

Many railroads have specific provisions regarding skew angle and type of construction for skewed railway bridges. Track support at the ends of skew bridges should be perpendicular to the track.

11.4.9 Camber

Rolled beam and plate girder spans less than 90 ft in length need not be cambered, unless specified by the railroad company. Plate girder spans in excess of 90 ft long should be cambered for dead-load deflection. Trusses should be cambered at each panel point for dead-load deflection plus the deflection from a uniform live load of 3000 lb/ft of track.

11.4.10 Safety Appliances

Safety devices required by the railroad and by regulations must be provided for in the design. Safety devices may include such items as walkways, hand railings, vandal fences, ladders, grab-irons, bridge end posts, clearance signs, refugee booths, stanchions, and fall-protection fittings. Most railroad companies have policies, based on federal and state regulations, regarding safety appliance requirements for bridges. The designer should consult with the railroad company concerning specific safety appliances that are required.

11.4.11 Bridge Bearings

Span expansion bearings are subject to the following considerations. Design of steel railway bridges should allow for a change in length due to temperature change of 1 in per 100 ft of span. Also, provision should be made for change of length from live load. In truss spans more than 300 ft long, allowance should be made for expansion of the floor system. The use of high-adhesion locomotives may justify a more rigorous review of floor system expansion in even shorter spans. Spans 50 ft or greater in length should have expansion bearings that also accommodate rotation due to span deflection. For specific recommendations concerning expansion bearing design and fabrication criteria, see AREMA *MRE* Chap. 15, Part 10 and Part 11.

Span fixed bearings that accommodate rotation due to span deflection are required for spans 50 ft or greater in length. For specific recommendations concerning fixed bearing design and fabrication criteria, see AREMA *MRE* Chap. Chap. 15, Part 10 and Part 11.

Bearings for viaduct towers, at the bases of columns, should be designed to allow for expansion and contraction of the tower or bent bracing system.

11.4.12 Protective Coatings

Steel bridges fabricated with modern atmospheric corrosion-resistant steels (weathering steel) are often not coated, with exception of specific areas that may be galvanized, metallized, or painted for localized corrosion protection (e.g., bearing areas, top flanges of open-deck spans). Where required, modern multiple-coat painting systems are used for steel railway bridge protection, and many railroads have developed their own cleaning and painting guidelines or specifications.

11.4.13 Secondary Stresses in Members

Secondary stresses should be minimized by design and details. Secondary stresses exceeding 4 ksi for tension members and 3 ksi for compression members must be superimposed on, and considered as, primary stresses.

11.4.14 Truss Web Members

To ensure that truss web members and their connections do not reach their capacity before other members and connections of the truss, it is recommended that truss web members and their connections be proportioned using a 33% increase in allowable stresses for total web member and connection forces, determined using the notional live load that increases the total stress in the most highly stressed chord by one-third.

11.5 DESIGN LOADINGS

Bridges must be designed to carry the specified dead loads, live loads, and impact, as well as centrifugal, wind, other lateral loads, loads from continuous welded rail, longitudinal loads, and earthquake loads. The forces and stresses from each of these specified loads should be a separate part of the design calculations. Also, because rail cars have changed in size and weight over the years and frequently are run in unit consists, the designer should be aware of live loadings that may be more severe than those used in some specifications (Art. 11.5.2).

11.5.1 Dead Loads

Dead loads should be calculated based on the weight of the materials actually specified for the structure. The dead load for rail and fastenings may be assumed as 200 lb/ft of track. Unit weights of other materials may be taken as follows:

Material	Weight, lb/ft^3
Timber	60
Ballast	120
Concrete	150
Steel	490

Note that walkway construction may add to the dead load. Also, when long-body rail castings, such as expansion joints, are specified for a bridge, the castings should be supported only on one span of the stringers.

11.5.2 Live Load

Railroad bridges have been designed for many years using specified Cooper E loadings. See Fig. 11.2*a* for the wheel arrangement and the trailing load for the Cooper E80 loading, which includes 80-kip axle loads on the drivers. This configuration can be moved in either direction across a span to determine the maximum moments and shears. With the continuing increase in car axle loads, AREMA has also adopted the **alternate live load** on four axles shown in Fig. 11.2*b*. It recommends that bridge

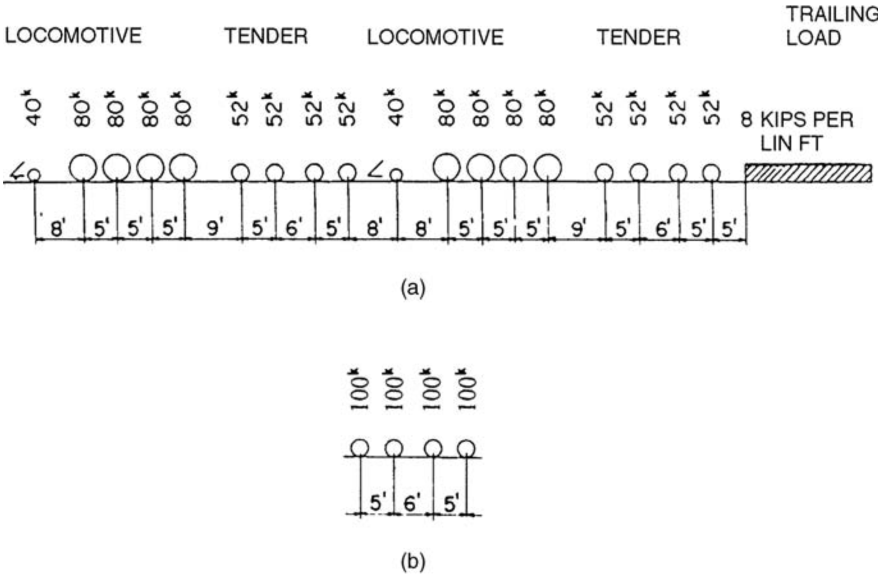


FIGURE 11.2 Loadings for design of railway bridges. (*a*) Cooper E80 load. (*b*) Alternate live load on four axles. (Adapted from *AREMA Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.)

design be based on the E80 or the alternate loading, whichever produces the greater stresses in the member. A table of live load moments, shears, and reactions for both the E80 and the alternate loading may be found in the Appendix to Chap. 15 of the AREMA *MRE*. The table values are presented in terms of wheel loads (one-half of an axle load).

Some railroads elect to use loadings other than E80 in some cases. Such loadings may be directly proportioned from the E80 loading according to the axle load on the drivers. For example, a railroad specifying a new through-truss or girder span may specify an E100 loading for the floor system and hangers, and an E80 loading for the rest of the structure.

11.5.3 Load on Multiple-Track Structures

To account for the effect of multiple tracks on a structure, the proportion of full live load on the tracks should be taken as follows:

Two tracks: full live load

Three tracks: full live load on two tracks, one-half live load on third track

Four tracks: full live load on two tracks, one-half live load on one track, one-quarter live load on remaining track

The tracks selected for these loads should be such that they produce the maximum live-load stress in the member under consideration. For bridges carrying more than four tracks, the track loadings should be specified by the railroad.

11.5.4 Impact Load

Impact loads, I, are expressed as a percentage of the specified axle load and should be applied vertically at the top of the rail. For open-deck bridge construction, the percentages are obtained from the applicable equations given below. For ballast-deck bridges designed according to specifications, use 90% of the impact load given for open-deck bridges.

For rolling equipment without hammer blow (diesel or electric locomotives, tenders, rolling stock):

For $L < 80$ ft:
$$I = RE + 40 - \frac{3L^2}{1600} \tag{11.1}$$

For $L \geq 80$ ft:
$$I = RE + 16 + \frac{600}{L - 30} \tag{11.2}$$

For steam locomotives (hammer blow):

For girders, beam spans, stringers, floor beams, floor beam hangers, and posts of deck trusses that carry floor beam loads only:

For $L < 100$ ft:
$$I = RE + 60 - \frac{L^2}{500} \tag{11.3}$$

For $L \geq 100$ ft:
$$I = RE + 10 + \frac{1800}{L - 40} \tag{11.4}$$

For truss spans:
$$I = RE + 15 + \frac{4000}{L - 25} \tag{11.5}$$

where L = length, ft, center to center of supports for stringers, transverse floorbeams without stringers, main longitudinal girders, and trusses, *or*

L = length, ft, of the longer adjacent supported stringers, longitudinal beam, girder or truss for impact in floorbeams, floorbeam hangers, subdiagonals of trusses, transverse girders, supports for longitudinal and transverse girders, and viaduct columns

The term *RE* in the above equations represents the rocking effect that is superimposed on the vertical effects given by the last two terms in the impact equations. It is created by the transfer of load from the wheels on one side of railway equipment to the other from periodic lateral rocking of the equipment. *RE* for a given member should be calculated as the reaction to a vertical force couple applied at the top of the rails from 20% of the wheel load, without vertical impact, acting upward on one rail and downwards on the other.

On multiple track bridges, the impact should be applied as follows:

When load is received from two tracks:

For $L \leq 175$ ft: full impact on two tracks

For 175 ft $\leq L \leq$ 225 ft: full impact on one track and a percentage of full impact on the other track as given by $(450\text{-}2L)$

For $L > 225$ ft: full impact on one track and no impact on other track

When load is received from more than two tracks:

For all values of *L:* full impact on any two tracks

For fatigue design, use the mean impact expressed as a percentage of the values given by the above equations, as follows:

Loaded length of member, ft	Mean impact, %
$L \leq 10$ (and no load sharing)	65
Truss hangers	40
Other truss members	65
Beams, stringers, girders, and floorbeams	35

Mean impact should be taken as 100% for all members with a loaded length not exceeding 80 ft and where railroad operating practice involves operating trains with equipment with a large percentage of flat or out-of-round wheels; or where track conditions are poor.

11.5.5 Longitudinal Forces

Longitudinal forces due to train braking and acceleration are considerable in modern railroad equipment (i.e., new braking systems and high-adhesion locomotives). The longitudinal force *LF* (kips) for Cooper's E80 loading is as follows.

From braking applied at 8 ft above top of rail:

$$LF_B = 45 + 1.2(L) \tag{11.6}$$

From traction applied at 3 ft above top of rail:

$$LF_T = 25\sqrt{L} \tag{11.7}$$

where L = length, ft, of the portion of the bridge under consideration. For specified live load other than E80, longitudinal force should be scaled proportionally.

Longitudinal force should be distributed to the various components of supporting structures in accordance with their relative stiffness. On multiple-track structures, longitudinal forces are applied in the same manner as other live loads. The Commentary to Chap. 15 of the AREMA *MRE* provides considerable information regarding the application of longitudinal forces for the design of steel railway bridges.

11.5.6 Centrifugal Force

On curves where a maximum design speed is not specified, a centrifugal force corresponding to 15% of each axle load without impact should be applied horizontally through a point 8 ft above the top of rail, measured in a line perpendicular to the plane at the top of the rails and equidistant from each rail, using a rail superelevation of 6 in.

On curves where a maximum design speed is specified, a centrifugal force corresponding to the percentage, C, of each axle load without impact should be applied horizontally through a point 8 ft above the top of rail measured in a line perpendicular to the plane at the top of the rails and equidistant from each rail, using the rail superelevation specified by the railroad.

$$C = 0.00117S^2D \tag{11.8}$$

where C = percentage of axle load
S = speed, mph
D = degree of curve = $5729.65/R$
R = radius of curve, ft

The horizontal centrifugal force stresses lateral bracing, cross frames, diaphragms, and bearings. The overturning moment associated with the application of the centrifugal force above the track will increase the vertical live load in members to the outside of the curve. Full impact is to be applied to the resulting vertical live load. Members inside the curve are recommended to be of the same section as those outside the curve and, therefore, the reduction in live load from centrifugal effects is neglected for these members. The vertical axle loads are shifted toward the inside of the curve in relation to the track superelevation, and this will decrease the vertical load on members outside of the curve. Both the increased live load from centrifugal effects and the decrease in live load from superelevation effects should be considered in determining the vertical live load axle forces on bridges with curved track. AREMA *MRE* Chap. 5, Track, provides information concerning the relationship of curvature, speed, and superelevation.

11.5.7 Lateral Loads from Equipment

In the design of bracing systems, the lateral force to provide for the effect of the nosing of equipment, such as locomotives (in addition to the other lateral forces specified), should be a single moving force equal to 25% of the heaviest axle load (E80 configuration). It should be applied at the base of the rail. This force may act in either lateral direction at any point of the span.

On spans supporting multiple tracks, the lateral force from only one track should be used. Resulting vertical forces should be disregarded.

The resulting stresses to be considered are axial stresses in the members bracing the flanges of stringers, beams and girders, axial stresses in the chords of trusses and in members of cross frames of these spans, and the stresses from lateral bending of flanges of longitudinal flexural members, which have no bracing system.

The effects of the lateral load should be disregarded in considering lateral bending between brace points of flanges, axial forces in flanges, and the vertical forces transmitted to the bearings.

11.5.8 Wind Forces

AREMA-recommended practices consider wind to be a moving force acting in any horizontal direction. On unloaded bridges, the specified lateral force is 50 lb/ft^2 acting on the following surfaces:

Girder spans: 1.5 times vertical projection

Truss spans: vertical projection of span plus any portion of leeward truss not shielded by the floor system

Viaduct towers and bents: vertical protection of all columns and tower bracing

On loaded bridges, a wind lateral force of 30 lb/ft^2 acting as described above, should be applied with a wind force of 0.30 kip/ft acting on the live load of one track at a distance of 8 ft above the top of the rail. On girder and truss spans, the wind force should be at least 0.20 kip/ft for the loaded chord or flange and 0.15 kip/ft for the unloaded chord or flange.

Longitudinal wind forces are considered as 25% and 50% of the specified lateral wind forces for girder and truss spans, respectively. For viaduct towers the recommended longitudinal wind force is 50 lb/ft^2 on the vertical projection of windward and leeward columns and bracing members.

The designer should consider specific locations where high wind velocities and gust forces combined with the use of high-profile equipment (such as double-stack container trains) may justify the use of greater design wind forces.

11.5.9 Seismic Forces

In some cases railway bridges designed in accordance with generally accepted practices for anchor bolts, bridge seat widths, edge distance on masonry plates, continuous rail, etc., may not require analysis for earthquake forces. In other cases, earthquake forces may be very important. Members and connections subjected to earthquake forces should be designed in accordance with AREMA *MRE* Chap. 9, Seismic Design for Railway Structures.

11.5.10 Forces from Continuous Welded Rail

Definitive evaluation of the forces due to thermal changes in continuous welded rail (CWR) is a difficult and complex problem. Factors affecting this complex behavior are span movements associated with temperature change, rail laying temperature, type of bridge (materials, open or ballasted deck), connection between rails and deck, connection between deck and span, and cross-sectional area of the rail. In particular, conditions such as the presence of adjacent span expansion bearings, must be carefully considered by the designer.

The rail must be adequately restrained against vertical and lateral movements. Most conventional rail-fastening systems also provide for longitudinal restraint of the rail. The type of longitudinal rail restraint will have a large effect on the magnitude of rail forces generated and transferred to the bridge. In general, and particularly on long open-deck spans, CWR installation should allow sufficient longitudinal movement of the rail to preclude the large forces that would damage or fail rail and/or deck anchorage systems, while precluding rail lateral instability. Therefore, longitudinal rail restraint on a span is sometimes limited to a maximum distance from fixed bearings. Rail expansion joints and special rail fasteners that allow longitudinal movement, while restraining vertical and lateral displacement, are sometimes used on longer spans.

Chapter 15, Part 8, of the AREMA *MRE* contains recommendations for the installation of CWR on railway bridges. Most railroad companies also have their own standards regarding the installation of CWR on bridges.

11.5.11 Load Combinations

Every component of substructure and superstructure should be proportioned to resist all combinations of forces applicable to the type of bridge and its site. Members subjected to stresses from dead, live, impact, and centrifugal loads should be designed for the basic allowable unit stress or the allowable fatigue stress, whichever governs.

With the exception of floorbeam hangers, members subjected to stresses from other lateral or longitudinal forces, as well as to dead, live, impact, and centrifugal loads, may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. However, the section should not be smaller than that required to satisfy basic unit stresses or the allowable fatigue stress range when those lateral or longitudinal forces are not present.

Components subject to stresses from wind loads only should be designed for the basic allowable stresses. Also, no increase in the basic allowable stresses in high-strength bolts should be taken for connections of members covered in this article.

11.5.12 Distribution of Live Load

The AREMA *MRE* contains recommended practices for distribution of live loads to the ties in open-deck construction and to the deck materials in ballast-deck bridges. Attention is called to the provision that, in the design of beams and girders, the live load must be considered as a series of concentrated loads, without any longitudinal distribution.

On open-deck bridges, ties within a length of 4 ft, but not more than three ties, may be assumed to support a wheel load. For ballasted-deck structures, live-load distribution is based on the assumption of standard cross ties at least 8 ft long, about 8 in wide, and spaced not more than 2 ft on centers, with at least 6 in of ballast under the ties. For deck design, each axle load should be uniformly distributed over a length of 3 ft plus the minimum distance from bottom of tie to top of beams or girders, but not more than 5 ft or the minimum axle spacing of the live load. In the lateral direction, the axle load should be uniformly distributed over a length equal to the length of tie plus the minimum distance from the bottom of tie to top of beams or girders. Deck thickness should be at least $^1/_2$ in for steel plate, 3 in for timber, and 6 in for reinforced concrete.

For ballasted concrete decks supported by transverse steel beams without stringers, the portion of the maximum axle load to be carried by each beam is given by

$$P = \frac{1.15A}{S} \qquad S \geq d \tag{11.9}$$

where A = axle load
S = axle spacing, ft
D = effective beam spacing, ft
d = beam spacing, ft

For bending moment, within the limitation that D may not exceed either axle or beam spacing, the effective beam spacing may be computed from

$$D = d\frac{1}{1 + d/aH}\left(0.4 + \frac{1}{d} + \sqrt{\frac{H}{12}}\right) \tag{11.10}$$

where a = beam span, ft
$H = nl_b/ah^3$
n = ratio of modulus of elasticity of steel to that of concrete
l_b = moment of inertia of beam, in^4
h = thickness of concrete deck, in

For end shear, $D = d$. At each rail, a concentrated load of *PI2* should be assumed acting on each beam, without any lateral distribution.

D should be taken equal to d for bridges without a concrete deck or where the concrete slab extends over less than the center 75% of the floorbeam.

If $d > S$, P should be the maximum reaction of the axle loads with the deck between beams acting as a simple span.

For ballasted decks supported on longitudinal girders, axle loads should be distributed equally to all girders whose centroids lie within a lateral width equal to length of tie plus twice the minimum distance from bottom of tie to top of girders.

Design requirements for use of timber and concrete for bridge decks are included in Chaps. 7 and 8 of the AREMA *MRE*.

The designer should be aware of any pertinent requirements of the railroad for such items as concrete slab overhang, derailment conditions, composite action, waterproofing, walkway connection, and drainage.

11.6 BASIC ALLOWABLE STRESSES

Table 11.1 lists the allowable stresses for steel railroad bridges recommended in the AREMA *MRE*. The stresses are related to the specified minimum yield stress F_y or the specified minimum tensile strength F_u of the material except where stresses are independent of the grade of steel. The basic

TABLE 11.1 Basic Allowable Stresses for Steel Railroad Bridges*

Loading condition	Allowable stress, psi
Tension:	
Axial, gross section	$0.55F_y$
Axial, effective net area	$0.50F_u$
Floorbeam hangers, including bending, gross section with high-strength bolts in end connections	$0.55Fy$
Bending, extreme fiber of rolled shapes, girders, and built-up sections, net section	$0.55F_y$
On high-strength bolts including prying action:	
A325 bolts	44,000
A490 bolts	54,000
Compression:	
Axial, gross section, in:	
Stiffeners of plate girders (check also as column)	$0.55F_y$
Splice material	$0.55F_y$
Compression members centrally loaded:	
When $KL/r \leq 0.629/\sqrt{F_y/E}$	$0.55F_y$
When $0.629/\sqrt{F_y/E} < KL/r < 5.034/\sqrt{F_y/E}$	$0.60F_y - (17{,}500F_y/E)^{3/2}\,(KL/r)$
When $KL/r \geq 5.034/\sqrt{F_y/E}$	$0.514\pi^2E/(KL/r)^2$
where KL = effective length of compression member, in $K = {}^7/_8$ for members with pin-end conditions $K = {}^3/_4$ for members with bolted or welded end connections r = applicable radius of gyration of compression member, in E = modulus of elasticity = 29,000,000 psi	
Compression in extreme fibers of I-type members subjected to loading perpendicular to the web	$0.55F_y$
Compression in extreme fibers of welded built-up plate or rolled-beam flexural members with solid rectangular flanges, symmetrical about the principal axis in the plane of the web (other than box-type flexural members), the larger of the values computed by $0.55F_y\left[1-\frac{(L/r_y)^2 F_y}{6.3\pi^2 E}\right]$ or $\frac{0.131\pi E}{Ld\sqrt{1+\mu}/A_f}$ but not to exceed $0.55F_y$	
where L = distance between points of lateral support for compression flange, in r_y = minimum radius of gyration of the compression flange and that portion of the web area on the compression side of the axis of bending, about an axis in the plane of the web, in A_f = area of the smaller flange excluding any portion of the web, in^2 d = overall depth of member, in μ = Poisson's ratio = 0.30	

(*Continued*)

TABLE 11.1 Basic Allowable Stresses for Steel Railroad Bridges* (*Continued*)

Loading condition	Allowable stress, psi
Compression in extreme fibers of hot-rolled channels, where column strength is negligible $\frac{0.131\pi E}{Ld\sqrt{1+\mu}/A_f} < 0.55F_y$	
Compression in extreme fibers of bolted built-up flexural members symmetrical about the principal axis in the plane of the web, other than box-type flexural members $0.55F_y\left[1-\frac{(L/r_y)^2 F_y}{6.3\pi^2 E}\right]$	
Compression in extreme fibers of box-type welded or bolted flexural members symmetrical about the principal axis midway between the webs and whose proportions meet the provisions of AREMA Arts. 1.6.1 and 1.6.2 $0.55F_y\left[1-\frac{(L/r_y)_e^2 F_y}{6.3\pi^2 E}\right]$	
where *(L/r)$_e$* is the effective slenderness ratio of the box-type flexural member as determined by $(L/r)_e = \sqrt{\frac{1.105\pi L S_x\sqrt{\sum s/t}}{A\sqrt{I_y/(1+\mu)}}}$	
where L = distance between points of lateral support for compression flange, in S_x = section modulus of box-type member about its major axis, in^3 A = total area enclosed within center lines of box-type member webs and flanges, in^2 s/t = ratio of width of any flange or depth of web component to its thickness (neglect any portion of flange that projects beyond the box section) I_y = moment of inertia of box-type member about its minor axis, in^4	
Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously	$0.55F_y$
Stress in extreme fibers of pins	$0.83F_y$
Shear in webs of rolled beams and plate girders, gross section	$0.35F_y$
Shear in ASTM A325 bolts	17,000 (Note 1)
Shear in ASTM A490 bolts	21,000 (Note 1)
Shear in pins	$0.42F_y$
Bearing on pins	$0.75F_y$
where F_y = yield point of the material on which the pin bears, or of the pin material, whichever is less	

TABLE 11.1 Basic Allowable Stresses for Steel Railroad Bridges* (*Continued*)

Loading condition	Allowable stress, psi
Bearing on ASTM A325 and ASTM A490 bolts: The smaller of (Note 2)	$LF_u/2d$ or $1.2F_u$
where L = distance, in, measured in line of force from the centerline of a bolt to the nearest edge of an adjacent bolt or to the end of the connected part toward which the force is directed d = diameter of bolts, in F_u = lowest specified minimum tensile strength of connected part, ksi	
Bearing on milled stiffeners and other steel parts in contact	$0.83F_y$
Bolts subjected to combined tension and shear	$F_v \le S_a(1 - f_t A_b/T_b)$
where F_v = allowable shear stress, reduced due to combined stress, psi S_a = allowable shear stress, when loaded in shear only, psi f_t = average tensile stress due to direct load, psi A_b = nominal bolt area, in^2 T_b = minimum tension of installed bolts, lb	

*For steel castings, allowable stresses in compression and bearing are the same as those of structural steel of the same yield point. Other allowable stresses are 75% of those of structural steel of the same yield point.

Note 1: Applicable for surfaces with clean mill scale free of oil, paint, lacquer, or other coatings and loose oxide, for standard-size holes as specified in AREMA *Manual,* Art. 3.2.5. Where the engineer has specified special treatment of surfaces or other than standard holes in a slip-critical connection, the allowable stresses in AREMA Commentary, Table 15-9-2, may be used if approved by the engineer.

Note 2: For single bolt in line of force or connected materials with long slotted holes, $1.0F_u$ is the limit. A value of allowable bearing pressure F_p on the connected material at a bolt greater than permitted can be justified provided deformation around the bolt hole is not a design consideration and adequate pitch and end distance, L, are provided according to $F_p = LF_u/2d \le 1.5F_u$.

Source: Adapted from *AREMA Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

stresses may be increased for loading combinations (Art. 11.5.11) or may be superseded by allowable fatigue stresses (Art. 11.7).

Allowable stresses for welds for railroad bridges are given in Table 11.2. These stresses may also be increased for loading combinations (Art. 11.5.11), or may be superseded by allowable fatigue stresses (Art. 11.7). The designer should review the AREMA *MRE* for complete provisions, including prohibited types of welds and joints. Special provisions may apply for fracture-critical members.

TABLE 11.2 Allowable Stresses on Welds

Type of weld	Electrode tensile strength class, psi	Allowable stress, psi[a]
Groove welds in tension or compression of base metal	*b*	$0.55F_y$
Groove welds in shear of base metal	*b*	$0.35F_y$
Fillet welds in shear (force applied in any direction)	60,000	16,500[c]
	70,000	19,000[c]
	80,000	22,000[c]

[a] F_y refers to yield point of base metal.

[b] Use matching weld metal.

[c] Also limited to 0.35 times F_y of base metal.

Source: Adapted from *AREMA Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

TABLE 11.3 Number of Constant Stress Cycles for Design of Railroad Bridges

Component*	Span length, ft	No. of loaded tracks	Specified no. of stress cycles (N)
Truss chord members and end posts,	$L > 100$	—	2,000,000
and longitudinal flexural members	$L \leq 100$	—	>2,000,000
Floorbeams	—	1	2,000,000
		2	>2,000,000
Truss hangers and subdiagonals that	—	1	2,000,000
carry floorbeam reactions only		2	>2,000,000
Truss web members	—	1	2,000,000
		2	>2,000,000

*Includes member connections.

Source: Adapted from *AREMA Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

11.7 FATIGUE DESIGN

Repetitive loading from locomotives and rolling stock can cause fatigue-damage accumulation and crack initiation at fatigue-prone details in steel bridges. Allowable fatigue stress ranges are specified for various details. Often, however, the damage is caused by secondary loadings that were not considered in design. For example, live loads may deflect one girder more than an adjacent girder in a multigirder bridge. This can cause the cross frames connecting the girders to induce large out-of-plane distortions and transverse bending stresses in the girder webs. Such conditions can usually be avoided by careful detailing.

The number of stress cycles (N) assigned to bridge members for design is based on the bridge span length or the number of loaded tracks, depending on the component. As indicated in Table 11.3, the number of stress cycles specified for the various components falls into one of two categories—either 2,000,000 cycles or over 2,000,000 cycles.

TABLE 11.4 Allowable Fatigue Stress Range for Details

	Allowable fatigue stress range, ksi, for number of constant stress cycles	
Stress category	2,000,000	>2,000,000
A	24 ksi	24 ksi
B	18	16
B′	14.5	12
C	13	10*
D	10	7†
E	8	4.5
E′	5.8	2.6
F	9	8

*12 ksi for transverse stiffener welds.

†6 ksi for base metal in low-slip-resistance bolted connections.

Source: Adapted from *AREMA Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

The allowable fatigue-stress range for various details has been determined by tests of large-scale members. The details have been classified in categories designated A through F. In design, the member must be proportioned so that the stress range at each detail does not exceed the allowable range, which depends on *N*. Table 11.4 lists the allowable fatigue-stress ranges for the various details. The allowable fatigue-stress ranges are for bridges designed for E80 live loads. The details for the various categories are given in the AREMA *MRE*.

11.8 FRACTURE-CRITICAL MEMBERS

Fracture-critical members (FCMs) are tension members or components of members in tension whose failure would result in the bridge's inability to safely and reliably carry railroad traffic. Also, welded attachments greater than 4 in in length in the direction of the tensile stress in a FCM should be considered as fracture-critical.

The AREMA *MRE* Chap. 15 fracture-control plan recognizes that FCMs have special material, fabrication, welding, inspection, and testing requirements. The designer must identify FCMs and select appropriate materials and specify welding requirements for their design. The designer should also recommend that appropriate fabrication methods, inspection, and testing are specified for FCMs.

The resistance to fracture of steel, for railway bridge design and fabrication, is generally specified in terms of the required minimum absorbed energy at a specific temperature measured by the Charpy V-notch (CVN) test. These toughness requirements are a function of material grade, thickness of material, fastenings employed, and service temperature. AREMA *MRE* Chap. 15, Table 15-1-2 and Table 15-1-14, outline the requirements for non-fracture-critical main load-carrying members and FCMs, respectively. The toughness requirements are more severe for FCMs. The submerged arc-welding (SAW) process is recommended for girder butt splices, flange-to-web welds, and box-member welds.

11.9 MEMBER DESIGN

11.9.1 Axial Tension Members

Most modern steel railway bridges use tension members comprised of solid rectangular flanges and webs, but occasionally built-up members utilizing lacing bars and cover plates are designed. The basic allowable stresses for tension members (Art. 11.6) ensure that yielding on the gross section is a limit state. The effective net section service-load capacity is based on the specified minimum tensile strength, F_U. The slenderness ratio (length of member divided by least radius of gyration, L/r) for tension members should not exceed 200.

Net area of tension members involves the following considerations. The nominal diameter of fasteners should be used in calculations. Fasteners should be arranged symmetrically about the axis of the member. The net section of a part should be taken as the thickness multiplied by the least net width of the part. The net section of a bolted tension member is the sum of the net sections of its parts, computed as the net width times the thickness.

The **net width** for a chain of holes extending across a part should be taken as the gross width, less the sum of the diameters of all holes in the chain, plus a quantity for each space in the chain, computed as

$$\frac{S^2}{4g} \tag{11.11}$$

where S = pitch of two successive holes in the chain in the direction of tensile stress and g = gage of the same two holes, in the transverse direction.

The net section of the part is determined by using the chain of holes that gives the least width. The net width should not be considered as more than 85% of the gross width. The diameter of the holes should be taken as $^1/_8$ in more than the nominal size of the fastener.

In the calculation of net width of angles, the gross width is the sum of the widths of the legs less the thickness; and the gage for holes in opposite legs is the sum of the gages (measured from back of angles) less the thickness. For splices, the effective thickness is only that part of the splice material developed by the fasteners.

An **effective net area** must be considered when tension loads are not transmitted to each element of the cross section, because shear lag can reduce the net area. The effective net area should be taken as the net area only when tension loads are transmitted to each element of the cross section of the member at connections.

For bolted connections, when tension loads are not transmitted to each element of the cross section of the member, the effective net area is

$$A_e = UA_n \tag{11.12}$$

where A_n = member net area
U = shear lag reduction coefficient = $(1 - x/L) \leq 0.90$
x = distance between centroid of connected area to shear plane of the connection
L = connection length in direction of loading between end fasteners

For bolted connections, U is 0.80 for angles with a minimum of four fasteners per line or 0.60 for angles with less than four fasteners per line. AREMA *MRE* Chap. 15, Art. 1.6.5, provides detailed information on the determination of x.

For welded connections, when tension loads are not transmitted to each element of the cross section of the member, the effective net area is

$$A_e = UA_g \tag{11.13}$$

For connections using only longitudinal welds on other than plate members or for connections using a combination of longitudinal and transverse welds,

A_g = member gross area
U = shear lag reduction coefficient = $(1 - x/L) \leq 0.90$
x = distance between centroid of connected area to shear plane of the connection
L = connection length in direction of loading

For connections using only transverse welds to the direction of loading,

A_g = area of only directly connected elements
$U = 1$

For connections using only longitudinal welds on plate members (welds must be on both edges of plates for a length not less than the distance between the welds),

A_g = area of plate member only
$U = 1.00$ for $L \geq 2w$
$= 0.87$ for $2w > L \geq 1.5w$
$= 0.75$ for $1.5w > L \geq w$
L = length of weld
w = distance between welds

In **built-up tension members,** lacing bars and cover plates should be designed for the shear force in the plane of the lacing bars or cover plates due to the weight of the member and other directly applied forces. Detailed recommendations for the design of lacing bars, stay plates, and perforated cover plates for tension members are given in AREMA *MRE* Chap. 15, Sec. 1.6.

11.9.2 Axial Compression Members

Compression members should be configured such that the main elements of the section are connected directly to gusset plates, pins, or other members. Most modern steel railway bridges use compression members comprised of solid rectangular flanges and webs, but occasionally built-up members utilizing lacing bars and cover plates are designed.

To prevent **elastic lateral buckling,** the slenderness ratio (length of member divided by least radius of gyration, L/r) should not exceed the following:

- 100 for main compression members
- 120 for wind and sway bracing
- 140 for single lacing and 200 for double lacing

Inelastic buckling and yielding are precluded by use of the allowable stress equations (Art. 11.6).

Built-up compression members must meet the following requirements. For members consisting of parts connected by lacing or solid cover plates, the minimum thickness of the web plate, should not be less than

$$t_m = \frac{0.90b\sqrt{F_Y/E}}{\sqrt{P_c/f}} \tag{11.14}$$

and $\sqrt{P_c/f}$ must not exceed 2.0.

The thickness of the cover plate should not be less than

$$t_m = \frac{0.72b\sqrt{F_Y/E}}{\sqrt{P_c/f}} \tag{11.15}$$

and $\sqrt{P_c/f}$ must not exceed 2.0.

In the above expressions,

t_m = minimum thickness, in

b = unsupported distance between the nearest line of fasteners or welds, or between the roots of rolled flanges, in

P_c = allowable stress for the member in axial compression, psi

f = calculated stress in compression, psi

F_y = yield point for the material, psi

Lacing bars and cover plates for built-up compression members should be designed for the shear force in the plane of the lacing bars and cover plates due to the weight of the member, other directly applied forces, and 2.5% of the compressive axial force in the member, but not less than

$$\frac{AF_Y}{150} \tag{11.16}$$

where A = member area required for axial compression, in^2

F_Y = yield point of member material, psi

Detailed recommendations for the design of lacing bars, stay plates, and perforated cover plates for compression members are given in AREMA *MRE* Chap. 15, Sec. 1.6.

Local buckling of outstanding elements in compression is an important design consideration. The width, in, of the outstanding elements of compression members should not exceed the following values expressed in terms of the element thickness t, in, and the material yield point F_y, psi.

Legs of angles or flanges of beams or tees:

For stringers and girders where ties rest on the flange: $1900t/\sqrt{F_y}$

For main members subject to axial force and for stringers and girders where ties do not rest on the flange: $2300t/\sqrt{F_y}$

For bracing and other secondary members: $2700t/\sqrt{F_y}$

Plates: $2300t/\sqrt{F_y}$

Stems of tees: $3000t/\sqrt{F_y}$

The width of the plate element should be taken as the distance from the free edge to the center of the first line of fasteners or welds. Angle legs and tee stems should be taken as the full nominal dimension. The flange of beams and tees should be measured from the free edge to the toe of the fillet. If the projecting element exceeds the above width but could be made to conform if a part of its width were considered removed, and if that reduced section would be satisfactory for stress requirements, the element should be considered acceptable.

11.9.3 Flexural Members

Steel rolled-beam and fabricated-plate girder spans provide economical bridges for railways. These spans generally require lateral bracing systems and diaphragms or cross frames for lateral strength and stability (Art. 11.9.8). Allowable stress design procedures using the moment-of-inertia method are used to proportion steel railway bridge flexural members as follows:

$$\frac{I}{C} = \frac{M}{f_b} \tag{11.17}$$

where I = moment of inertia, in^4, of gross section for compressive stress and of net section for tensile stress

c = distance, in, from neutral axis (taken at center of gravity of the gross section) to extreme fiber in flexure

M = bending moment at section, in·kips

f_b = allowable compressive or tensile bending stress, ksi

Flanges for plate girders and rolled beams are subject to the following considerations. Where not fully supported laterally, the compression flange of a flexural member should be supported at points so that the ratio of the distance between points, to the radius of gyration of the flange plus the part of the web on the compression side of the neutral axis, does not exceed $5.55\sqrt{E/F_y}$, where F_y is the yield point of the material, psi, and E = 29,000,000 psi.

In open-deck construction, ties may be seated on the top flange. Tie deflection loads the flange nonuniformly with the passage of each wheel. The minimum thickness for flange angles should be $^5/_8$ in if cover plates are used and $^3/_4$ in where cover plates are not used. Flanges of plate girders should be proportioned without the use of side plates.

Where cover plates are used, at least one plate of each flange should run the full length of the span. Partial-length cover plates should be avoided, but where they are used, they should extend far enough beyond the theoretical end to develop the plate and to a section where the stress in the flange, without the cover plate, is not greater than the allowable fatigue stress.

In welded construction, only one plate should be used for the flange. Side plates should not be used. Flange plate width and thickness may be varied in the length of the member using appropriate butt welds and transitions. Where the ties will sit on the top flange, consider the following:

1. Wider flanges are subject to more flexure as the tie deflects.
2. If the flange width or thickness changes, adjust tie (dapping) to fit the sections.
3. The flange width should accommodate tie hold-down devices without fouling guard timbers. Tie hold-downs should preferably go on the field side of the flange to avoid tie skew.

Only one cover plate should be used on the flange of a rolled beam. The cover plate should be of one thickness, full-length, and should be connected to the beam flange with continuous fillet welds. The cover-plate thickness should be not more than 1.5 times the thickness of the beam flange and should meet the minimum thickness requirements. Beam flanges supporting ties frequently experience mechanical wear and corrosion, and may require supplemental protection. Tie-bearing area on the cover plate should be at least as great as the bearing area of the tie plate.

Flange splices for plate girders and rolled beams may be made by bolting or shop welding and should have a capacity not less than the capacity of the member. Bolted splices should be made with components having a section not less than the section of the member being spliced. The number of fasteners used on each side of the splice must develop the force in the cut member. Flange angles should be spliced with angles, and no two elements in the same flange should be spliced at the same cross section. Welded shop splices should be made with complete-penetration groove welds.

Webs for plate girders must meet the minimum thickness requirements. Depending on detailing and service conditions, web plates are prone to spot corrosion. The thickness of the web plate should not be less than $^1/_6$ the thickness of the flange, nor, for webs without longitudinal stiffeners, less than a thickness (in) of $0.18\sqrt{F_y/E}$ times the clear distance between the flanges, (in) where F_y is the yield point of the material, psi, and $E = 29{,}000{,}000$ psi. The minimum thickness of web plates with longitudinal stiffeners may be taken as one-half the value determined for webs without longitudinal stiffeners.

Web splices for plate girders and rolled beams may be made by bolting or welding and should have a capacity not less than the capacity of the member. Web splices should be made using splice plates on each side of the web. The net moment of inertia of the splices plates must not be less than the net moment of inertia of the web. Shop-weld splices may be made with complete-penetration groove welds over the entire cross section without using splice plates. Splices in webs should be designed for the greater of the shear strength of the web gross section, or the combined strength of the net section flexural strength of the web in conjunction with the maximum shear force at the splice.

Intermediate transverse stiffeners are often spaced such that they may be used as connection plates for cross frames and diaphragms. The designer should be cognizant of issues concerning the fatigue strength of welded stiffener connections near the tension flange and out-of-plane displacements at the web gap. The web gap for welded transverse stiffeners is recommended to be a minimum of six times the web thickness from the toe of the tension flange-to-web weld. Wrap-around fillet welds are prohibited for intermediate transverse stiffeners.

To preclude elastic flexural and shear buckling of girder web plates, AREMA *MRE* Chap. 15 recommends that pairs of transverse stiffeners be used when

$$\frac{h_w}{t_w} \geq 2.12\sqrt{\left(\frac{E}{F_Y}\right)} \tag{11.18}$$

where h_w = depth of web between flanges, in
t_w = thickness of web, in
F_Y = yield point of member material, psi

The clear distance between intermediate transverse stiffeners is recommended not to exceed the following: 96 in, h_w, and d. The distance d is defined as

$$d = 1.95t_w\sqrt{\frac{E}{S}} \tag{11.19}$$

where S = calculated shear stress on the gross section of the web at location under consideration, psi.

The moment of inertia of the intermediate stiffeners should be at least

$$I = 2.5dt_w^3\left(\frac{h_w^2}{d^2} - 0.7\right) \tag{11.20}$$

where I = minimum required transverse stiffener moment of inertia, in^4, taken about the centerline of web for pairs of stiffeners and about the face of web in contact with stiffener for single stiffeners (where single stiffeners are used, AREMA recommends they be connected to the compression flange)

The minimum width of the outstanding element of a transverse stiffener should not exceed 16 times its thickness nor be less than 2 in plus $^1/_{30}$ of girder depth. Chapter 15 of the AREMA *MRE* provides specific recommendations concerning connection of intermediate transverse stiffeners to the compression and tension flanges of through-plate girder spans.

Longitudinal web stiffeners should be used at $h_w/5$ from the inner surface of the compression flange when

$$\frac{h_w}{t_w} \geq 4.18\sqrt{\left(\frac{E}{f}\right)} \tag{11.21}$$

and f = calculated compressive flexural stress in flange, psi.

The moment of inertia of the longitudinal stiffener should be at least

$$I_E = h_w t_w^3\left(2.4\frac{d_a^2}{h_w^2} - 0.13\right) \tag{11.22}$$

where I_E = minimum required longitudinal stiffener moment of inertia, in^4, taken about the face of the web in contact with the stiffener for stiffeners used on one side of the web and about the centerline of web for stiffeners used on each side of the web.

The minimum thickness of longitudinal stiffeners, in, should be

$$t_l = 2.39b'\sqrt{\frac{f}{E}} \tag{11.23}$$

where t_l = minimum thickness of longitudinal stiffener
b' = width of outstanding stiffener element

Longitudinal stiffeners are usually used on one side of the web plate and are discontinuous at intersections with transverse stiffeners. The stress in the stiffener must not exceed the allowable stress of the stiffener material, assuming the longitudinal stiffener participates in resisting girder loads.

Although not outlined as recommended practice in Chap. 15 of the AREMA *MRE*, some designers calculate the minimum required moment of inertia of the intermediate transverse stiffeners using 0.80 h_w, based on the subpanel depth created by the use of longitudinal stiffeners. In this case, the maximum clear distance between the intermediate transverse stiffeners should not exceed d, as calculated for transverse stiffeners without longitudinal stiffeners, nor 1.20 h_w.

Bolted flange angle-to-web plate connections must transmit both horizontal shear force and directly applied forces to the flange. For open-deck construction, where ties bear directly on the flange, one wheel load plus 80% impact is assumed to be distributed over 3 ft. For ballasted-deck construction with steel plate or concrete decks, one wheel load plus 80% impact is assumed to be distributed over 5 ft.

Welded flange plate-to-web plate connections can be made with either continuous-complete-penetration groove welds or continuous fillet welds. For deck-plate girders and stringers of open-deck or noncomposite-deck construction, complete-penetration groove welds are recommended. For box girders, continuous fillet welds are recommended. It is also recommended that identical welds be used for both tension and compression flanges of girders.

Bearing stiffeners should be provided in pairs, opposite each other, at the centerline of the end bearings of plate girders and beams. Appropriately positioned pairs of stiffeners should be placed at all points of concentrated loads. Stiffener width should be as much as the flange will accommodate, and the stiffener connection to the web should have the capacity to transmit the load. Angle stiffeners should not be crimped. Plate stiffeners should be clipped top and bottom to clear the fillet of the flange-to-web interface.

The outstanding element of the bearing stiffener should meet the width-to-thickness requirements for compression elements. Bearing stiffeners may be designed as a column, using the pair of stiffeners and a strip of the web whose width is equal to 25 times the thickness of the web. For stiffeners located at the end of the web, the web column width should be taken as 12 times its thickness.

The effective length, L, should be taken as three-fourths of the stiffener length in determining L/r, where r is the radius of gyration. Stiffeners should also be designed for bearing, without considering any part of the web and using only the area of the stiffener in contact with the flange. Where stiffeners are welded to the flange with complete-penetration groove welds, the bearing area should be taken as the length of the weld times the thickness of the stiffener.

11.9.4 Composite Steel and Concrete Flexural Members

Simple-span steel railway bridges with concrete decks are sometimes designed as composite for structural efficiency. Allowable stress design procedures using the moment-of-inertia method are used to proportion composite steel-and-concrete railway bridge flexural members. The designer must be cognizant of the railroad's preference or site constraints that may require unshored composite construction.

For unshored construction, the dead load (usually prior to installation of superimposed dead loads of waterproofing, ballast, and track) must be carried by the steel section alone. The composite section will carry superimposed dead loads and the design must consider the effects of concrete creep. Where shoring is provided, it should not be removed until concrete has reached a minimum of 75% of specified 28-day strength. In this case, all dead loads can be assumed to be resisted by the composite section.

Section properties at the various stages of unshored construction are as follows:

- Unshored dead load: $S_1 = I/c$
- Application of live load: $S_2 = I_c/c_c$
- Dead load on composite section: $S_3 = I_c'/c_c'$

where S_1 = section modulus at top of steel or bottom of steel
S_2, S_3 = section modulus at top of concrete, top of steel, or bottom of steel
I = steel gross or net section moment of inertia
c = distance from neutral axis to top of steel or bottom of steel
I_c = composite-section net moment of inertia using elastic modular ratio
c_c = distance from neutral axis to top of concrete, top of steel, or bottom of steel using elastic modular ratio
I_c' = composite-section net moment of inertia using plastic modular ratio
c_c' = distance from neutral axis to top of concrete, top of steel, or bottom of steel using plastic modular ratio

In calculating section properties of the composite section, the effective width of flanges on either side of a beam should not exceed the following:

- $^1/_2$ distance to centerline of adjacent beam
- $^1/_8$ beam span for beams with concrete slab on both sides
- $^1/_{12}$ beam span for beams with concrete slab on one side only without overhang that contributes to effective width
- 6 times the slab thickness

For exterior beams the effective flange width should not exceed the overhang. The neutral axis of the composite section should be below the top of steel beams to preclude concrete tension.

Horizontal shear stress calculated in accordance with conventional beam theory is used to design shear transfer devices (welded studs or channels) for the maximum horizontal shear and the range of horizontal shear (fatigue) at the steel to concrete interface. AREMA *MRE* Chap. 15, Part 5, provides detailed recommendations concerning shear-transfer-device design strengths, materials, welding, and installation details. Shear transfer device spacing should not exceed 24 in.

Composite beams should be cambered for dead-load deflections exceeding 1 in. Calculated dead-load deflections depend on whether shored or unshored construction is used.

Some railroads may have supplemental requirements for the maximum allowable live load-plus-impact deflection of composite spans. Some railroads also have requirements concerning the noncomposite strength of steel-and-concrete deck bridges based on deck maintenance and replacement requirements associated with the age of the structure or in the event of damage.

11.9.5 Continuous and Cantilever Flexural Members

Continuous spans are prohibited by some railroads, and others limit use to situations where intermediate supports are unyielding (such as piers founded on solid bedrock). Recommendations concerning the loading patterns, impact load, structural analysis, support conditions, deflections, camber, bracing, girder stiffeners, cover plates, and splices are provided in AREMA *MRE* Chap. 15, Part 5.

Cantilever spans using suspended simple spans and, in some cases with shear connection between cantilevers without suspended spans, are used for long-span railway bridge construction. Recommendations concerning the loading patterns, impact load, structural analysis, support conditions, deflections, camber, girder stiffeners, cover plates, and splices are provided in AREMA *MRE* Chap. 15, Part 5. The deflection at the end of cantilever arms is limited to $^1/_{250}$ of the length of the cantilever arm.

11.9.6 Combined Axial Tension and Bending Members

Members subject to both axial tension and flexural stress should be designed such that the maximum superimposed tensile stress does not exceed $0.55F_Y$. If bending with respect to either axis of the member creates compressive stress in conjunction with minimum axial tension, the member should be designed considering the allowable compressive stress at extreme fibers of flexural members (Art. 11.6).

11.9.7 Combined Axial Compression and Bending Members

Members subject to both axial compression and flexural stress should be designed to satisfy the following interaction equations.

- When the calculated axial compressive stress, f_a, is equal to or less than 15% of the allowable axial compressive stress, F_a (Art. 11.6),

$$\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}} + \frac{f_{b2}}{F_{b2}} \leq 1.0 \qquad (11.24)$$

- When the calculated axial compressive stress f_a exceeds 15% of the allowable axial compressive stress F_a (Art. 11.6),

$$\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}[1-(f_a/0.514\pi^2 E)(k_1 L_1/r_1)^2]} + \frac{f_{b2}}{F_{b2}[1-(f_a/0.514\pi^2 E)(k_2 L_2/r_2)^2]} \leq 1.0 \qquad (11.25)$$

where F_{b1}, F_{b2} = allowable compressive bending stress about axes 1–1 and 2–2 (Art. 11.6)
f_{b1}, f_{b2} = calculated compressive bending stress about axes 1–1 and 2–2
$\frac{k_1 L_1}{r_1}, \frac{k_2 L_2}{r_2}$ = effective length to radius of gyration ratio for axes 1–1 and 2–2

11.9.8 Bracing

Top lateral bracing for all deck spans and through spans should be provided where possible. The top flanges of through-girder spans should be braced at the panel points by brackets (knee braces) of solid construction. The knee braces should extend from the floorbeams to the girder top flange and be as wide as clearances (Art. 11.4.7) will allow. For through-girder spans with solid floors (e.g., ballasted steel-plate or concrete-deck), the knee-brace spacing should not exceed 12 ft. Lateral bracing of girder compression flanges should be designed for a transverse shear force, in any panel equal to 2.5% of the total axial force acting in both members of the panel under consideration, in addition to forces from the specified lateral loads (i.e., centrifugal, equipment, wind forces).

In addition to top lateral bracing, there should be bottom lateral bracing for all spans longer than 50 ft. However, for deck spans with four or more beams per track, with a beam depth of less than 6 ft and where adequate shear transfer is provided by a reinforced-concrete deck (i.e., shear transfer connectors), bottom lateral bracing is not required.

Span floor systems may be used as lateral bracing in their plane, provided the floor construction is such that it provides adequate resistance to the specified lateral loads. Double bracing systems should be designed such that each member is proportioned for the compressive and tensile forces in the panel under consideration.

Cross frames and diaphragms for deck girder spans have generally not been specifically designed for lateral distribution of loads, but such distribution is inherent in typical construction. The following are recommended practices.

1. Longitudinal girders and beams that are more than 42 in deep and that are spaced more than 48 in apart should be braced with cross frames. Cross-frame diagonals should make an angle with the vertical that does not exceed 60°. Minimum steel thickness and number of fasteners should be indicated in the design. Cross frames or diaphragms should be used at the ends of spans and should be proportioned for lateral and centrifugal forces, as well as jacking loads, if required. Where girders or beam ends frame into a floorbeam, cross frames or diaphragms are not required.

2. Cross frames and diaphragms, and their connections, should be adequate to resist forces induced by out-of-plane bending and lateral loads. Connection plates for cross frames and diaphragms between beams or girders subject to out-of-plane bending should be adequately fastened to the web and both the top and bottom flanges of the beams or girders. Diaphragm and cross-frame spacing may be made coincident with the stiffener spacing. The requirement to fasten the connection plate to the tension flange of the girder requires special attention in welded fabrication.

3. Longitudinal beams and girders of depth and spacing that do not require cross frames should be braced with rolled-shape diaphragms that are as deep as the beam or girder will permit. Wide flange sections are frequently used, but channels may be an economical alternative. The connection for these diaphragms should be designed to carry shear of at least 50% of the shear capacity of the diaphragm.

4. On ballasted-deck bridges utilizing closely spaced transverse floorbeams, the beams should be connected with one or more lines of longitudinal diaphragms for each track.

5. The spacing of diaphragms or cross frames should be as follows:

For open-deck construction: 18 ft maximum

For ballast-deck construction with top lateral bracing: 18 ft maximum

For steel-plate or noncomposite concrete ballast-deck construction without top lateral bracing: 12 ft maximum

For ballast-deck construction with cast-in-place concrete decks that are integrated with the beams or girders: 24 ft maximum

Where a cast-in-place concrete deck is used and the girders and beams are 54 in deep or less, a concrete diaphragm may be used, provided the reinforcing extends through the web and is developed in the adjacent concrete.

Portal bracing and sway bracing is required for through-truss spans. Portal bracing should incorporate knees and be as deep as clearances (Art. 11.4.7) will allow. Sway bracing is required at intermediate panel points of the compression chord. Sway bracing should not be less than 6 ft deep. If clearances do not allow for sway bracing, top lateral struts as deep as the compression chord should be used with knee braces connected to posts. The knee braces should be as deep as clearances will allow.

Deck truss spans should be designed with sway bracing at panel points in conjunction with top lateral bracing to carry transverse shear at the compression flange to the end supports. The top lateral bracing should be designed for a transverse shear force in any panel equal to 2.5% of the total axial force acting in both members of the panel under consideration, in addition to forces from the specified lateral loads (i.e., centrifugal, equipment, wind forces).

Bracing for towers and bents should be comprised of a double system of diagonal braces in conjunction with horizontal struts at the top, intermediate panel points, and bottom of the tower or bent. To resist horizontal forces, towers carrying multiple tracks require horizontal bracing in the plane of the top of the towers. Struts at the bottom of towers should be proportioned for the greater of their calculated forces, or tension or compression forces equal to 25% of the dead-load reaction on one pedestal. Lateral bracing between the posts of viaduct towers should be designed for a transverse shear force in any panel equal to 2.5% of the total axial force acting in both members of the panel under consideration, in addition to forces from the specified lateral loads.

Bracing members used as ties or struts to reduce the unsupported length of a member should be designed for 2.5% of the force in the supported member.

11.10 CONNECTION AND SPLICE DESIGN

11.10.1 Connection Design Criteria

Connections of main members should be designed for the following:

- Splices (except milled compression splices): capacity of the member being spliced (Art. 11.9.3)
- Milled compression splices: 50% of the force transmitted to four splice plates, provided members are faced to bear on all surfaces
- Beam end connections: combined effect of moment and shear
- Beam end connections for simply supported beams: 1.25 times the calculated shear
- End connections receiving forces from combined effect of floor system and truss behavior: capacity of the member
- Other connections [i.e., bearing stiffener, flange-to-web (Art. 11.9.3), built-up sections]: the maximum calculated force in the connection

Connections of secondary and bracing members should be designed for the following.

- End connections of secondary and bracing members: the lesser of 1.5 times the maximum calculated force on the connection, or the member strength

- End connections of bracing members used only as ties or struts (Art. 11.9.8): 2.5% of force in supported member

Block shear should be evaluated for all beam end connections with coped top flanges, tension-member end connections, gusset plates, and other connections that may be subject to conditions conducive to block shear failure. The evaluation of block shear depends on the net area of connection subject to tension and shear. The allowable block-shear rupture strength P_{bs} is determined as follows:

When $F_U A_{\mathrm{nt}} \geq 0.60 F_U A_{nv}$,

$$0.35 F_Y A_{gv} + 0.50 F_U A_{\mathrm{nt}} \leq 0.30 F_U A_{nv} + 0.50 F_U A_{\mathrm{nt}} \tag{11.26}$$

when $F_U A_{\mathrm{nt}} < 0.60 F_U A_{nv}$,

$$0.30 F_Y A_{nv} + 0.55 F_Y A_{\mathrm{gt}} \leq 0.30 F_U A_{nv} + 0.50 F_U A_{\mathrm{nt}} \tag{11.27}$$

where A_{gv}, A_{gt} = gross area subject to shear and tension, respectively
A_{nv}, A_{nt} = net area subject to shear and tension, respectively

11.10.2 Requirements for Bolted Connections

Chapter 15 of the AREMA *MRE* recommends that all bolted connections be slip-critical because of the stress reversals, large impact, and vibrations present in railway bridges. The allowable shear stress given in Art. 11.6 is for Class A slip-critical connections. The designer should consult AREMA *MRE* Chap. 15, Part 9, Commentary, for allowable stresses for other classes of slip-critical bolted connections. High-strength bolts for the slip-critical connections specified by AREMA *MRE* Chap. 15, Part 1, Design, must be installed in accordance with the minimum tension and methods specified in AREMA *MRE* Chap. 15, Part 3, Fabrication.

The effective diameter of bolts is the nominal diameter and the effective bearing area is taken as the diameter times the thickness of the steel on which the bolt bears (except for countersunk bolts, where one-half the countersink depth, and counterbored bolts, where the counterbore depth should be subtracted from the thickness). Allowable bearing stress is given in Art. 11.6. Threads should be excluded from shear planes.

Bolts should be ASTM A325 or A490 bolts. Most steel railway bridge designers use $^7/_8$-in-diameter bolts in $^{15}/_{16}$-in holes, by preference of many railroads. Railroad companies may also have requirements relating to the use of oversize or slotted holes in fabrication. Bolted construction should have a minimum of three fasteners per plane of connection or the equivalent fillet welds made parallel and symmetric with the applied force. AREMA *MRE* Chap. 15, Sec. 1.9, provides direction on minimum bolt spacing and edge distance; and maximum fastener diameter for use in connection angles for steel railway bridge design and fabrication.

The sealing requirements of AREMA *MRE* Chap. 15, Sec. 1.5, outline maximum fastener spacing and weld requirements based on the thickness of members being connected. Air-tightness and water-tightness requirements for welded box members are also included. In order that built-up members behave as single members, AREMA *MRE* Chap. 15, Sec. 1.5, also outlines minimum requirements for stitch bolting based on the thickness of sections being connected. Additional bolts are not required for development of fillers.

Connections utilizing a combination of bolts and welds should be proportioned such that the weld carries the entire force.

The effect of prying action should be included in all tension-connection design (Art. 11.6). Prying action is a function of the direct tension force and connection geometry, and is additive to the direct tension force in a bolt. Fatigue effects can be neglected for all bolted connections subject to tensile cyclical loading where the prying force does not exceed 5% of the externally applied tension force, and also for connections subject to less than 500,000 cycles of direct tension where the prying force does not exceed 10% of the externally applied tension force. For connections subject to less than 500,000 cycles of direct tension where the prying force does not exceed 20% of the externally applied tension force, the allowable stress should be taken as 60% of the allowable tensile stress (Art. 11.6). For connections subject to more than 500,000 cycles of direct tension where prying force does not exceed 15% of externally applied tension force, the allowable stress should be taken as 50% of the allowable tensile stress (Art. 11.6).

11.10.3 Requirements for Welded Connections

Welding design, procedures, and inspection requirements for railway bridges should be in accordance with *ANSI/AASHTO/AWS D1.5, Bridge Welding Code*. Welding of ASTM A709 Grade HPS 70W steel is recommended to be in accordance with *AASHTO Guide for Highway Bridge Fabrication with HPS 70W Steel*. (Available from American Association of State Highway and Transportation Officials, 444 N. Capitol St., NW, Washington, DC 20001.)

Chapter 15 of the AREMA *MRE* recommends field welding of only minor connections not subject to live load. Welded field connections of any type are sometimes prohibited by railroad companies. Prohibited welds in AWS D1.5, as well as plug welds, slot welds, intermittent welds, and transverse tack welds on members subjected to flexural tension, are prohibited by Chap. 15 of the AREMA *MRE.* Chapter 15 also prohibits some butt welds and groove welds.

Attention is drawn to the allowable fatigue stresses that apply for stiffeners, gussets, and other details welded to members subjected to fatigue.

Welded butt joints, used to connect materials of different width or thickness in joints subjected to flexural or axial tensile stress, should be transitioned with a slope not exceeding 1 to 2.5. The thicker plate should not be more than twice as thick as the thinner plate. The weld face in welded butt joints used to connect materials of different thickness in joints subjected to axial compressive stress should be transitioned with a slope not exceeding 1 to 2.5. Welded butt joints used to connect materials of different width in joints subjected to axial compressive stress are recommended to be transitioned. Welded butt joints used to connect materials of both different width and thicknesses in joints subjected to flexural stress are prohibited.

Groove welds should be complete-penetration welds. Partial-penetration groove welds are generally not recommended, and partial-penetration groove welds transverse to the direction of stress are specifically prohibited in steel railway bridge fabrication.

Fillet welds subjected to tensile stresses not parallel to the weld or cyclical stress should have continuous returns at corners of joints. The length of the return weld should not be less than twice the weld size. Wrap-around fillet welds should not be used at intermediate transverse stiffeners.

11.10.4 Requirements for Pin Connections

Pin-connected members are not often used in modern steel railway bridge design. However, Chap. 15 of the AREMA *MRE* outlines recommendations concerning pin materials and geometry, as well as minimum net section and reinforcing plates for members.

CHAPTER 12
BEAM AND GIRDER BRIDGES

Mahir Sen, P.E.
Manager, Structures Department
Michael Baker Jr., Inc.
Princeton, New Jersey

Alfred Hedefine, P.E.
Former President
Parsons Brinckerhoff Quade & Douglas Inc.
New York, New York

John Swindlehurst, P.E.
Former Senior Professional Associate
Parsons Brinckerhoff Quade & Douglas Inc.
Newark, New Jersey

Steel beam and girder bridges are often the most economical type of framing. Contemporary capabilities for extending beam construction to longer and longer spans safely and economically can be traced to the introduction of steel and the availability, in the early part of the twentieth century, of standardized rolled beams. By the late thirties, after wide-flange shapes became generally available, highway stringer bridges were erected with simply supported, wide-flange beams on spans up to about 110 ft. Riveted plate girders were used for highway-bridge spans up to about 150 ft. In the fifties, girder spans were extended to 300 ft by taking advantage of welding, continuity, and composite construction. And in the sixties, spans two and three times as long became economically feasible with the use of high-strength steels and box girders, or orthotropic-plate construction, or stayed girders. Thus, now, engineers, as a matter of common practice, design girder bridges for medium and long spans as well as for short spans.

12.1 CHARACTERISTICS OF BEAM BRIDGES

Rolled wide-flange shapes generally are the most economical type of construction for short-span bridges. The beams usually are used as stringers, set, at regular intervals, parallel to the direction of traffic, between piers or abutments (Fig. 12.1). A concrete deck, cast on the top flange, provides lateral support against buckling. Diaphragms between the beams offer additional bracing and also distribute loads laterally to the beams before the concrete deck has cured.

Spacing. For railroad bridges, two stringers generally carry each track. They may, however, be more widely spaced than the rails, for stability reasons. If a bridge contains only two stringers, the

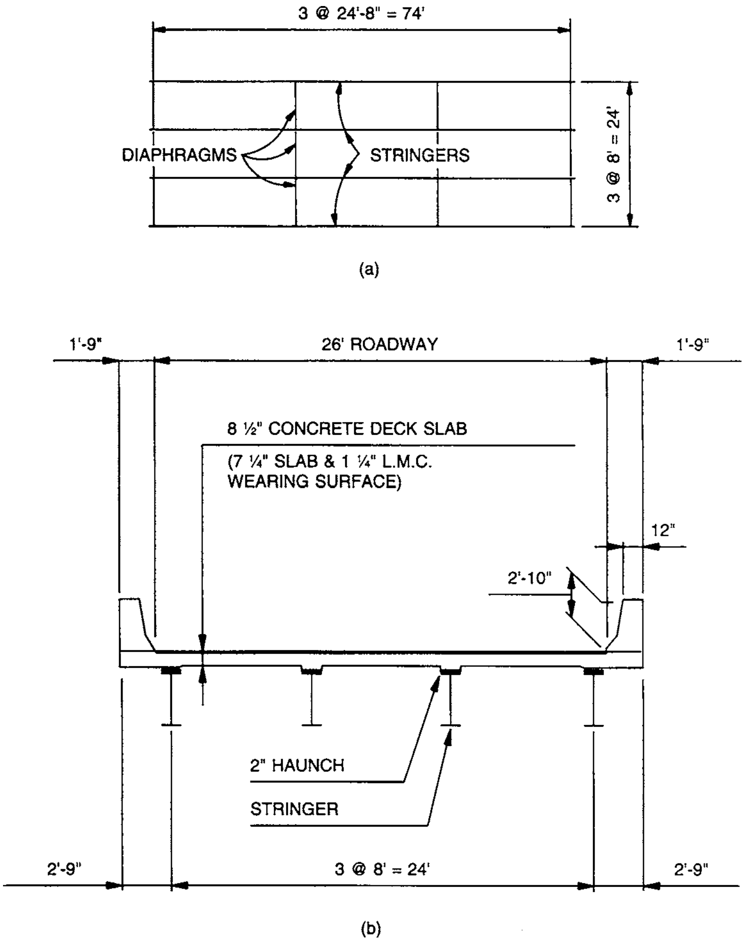


FIGURE 12.1 Two-lane highway bridge with rolled-beam stringers. (*a*) Framing plan. (*b*) Typical cross section.

distance between their centers should be at least 6 ft 6 in. When more stringers are used, they should be placed to distribute the track load uniformly to all beams.

For highway bridges, one factor to be considered in selection of stringer spacing is the minimum thickness of concrete deck permitted. For the deck to serve at maximum efficiency, its span between stringers should be at least that requiring the minimum thickness. But when stringer spacing requires greater than minimum thickness, the dead load is increased, cutting into the savings from use of fewer stringers. For example, if the minimum thickness of concrete slab is about 8 in, the stringer spacing requiring this thickness is about 8 ft for 4000-psi concrete. Thus, a 29-ft 6-in–wide bridge, with a 26-ft roadway, could be carried on four girders with this spacing. The outer stringers then would be located 1 ft from the curb into the roadway, and the outer portion of the deck, with parapet, would cantilever 2 ft 9 in beyond the stringers.

If an outer stringer is placed under the roadway, the distance from the center of the stringer to the curb preferably should not exceed about 1 ft.

Stringer spacing usually lies in the range 6 to 15 ft. The smaller spacing generally is desirable near the upper limits of rolled-beam spans.

The larger spacing is economical for the longer spans where deep, fabricated, plate girders are utilized. Wider spacing of girders has resulted in development of long-span stay-in-place forms. This improvement in concrete-deck forming has made steel girders with a concrete deck more competitive.

Regarding deck construction, while conventional cast-in-place concrete decks are commonplace, precast-concrete deck slab bridges are often used and may prove practical and economical if stage construction and maintenance of traffic are required. Additionally, use of lightweight concrete, a durable and economical product, may be considered if dead weight is a problem.

Other types of deck are available such as steel orthotropic plates. Also, steel grating decks may be utilized, whether unfilled, half-filled, or fully filled with concrete. The latter two deck-grating construction methods make it possible to provide composite action with the steel girder.

Short-Span Stringers. For spans up to about 40 ft, noncomposite construction, where beams act independently of the concrete slab, and stringers of American Association of State Highway and Transportation Officials (AASHTO) M270 (ASTM A709), Grade 36 steel often are economical. If a bridge contains more than two such spans in succession, making the stringers continuous could improve the economy of the structure. Savings result primarily from reduction in number of bearings and expansion joints, as well as associated future maintenance costs. A three-span continuous beam, for example, requires four bearings, whereas three simple spans need six bearings.

For such short spans, with relatively low weight of structural steel, fabrication should be kept to a minimum. Each fabrication item becomes a relatively large percentage of material cost. Thus, cover plates should be avoided. Also, diaphragms and their connections to the stringers should be kept simple. For example, they may be light channels field bolted or welded to plates welded to the beam webs (Fig. 12.2).

For spans 40 ft and less, each beam reaction should be transferred to a bearing plate through a thin sole plate welded to the beam flange. The bearing may be a flat steel plate or an elastomeric pad. At interior supports of continuous beams, sole plates should be wider than the flange. Then, holes needed for anchor bolts can be placed in the parts of the plates extending beyond the flange. This not only reduces fabrication costs by avoiding holes in the stringers but also permits use of lighter stringers, because the full cross section is available for moment resistance.

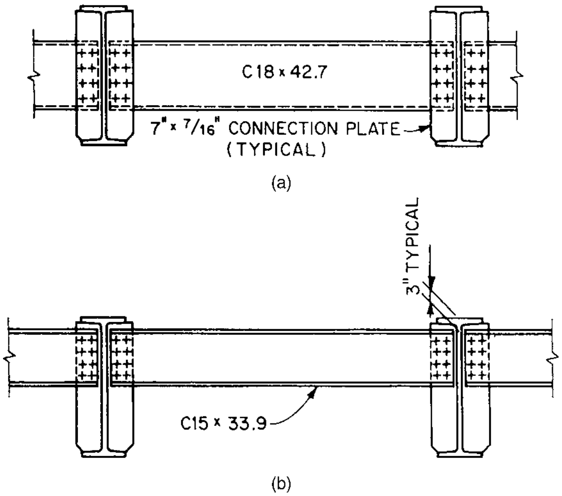


FIGURE 12.2 Diaphragms for rolled-beam stringers. (*a*) Intermediate diaphragm. (*b*) End diaphragm.

At each expansion joint, the concrete slab should be thickened to form a transverse beam, to protect the end of the deck. Continuous reinforcement is required for this beam. For the purpose, slotted holes should be provided in the ends of the steel beams to permit the reinforcement to pass through.

Live Loads. Although AASHTO "Standard Specifications for Highway Bridges" specify for allowable stress design (ASD) and load factor design (LFD) H15-44, HS15-44, H20-44, and HS20-44 truck and lane loadings (Art. 10.5), many state departments of transportation are utilizing larger live loadings. The most common is HS20-44 plus 25% (HS25). An alternative military loading of two axles 4 ft apart, each axle weighing 24 kips, is usually also required and should be used if it causes higher stresses. Some states prefer 30 kip axles instead of 24 kips. Also see Art. 10.5 for LRFD design loads.

Dead Loads. Superstructure design for bridges with a one-course deck slab should include a 25-lb/ft^2 additional dead load to provide for a future 2-in-thick overlay wearing surface. Bridges with a two-course deck slab generally do not include this additional dead load. The assumption is that during repaving of the adjoining roadway, the $1^1/_4$-in wearing course (possibly latex-modified concrete) will be removed and replaced only if necessary.

If metal stay-in-place forms are permitted for deck construction, consideration should be given to providing for an additional 8 to 12 lb/ft^2 to be included for the weight of the permanent steel form plus approximately 5 lb/ft^2 for the additional thickness of deck concrete required. The specific additional dead load should be determined for the form to be utilized. Wider stringer spacings may require deeper forms and additional weights. The additional dead load is considered secondary and may be included in the superimposed dead load supported by composite construction, when shoring is used.

Long-Span Stringers. Composite construction with rolled beams (Art. 10.15) may become economical when simple spans exceed about 40 ft, or the end span of a continuous stringer exceeds 50 ft, or the interior span of a continuous stringer exceeds 65 ft. W36 rolled wide-flange beams of Grade 36 steel designed for composite action with the concrete slab are economical for spans up to about 85 ft, though such beams can be used for longer spans. When spans exceed 85 ft, consideration should be given to rolled beams made of high-strength steels, W40 rolled wide-flange beams, or plate-girder stringers. In addition to greater economy than with noncomposite construction, composite construction offers smaller deflections or permits use of shallower stringers, and the safety factor is larger.

For long-span, simply supported, composite, rolled beams, costs often can be cut by using a smaller rolled section than required for maximum moment and welding a cover plate to the bottom flange in the region of maximum moment (partial-length cover plate). For the purpose, one plate of constant width and thickness should be used. It also is desirable to use cover plates on continuous beams. The cover plate thickness should generally be limited to about 1 in and be either 2 in narrower or 2 in maximum wider than the flange. Longitudinal fillet welds attach the plate to the flange. Cover plates may be terminated and end-welded within the span at a developed length beyond the theoretical cutoff point. AASHTO specifications provide for a Category E′ allowable fatigue-stress range, which must be utilized in the design of girders at this point.

Problems with fatigue cracking of the end weld and flange plate of older girders has caused designers to avoid terminating the cover plate within the span. Some state departments of transportation specify that cover plates be full-length or terminated within 2 ft of the end bearings. The end attachments may be either special end welds or bolted connections.

Similarly, for continuous, noncomposite, rolled beams, costs often can be cut by welding cover plates to flanges in the regions of negative moment. Savings, however, usually will not be achieved by addition of a cover plate to the bottom flange in positive-moment areas. For composite construction, though, partial-length cover plates in both negative-moment and positive-moment regions can save money. In this case, the bottom cover plate is effective because the tensile forces applied to it are balanced by compressive forces acting on the concrete slab serving as a top cover plate.

For continuous stringers, composite construction can be used throughout or only in positive-moment areas. Costs of either procedure are likely to be nearly equal.

Design of composite stringers usually is based on the assumption that the forms for the concrete deck are supported on the stringers. Thus, these beams have to carry the weight of the uncured concrete. Alternatively, they can be shored, so that the concrete weight is transmitted directly to the ground. The shores are removed after the concrete has attained sufficient strength to participate in composite action. In that case, the full dead load may be assumed applied to the composite section. Hence, a slightly smaller section can be used for the stringers than with unshored erection. The savings in steel, however, may be more than offset by the additional cost of shoring, especially when provision has to be made for traffic below the span.

Diaphragms for long-span rolled beams, as for short-span, should be of minimum permitted size. Also, connections should be kept simple (Fig. 12.2). At span ends, diaphragms should be capable of supporting the concrete edge beam provided to protect the end of the concrete slab. Consideration should also be given to designing the end diaphragms for jacking forces for future bearing replacements.

For simply supported, long-span stringers, one end usually is fixed, whereas arrangements are made for expansion at the other end. Bearings may be built up of steel or they may be elastomeric pads. A single-thickness pad may be adequate for spans under 85 ft. For longer spans, laminated pads will be needed. Expansion joints in the deck may be made economically with extruded or preformed plastics.

Cambering of rolled-beam stringers is expensive. It often can be avoided by use of different slab-haunch depths over the beams.

12.2 EXAMPLE—ALLOWABLE STRESS DESIGN OF COMPOSITE ROLLED-BEAM STRINGER BRIDGE

To illustrate the design procedure, a two-lane highway bridge with simply supported, composite, rolled-beam stringers will be designed. As indicated in the framing plan in Fig. 12.1*a*, the stringers span 74 ft center to center (c to c) of bearings. The typical cross section in Fig. 12.1*b* shows a 26-ft-wide roadway flanked by 1-ft 9-in parapets. Structural steel to be used is Grade 36. Loading is HS25. Appropriate design criteria given in Chap. 10 will be used for this structure. Concrete to be used for the deck is Class A, with 28-day compressive strength $f_c' = 4000$ psi and allowable compressive strength $f_c' = 1400$ psi. Modulus of elasticity $E_c = 33w^{1.5}\sqrt{f_c'} = 33(145)^{1.5}\sqrt{4000} = 3{,}644{,}000$ psi, say 3,600,000 psi.

Assume that the deck will be supported on four rolled-beam stringers, spaced 8 ft c to c, as shown in Fig. 12.1.

Concrete Slab. The slab is designed to span transversely between stringers, as in noncomposite design. The effective span S is the distance between flange edges plus half the flange width, ft. In this case, if the flange width is assumed as 1 ft, $S = 8 - 1 + {}^1\!/_2 = 7.5$ ft. For computation of dead load, assume a 9-in-thick slab, weight 112 lb/ft^2 plus 5 lb/ft^2 for the additional thickness of deck concrete in the stay-in-place forms. The 9-in-thick slab consists of a $7^3\!/_4$-in base slab plus a $1^1\!/_4$-in latex-modified concrete (LMC) wearing course. Total dead load then is 117 lb/ft^2. With a factor of 0.8 applied to account for continuity of the slab over the stringers, the maximum dead-load bending moment is

$$M_D = \frac{w_D S^2}{10} = \frac{117(7.5)^2}{10} = 660\,\text{ft}\cdot\text{lb/ft}$$

From Table 10.28, the maximum live-load moment, with reinforcement perpendicular to traffic, plus a 25% increase for conversion to HS25 loading, equals

$$M_L = 1.25 \times 400(S + 2) = 500(7.5 + 2) = 4750\ \text{ft}\cdot\text{lb/ft}$$

Allowance for impact is 30% of this, or 1425 ft·lb/ft. The total maximum moment then is

$$M = 660 + 4750 + 1425 = 6835\ \text{ft}\cdot\text{lb/ft}$$

For balanced design of the concrete slab, the depth $k_b d_b$ of the compression zone is determined from

$$k_b = \frac{1}{1 + f_s/nf_c} = \frac{1}{1 + 24{,}000/8(1400)} = 0.318$$

where d_b = effective depth of slab, in, for balanced design
f_s = allowable tensile stress for reinforcement, psi = 24,000 psi
n = modular ratio = E_s/E_c = 8
E_s = modulus of elasticity of the reinforcement, psi = 29,000,000 psi
E_c = modulus of elasticity of the concrete, psi = 3,600,000 psi

For determination of the moment arm $j_b d_b$ of the tensile and compressive forces on the cross section,

$$j_b = 1 - \frac{k_b}{3} = 1 - \frac{0.318}{3} = 0.894$$

Then the required depth for balanced design, with width of slab b taken as 1 ft, is

$$d_b = \sqrt{\frac{2M}{f_c bjk}} = 5.86 \text{ in}$$

For the assumed dimensions of the concrete slab, the depth from the top of slab to the bottom reinforcement is

$$d = 9 - 0.5 - 1 - 0.38 = 7.12 \text{ in}$$

The depth from bottom of slab to top reinforcement is

$$d = 7.75 + 1.25 - 2.75 - 0.38 = 5.88 \text{ in}$$

Since $d > d_b$, this will be an underreinforced section. Use d = 5.88 in. Then, the maximum compressive stress on a slab of the assumed dimensions is

$$f_c = \frac{M}{(kd)(jd)b/2} = \frac{6835 \times 12}{1.87 \times 5.26 \times {}^{12}\!/_2} = 1390 < 1400 \text{ psi}$$

Hence, a 9-in-thick concrete slab is satisfactory.

Required reinforcement area transverse to traffic is

$$A_s = \frac{12M}{f_s jd} = \frac{12 \times 6835}{24{,}000 \times 5.26} = 0.65 \text{ in}^2/\text{ft}$$

Use No. 6 bars at 8-in intervals. These supply 0.66 in^2/ft. For distribution steel parallel to traffic, use No. 5 bars at 9 in, providing an area about two-thirds of 0.65 in^2/ft.

Stringer Design Procedure. A composite stringer bridge may be considered to consist of a set of T beams set side by side. Each T beam comprises a steel stringer and a portion of the concrete slab (Art. 10.15). The usual design procedure requires that a section be assumed for the steel stringer. The concrete is transformed into an equivalent area of steel. This is done for a short-duration load by dividing the effective area of the concrete flange by the ratio n of the modulus of elasticity of steel to the modulus of elasticity of the concrete, and for a long-duration load, under which the concrete may creep, by dividing by $3n$. Then, the properties of the transformed section are computed. Next, bending stresses are checked at top and bottom of the steel section and top of concrete slab. After that, cover-plate lengths are determined, web shear is investigated, and shear connectors are provided to bond the concrete slab to the steel section. Finally, other design details are taken care of, as in noncomposite design.

Fabrication costs often will be lower if all the stringers are identical. The outer stringers, however, carry different loads from those on interior stringers. Sometimes girder spacing can be adjusted to equalize the loads. If not, and the load difference is large, it may be necessary to provide different designs for inner and outer stringers. Exterior stringers, however, should have at least the same load capacity as interior stringers. Since the design procedure is the same in either case, only a typical interior stringer will be designed in this example.

Loads, Moments, and Shears. Assume that the stringers will not be shored during casting of the concrete slab. Hence, the dead load on each stringer includes the weight of an 8-ft-wide strip of concrete slab as well as the weights of steel shape, cover plate, and framing details. This dead load will be referred to as *DL*.

DEAD LOAD CARRIED BY STEEL BEAM, KIPS/FT

Slab: $0.150 \times 8 \times 7.75 \times {}^1/_{12}$	= 0.775
Haunch—12 × 1 in: $0.150 \times 1 \times {}^1/_{12}$	= 0.013
Stay-in-place forms: 0.013×7	= 0.091
Rolled beam and details—assume	0.296
DL per stringer	1.175

Maximum moment occurs at the center of the 74-ft span:

$$M_{DL} = \frac{1.175(74)^2}{8} = 804 \text{ ft} \cdot \text{kips}$$

Maximum shear occurs at the supports:

$$V_{DL} = 1.175 \times \frac{74}{2} = 43.5 \text{ kips}$$

The safety-shaped parapets will be placed after the concrete has cured. Their weights may be equally distributed to all stringers. No allowance will be made for a future wearing surface, but provision will be made for the weight of the $1^1/_4$-in LMC wearing course. The total superimposed dead load will be designated *SDL*.

DEAD LOAD CARRIED BY COMPOSITE SECTION, KIPS/FT

Two parapets: 1.060/4	= 0.265
LMC wearing course: $0.150 \times 8 \times 1.25/12$	0.125
SDL per stringer:	0.390

Maximum moment occurs at mid-span and equals

$$M_{SDL} = \frac{0.390(74)^2}{8} = 267 \text{ ft} \cdot \text{kips}$$

Maximum shear occurs at the supports:

$$V_{SDL} = 0.390 \times \frac{74}{2} = 14.4 \text{ kips}$$

The HS25 live load imposed may be a truck load or a lane load. For maximum effect with the truck load, the two 40-kip axle loads, with variable spacing *V*, should be placed 14 ft apart, the minimum permitted (Fig. 12.3*a*). Then the distance of the center of gravity of the three axle loads from the center load is found by taking moments about the center load.

$$a = \frac{40 \times 14 - 10 \times 14}{40 + 40 + 10} = 4.66 \text{ ft}$$

Maximum moment occurs under the center axle load when its distance from mid-span is the same as the distance of the center of gravity of the loads from mid-span, or 4.66/2 = 2.33 ft. Thus, the center load should be placed ${}^{74}/_2 - 2.33 = 34.67$ ft from a support (Fig. 12.3*a*). Then, the maximum moment due to the 90-kip truck load is

$$M_T = \frac{90({}^{74}/_2 + 2.33)^2}{74} - 40 \times 14 = 1321 \text{ ft} \cdot \text{kips}$$

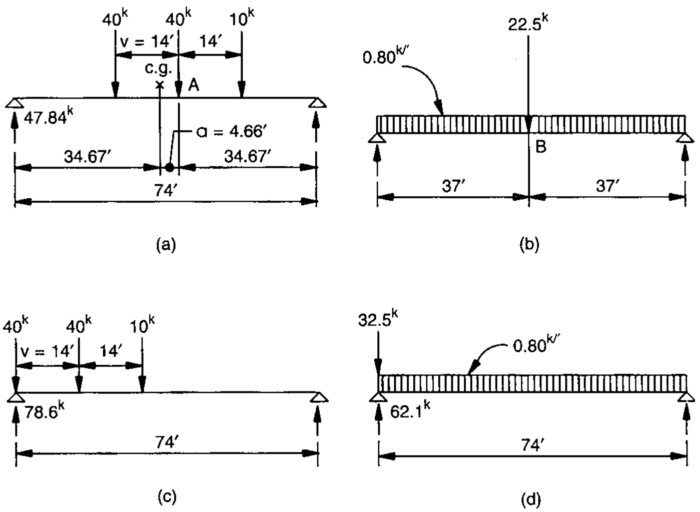


FIGURE 12.3 Positions of load for maximum stress in a simply supported stringer. (*a*) Maximum moment in the span with truck loads. (*b*) Maximum moment in the span with lane loading. (*c*) Maximum shear in the span with truck loads. (*d*) Maximum shear in the span with lane loading.

This loading governs, because the maximum moment due to lane loading (Fig. 12.3*b*) is smaller:

$$M_L = 0.80\frac{(74)^2}{8} + 22.5 \times {}^{74}\!/_{4} = 964 < 1321\ \text{ft}\cdot\text{kips}$$

The distribution of the live load to a stringer may be obtained from Table 10.14, for a bridge with two traffic lanes.

$$\frac{S}{5.5} = \frac{8}{5.5} = 1.454\ \text{wheels} = 0.727\ \text{axle}$$

Hence, the maximum live-load moment is

$$M_{LL} = 0.727 \times 1321 = 960\ \text{ft}\cdot\text{kips}$$

While this moment does not occur at mid-span as do the maximum dead-load moments, stresses due to M_{LL} may be combined with those from M_{DL} and M_{SDL} to produce the maximum stress, for all practical purposes.

For maximum shear with the truck load, the outer 40-kip load should be placed at the support (Fig. 12.3*c*). Then, the shear is

$$V_T = \frac{90(74 - 14 + 4.66)}{74} = 78.6\ \text{kips}$$

This loading governs, because the shear due to lane loading (Fig. 12.3*d*) is smaller:

$$V_L = 32.5 + 0.80 \times {}^{74}\!/_{2} = 62.1 < 78.6\ \text{kips}$$

Since the stringer receives 0.727 axle loads, the maximum shear on the stringer is

$$V_{LL} = 0.727 \times 78.6 = 57.1\ \text{kips}$$

Impact is the following fraction of live-load stress:

$$I = \frac{50}{L+125} = \frac{50}{74+125} = 0.251$$

Hence, the maximum moment due to impact is

$$M_I = 0.251 \times 960 = 241 \text{ ft}\cdot\text{kips}$$

and the maximum shear due to impact is

$$V_I = 0.251 \times 57.1 = 14.3 \text{ kips}$$

MID-SPAN BENDING MOMENTS, FT · KIPS

M_{DL}	M_{SDL}	$M_{LL} + M_I$
804	267	1201

END SHEAR, KIPS

V_{DL}	V_{SDL}	$V_{LL} + V_I$	Total V
43.5	14.4	71.4	129.3

Properties of Composite Section. The 9-in-thick roadway slab includes an allowance of 0.5 in for a wearing surface. Hence, the effective thickness of the concrete slab for composite action is 8.5 in.

The effective width of the slab as part of the top flange of the T beam is the smaller of the following:

$^1/_4$ span = $^1/_4 \times 74 = 222$ in

Stringer spacing, c to c = $8 \times 12 = 96$ in

12 × slab thickness = $12 \times 8.5 = 102$ in

Hence, the effective width is 96 in (Fig. 12.4).

To complete the T beam, a trial steel section must be selected. As a guide in doing this, formulas for estimated required flange area given in I. C. Hacker, "A Simplified Design of Composite Bridge Structures," *Journal of the Structural Division, ASCE, Proceedings Paper* 1432, November, 1957, may

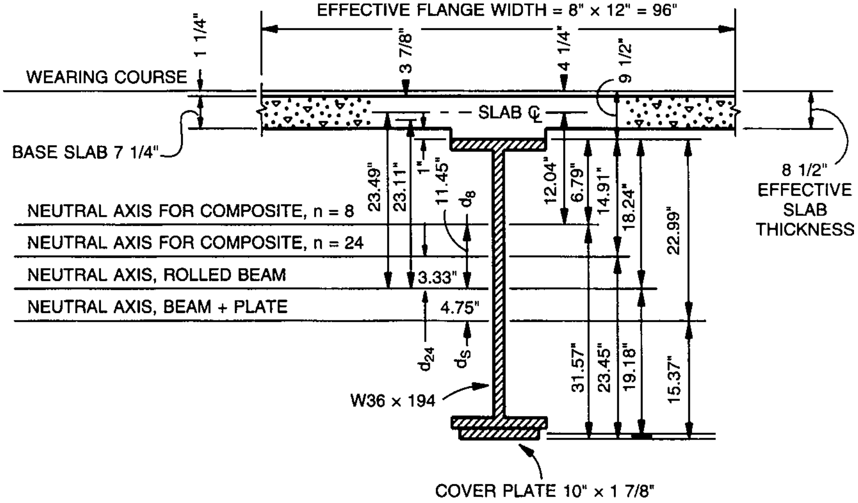


FIGURE 12.4 Cross section of composite stringer at mid-span.

be used. To start, assume the rolled beam will be a 36-in-deep wide-flange shape, and take the allowable bending stress F_b as 20 ksi. The required bottom-flange area, in^2, then may be estimated from

$$A_{sb} = \frac{12}{F_b}\left(\frac{M_{DL}}{d_{cg}} + \frac{M_{SDL} + M_{LL} + M_I}{d_{cg} + t}\right) \tag{12.1a}$$

where d_{cg} = distance, in, between center of gravity of flanges of steel shape and t = thickness, in, of concrete slab. With d_{cg} assumed as 36 in, the estimated required bottom-flange area is

$$A_{sb} = \frac{12}{20}\left(\frac{804}{36} + \frac{267 + 1201}{36 + 8.5}\right) = 33.2 \text{ in}^2$$

The ratio $R = A_{st}/A_{sb}$, where A_{st} is the area, in^2, of the top flange of the steel beam, may be estimated to be

$$R = \frac{50}{190 - L} = \frac{50}{190 - 74} = 0.43 \tag{12.1b}$$

Then, the estimated required area of the top flange is

$$A_{st} = RA_{sb} = 0.43 \times 33.2 = 14.3 \text{ in}^2$$

A W36 × 194 provides a flange with width 12.117 in, thickness 1.26 in, and area

$$A_{st} = 12.117 \times 1.26 = 15.27 > 14.3 \text{ in}^2 \qquad \text{OK}$$

With this shape, a bottom cover plate with an area of at least 33.2 − 15.27 = 17.9 in^2 is required. Maximum thickness permitted for a cover plate on a rolled beam is 1.5 times the flange thickness. In this case, therefore, plate thickness should not exceed 1.5 × 1.26 = 1.89 in. These requirements are met by a 10 × 1$^7/_8$-in plate, with an area of 18.75 in^2.

The trial section chosen consequently is a W36 × 194 with a partial-length cover plate 10 × 1$^7/_8$ in on the bottom flange (Fig. 12.4). Its neutral axis can be located by taking moments about the neutral axis of the rolled beam. This computation and that for the section moduli S_{st} and S_{sb} of the steel section are conveniently tabulated in Table 12.1.

In computation of the properties of the composite section, the concrete slab, ignoring the haunch area, is transformed into an equivalent steel area. For the purpose, for this bridge, the concrete area is

TABLE 12.1 Steel Section for Maximum Moment

Material	A	d	Ad	Ad^2	I_o	I
W36 × 194	57.00				12,100	12,100
Cover plate 10 × 1$^7/_8$	18.75	−19.18	−359.6	6,898		6,898
	75.75		−359.6			18,998
$d_s = -359.6/75.75 = -4.75$ in						$-4.75 \times 359.6 = -1{,}708$
						$I_{NA} = 17{,}290$

Distance from the neutral axis of the steel section to:

Top of steel = 18.24 + 4.75 = 22.99 in

Bottom of steel = 18.24 − 4.75 + 1.88 = 15.37 in

Section moduli

Top of steel	Bottom of steel
$S_{st} = 17{,}290/22.99 = 752 \text{ in}^3$	$S_{sb} = 17{,}290/15.37 = 1{,}125 \text{ in}^3$

divided by the modular ratio $n = 8$ for short-time loading, such as live loads and impact. For long-time loading, such as superimposed dead loads, the divisor is $3n = 24$, to account for the effects of creep. The computations of neutral-axis location and section moduli for the composite section are tabulated in Table 12.2. To locate the neutral axis, moments are taken about the neutral axis of the rolled beam.

Stresses in Composite Section. Since the stringers will not be shored when the concrete is cast and cured, the stresses in the steel section for load *DL* are determined with the section moduli of the steel section alone (Table 12.1). Stresses for load *SDL* are computed with section moduli of the composite section when $n = 24$ from Table 12.2*a*. And stresses in the steel for live loads and impact are calculated with section moduli of the composite section when $n = 8$ from Table 12.2*b* (Table 12.3*a*).

Stresses in the concrete are determined with the section moduli of the composite section with $n = 24$ for *SDL* from Table 12.2*a* and $n = 8$ for $LL + I$ from Table 12.2*b* (Table 12.3*b*).

TABLE 12.2 Composite Section for Maximum Moment

(*a*) For superimposed dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	75.75		−360			18,998
Concrete 96 × 7.75/24*	31.00	23.11	716	16,556	155	16,711
	106.75		356			35,709
$d_{24} = 356/106.75 = 3.33$ in						$-3.33 \times 356 = -1{,}185$
						$I_{NA} = 34{,}534$

Distance from the neutral axis of the composite section to:

Top of steel = 18.24 − 3.33 = 14.91 in

Bottom of steel = 18.24 + 3.33 + 1.88 = 23.45 in

Top of concrete = 14.91 + 1 + 7.75 = 23.66 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 34{,}534/14.91$ $= 2{,}316$ in^3	$S_{sb} = 34{,}534/23.45$ $= 1{,}473$ in^3	$S_c = 34{,}534/23.66$ $= 1{,}460$ in^3

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	75.75		−360			18,998
Concrete 96 × 8.5/8	102.00	23.49	2,396	56,280	615	56,895
	177.75		2,036			75,893
$d_8 = 2{,}036/177.75 = 11.45$ in						$-11.45 \times 2{,}036 = -23{,}312$
						$I_{NA} = 52{,}581$

Distance from the neutral axis of the composite section to:

Top of steel = 18.24 − 11.45 = 6.79 in

Bottom of steel = 18.24 + 11.45 + 1.88 = 31.57 in

Top of concrete = 6.79 + 1 + 8.5 = 16.29 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 52{,}580/6.79$ $= 7{,}744$ in^3	$S_{sb} = 52{,}580/31.57$ $= 1{,}666$ in^3	$S_c = 52{,}580/16.29$ $= 3{,}228$ in^3

*Depth of the top slab is taken as 7.75 in, inasmuch as the $1^1/_4$-in wearing course is included in the superimposed load.

TABLE 12.3 Stresses in the Composite Section, ksi, at Section of Maximum Moment

(*a*) Steel stresses		
	Top of steel (compression)	Bottom of steel (tension)
DL:	$f_b = 804 \times 12/752 = 12.83$	$f_b = 804 \times 12/1125 = 8.58$
SDL:	$f_b = 267 \times 12/2316 = 1.38$	$f_b = 267 \times 12/1473 = 2.18$
LL + *I*:	$f_b = 1201 \times 12/7744 = 1.86$	$f_b = 1201 \times 12/1666 = 8.66$
Total:	16.07 < 20	19.42 < 20
(*b*) Stresses at top of concrete		
SDL:	$f_c = 267 \times 12/(1460 \times 24) = 0.09$	
LL + *I*:	$f_c = 1201 \times 12/(3228 \times 8) = 0.56$	
	0.65 < 1.6	

Since the bending stresses in steel and concrete are less than the allowable, the assumed steel section is satisfactory. Use the W36 × 194 with 10 × $1^7/_8$-in bottom cover plate. Total weight of steel will be about 0.274 kip/ft, including 0.016 kip/ft for diaphragms, whereas 0.296 kip/ft was assumed in the dead-load calculations.

Maximum Shear Stress. Though shear rarely is critical in wide-flange shapes adequate in bending, the maximum shear in the web should be checked. The total shear at the support has been calculated to be 129.3 kips. The web of the steel beam is about 36 in deep and the thickness is 0.770 in. Thus, the web area is

$$36 \times 0.770 = 27.7 \text{ in}^2$$

and the average shear stress is

$$f_v = \frac{129.3}{27.7} = 4.7 < 12 \text{ ksi}$$

This indicates that the beam has ample shear capacity.

End bearing stiffeners are not required for a rolled beam if the web shear does not exceed 75% of the allowable shear for girder webs, 12 ksi. The ratio of actual to allowable shears is

$$\frac{f_v}{F_v} = \frac{4.7}{12} = 0.39 < 0.75$$

Hence, bearing stiffeners are not required.

Cover-Plate Cutoff. Bending moments decrease almost parabolically with distance from mid-span, to zero at the supports. At some point on either side of the center, therefore, the cover plate is not needed for carrying bending moment. For locating this cutoff point, the properties of the composite section without the cover plate are needed, with $n = 24$ and $n = 8$ (Fig. 12.5). The computations are tabulated in Table 12.4.

The length L_{cp}, ft, required for the cover plate may be estimated by assuming that the curve of maximum moments is a parabola. Approximately,

$$L_{cp} = L\sqrt{1 - \frac{S'_{sb}}{S_{sb}}} \qquad (12.2)$$

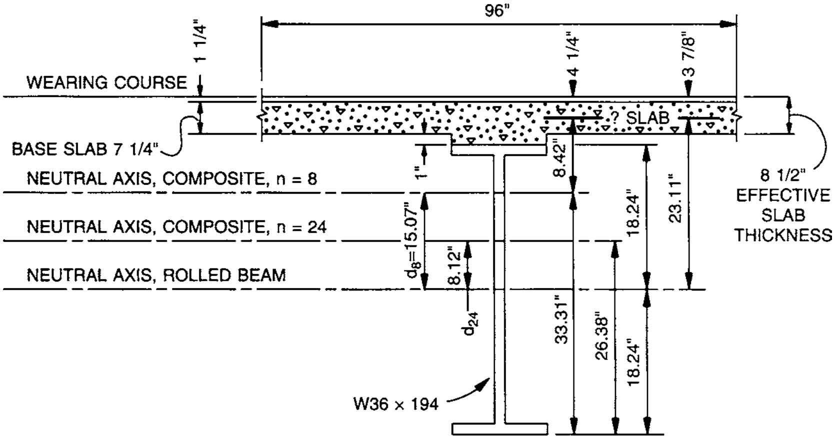


FIGURE 12.5 Cross section of composite stringer near supports.

where L = span, ft

S'_{sb} = section modulus with respect to bottom of steel shape with lighter flange (without cover plate), in^3

S_{sb} = section modulus with respect to bottom of steel shape with heavier flange (with cover plate), in^3

For the W36 × 194, $S'_{sb} = 665$. Hence,

TABLE 12.4 Composite Section Near Supports

(*a*) For dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
W36 × 194	57.0				12,100	12,100
Concrete 96 × 7.75/24	31.0	23.11	716	16,556	155	16,711
	88.0		716			28,811
$d_{24} = 716/88.0 =$ 8.14 in						$-8.14 \times 716 = -5{,}826$
Half-beam depth = 18.24						$I_{NA} = 22{,}985$
26.38 in						

$S_{sb} = 22{,}985/26.38 = 871$ in^3

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
W36 × 194	57.0				12,100	12,100
Concrete 96 × 8.5/8	102.0	23.49	23.96	56,282	615	56,900
	159.0		2,396			69,000
$d_8 = 2{,}396/159 = 15.07$ in						$-15.07 \times 2{,}396 = -36{,}110$
Half-beam depth = 18.24						$I_{NA} =$ 32,890
33.31 in						

$S_{sb} = 32{,}890/33.31 = 987$ in^3

$$L_{cp} = 74\sqrt{1-\frac{665}{1125}} = 48 \text{ ft}$$

If the cover plate is welded along its ends, the terminal distance that the plate must be extended beyond its theoretical cutoff point is about 1.5 times the plate width. For the 10-in plate, therefore, the terminal distance is 1.5 × 10 = 15 in. Use 1.5 ft. Thus, L_{cp} must be increased by 2 × 1.5, to 51 ft.

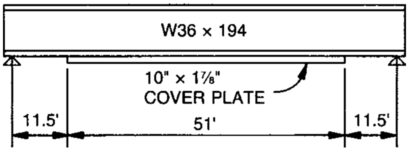


FIGURE 12.6 Elevation view of stringer.

Assume a 51-ft-long cover plate. It would then terminate 11.5 ft from each support (Fig. 12.6). The theoretical cutoff point is therefore 11.5 + 1.5 = 13.0 ft from each support. The stresses at that point should be checked to ensure that allowable bending stresses in the composite section without the cover plate are not exceeded. Table 12.5*a* presents the calculations for maximum flexural tensile stress at the theoretical cutoff points, 13 ft from the supports, and Table 12.5*b*, calculations for stresses at the actual terminations of the cover plate, 11.5 ft from the supports. The composite section without the cover plate is adequate at the theoretical cutoff point. But fatigue stresses in the beam should be checked at the actual termination of the plate, 11.5 ft from each support.

From Table 12.5*b*, the stress range equals the stress due to live load plus impact, 8.23 ksi. On the assumption that the bridge is a redundant-load-path structure, for base metal adjacent to a fillet weld (Category E′) subjected to 500,000 loading cycles, the allowable fatigue stress range permitted by AASHTO standard specifications is $F_{sr} = 9.2$ ksi > 8.23. The cover plate is satisfactory. (Because of past experience with fatigue cracking at termination welds for cover plates, however, the usual practice, when a cover plate is specified, is to extend it the full length of the beam.)

Cover-Plate Weld. The fillet weld connecting the cover plate to the bottom flange must be capable of resisting the shear at the bottom of the flange. The shear is a maximum at the end of the cover

TABLE 12.5 Stresses in Composite Steel Beam without Cover Plate

(*a*) At theoretical cutoff point, 13 ft from supports		
Bending moments, ft·kips		
M_{DL}	M_{SDL}	$M_{LL}+M_I$
466	155	744 (Fig. 12.7)
Stresses at bottom of steel (tension), ksi		
$DL: f_b = 466 \times 12/665 =$	8.41	(S_{sb} for W36 × 194)
$SDL: f_b = 155 \times 12/871 =$	2.14	(S_{sb} from Table 12.4*a*)
$LL + I: f_b = 744 \times 12/987 =$	9.04	(S_{sb} from Table 12.4*b*)
Total:	19.59 < 20	
(*b*) At cover-plate terminal, 11.5 ft from support		
Bending moments, ft·kips		
M_{DL}	M_{SDL}	$M_{LL}+M_I$
422	140	677 (Fig. 12.8)
Stresses at bottom of steel (tension), ksi		
$DL: f_b = 422 \times 12/665 =$	7.62	(S_{sb} for W36 × 194)
$SDL: f_b = 140 \times 12/871 =$	1.93	(S_{sb} from Table 12.4*a*)
$LL + I: f_b = 677 \times 12/987 =$	8.23	(S_{sb} from Table 12.4*b*)
Total:	17.78	

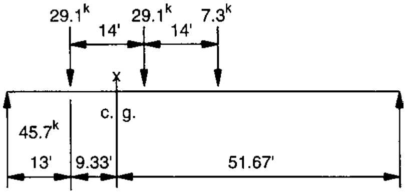


FIGURE 12.7 Position of truck load for maximum moment 13 ft from the support.

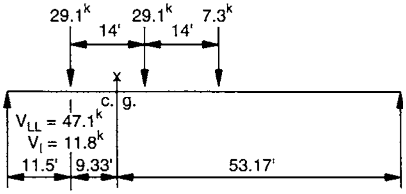


FIGURE 12.8 Position of truck load for maximum moment 11.5 ft from the support.

plate, 11.5 ft from the supports. The position of the truck load to produce maximum shear there is the same as that for maximum movement at those points (Fig. 12.8). Maximum shears and resulting shear stresses are given in Table 12.6.

The shear stress at the section is computed from

$$v = \frac{VQ}{I} \tag{12.3}$$

where v = horizontal shear stress, kips/in
V = vertical shear on cross section, kips
Q = static moment about neutral axis of area of cross section on one side of axis and not included between neutral axis and horizontal line through given point, in^3
I = moment of inertia, in^4, of cross section about neutral axis

AASHTO specifications permit a stress $F_v = 0.27F_u = 15.7$ ksi in fillet welds when the base metal is Grade 36 steel. The minimum size of fillet weld permitted with the $1^7/_8$-in-thick cover plate is $^5/_{16}$ in. If a $^5/_{16}$-in weld is used on opposite sides of the plate, the two welds would be allowed to resist a shear stress of

$$v_a = 2 \times 0.313 \times 0.707 \times 15.7 = 6.9 > 1.23 \text{ kips/in}$$

Therefore, use $^5/_{16}$-in welds.

Shear Connectors. To ensure composite action of concrete deck and steel stringer, shear connectors welded to the top flange of the stringer must be embedded in the concrete (Art. 10.15). For this structure, $^3/_4$-in-diameter welded studs are selected. They are to be installed in groups of three at specified locations to resist the horizontal shear at the top of the steel stringer (Fig. 12.9). With height $h = 6$ in, they satisfy the requirement $h/d \geq 4$, where d = stud diameter, in.

TABLE 12.6 Shear Stress 11.5 ft from Support

Shear, kips		
V_{DL}	V_{SDL}	$V_{LL} + V_I$
30.0	9.9	58.9

Shear stress, kips/in

DL: $v = 30.0 \times 18.75 \times 14.43/17{,}290 = 0.47$ (I from Table 12.1)
SDL: $v = 9.9 \times 18.75 \times 22.51/34{,}530 = 0.12$ (I from Table 12.2a)
$LL + I$: $v = 58.9 \times 18.75 \times 30.63/52{,}580 = 0.64$ (I from Table 12.2b)
Total: 1.23

With $f'_c = 4000$ psi for the concrete, the ultimate strength of a $^3/_4$-in-diameter welded stud is

$$S_u = 0.4d^2\sqrt{f'_cE_c} = 0.4(0.75)^2\sqrt{4{,}000 \times 3{,}600{,}000} = 27 \text{ kips}$$

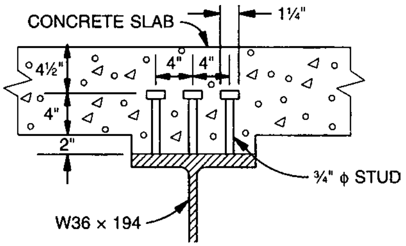


FIGURE 12.9 Welded studs on beam flange.

This value is needed for determining the number of shear connectors required to develop the strength of the steel stringer or the concrete slab, whichever is smaller. At mid-span, the strength of the rolled beam and cover plate, with area $A_s = 75.75$ in^2 from Table 12.1, is

$$P_1 = A_sF_y = 75.75 \times 36 = 2727 \text{ kips}$$

The compressive strength of the concrete slab is

$$P_2 = 0.85f'_cbt = 0.85 \times 4.0 \times 96 \times 8.5$$
$$= 2774 > 2727 \text{ kips}$$

Steel strength, governs. Hence, the number of studs provided between mid-span and each support must be at least

$$N_1 = \frac{P_1}{\phi S_u} = \frac{2727}{0.85 \times 27} = 119$$

With the studs placed in groups of three, therefore, there should be at least 40 groups on each half of the stringer.

Between the end of the cover plate and the support, the strength of the rolled beam alone, with $A_s = 57.0$, is

$$P_1 = A_sF_y = 57.0 \times 36 = 2052 > 2727 \text{ kips}$$

Steel strength still governs.

Pitch is determined by fatigue requirements. The allowable load range, kips per stud, may be computed from

$$Z_r = \alpha d^2 \tag{12.4}$$

With $\alpha = 10.6$ for 500,000 cycles of load (AASHTO specifications),

$$Z_r = 10.6(0.75)^2 = 5.97 \text{ kips/stud}$$

At the supports, the shear range $V_r = 71.4$ kips, the shear produced by live load plus impact. Consequently, with $n = 8$ for the concrete, and the transformed concrete area equal to 102 in^2 and $I = 32{,}980$ in^4 from Table 12.4*b*, the range of horizontal shear stress is

$$S_r = \frac{V_rQ}{I} = \frac{71.4 \times 102.0 \times 8.42}{32{,}890} = 1.864 \text{ kips/in}$$

Hence, the pitch required for stud groups near the supports is

$$p = \frac{3Z_r}{S_r} = \frac{3 \times 5.97}{1.864} = 9.61 \text{ in}$$

At 5 ft from the supports, the shear range $V_r = 66.1$ kips, produced by live load plus impact. Since the cross section is the same as the support, the pitch required for the studs is

$$p = \frac{9.63 \times 71.4}{66.1} = 10.40 \text{ in}$$

At 25 ft from the supports, $V_r = 46.1$ kips (Fig. 12.10). With $I = 52{,}580 \text{ in}^4$ from Table 12.2*b*, the range of horizontal shear stress is

$$S_r = \frac{V_r Q}{I} = \frac{46.1 \times 102.0 \times 12.04}{52{,}580} = 1.077 \text{ kips/in}$$

Hence, the pitch required is

$$p = \frac{3 \times 5.97}{1.077} = 16.6 \text{ in}$$

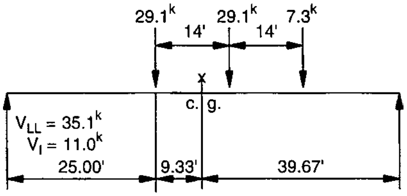


FIGURE 12.10 Position of loads for maximum shear 25 ft from the support.

The shear connector spacing selected to meet the preceding requirements is shown in Fig. 12.11.

Deflections. Dead-load deflections may be needed so that concrete for the deck may be finished to specified elevations. Cambering of rolled beams to offset dead-load deflections usually is undesirable because of the cost. The beams may, however, be delivered from the mill with a slight mill camber. If so, advantage should be taken of this, by fabricating and erecting the stringers with the camber upward.

The dead-load deflection has two components, one corresponding to *DL* and one to *SDL*. For computation for *DL*, the moment of inertia *I* of the steel section alone should be used. For *SDL*, *I* should apply to the composite section with $n = 24$ (Table 12.2*a*). Both components can be computed from

$$\delta = \frac{22.5 w L^4}{EI} \tag{12.5}$$

where w = uniform load, kips/in
L = span, ft
E = modulus of elasticity of steel, ksi
I = moment of inertia of section about neutral axis

For *DL*, $w = 1.175$ kips/ft, and for *SDL*, $w = 0.390$ kip/ft.

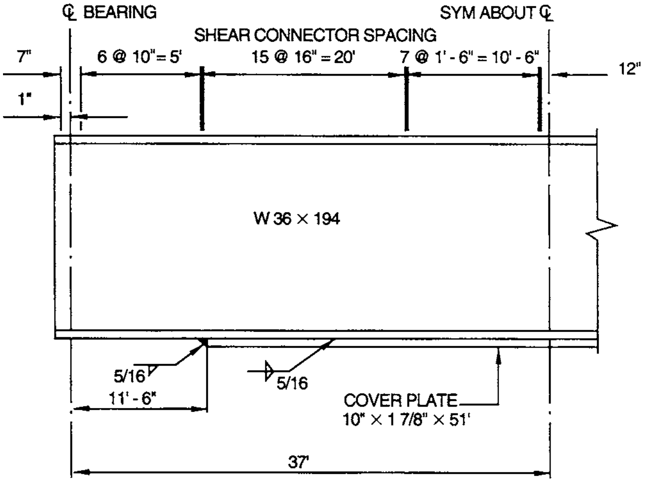


FIGURE 12.11 Shear connector spacing along the top flange of a stringer.

DEAD-LOAD DEFLECTION

$$\begin{aligned} DL&: \delta = 22.5 \times 1.175(74)^4/(29{,}000 \times 17{,}290) = 1.60 \text{ in} \\ SDL&: \delta = 22.5 \times 0.390(74)^4/(29{,}000 \times 34{,}530) = \underline{0.27} \\ \text{Total}&: \qquad 1.87 \text{ in} \end{aligned}$$

Maximum live-load deflection should be checked and compared with 12L/800. If desired, this deflection can be calculated accurately by classical methods, including the effects of changes in moments of inertia. Or the mid-span deflection of a simply supported stringer under AASHTO HS truck loading may be obtained with acceptable accuracy from the approximate formula

$$\delta = \frac{324}{EI} P_T (L^3 - 555L + 4780) \tag{12.6}$$

where P_T = weight, kips, of one front truck wheel multiplied by the live-load distribution factor, plus impact, kips. In this case, $P_T = 10 \times 0.727 + 0.251 \times 10 \times 0.727 = 9.1$ kips. From Table 11.2*b*, for $n = 8$, $I = 52{,}580$. Hence,

$$\delta = \frac{324 \times 9.1}{29{,}000 \times 52{,}580}(74^3 - 555 \times 74 + 4{,}780) = 0.70 \text{ in}$$

And the deflection–span ratio is

$$\frac{0.70}{74 \times 12} = \frac{1}{1200} < \frac{1}{800}$$

Thus, the live-load deflection is acceptable.

12.3 CHARACTERISTICS OF PLATE-GIRDER STRINGER BRIDGES

For simple or continuous spans exceeding about 85 ft, plate girders may be the most economical type of construction. Used as stringers instead of rolled beams, they may be economical even for long spans (350 ft or more). Design of such bridges closely resembles that for bridges with rolled-beam stringers (Arts. 12.1 and 12.2). Important exceptions are noted in this and following articles.

The decision whether to use plate girders often hinges on local fabrication costs and limitations imposed on the depth of the bridges. For shorter spans, unrestricted depth favors plate girders over rolled beams. For long spans, unrestricted depth favors deck trusses or arches. Even then, cable-supported girders may be competitive in cost. Stringent depth restrictions, however, favor through trusses or arches.

Composite construction significantly improves the economy and performance of plate girders and should be used wherever feasible. (See also Art. 12.1.) Advantage also should be taken of continuity wherever possible, for the same reasons.

Spacing. For stringer bridges with spans up to about 175 ft, two lanes may be economically carried on four girders. Where there are more than two lanes, five or more girders should be used at spacings of 7 ft or more. With increase in span, economy improves with wider girder spacing because of the increase in load-carrying capacity with increase in depth for the same total girder area.

For stringer bridges with spans exceeding 175 ft, girders should be spaced about 14 ft apart. Consequently, this type of construction is more advantageous where roadway widths exceed about 40 ft. For two-lane bridges in this span range, box girders may be less costly.

Steel Grades. In spans under about 100 ft, Grade 36 steel often will be more economical than higher-strength steels. For longer spans, however, designers should consider use of stronger steels because some offer maintenance benefits as well as a favorable strength–cost ratio. But in small quantities, these steels may be expensive or unavailable. So where only a few girders are required, it may be uneconomical to use a high-strength steel for a light flange plate extending only part of the length of a girder.

In spans between 100 and 175 ft, hybrid girders, with stronger steels in the flanges than in the web (Art. 10.16), often will be more economical than girders completely of Grade 36 steel. For longer spans, economy usually is improved by making the web of higher-strength steels than Grade 36. In such cases, the cost of a thin web with stiffeners should be compared with that of a thicker web with fewer stiffeners and thus lower fabrication costs. Though high-strength steels may be used in flanges and web, other components, such as stiffeners, bracing, and connection details, should be of Grade 36 steel, because size is not determined by strength.

Haunches. In continuous spans, bending moments over interior supports are considerably larger than maximum positive bending moments. Hence, theoretically, it is advantageous to make continuous girders deeper at interior supports than at mid-span. This usually is done by providing a haunch, usually a deepening of the girders along a pleasing curve in the vicinity of those supports.

For spans under about 175 ft, however, girders with straight soffits may be less costly than with haunches. The expense of fabricating the haunches may more than offset savings in steel obtained with greater depth. With long spans, the cost of haunching may be further increased by the necessity of providing horizontal splices, which may not be needed with straight soffits. So before specifying a haunch, designers should make cost estimates to determine whether its use will reduce costs.

Web. In spans up to about 100 ft, designers may have the option of specifying a web with stiffeners or a thicker web without stiffeners. For example, a $^5/_{16}$-in-thick stiffened plate or a $^7/_{16}$-in-thick unstiffened plate often will satisfy shear and buckling requirements in that span range. A girder with the thinner web, however, may cost more than with the thicker web, because fabrication costs may more than offset savings in steel. But if the unstiffened plate had to be thicker than $^7/_{16}$ in, the girder with stiffeners probably would cost less.

For spans over 100 ft, transverse stiffeners are necessary. Longitudinal stiffeners, with the thinner webs they permit, may be economical for Grade 36 as well as for high-strength steels.

Flanges. In composite construction, plate girders offer greater flexibility than rolled beams, and thus can yield considerable savings in steel. Flange sizes of plate girders, for example, can be more closely adjusted to variations in bending stress along the span. Also, the grade of steel used in the flanges can be changed to improve economy. Furthermore, changes may be made where stresses theoretically permit a weaker flange, whereas with cover-plated rolled beams, the cover plate must be extended beyond the theoretical cutoff location.

Adjoining flange plates are spliced with a groove weld. It is capable of developing the full strength of the weaker plate when a gradual transition is provided between groove-welded material of different width or thickness. AASHTO specifies transition details that must be followed.

Designers should avoid making an excessive number of changes in sizes and grades of flange material. Although steel weight may be reduced to a minimum in that manner, fabrication costs may more than offset the savings in steel.

For simply supported, composite girders in spans under 100 ft, it may be uneconomical to make changes in the top flange. For spans between 100 and 175 ft, a single reduction in thickness of the top flange on either side of mid-span may be economical. Over 175 ft, a reduction in width as well as thickness may prove worthwhile. More frequent changes are economically justified in the bottom flange, however, because it is more sensitive to stress changes along the span. In simply supported spans up to about 175 ft, the bottom flange may consist of three plates of two sizes—a center plate extending over about the middle 60% of the span and two thinner plates extending to the supports. (See Art. 10.16.)

Note that even though high-strength steels may be specified for the bottom flange of a composite girder, the steel in the top flange need not be of higher strength than that in the web. In a continuous girder, however, if the section is not composite in negative-moment regions, the section should be symmetrical about the neutral axis.

In continuous spans, sizes of top and bottom flanges may be changed economically once or twice in a negative-moment region, depending on whether only thickness need be changed or both width and thickness have to be decreased. Some designers prefer to decrease thickness first and then narrow the flange at another location. But a constant-width flange should be used between flange splices. In positive-moment regions, the flanges may be treated in the same way as flanges of simply supported spans.

Welding of stiffeners or other attachments to a tension flange usually should be avoided. Transverse stiffeners used as cross-frame connections should be connected to both girder flanges (Art. 10.11.6). The flange stress should not exceed the allowable fatigue stress for base metal adjacent to or connected by fillet welds. Stiffeners, however, should be welded to the compression flange. Though not required for structural reasons, these welded connections increase lateral rigidity of a girder, which is a desirable property for transportation and erection.

Bracing. Intermediate cross frames usually are placed in all bays and at intervals as close to 25 ft as practical, but not farther apart than 25 ft. Consisting of minimum-size angles, these frames provide a horizontal angle near the bottom flange and V bracing (Fig. 12.12) or X bracing. The angles usually are field-bolted to connection plates welded to each girder web. Eliminating gusset plates and bolting directly to stiffeners is often economical.

Cross frames also are required at supports. Those at interior supports of continuous girders usually are about the same as the intermediate cross frames. At end supports, however, provision must be made to support the end of the concrete deck. For the purpose, a horizontal channel of minimum weight, consistent with concrete edge-beam requirements, often is used near the top flange, with V or X bracing, and a horizontal angle near the bottom flange.

Lateral bracing in a horizontal plane near the bottom flange is sometimes required. The need for such bracing must be investigated, based on a wind pressure of 50 lb/ft^2. (Spans with nonrigid decks may also require a top lateral system.) This bracing usually consists of crossing diagonal angles and the bottom angles of the cross frames.

Bearings. Laminated elastomeric pads may be used economically as bearings for girder spans up to about 175 ft. Welded steel rockers or rollers do not meet seismic requirements and are no longer used. Seismic attenuation bearings, pot bearings, or spherical bearings with teflon guided surfaces for expansion are other alternatives.

Camber. Plate girders should be cambered to compensate for dead-load deflections. When the roadway is on a grade, the camber should be adjusted so that the girder flanges will parallel the profile grade line. For the purpose, designers should calculate dead-load deflection at sufficient points along each span to indicate to the fabricator the desired shape for the unloaded stringer.

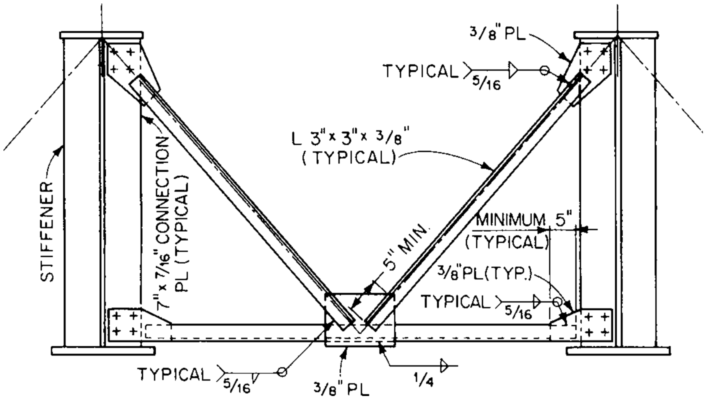


FIGURE 12.12 Intermediate cross frame for a stringer bridge.

12.4 EXAMPLE—LOAD FACTOR DESIGN OF COMPOSITE PLATE-GIRDER BRIDGE

The AASHTO "Standard Specifications for Highway Bridges" allow load factor design (LFD) as an alternative method to allowable stress design for design of simple and continuous beam and girder structures of moderate length, and it is widely used for highway bridges.

Load factor design is a method of proportioning structural members for multiples of the design loads. The moments, shears, and other forces are determined by assuming elastic behavior of the structure. To ensure serviceability and durability, consideration is given to control of permanent deformations under overloads, to fatigue characteristics under service loadings, and to control of live-load deflections under service loadings. To illustrate load factor design, a simply supported, composite, plate-girder stringer of a two-lane highway bridge will be designed. As indicated in the framing plan, Fig. 12.13*a*, the stringers span 100 ft c to c of bearings. The typical cross section, Fig. 12.13*b*, shows a 26-ft-wide roadway flanked by 1-ft 9-in-wide barrier curbs.

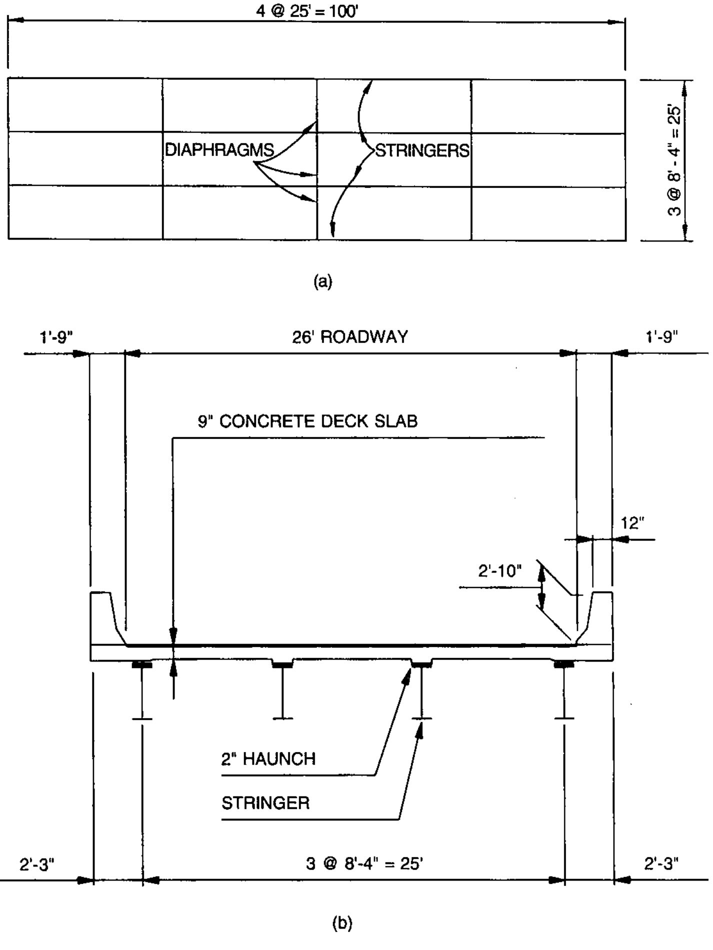


FIGURE 12.13 Two-lane highway bridge with plate-girder stringers. (*a*) Framing plan. (*b*) Typical cross section.

TABLE 12.7 Composite Section Near Supports

(*a*) For dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	55.8		−168			35,170
Concrete 100 × 8.5/24	35.4	37.0	1,310	48,470	210	48,680
	91.2		1,142			83,850
$d_{24} = 1{,}142/91.2 = 12.52$ in						$-12.52 \times 1{,}142 = -14{,}300$
						$I_{NA} =$ 69,550

Distance from neutral axis of steel section to:

Bottom of steel = 30 + 0.88 + 12.52 = 43.40 in

Section modulus, bottom of steel

$S_{sb} = 69{,}550/43.40 = 1{,}602$ in^3

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	55.8		−168			35,170
Concrete 100 × 8.5/8	106.3	37.0	3,933	145,520	640	146,160
	162.1		3,765			181,330
$d_8 = 3{,}765/162.1 = 23.23$ in						$-23.23 \times 3{,}765 = -87{,}460$
						$I_{NA} =$ 93,870

Distance from neutral axis of steel section to:

Bottom of steel = 30 + 0.88 + 23.23 = 54.11 in

Section modulus, bottom of steel

$S_{sb} = 93{,}870/54.11 = 1{,}735$ in^3

Structural steel is Grade 36, with yield strength $f_y = 36$ ksi, and concrete for the deck slab is Class A, with 28-day strength $f'_c = 4000$ psi. Loading is HS25.

12.4.1 Stringer Design Procedure

In the usual design procedure, the concrete deck slab is designed to span between the girders. A section is assumed for the steel stringer and classified as either symmetrical or unsymmetrical, compact or noncompact, braced or unbraced, and transversely or longitudinally stiffened. Section properties

TABLE 12.8 Stresses in Composite Plate Girder 17 ft from Supports

Bending moments, ft·kips

M_{DL}	M_{SDL}	$M_{LL} + M_I$
981	335	1044 (Fig. 12.19)

Stresses at top of steel (compression), ksi

DL: 981 × 12/1244 =	9.46 (S_{st} from Table 12.13)
SDL: 335 × 12/1602 =	2.51 (S_{st} from Table 12.14*a*)
LL + *I*: 1044 × 12/1735 =	7.22 (S_{st} from Table 12.14*b*)
Total:	19.19 < 20

TABLE 12.9 Shear Stresses in Composite Plate Girder, ksi, at Supports

(*a*) Static moment Q, in^3, of flange

Top flange: $Q_t = 12.0 \times 33.39 = 401$
Bottom flange: $Q_b = 17.5 \times 27.43 = 480$

Composite section, $n = 8$

Steel top flange: $Q_{st} = 12.0 \times 7.15 = 86$
Concrete slab: $Q_c = 106.3 \times 13.77 = 1{,}464$

Total: $Q_t = 1{,}550$
Steel bottom flange: $Q_b = 17.5 \times 53.67 = 939$

(*b*) Maximum shears, kips, at supports

V_{DL}	V_{SDL}	$V_{LL} + V_I$
69.5	23.7	75.7

(*c*) Shear stresses, kips/in

Top-flange welds	Bottom-flange welds
DL: $69.5 \times 401/34{,}660 = 0.804$	DL: $69.5 \times 480/34{,}660 = 0.962$
SDL: $23.7 \times 1{,}550/93{,}870 = 0.391$	SDL: $23.7 \times 939/93{,}870 = 0.237$
$LL + I$: $75.7 \times 1{,}550/93{,}870 = 1.250$	$LL + I$: $75.7 \times 939/93{,}870 = 0.757$
$v = 2.445$	$v = 1.956$

of a steel girder alone, and composite section properties of the steel girder and concrete slab are then determined, in a similar way as for allowable stress design, for long- and short-duration loads. Next, flange local buckling is checked for the composite section. Fatigue stress checks are made for the most common connections found in a welded plate girder, such as those for transverse stiffeners, flange plate splices, gusset plates for lateral bracing, and flanges to webs.

The trial section is checked for compactness. The allowable stresses may have to be reduced if the section is noncompact and unbraced. Next, bending strength and shear capacity of the section are checked, and the section is adjusted as necessary. Then, transverse and longitudinal stiffeners are designed, if required. In addition, for a complete design, flange-web welds and shear connectors (fatigue to be included), bearing stiffeners (as concentrically loaded columns), and lateral bracing (for wind loading) are designed and a deflection check is made.

12.4.2 Concrete Slab

The slab is designed to span transversely between stringers in the same way as for the allowable stress method (Art. 12.2) in this example. A 9-in-thick, one-course concrete slab is used. The effective span S, the distance, ft, between flange edges plus half the flange width, is, for an assumed flange width of 16 in (1.33 ft),

$$S = 8.33 - 1.33 + \frac{1.33}{2} = 7.67 \text{ ft}$$

For computation of dead load,

$$\begin{array}{ll}
\text{Weight of concrete slab: } 0.150 \times {}^9/_{12} = 0.113 & \\
{}^3/_8\text{ -in extra concrete in stay-in-place} & \\
\quad\text{forms: } 0.150\ ({}^3/_8)/12 & = 0.005 \\
\text{Future wearing surface} & = \underline{0.025} \\
\text{Total dead load } w_D\text{:} & \quad 0.143 \text{ kip/ft}
\end{array}$$

With a factor of 0.8 applied to account for continuity of slab over more than three stringers, the maximum dead-load bending moment is

$$M_D = \frac{w_D S^2}{10} = \frac{0.143(7.67)^2}{10} = 0.84 \text{ ft} \cdot \text{kips/ft}$$

Maximum live-load moment, with reinforcement perpendicular to traffic, using a factor of 0.8 applied to account for continuity, equals

$$M_L = \frac{(S+2)}{32} P(0.8) \tag{12.7}$$

where P is the load on one rear wheel of a truck. Since $P = 16 \times 1.25 = 20$ kips for an HS25 truck,

$$M_L = \frac{(7.67+2)}{32} 20 \times 0.8 = 4.84 \text{ ft} \cdot \text{kips/ft}$$

Allowance for impact is 30% of this, or 1.45 ft·kips/ft. The total live-load moment then is

$$M_L = 4.84 + 1.45 = 6.29 \text{ ft} \cdot \text{kips/ft}$$

The factored total moment for AASHTO Group I loading on a straight bridge is

$$\begin{aligned} M_T &= 1.3[DL + 1.67(LL + I)] = 1.3(0.84 + 1.67 \times 6.29) \\ &= 14.75 \text{ ft} \cdot \text{kips/ft} \end{aligned}$$

For a strip of slab $b = 12$ in wide, the effective depth d of the steel reinforcement is determined based on the assumption that No. 6 bars with 2.5 in of concrete cover will be used:

$$d = 9 - 2.5 - \frac{({}^6/_8)}{2} = 6.13 \text{ in}$$

For determination of the moment capacity of the concrete slab, the depth of the equivalent rectangular compressive-stress block is given by

$$a = \frac{A_s f_y}{0.85 f_c' b} \tag{12.8}$$

where A_s = the area, in^2, of the reinforcing steel.

For $f_c' = 4$ ksi and the yield strength of the reinforcing steel $F_y = 60$ ksi,

$$a = \frac{60 A_s}{0.85 \times 4 \times 12} = 1.47 A_s$$

Design moment strength ϕM_n is given by

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) \tag{12.9}$$

where the strength reduction factor $\phi = 0.90$ for flexure. If the nominal moment capacity ϕM_n is equated to the total factored moment M_T, the required area of reinforcement steel A_s can be obtained with Eq. (12.9) by solving a quadratic equation:

$$14.75 \times 12 = 0.9 \times 60 A_s \left(6.13 - \frac{1.47 A_s}{2} \right)$$

from which $A_s = 0.58\ \text{in}^2/\text{ft}$. Number 6 bars at 9-in intervals supply 0.59 in^2/ft and will be specified. The area provided should be checked to ensure that its ratio ρ to the concrete area does not exceed 75% of the balanced reinforcement ratio ρ_b.

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{12.10}$$

where the factor $\beta_1 = 0.85$ for $f_c' = 4$-ksi concrete.

$$\rho_b = \frac{0.85 \times 0.85 \times 4}{60}\left(\frac{87}{87 + 60}\right) = 0.0285$$

For the steel area provided,

$$\rho = \frac{0.59}{12 \times 6.13} = 0.008 < (0.75\rho_b = 0.0214) \qquad \text{OK}$$

AASHTO standard specifications state that, at any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment M_u calculated on the basis of modulus of rupture f_r for normal-weight concrete.

$$f_r = 7.5\sqrt{f_c'} \tag{12.11}$$

For $f_c' = 4$ ksi, $f_r = 7.5\sqrt{4000} = 474$ psi. The cracking moment is obtained from

$$M_u = f_r S \tag{12.12}$$

where the section modulus $S = bh^2/6 = 12 \times 8.5^2/6 = 144.5\ \text{in}^3$. (One-half inch is deducted from the section for the wearing course.)

$$M_u = 474 \times \frac{144.5}{12{,}000} = 5.71\ \text{ft} \cdot \text{kips/ft}$$

From Eq. (12.8), the depth of the equivalent rectangular stress block is

$$a = \frac{60 \times 0.59}{0.85 \times 4 \times 12} = 0.87\ \text{in}$$

Substitution of the preceding values in Eq. (12.9) yields the moment capacity

$$\begin{aligned}\phi M_n &= 0.90 \times 0.59 \times \frac{60(6.13 - 0.87/2)}{12} \\ &= 15.12 > (1.2M_u = 6.85\ \text{ft} \cdot \text{kips/ft})\end{aligned}$$

Therefore, the minimum reinforcement requirement is satisfied.

For a complete slab design, serviceability requirements in the AASHTO standard specifications for fatigue and distribution of reinforcement in flexural members also need to be satisfied. Only a typical interior stringer will be designed in this example.

12.4.3 Loads, Moments, and Shears

Assume that the girders will not be shored during casting of the concrete slab. Hence, the dead load on each steel stringer includes the weight of an 8.33-ft-wide strip of slab as well as the weights of steel girder and framing details. This dead load will be referred to as *DL*.

DEAD LOAD CARRIED BY STEEL BEAM, KIPS/FT

Slab: $0.150 \times 8.33 \times {}^{9}/_{12}$	= 0.938
Haunch—16 × 2 in: $0.150 \times 1.33 \times 0.167$	= 0.034
Steel stringer and framing details—assume:	0.327
Stay-in-place forms and additional concrete in forms:	0.091
DL per stringer:	1.390

Maximum moment occurs at the center of the 100-ft span and equals

$$M_{DL} = \frac{1.39(100)^2}{8} = 1,738 \text{ ft}\cdot\text{kips}$$

Maximum shear occurs at the supports and equals

$$V_{DL} = \frac{1.39 \times 100}{2} = 69.5 \text{ kips}$$

Barrier curbs will be placed after the concrete slab has cured. Their weights may be equally distributed to all stringers. In addition, provision will be made for a future wearing surface, weight 25 lb/ft^2. The total superimposed dead load will be designated *SDL*.

DEAD LOAD CARRIED BY COMPOSITE SECTION, KIPS/FT

Two barrier curbs: $2 \times 0.530/4$	= 0.265
Future wearing surface: 0.025×8.33	= 0.208
SDL per stringer:	0.473

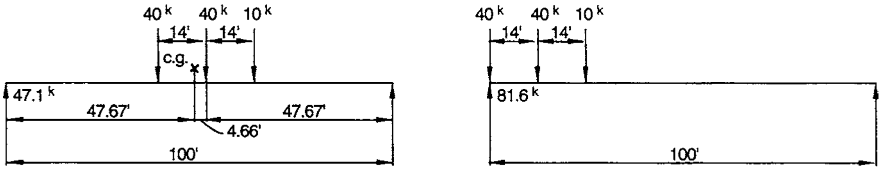
Maximum moment occurs at mid-span and equals

$$M_{SDL} = \frac{0.473(100)^2}{8} = 592 \text{ ft}\cdot\text{kips}$$

Maximum shear occurs at supports and equals

$$V_{SDL} = \frac{0.473 \times 100}{2} = 23.7 \text{ kips}$$

The HS25 live load imposed may be a truck load or lane load. But for this span, the truck load shown in Fig. 12.14*a* governs. The center of gravity of the three axles lies between the two heavier loads and is 4.66 ft from the center load. Maximum moment occurs under the center-axle load when its distance from mid-span is the same as the distance of the center of gravity of the loads from mid-span, or 4.66/2 = 2.33 ft. Thus, the center load should be placed 100/2 − 2.33 = 47.67 ft from a support (Fig. 12.14*a*). Then, the maximum moment is



(a) (b)

FIGURE 12.14 Positions of loads on a plate girder for maximum stress. (*a*) For maximum moment in the span. (*b*) For maximum shear in the span.

$$M_T = \frac{90(100/2 + 2.33)^2}{100} - 40 \times 14 = 1905 \text{ ft} \cdot \text{kips}$$

The distribution of the live load to a stringer may be obtained from Table 10.15 for a bridge with two traffic lanes.

$$\frac{S}{5.5} = \frac{8.33}{5.5} = 1.516 \text{ wheels} = 0.758 \text{ axle}$$

Hence, the maximum live-load movement is

$$M_{LL} = 0.758 \times 1905 = 1444 \text{ ft} \cdot \text{kips}$$

While this moment does not occur at mid-span as do the maximum dead-load moments, stresses due to M_{LL} may be combined with those from M_{DL} and M_{SDL} to produce the maximum stress, for all practical purposes.

For maximum shear with the truck load, the outer 40-kip load should be placed at the support (Fig. 12.14*b*). Then, the shear is

$$V_T = \frac{90(100 - 14 + 4.66)}{100} = 81.6 \text{ kips}$$

Since the stringer receives 0.758 axle load, the maximum shear on the stringer is

$$V_{LL} = 0.758 \times 81.6 = 61.9 \text{ kips}$$

Impact is taken as the following fraction of live-load stress:

$$I = \frac{50}{L + 125} = \frac{50}{100 + 125} = 0.222$$

Hence, the maximum moment due to impact is

$$M_I = 0.222 \times 1444 = 321 \text{ ft} \cdot \text{kips}$$

and the maximum shear due to impact is

$$V_I = 0.222 \times 61.9 = 13.8 \text{ kips}$$

MID-SPAN BENDING MOMENTS, FT · KIPS

M_{DL}	M_{SDL}	$M_{LL} + M_I$
1738	592	1765

END SHEAR, KIPS

V_{DL}	V_{SDL}	$V_{LL} + V_I$	Total V
69.5	23.7	75.7	168.9

The factored moments and shears will be obtained from the combination of dead load (*DL*) plus live load and impact (*LI* + *I*). For the AASHTO Group I loading combination, the factored moment is

$$M_f = \gamma[\beta_D M_{DL} + 1.67(M_L + M_I)] \tag{12.13}$$

and the factored shear is

$$V_f = \gamma[\beta_D V_{DL} + 1.67(V_L + V_I)] \tag{12.14}$$

where γ is the load factor ($\gamma = 1.30$ for moment and 1.41 for shear) and β is 1.0. Then,

$$M_{fT} = 1.30[1.0 \times 1738 + 1.0 \times 592 + 1.67 \times (1444 + 321)] = 6862$$
$$V_{fT} = 1.41[1.0 \times 69.5 + 1.0 \times 23.7 + 1.67 \times (61.9 + 13.8)] = 309.7$$

FACTORED BENDING MOMENTS AT MID-SPAN, FT · KIPS

M_{fDL}	M_{fSDL}	$M_{fLL} + M_{fI}$	M_{fT}
2260	770	3832	6862

FACTORED END SHEAR, KIPS

V_{fDL}	V_{fSDL}	$V_{fLL} + V_{fI}$	V_{fT}
98.0	33.4	178.3	309.7

12.4.4 Trial Girder Section

A trial section with a web plate 60 × 7/16 in is assumed. Bottom-flange area can be estimated from

$$A_{sb} = \frac{12(M_{DL} + M_{LL} + M_I)}{F_y d} \qquad (12.15)$$

For the preceding bending moments,

$$A_{sb} = \frac{12(2260 + 770 + 3832)}{36 \times 60} = 38.1 \text{ in}^2 \qquad (12.16)$$

Since the part of the web below the neutral axis will also carry some force, a bottom flange 20 × 1½ in ($A_{sb} = 30.0$ in^2) will be tried first. For the top flange plate, $A_{sb}/2 = 15.0$ in^2, a top flange of 16 × 1 in will be tried.

The concrete section for an interior stringer, not including the concrete haunch, is 8 ft 4 in wide (c to c of stringers) and 8½ in deep (½ in of slab is deducted from the concrete depth for the wearing course). The concrete area $A_c = 8.33 \times 12 \times 8.50 = 850$ in^2. Thus, this is an unsymmetrical composite section.

Check for Local Buckling. The trial section is assumed to be braced and noncompact. The width–thickness ratio b'/t of the projecting compression-flange element may not exceed

$$\frac{b'}{t} = \frac{69.6}{\sqrt{F_y}} \qquad (12.17)$$

where b' is the width of the projecting element, t is the flange thickness, and F_y is the specified yield stress, ksi. For flange with $b = 16$ in and $F_y = 36$ ksi, the thickness should be at least

$$t = \frac{\sqrt{36}}{69.6} \times \frac{16}{2} = 0.69 \text{ in}$$

The 1-in-thick top flange is satisfactory.

Properties of Trial Section. The trial section is shown in Fig. 12.15. The computations for the location of the neutral axis and for the section moduli S_{st} and S_{sb} of the trial plate-girder section are tabulated in Table 12.10.

For unsymmetrical girders with transverse stiffeners but without longitudinal stiffeners, the minimum thickness of the web is obtained from

$$\frac{D_c}{t_w} \le \frac{577}{\sqrt{F_y}} \qquad D_c > D/2 \qquad (12.18)$$

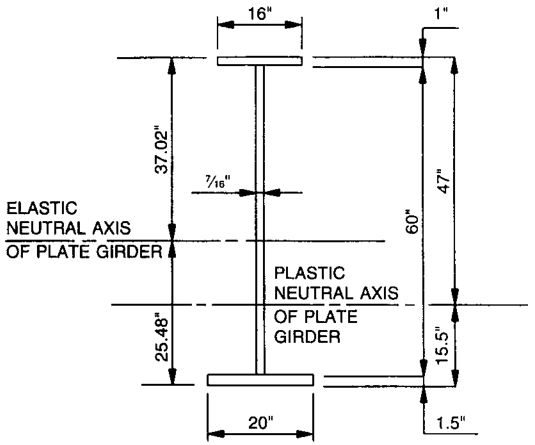


FIGURE 12.15 Cross section assumed for plate girder for load factor design.

where D_c is the clear distance, in, between the neutral axis and the compression flange; D is the web depth, in; and t_w is the web thickness, in. For the trial section, $D_c = 36.02 > (D/2 = 30)$. Hence, from Eq. (12.18), the web thickness should be at least

$$t_w = \frac{D_c\sqrt{F_y}}{577} = \frac{36.02\sqrt{36}}{577} = 0.38 \text{ in, or } {}^3/_8 \text{ in}$$

Since the assumed ${}^7/_{16}$-in web thickness exceeds ${}^3/_8$ in, the requirement for minimum web thickness without longitudinal stiffeners is met.

The computations for the location of the neutral axis and for the section moduli are given in Table 12.11 for the composite section, with $n = 8$ for short-time loading, such as live load and impact, and $n = 3 \times 8 = 24$ for long-time loading, such as superimposed dead loads. To locate the neutral axis, moments are taken about middepth of the girder web. Depth of the concrete haunch atop the

TABLE 12.10 Steel Section for Maximum Factored Moment

Material	A	d	Ad	Ad^2	I_o	I
Top flange 16 × 1	16.0	30.50	488	14,880		14,880
Web 60 × ${}^7/_{16}$	26.3				7,880	7,880
Bottom flange 20 × $1^1/_2$	30.0	−30.75	−923	28,370		28,370
	72.3		−435			51,130
$d_s = -435/72.3 = -6.02$ in					−6.02 × 435 =	−2,620
						I_{NA} = 48,510

Distance from neutral axis of steel section to:

Top of steel = 30 + 1 + 6.02 = 37.02 in

Bottom of steel = 30 + 1.50 − 6.02 = 25.48 in

Section moduli

Top of steel	Bottom of steel
S_{st} = 48,510/37.02 = 1,310 in^3	S_{sb} = 48,510/25.48 = 1,904 in^3

TABLE 12.11 Composite Section for Maximum Factored Moment

(*a*) For superimposed dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	72.3		–435			51,130
Concrete 100 × 8.5/24	35.4	37.25	1319	49,120	210	49,330
	107.7		884			100,460
$d_{24} = 884/107.7 = 8.21$ in						$-8.21 \times 884 = -7,260$
						$I_{NA} = 93,200$

Distance from neutral axis of composite section to:

Top of steel = 31.00 – 8.21 = 22.79 in

Bottom of steel = 31.50 + 8.21 = 39.71 in

Top of concrete = 22.79 + 2 + 8.5 = 33.29 in

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 93,200/22.79$ $= 4,089$ in^3	$S_{sb} = 93,200/39.71$ $= 2,347$ in^3	$S_c = 93,200/33.29$ $= 2,800$ in^3

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	72.3		–435			51,130
Concrete 100 × 8.5/8	106.3	37.25	3,958	147,440	640	148,080
	178.6		3,523			199,210
$d_8 = 3523/178.6 = 19.73$ in						$-19.73 \times 3,523 = -69,510$
						$I_{NA} = 129,700$

Distance from neutral axis of composite section to:

Top of steel = 31.00 – 19.73 = 11.27 in

Bottom of steel = 31.00 + 19.73 = 50.73 in

Top of concrete = 11.27 + 2 + 8.5 = 21.77 in

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 129,700/11.27$ $= 11,510$ in^3	$S_{sb} = 129,700/50.73$ $= 2,557$ in^3	$S_c = 129,700/21.77$ $= 5,958$ in^3

girder is assumed to be 2 in. In addition, since the girder is composite, for prevention of flange buckling, the width–thickness ratio of the projecting element of the compression flange may not exceed

$$\frac{b'}{t} = \frac{69.9}{\sqrt{1.3 f_{dl1}}} \tag{12.19}$$

where f_{dl1} is the compression stress ksi, in the top flange due to noncomposite dead load.

$$f_{dl1} = \frac{2260 \times 12}{1310} = 20.70 \text{ ksi}$$

From Eq. (12.19), for a flange width of 16 in, the flange thickness t_1 should be at least

$$t_1 = \frac{\sqrt{1.3 \times 20.7}}{69.6} \times \frac{16}{2} = 0.6 \text{ in}$$

The 1-in-thick top flange is satisfactory.

TABLE 12.12 Categories and Allowable Fatigue Stress Ranges for Connections*

Connection type	Stress type	Category	Allowable stress range F_{sr}, ksi 500,000 cycles	 100,000 cycles
Toe of transverse stiffener	Tension or reversal	C	21	35.5
Groove weld at flanges	Tension of reversal	B	29	49
Gusset plate for lateral bracing	Tension or reversal	B		
Flange-to-web weld	Shear	F	12	15

*See AASHTO Specifications for full requirements that apply.

12.4.5 Fatigue Stresses

In the next step of the design procedure, fatigue stresses will be investigated. The four-stringer system is considered to have multiple load paths. A single fracture in a member cannot lead to collapse of the bridge. Hence, the structure is not fracture-critical.

Determination of the allowable stress range F_{sr} for fatigue is based on the stress category for the connection under consideration, the type of load path (redundant or nonredundant), and the stress cycle.

The bridge is located on a major highway (Case II) with an average daily truck traffic in one direction (ADTT) less than 2500. The plate girders incorporate the four connection types tabulated in Table 12.12 with corresponding stress types and categories. For main (longitudinal) load-carrying members, the number of stress cycles of the maximum stress range for Case II, with ADTT < 2500, the AASHTO standard specifications specify 500,000 loading cycles for truck loading and 100,000 for lane loading. Table 12.13 also lists for the four types of connections the allowable stress ranges F_{sr} for the redundant-load-path structure based on the connection stress category and the number of stress cycles. The fatigue stress in the bottom flange is checked for unfactored HS20 loading, as stated by AASHTO, on the composite section. For live-load moment plus impact, $M = 1412$ ft·kips, and the corresponding stress is

$$f_b = \frac{1412 \times 12}{2557} = 6.6 \text{ ksi}$$

Since the plate girder under consideration is simply supported, the minimum live-load moment would be zero and the live-load stress range becomes $f_{sr} = 6.6 \text{ ksi} < F_{sr} = 21$ ksi. The section is OK for fatigue.

12.4.6 Check for Compactness

The allowable stresses may have to be reduced if the section is noncompact and unbraced. Composite beams in positive bending qualify as compact when the web depth–thickness ratio D/t_w of the steel section meets the following requirement:

TABLE 12.13 Stresses, ksi, in Composite Girder at Section of Maximum Moment

Top of steel (compression)		Bottom of steel (tension)	
$DL: f_b = 2{,}260 \times 12/1{,}310 =$	20.70	$DL: f_b = 2{,}260 \times 12/1{,}904 =$	14.24
$SDL: f_b = 770 \times 12/4{,}089 =$	2.26	$SDL: f_b = 770 \times 12/2{,}347 =$	3.94
$LL + I: f_b = 3{,}832 \times 12/11{,}510 =$	3.99	$LL + I: f_b = 3{,}832 \times 12/2{,}557 =$	17.98
Total:	26.95 < 36	Total:	36.16 ≈ 36
Top of concrete			
$SDL: f_c = 770 \times 12/(2{,}800 \times 8) =$	0.41		
$LL + I: f_c = 3{,}832 \times 12/(5{,}958 \times 8) =$	0.96		
	1.37 < 4.0		

$$\frac{D}{t_w} \le \frac{608}{\sqrt{F_y}} \tag{12.20}$$

where t_w is the web thickness, in, and D is the clear distance, in, between the flanges. For composite beams used in simple spans, D may be replaced by $2D_{cp}$, the distance, in, from the compression flange to the neutral axis in plastic bending. The compression depth of the composite section in plastic bending, including the slab, may not exceed

$$d_c = \frac{d + t_s}{7.5} \tag{12.21}$$

where d is the depth of the steel girder, in, and t_s is the thickness, in, of slab.

$$d_c = \frac{d + t_s}{7.5} = \frac{62.5 + 8.5}{7.5} = 9.47 \text{ in}$$

Therefore, the maximum allowed $D_{cp} = 9.47 - 8.5 = 0.97$ in. From Eq. (12.20), with D replaced by $2D_{cp} = 2 \times 0.97 = 1.94$,

$$t_w = \frac{\sqrt{36}}{608} \times 1.94 = 0.02 < {}^7/_{16} \text{ in}$$

The section meets the requirement for compactness.

Check of Unbraced Length of Top Flange. For live loads, the top flange of the girder is continuously supported by the concrete deck slab. But it is necessary to check the unbraced length L_b of the top flange for dead loads on the noncomposite section. For compact sections, spacing of lateral bracing of the compression flange may not exceed

$$\frac{L_b}{r_y} = \frac{[3.6 - 2.2(M_1/M_u)]10^3}{F_y} \tag{12.22}$$

where r_y is the radius of gyration with respect of the y–y axis, in, M_1 is the smaller moment at the end of the unbraced length of the member, M_u is the maximum bending strength $= F_y Z$, and Z is the plastic section modulus, in^3. For the 16 × 1-in top flange,

$$r_y = \sqrt{\frac{I}{A}} = \sqrt{\frac{1 \times 16^3/12}{1 \times 16}} = 4.62 \text{ in}$$

To determine the plastic modulus, Z, the location of the axis that divides the section into two equal areas has to be found. The total area A of the girder is 72.25 in^2 (Table 12.10), and $A/2 = 36.13$ in^2. If $\bar{y}$ is the distance from top of steel to the axis, then $\bar{y} - 1$ is the web length from the axis to the flange. Since the web thickness is $^7/_{16}$ in and the flange area $A_f = 16$ in^2, $16 + (\bar{y} - 1)(^7/_{16}) = 36.83$, and $\bar{y} = 47.0$ in (Fig. 12.15). Z is computed by taking moments about the axis:

$$\begin{aligned} Z &= (16 \times 46.5 + 20.13 \times 23) + (30 \times 14.75 + 6.13 \times 7) \\ &= 1692 \text{ in}^2 \end{aligned}$$

The bending strength then is

$$M_u = F_y Z = 36 \times \frac{1692}{12} = 5076 \text{ ft} \cdot \text{kips}$$

From Eq. (12.22) with $M_1 = M_{DL} = 1738$ ft·kips, the maximum allowable unbraced length is

$$L_b = \frac{4.62[3.6 - 2.2(1738/5076)]10^3}{36} = 365 \text{ in}$$

Since the spacing of bracing cross frames is 25 ft = 300 in and L_b is larger, the section may be treated as braced and compact.

12.4.7 Bending Strength of Girder

The flexural stresses in the composite section of the interior girder are checked for all the factored loads to ensure that the maximum stresses do not exceed $F_y = 36$ ksi. The computations in Table 12.13 indicate that the composite section is OK.

12.4.8 Shear Capacity of Girder

For girders with transverse stiffeners, shear capacity V_u, ksi, is given by

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \qquad \frac{d_o}{D} \le 3 \qquad \frac{d_o}{D} \le \frac{67,600}{(D/t_w)^2} \tag{12.23}$$

where V_p = shear yielding strength of web, ksi = $0.58Dt_wF_y$
d_o = spacing, in, of intermediate stiffeners
D = clear distance, in, between flanges
t_w = web, thickness, in
C = web buckling coefficient (Art. 10.11.4)

Stiffeners are usually equally spaced between cross frames. Spacing ranges up to the maximum of 1.5D for the first stiffener. The cross frames in the example are spaced about 25 ft apart. For a first trial, $d_o = 25/2 = 12.50$ ft = 150 in.

$$\frac{d_o}{D} = \frac{150}{60} = 2.5 < 3 \qquad \text{OK}$$

$$\frac{67,600}{[60/(^7/_{16})]^{12}} = 3.6 > \frac{d_o}{D} \qquad \text{OK}$$

The plastic shear force is

$$V_p = 0.58 \times 36 \times 60 \times {}^7/_{16} = 548 \text{ kips}$$

The coefficient C, the buckling shear stress divided by the shear yield stress, is computed from

$$C = \frac{45,000k}{(D/t_w)^2 F_y} \qquad \frac{D}{t_w} > 237\sqrt{\frac{k}{F_y}} \tag{12.24}$$

where

$$k = 5\left[1 + \frac{1}{(d_o/D)^2}\right] = 5\left[1 + \frac{1}{(150/60)^2}\right] = 5.8$$

and

$$D/t_w = 60/(^7/_{16}) = 137$$

$$C = \frac{45{,}000 \times 5.8}{137^2 \times 36} = 0.39$$

From Eq. (12.23), the shear capacity of the girder is

$$V_u = 548\left[0.39 + \frac{0.87(1-0.39)}{\sqrt{1-(2.5)^2}}\right] = 322 \text{ kips}$$

The total factored end shear $V_{\max} = 309.7$ kips < 322 kips. Thus, the section is adequate for shear.

12.4.9 Transverse Stiffener Design

For girders that do not meet the shear capacity requirement $V_u = CV_p$, transverse stiffeners are required. The trial section in this example meets this requirement. Girders designed to meet the shear requirement without transverse stiffeners normally have thicker webs but usually cost less than girders designed with thinner webs and transverse stiffeners, because of high welding costs for attaching stiffeners to webs. Another added advantage of a design without transverse stiffeners is elimination of the fatigue-prone welds between webs and stiffeners.

12.4.10 Shear Connectors

The horizontal shears at the interface of the concrete slab and steel girder are resisted by shear connectors throughout the simply supported span to develop composite action. Shear connectors are mechanical devices, such as welded studs or channels, placed in transverse rows across the top flange of the girder and embedded in the slab. The shear connectors for the girder will be designed for ultimate strength and the number of connectors provided for that purpose will be checked for fatigue.

For ultimate strength, the number N_1 of shear connectors required between a section of maximum positive moment and an adjacent end support should be at least

$$N_1 = \frac{P}{\phi S_u} \tag{12.25}$$

where S_u = ultimate strength of a shear connector, kips
ϕ = reduction factor = 0.85
P = force, kips in the concrete slab taken as the smaller of P_1 and P_2
$P_1 = A_s F_y$
$P_2 = 0.85 f'_c b t_s$
A_s = area, in^2, of the steel section
b = effective width, in slab for composite action
t_s = slab thickness, in
$P_1 = 72.3 \times 36 = 2603$ kips
$P_2 = 0.85 \times 4 \times 100 \times 8.5 = 2890 > 2603$ kips (Steel strength governs)

Hence, P for Eq. (12.25) = 2603 kips.

For shear connectors, welded studs $^7/_8$ in in diameter and 6 in long will be used. According to AASHTO standard specifications, the ultimate strength S_u, kips, of welded studs for $h/d > 4$, where h is stud height, in, and d = stud diameter, in, may be determined from

$$S_u = 0.4d^2\sqrt{f'_c E_c} \tag{12.26}$$

where E_c = modulus of elasticity of the concrete, $\text{ksi} = 1800\sqrt{f'_c} = 3600$ ksi. For the $^7/_8$-in-diameter welded studs,

$$S_u = 0.4(^7/_8)^2\sqrt{4 \times 3600} = 36.75 \text{ kips}$$

from which the number of studs required is

$$N_1 = \frac{2603}{0.85 \times 36.75} = 83$$

With the studs placed in groups of three, there should be at least 28 groups on each half of the girder.

Pitch is determined by fatigue requirements. The allowable load range, kips per stud, is given by Eq. (12.4), with $\alpha = 10.6$ for 500,000 cycles of loading. Hence, the allowable load range is

$$Z_r = 10.6(^7/_8)^2 = 8.12 \text{ kips}$$

At the supports, the shear range $V_r = 75.7$ kips, the shear produced by live load plus impact service loads. Consequently, with $n = 8$ for the concrete, the transformed concrete area equal to 106.3 in^2, and $I = 129{,}700$ in^4 from Table 12.11*b*, the range of horizontal shear is

$$S_r = \frac{V_r O}{I} = \frac{75.7 \times 106.3 \times 17.52}{129{,}700} = 1.087 \text{ kips/in}$$

The pitch required for stud groups near a support is

$$p = \frac{3Z_r}{S_r} = \frac{3 \times 8.12}{1.087} = 22.41 \text{ in}$$

The average pitch required for ultimate strength for 28 groups between mid-span and a support is $^1/_2 \times 100 \times {}^{12}/_{28} = 21$ in. Use three $^7/_8$-in-diameter by 6-in-long studs per row, spaced at 18 in.

See also Arts. 12.8.4 to 12.8.6.

12.5 CHARACTERISTICS OF CURVED-GIRDER BRIDGES

Past practice in design of new highways often located bridges first, then aligned the roadway with them. Current practice, in contrast, usually fits bridges into the desired highway alignment. Since curved crossings are sometimes unavoidable, and curved ramps at interchanges often must span other highways, railroads, or structures, bridges in those cases must be curved. Plate or box girders usually are the most suitable type of framing for such bridges.

Though the deck may be curved in accordance with the highway alignment, the girders may be straight or curved between skewed supports. Straight girders require less steel and have lower fabrication costs. But curved girders offer better appearance, and often the overall cost of a bridge with such girders may not be greater than that of a structure with straight members. Curved girders may reduce the number of foundations required because longer spans may be used; deck design and construction is simpler, because girder spacing and deck overhangs may be kept constant throughout the span; and cost savings may accrue from use of continuous girders, which may not be feasible with straight, skewed girders. Consequently, curved girders are generally used in curved bridges.

Curved girders introduce a new dimension in bridge design. The practice used for straight stringers of distributing loads to an individual stringer, as indicated in a standard specification, and then analyzing and designing the stringer by itself, cannot be used for curved-girder bridges. For these structures, the entire superstructure must be designed as a unit. Diaphragms or cross frames as well as the stringers serve as main load-carrying members, because of the torsion induced by the curvature.

Analyses of such grids are very complicated, because they are statically indeterminate to a high degree. Numerous computer programs, however, are available for performing the analyses. In addition, experience with rigorous analyses indicates that under certain conditions approximate methods give sufficiently accurate results.

The approximate methods described in this article are suitable for manual computations. They appear to be applicable to concentric, circular stringers where the arc between supports subtends an angle not much larger than about 0.5 radian, or about 30°. Also, where the spans are continuous, the methods may be used if the sum of the central angles subtended by each span does not exceed 90°. Accuracy of these methods, however, also seems to depend on the flexural rigidity of the deck in the radial direction and of the diaphragms.

The limitation of central angle indicates that the maximum span, along the arc, for a radius of curvature of 300 ft is about 300 × 0.5 = 150 ft for the approximate analysis. If the curved span is 200 ft, the approximate method should not be used unless the radius is at least 200/0.5 = 400 ft.

Each simply supported or continuous girder should have at least one torsionally fixed support.

For box girders, in addition, accuracy depends on the ratio of bending stiffness to torsional rigidity EI/GK, where E is the modulus of elasticity, G is the shearing modulus, I is the moment of inertia for longitudinal bending, and K is the torsional constant for the radial cross section.

For a hollow, rectangular tube,

$$K = \frac{4A^2}{\Sigma(l/t)} \tag{12.27}$$

where A = area, in^2, enclosed within the mean perimeter of tube
l = length of a side, in
t = thickness of that side, in

For inclusion in the summation in the denominator, a concrete slab in composite construction should be transformed into an equivalent steel plate by dividing the concrete cross-sectional area by the modular ratio n.

If the central angle of a curved span is about 0.5 rad, the approximate method should give satisfactory results if the weighted average of EI/GK in the span does not exceed 2.5.

A curved-girder bridge may have open framing, closed framing, or a combination of the two types. In open framing, curved plate girders are assisted in resisting torsion only by cross frames, diaphragms, or floorbeams at intervals along the span. In closed framing, the curved members may be box girders or plate girders assisted in resisting torsion by horizontal lateral bracing as well as by cross frames, diaphragms, or floorbeams.

12.5.1 Approximate Analysis of Open Framing

The approximate method for open framing derives from a rigorous method based on consistent deformations. Various components of the structure when distorted by loads must retain geometric compatibility with each other and simultaneously stay in equilibrium. The equations developed for these conditions can be satisfied only by a unique set of internal forces. In the rigorous method, a large number of such equations must be solved simultaneously. In the approximate method, considerable simplification is achieved by neglecting the stiffness of the plate girders in St. Venant (pure) torsion.

In the following, girders between the bridge centerline and the center of curvature are called **inner girders.** The rest are called **outer girders.**

The method will be described for a bridge with concentric circular stringers, equally spaced. Thus, for the four girders shown in Fig. 12.16*a*, if the distance from outer girder G_1 to inner girder G_4 is D, the girder spacing is $D/3$. The radius of the bridge centerline is R and of any girder G_n, R_n. Diaphragms are equally spaced at distance d apart along the centerline and placed radially between the girders.

Initially, the girders are assumed to be straight, and the span of each girder is taken as its developed length between supports. Preliminary moments M_p and shears V_p are computed as for straight girders.

These values must be corrected for the effects of curvature. The primary effect is a torque acting on every radial cross section of each girder. The torque per unit length at any section of a girder G_n is given approximately by

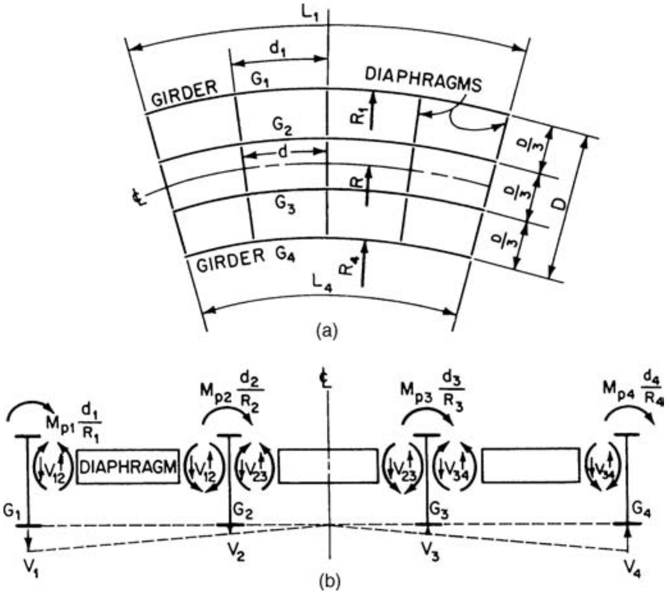


FIGURE 12.16 Curved-girder highway bridge. (*a*) Framing plan. (*b*) Cross section through the bridge at a diaphragm.

$$T_n = \frac{M_{pn}}{R_n} \tag{12.28}$$

where M_{pn} = preliminary bending moment at section and R_n = radius of G_n. If the diaphragm spacing along G_n is d_n and M_{pn} is taken as the preliminary moment at a diaphragm, then the total torque between diaphragms is

$$M_{tn} = \frac{M_{pn} d_n}{R_n} = \frac{M_{pn} d}{R} \tag{12.29}$$

This torque must be resisted by end moments in the diaphragms (Fig. 12.16*b*).

For equilibrium, the end moments on a diaphragm must be balanced by end shears forming an oppositely directed couple. For example, the diaphragm between G_2 and G_3 in Fig. 12.16*b* is subjected to end shears V_{23}. Also, the diaphragm between G_1 and G_2 is subjected to shears V_{12}. Consequently, G_2 is acted on by a net downward force V_2, called a *V* load, at the diaphragm,

$$V_2 = V_{12} + V_{23} \tag{12.30}$$

where upward forces are taken as positive and downward forces as negative.

The *V* loads applied by the diaphragms are treated as additional loads on the girders.

For a bridge with two girders, the *V* load on the inner girder equals that on the outer girder, at a specific diaphragm, but is oppositely directed. Determined by equilibrium conditions at the diaphragm, this *V* load may be computed from

$$V = \frac{M_{p1} + M_{p2}}{K} \tag{12.31}$$

where $K = RD/d$ and D = girder spacing in two-girder bridges and distance between inner and outer girders in bridges with more than two girders.

TABLE 12.14 Values of C for Eq. (12.32)

No. of girders	2	3	4	5	6	7	8	9	10
C	1.00	1.00	1.11	1.25	1.40	1.56	1.72	1.88	2.04

For a bridge with more than two girders, the method assumes that the V load on a girder at a diaphragm is proportional to the distance of the girder from the centerline of the bridge. Then, equilibrium conditions require that the V load on the outer girder of a multigirder bridge be computed from

$$V = \frac{\sum M_{pn}}{CK} \tag{12.32}$$

where C = constant given in Table 12.14. The numerator in Eq. (12.31) consists of the sum of the preliminary moments in the girders at the line of diaphragms. Thus, for the four-girder bridge in Fig. 12.16*b*, the V load on G_1 and G_4 equals

$$-V_1 = V_4 = \frac{M_{p1} + M_{p2} + M_{p3} + M_{p4}}{1.11K} \tag{12.33}$$

By proportion, $-V_2 = V_3 = V_4/3$.

The bending moment produced by the V loads at any section of a girder G_n must be added to the preliminary moment at that section to produce the final bending moment M_n there. Thus,

$$M_n = M_{pn} + M_{vn} \tag{12.34}$$

where M_{vn} = bending moment produced by V loads. Similarly, the shear due to the V loads must be added to the preliminary shears to yield the final shears. Stresses are computed in the same way as for straight girders.

Between diaphragms, the girder flanges resist the torsion. At any section, the stresses in the top and bottom flanges of a girder provide a couple equal to the torque but oppositely directed. The forces comprising this couple induce lateral bending in the flanges. If q_n is the force per unit length of flange in girder G_n resisting torque,

$$q_n = \frac{M_n}{R_n h_n} \tag{12.35}$$

where h_n = distance between centroids of flanges. Each flange may be considered to act under this loading as a continuous beam spanning between diaphragms. The maximum negative moment for design purposes may be taken as

$$M_{Ln} = -\frac{0.1 M_n d_n^2}{R_n h_n} \tag{12.36}$$

The stress due to lateral bending should be added to that due to M_n to obtain the maximum stress in each flange. Where provision is made for composite action, however, the lateral bending stress in that flange may be neglected.

For preliminary design purposes, a rough approximation of the effects of curvature may be obtained by use of

$$p = 5.25 \frac{(1+r) m L_c^2}{CRD} \tag{12.37}$$

where p = percent increase in moment in outer girder due to curvature

$$r = \frac{\text{loading on inner girder}}{\text{loading on outer girder}}\left(\frac{R'}{R}\right)^2$$

R' = radius of curvature of inner girder
R = radius of curvature of outer girder
m = number of girders
L_c = developed length of outer girder between supports when simply supported or between inflection points when continuous
C = constant given by Table 12.14
D = distance between inner and outer girders

12.5.2 Approximate Analysis of Closed Framing

Analysis of bridges with box girders or similar boxlike framing must take into account the torsional stiffness of these members. The method to be described is based on the following assumptions.

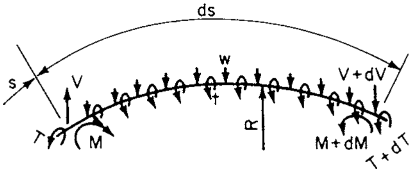


FIGURE 12.17 Forces acting on a differential length *ds* of a girder curved to radius *R*. The distributed vertical load *w* and torque *t* cause vertical end shears *V*, end moments *M*, and end torques *t*.

Girder cross sections are symmetrical about the vertical axis. Supports are radial. Curvature may vary so long as it does not change direction within a span. Diaphragms prevent distortion of the cross sections. Secondary stresses due to torsional warping are negligible.

Differential equations for determining the internal forces acting on a curved girder can be obtained from the equilibrium conditions for a differential segment (Fig. 12.17). Because upward and downward vertical forces must balance, the shear V is related to the loading w by

$$\frac{dV}{ds} = -w \tag{12.38}$$

Thus, as for a straight beam, the change in shear between any two sections of the girder equals the area of the load diagram between those sections. Because the sum of the moments about a radial plane must equal zero, bending moments M, torques T, and shears V are related by

$$\frac{dM}{ds} = -\frac{T}{R} + V \tag{12.39}$$

where R = radius of curvature of girder. In the approximate method, with the limitations on central angle subtended by the span and on the ratio of bending stiffness to torsional stiffness, the T/R term can be ignored. Thus, the equation becomes

$$\frac{dM}{ds} = V \tag{12.40}$$

As for straight beams, the change in bending moments between any two sections of the girder equals the area of the shear diagram between those sections.

Hence, bending moments in a curved girder of the closed-framing type may be computed approximately by treating it as a straight beam with span equal to the developed length of the curve.

A third equation is obtained by taking moments about a tangential plane:

$$\frac{dT}{ds} = \frac{M}{R} - t \tag{12.41}$$

where t = applied torque. With the bending moments throughout the girder known, the torque at any section can be found from Eq. (12.41). [For a more rigorous solution, Eqs. (12.38), (12.39), and (12.41) may be solved simultaneously. This can be done by differentiating Eq. (12.39), solving for dT/ds, substituting the result in Eq. (12.41), and then solving the resulting second-order differential equations.]

Equation (12.41) indicates that the change in torque between any two sections of the girder equals the area of the $M/R–t$ diagram between those sections. Consequently, the torque on a curved girder of the closed-framing type can be determined by a method similar to the conjugate-beam method for determining deflections. In the approximate method, however, the moments M determined from Eq. (12.40) are used instead of those from the more complex rigorous solution.

Thus, first the bending-moment diagram (Fig. 12.18*b*) is obtained for the vertical loading on the developed length of the girder (Fig. 12.18*a*). Then, all ordinates are divided by the radius R. Next, the applied-torque diagram (Fig. 12.18*d*) is plotted for the twisting moments applied by the loading to the girder (Fig. 12.18*c*). The ordinates of this diagram are subtracted from the corresponding ordinates of the M/R diagram. The resulting $M/R–t$ diagram then is used as a loading diagram on the developed length of the girder (Fig. 12.18*e*). The resulting shears (Fig. 12.18*f*) equal the torques T in the curved girder. Note that positive $M/R–t$ is equivalent to an upward load on the conjugate beam.

The conjugate beam shown in Fig. 12.18*c* is simply supported. This requires that the angle of twist at the supports be zero. Hence, for this case, the curved girder is torsionally fixed at the supports. This condition is attained with a line of diaphragms at each support and a bearing under each web capable of resisting uplift, a common practice. Sometimes, interior supports of a continuous box girder are not fixed against torsion, for example, where a single bearing is placed under a diaphragm. In such cases, the span of the conjugate beam should be taken as the developed length of girder between supports that are fixed against torsion.

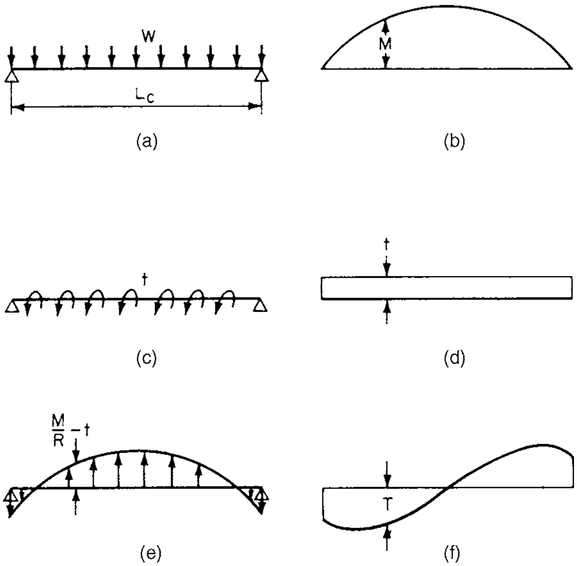


FIGURE 12.18 Loading diagrams for a curved box girder. (*a*) Uniform load w on the developed length. (*b*) Bending moment diagram for the uniform load. (*c*) Applied torque t. (*d*) Torsion diagram. (*e*) $M/R–t$ diagram applied as a load to the conjugate beam. (*f*) Shear diagram for the loading in (*e*).

12.5.3 Loading

Dead loads may be distributed to curved girders in the same way as for straight girders. For live loads, the designer also may use any method commonly used for straight girders. If the distribution procedure of the AASHTO "Standard Specifications for Highway Bridges" is used, however, a correction factor should be applied. The sum of the AASHTO live-load distribution factors for all the girders in a curved grid will usually exceed the number of wheel loads required for the roadway width. Hence, if these factors were used to compute the live-load moments in the girders at a line of diaphragms, the *V* loads there would be too large, because they are proportional to the sum of those moments. One way to correct the *V* loads determined with the AASHTO factors is to multiply the *V* loads by the ratio of the number of wheel loads required for the roadway width to the number of wheel loads determined by the sum of the AASHTO factors.

Impact may be taken into account, in the same way as for straight girders, as a percent increase in live load.

Centrifugal forces comprise a horizontal, radial loading on curved structures that does not apply to straight bridges. These forces are determined as a percentage of the live load, without impact (Art. 10.5). But the live load is restricted to one standard truck placed for maximum loading in each design lane.

Assumed to act 6 ft above the roadway surface, measured from the roadway centerline, centrifugal forces induce torques and horizontal shears in the superstructure. The shears may be assumed to be resisted by the concrete deck within its plane. The torques, however, must be resisted by the girders. In open-framing systems, the primary effect is on the preliminary bending moments. Resisting couples comprise upward and downward vertical forces in the girders. These forces increase bending moments in the outer girders (those farthest from the center of curvature) and decrease moments in the inner girders. The effect of centrifugal forces on *V* loads, however, is small, because *V* loads are determined by the sum of girder moments at a line of diaphragms and this sum is not significantly changed by centrifugal forces.

12.5.4 Sizing of Girders

Design rules for proportioning straight girders generally are applicable to curved girders, depth-span ratios, for example. But curvature does produce effects that should be considered for maximum economy. For instance, girder flanges in open-framing systems should be made as wide as practical to minimize lateral bending stresses. In some cases, where these stresses become too large, a reduction in spacing of diaphragms or cross frames may be desirable.

If curvature causes large adjustments to the preliminary moments in open framing, deepening of girders farthest from the center of curvature may be advantageous. This may be done without overall increase of the floor system, because of the superelevation of the deck.

Girder webs, in some cases, may have to be thicker than for straight girders with corresponding span, spacing, and loadings, because of the effects of curvature on shear. Reactions, too, may be significantly changed, and the effects on substructure design should be taken into account. For some sharply curved bridges, tie-downs may be required to prevent uplift as supports of girders closest to the center of curvature.

If horizontal lateral bracing is placed in an open-framing system, the effects of curvature should be examined more closely. Connections at frequent intervals can convert the system into the closed-framing type.

12.5.5 Fabrication

Curved plate girders usually are produced in one of two ways. One way is to mechanically bend the web to the desired curvature and then weld to it flange plates that have been flame-cut to the required shape. The procedure differs from fabrication of plate girders in handling procedures, layout for fabrication, and web-to-flange welding methods.

Alternatively, girders may be curved by selectively heating the flanges of members initially fabricated straight. In this method, less steel is required. The heating and cooling induce residual stresses, but research indicates that they do not affect fatigue strength.

Mechanical bending is sometimes used for curving rolled beams.

(L. C. Bell and C. P. Heins, "Analysis of Curved Bridges," *Journal of the Structural Division, ASCE,* vol. 96, no. ST8, pp. 1657–1673, August 1970.

P. P. Christiano and C. G. Culver, "Horizontally Curved Bridges Subject to Moving Load," *Journal of the Structural Division, ASCE*, vol. 95, no. ST8, pp. 1615–1643, August 1969.

"LRFD Bridge Design Specifications," "Guide Specifications for Horizontally Curved Steel Girder Highway Bridges," and "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

R. L. Brockenbrough, "Distribution Factors for Curved I-Girder Bridges," *Journal of Structural Engineering, ASCE*, vol. 112, no. ST10, pp. 2200–2215, October 1986.

M. A. Grubb, "Horizontally Curved I-Girder Bridge Analysis: V-Load Method," Transportation Research Board, 1984.)

12.6 EXAMPLE—ALLOWABLE STRESS DESIGN OF CURVED-STRINGER BRIDGE

The basic design procedures that apply to bridges with straight stringers apply also to bridges with curved stringers (Arts. 12.1 to 12.4). In determination of stresses, however, the effects of curvature must be taken into account (Art. 12.5).

To illustrate the design procedure, a curved, two-lane highway bridge with simply supported, composite, plate-girder stringers will be designed. As indicated in the framing plan in Fig. 12.19*a*, the stringers are concentric and the supports and diaphragms are placed radially. Outer girder G_1 spans 90 ft and has a radius of curvature R_1 of 300 ft. Spacing of diaphragms along this span is $d_1 = 15$ ft. Distance between inner and outer grids G_1 and G_3 is $D = 22$ ft c to c, and G_2 is midway between them. The typical cross section in Fig. 12.19*b* shows a 22-ft-wide roadway flanked by two 3-ft 3-in-wide safety walks.

Structural steel to be used is Grade 36. Concrete to be used for the deck is Class A, with 28-day strength $f'_c = 4000$ psi. Appropriate design criteria given in Art 10.7 will be used for this structure. The approximate analysis described in Art. 12.5 for open framing will be applied to the design of the girders.

Concrete Slab. The slab is designed, to span transversely between stringers, in the same way as for straight stringers (Art. 12.2). A 7.5-in-thick slab will be used with the curved plate-girder stringers.

Loads, Moments, and Shears for Stringers. Assume that the girders will not be shored during casting of the concrete slab. Hence, the dead load on each steel stringer includes the weight of the concrete slab as well as the weight of stringer and framing details. This dead load will be referred to as *DL* (see Table 12.15).

For design of the grid composed of the stringers and diaphragms, not only is the maximum bending moment needed for each stringer but also the bending moment at each line of diaphragms (Table 12.16).

For computation of *V* loads,

$$K = \frac{RD}{d} = \frac{300 \times 22}{15} = 440$$

From Table 12.14, $C = 1.00$. *V* loads now can be computed from Eq. (12.32) and are listed in Table 12.17. They act at the diaphragms, downward on G_1, upward on G_3 to resist the torque due to curvature. The *V* load on G_2, the central girder in a three-girder grid, is assumed to be zero. The reaction due to these loads is

$$R_v = 4.64 + 7.43 + \frac{8.36}{2} = 16.25 \text{ kips}$$

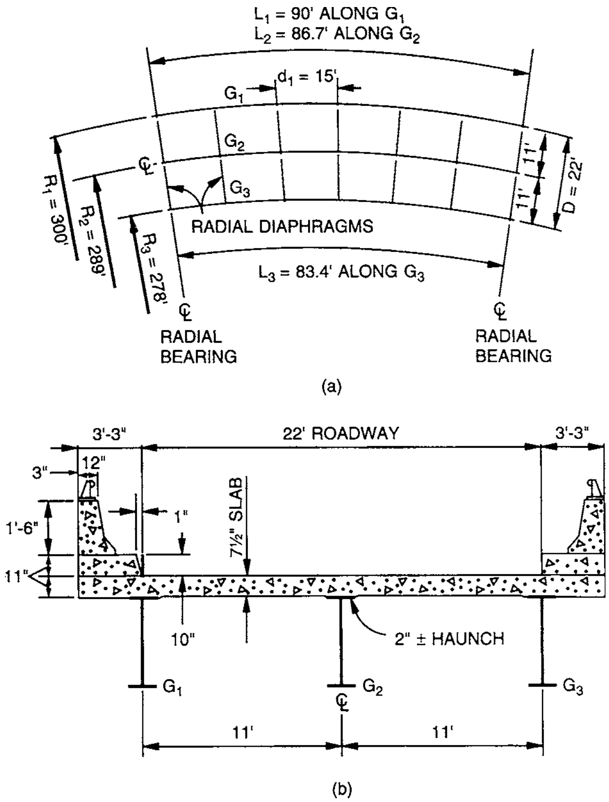


FIGURE 12.19 Two-lane highway bridge with curved stringers. (*a*) Framing plan. (*b*) Typical cross section.

TABLE 12.15 Dead Load Carried by Steel Beams, kips/ft

Stringers G_1 and G_3	
Slab: $0.150(^{11}/_{2} + 3.25)7.5/12$	= 0.82
Haunch and extra concrete: $0.150 \times (3.25 + 0.75) \times {}^{3}/_{12}$	= 0.15
Steel stringer and framing details—assume:	0.30
DL per stringer:	1.27
Stringer G_2	
Slab: $0.150 \times 11 \times 7.5/12$	= 1.03
Haunch—18 × 2 in: $0.150 \times 1.5 \times {}^{2}/_{12}$	= 0.04
Steel stringer and framing details—assume:	0.30
DL per stringer:	1.37

TABLE 12.16 Initial-Dead-Load Preliminary Moments, ft·kips

		Distance from support, ft		
	Span, ft	15, 75	30, 60	45
M_{p1} in G_1	90	714	1143	1286
M_{p2} in G_2	86.7	715	1144	1287
M_{p3} in G_3	83.4	613	981	1104
ΣM_{pn}		2042	3268	3677

TABLE 12.17 V Loads from $\sum M_{pn}/440$

Distance from support, ft	15, 75	30, 60	45
V loads on G_1 and G_3, kips	4.64	7.43	8.36

The resulting bending moments are given in Table 12.18.

Final moments are the sum of the preliminary bending moments and the moments due to the V loads (Table 12.19).

Maximum shear occurs at the supports. For G_1, the maximum dead-load shear is the sum of the preliminary shear and V-load shear:

$$V_{DL} = \frac{1.27 \times 90}{2} + 16.25 = 73.4 \text{ kips}$$

Parapets, railings, and safety walks will be placed after the concrete slab has cured. Their weights may be equally distributed to all stringers. In addition, provision will be made for a future wearing surface, weight 20 lb/ft^2. The total superimposed dead load will be designated *SDL*. Table 12.20 lists for the superimposed load on the composite section the dead loads, the preliminary bending moments, and the V loads. Table 12.21 gives the bending moments due to the V loads, and Table 12.22 gives the final superimposed dead-load moments. For G_1, the maximum shear due to the superimposed dead load is

$$V_{SDL} = \frac{0.607 \times 90}{2} + 7.57 = 34.9 \text{ kips}$$

The HS20-44 live load imposed may be a truck load or lane load. For these girder spans, however, truck loading governs. A standard truck should be placed within each 11-ft lane to produce maximum stresses in the stringers. The extreme left and right positions of the loading are shown in Fig. 12.20. The loads are distributed to the girders on the assumption that the concrete slab is simply supported on them (Table 12.23).

The trucks also should be positioned to produce maximum bending moment in each stringer for design of its central portion. Maximum dead-load moments, however, have been computed at mid-span. Since moments are needed at diaphragm locations for computation of V loads and maximum moment occurs near mid-span, it is convenient to place the trucks for maximum moment at mid-span, where a diaphragm is located, and the error in so doing will be small. Hence, the central 32-kip axle of each truck is placed at mid-span, with the other axles, 32 kips and 8 kips, 14 ft on either side.

The mid-span moments M_n for a truck in one lane only are used to produce the maximum mid-span moments in each girder for a truck in each of the two lanes. The calculations are given in Tables 12.24 to 12.27.

For maximum live-load moments at other sections of each girder, the trucks should be placed in each lane as for the mid-span moments and positioned to produce maximum preliminary moments at the sections. Then, V loads and the moments they cause should be calculated and added to the preliminary moments to yield M_{LL} at each section.

TABLE 12.18 Initial-Dead-Load V-Load Moments M_{vn}, ft·kips

	Distance from support, ft		
	15, 75	30, 60	45
M_{v1} in G_1	244	418	481
M_{v3} in G_3	–226	–387	–445

TABLE 12.19 Dead-Load Final Moments, ft·kips

	Distance from support, ft		
	15, 75	30, 60	45
M_1 in G_1	958	1561	1767
M_2 in G_2	715	1144	1287
M_3 in G_3	387	594	659

TABLE 12.20 Dead Load Carried by Composite Section, kips/ft

Two parapets: $(^1/_3)2 \times 0.150 \times 1.5(1 + 1.25)/2$	= 0.169
Two railings: $(^1/_3)2 \times 0.015$	= 0.010
Two safety walks: $(^1/_3)2 \times 0.150[3.25(0.917 + 0.833)/2 - 0.833 \times 0.083/2]$	= 0.281
Future wearing surface: $(^1/_3)0.020 \times 22$	= 0.147
SDL per stringer:	= 0.607

M_{pn} for superimposed dead load, ft·kips

		Distance from support, ft		
	Span, ft	15, 75	30, 60	45
M_{p1} in G_1	90	342	546	615
M_{p2} in G_2	86.7	317	507	571
M_{p3} in G_3	83.4	293	469	528
$\sum M_{pn}$		952	1522	1714

V loads from $\sum M_{pn}/440$

Distance from support, ft	15, 75	30, 60	45
V load on G_1 G_3, kips	2.16	3.46	3.90

$R_v = 2.16 + 3.46 + 3.90/2 = 7.57$ kips

TABLE 12.21 Superimposed Dead-Load *V*-Load Moments M_{vn}, ft·kips

	Distance from support, ft		
	15, 75	30, 60	45
M_{v1} in G_1	114	194	223
M_{v3} in G_3	−104	−180	−205

TABLE 12.22 Superimposed Dead-Load Final Moments, ft·kips

	Distance from support, ft		
	15, 75	30, 60	45
M_1 in G_1	456	740	838
M_2 in G_2	317	507	571
M_3 in G_3	189	289	323

TABLE 12.23 Fraction of Axle Load on Girders

	G_1	G_2	G_3
Load in lane 1 only, at extreme left	0.55	0.45	0
Load in lane 1 only, at extreme right	0.45	0.55	0
Load in lane 2 only, at extreme left	0	0.55	0.45
Load in lane 2 only, at extreme right	0	0.45	0.55

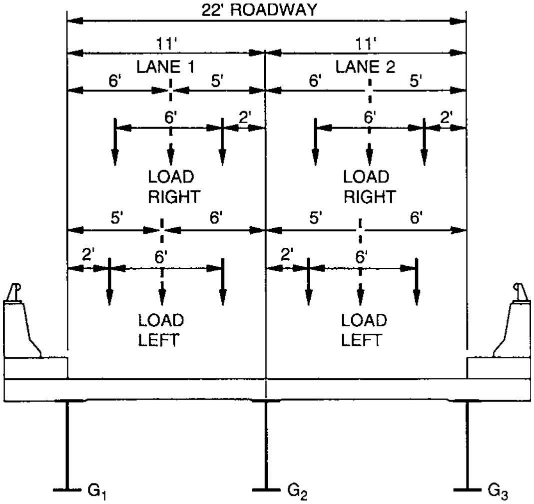


FIGURE 12.20 Position of truck wheel loads in design lanes.

Maximum preliminary shear occurs at a support with a 32-kip axle at the support and the center of gravity of the loading 14 – 4.66 = 9.34 ft from the support. For girder G_1, a truck should be placed at the left in both lanes 1 and 2 for maximum shear due to curvature. The truck in lane 1 should be located at a support for maximum shear, while the truck in lane 2 should be positioned for maximum mid-span moments in G_2 and G_3. The maximum shear in G_1 caused by the truck in lane 2 equals the reaction $R_v = 5.07$, previously computed in determining maximum mid-span moments (Table 12.26). The truck in lane 1 produces a maximum preliminary shear in G_1 of

$$V_{p1} = \frac{72(90 - 9.34)}{90} \times 0.55 = 35.5 \text{ kips}$$

TABLE 12.24 M_{pn} for Maximum Live-Load Moment at Mid-span, ft·kips

	Distance from support, ft				
	15	30	45	60	75
	Girder G_1, span 90 ft				
Full axle load	596	1192	1340	968	484
0.55 axle load	328	656	737	532	266
0.45 axle load	268	536	603	436	218
	Girder G_2, span 86.7 ft				
Full axle load	576	1152	1281	928	464
0.55 axle load	317	634	704	510	255
0.45 axle load	259	518	577	418	209
	Girder G_3, span 83.4 ft				
Full axle load	556	1109	1221	889	444
0.55 axle load	306	610	671	489	244
0.45 axle load	250	499	550	400	200

TABLE 12.25 M_{pn} for Truck in One Lane Only, ft·kips, and V Load, kips

	Distance from support, ft				
	15	30	45	60	75
At left in lane 1					
M_{p1} for G_1 (0.55 axle load)	328	656	737	532	266
M_{p2} for G_2 (0.45 axle load)	259	518	577	418	209
M_{p3} for G_3 (no load)	0	0	0	0	0
ΣM_{pn}	587	1174	1314	950	475
$V = \Sigma M_{pn}/440$	1.33	2.67	2.99	2.16	1.08
At right in lane 1					
M_{p1} for G_1 (0.45 axle load)	268	536	603	436	218
M_{p2} for G_2 (0.55 axle load)	317	634	704	510	255
M_{p3} for G_3 (no load)	0	0	0	0	0
ΣM_{pn}	585	1170	1307	946	473
$\text{V} = \Sigma M_{pn}/440$	1.33	2.66	2.97	2.15	1.08
At left in lane 2					
M_{p1} for G_1 (no load)	0	0	0	0	0
M_{p2} for G_2 (0.55 axle load)	318	634	704	510	255
M_{p3} for G_3 (0.45 axle load)	250	499	550	400	200
ΣM_{pn}	567	1133	1254	910	455
$V = \Sigma M_{pn}/440$	1.29	2.57	2.85	2.07	1.03
At right in lane 2					
M_{p1} for G_1 (no load)	0	0	0	0	0
M_{p2} for G_2 (0.45 axle load)	259	518	577	418	209
M_{p3} for G_3 (0.55 axle load)	306	610	671	489	244
ΣM_{pn}	565	1128	1248	907	453
$V = \Sigma M_{pn}/440$	1.28	2.56	2.84	2.06	1.03

This truck also induces the preliminary bending moments given in Table 12.28 in G_1 and G_2 at the diaphragms.

The reaction due to the V loads is

$$R_v = 1.03 \times {}^5\!/_6 + 1.02 \times {}^4\!/_6 + 0.76 \times {}^3\!/_6 + 0.51 \times {}^2\!/_6 + 0.25 \times {}^1\!/_6 = 2.12 \text{ kips}$$

Hence, the final shear in G_1 due to the trucks in both lanes is

$$V_{LL} = 35.5 + 2.12 + 5.07 = 42.7 \text{ kips}$$

Impact is given as 25% of truck loading by AASHTO for girder bending moment, torsion, and deflection. Thus, the maximum moments due to impact are

$$G_1{:}M_I = 0.25 \times 1046 = 262 \text{ ft}\cdot\text{kips}$$
$$G_2{:}M_I = 0.25 \times 1408 = 352 \text{ ft}\cdot\text{kips}$$
$$G_3{:}M_I = 0.25 \times 531 = 133 \text{ ft}\cdot\text{kips}$$

The maximum shear is given by AASHTO as 30%. Thus, for G_1

TABLE 12.26 *V*-Load Reactions, kips, and Final Mid-span Moments M_n for Truck in One Lane Only, ft·kips

At left in lane 1

$R_v = 1.33 \times {}^5/_6 + 2.67 \times {}^4/_6 + 2.99 \times {}^3/_6 + 2.16 \times {}^2/_6 + 1.08 \times {}^1/_6 = 5.28$
Mid-span $M_{v1} = 5.28 \times 45 - 1.33 \times 30 - 2.67 \times 15 = 158$
Mid-span $M_{v2} = 0$
Mid-span $M_{v3} = -158 \times 83.4/90 = -146$
Mid-span $M_1 = M_{p1} + M_{v1} = 737 + 158 = 895$
Mid-span $M_2 = M_{p2} + M_{v2} = 577$
Mid-span $M_3 = M_{p3} + M_{v3} = -146$

At right in lane 1

$R_v = 1.33 \times {}^5/_6 + 2.66 \times {}^4/_6 + 2.97 \times {}^3/_6 + 2.15 \times {}^2/_6 + 1.08 \times {}^1/_6 = 5.26$
Mid-span $M_{v1} = 5.26 \times 45 - 1.33 \times 30 - 2.66 \times 15 = 157$
Mid-span $M_{v2} = 0$
Mid-span $M_{v3} = -157 \times 83.4/90 = -145$
Mid-span $M_1 = M_{p1} + M_{v1} = 603 + 157 = 760$
Mid-span $M_2 = M_{p2} + M_{v2} = 704$
Mid-span $M_3 = M_{p3} + M_{v3} = -145$

At left in lane 2

$R_v = 1.29 \times {}^5/_6 + 2.57 \times {}^4/_6 + 2.85 \times {}^3/_6 + 2.07 \times {}^2/_6 + 1.03 \times {}^1/_6 = 5.07$
Mid-span $M_{v1} = 5.07 \times 45 - 1.29 \times 30 - 2.57 \times 15 = 151$
Mid-span $M_{v2} = 0$
Mid-span $M_{v3} = -151 \times 83.4/90 = -140$
Mid-span $M_1 = M_{p1} + M_{v1} = 151$
Mid-span $M_2 = M_{p2} + M_{v2} = 704$
Mid-span $M_3 = M_{p3} + M_{v3} = 550 - 140 = 410$

At right in lane 2

$R_v = 1.28 \times {}^5/_6 + 2.56 \times {}^4/_6 + 2.84 \times {}^3/_6 + 2.06 \times {}^2/_6 + 1.03 \times {}^1/_6 = 5.05$
Mid-span $M_{v1} = 5.05 \times 45 - 1.28 \times 30 - 2.56 \times 15 = 150$
Mid-span $M_{v2} = 0$
Mid-span $M_{v3} = -150 \times 83.4/90 = -140$
Mid-span $M_1 = M_{p1} + M_{v1} = 150$
Mid-span $M_2 = M_{p2} + M_{v2} = 577$
Mid-span $M_3 = M_{p3} + M_{v3} = 671 - 140 = 531$

TABLE 12.27 Mid-span Live-Load Moments M_{LL}, ft·kips

Girder	Truck position	M_{LL}
G_1	At left in lanes 1 and 2	895 + 151 = 1046
G_2	At right in lane 1, left in lane 2	704 + 704 = 1408
G_3	At right in lane 2	531

TABLE 12.28 M_p for Truck in Lane 1 Placed for Maximum Shear, ft·kips

		Distance from support, ft				
	Span, ft	15	30	45	60	75
M_{p1} for G_1	90	251	246	185	123	62
M_{p2} for G_2	86.7	203	201	151	100	50
ΣM_{pn}		454	447	336	223	112
$V = M_{pn}/440$		1.03	1.02	0.76	0.51	0.25

$$V_I = 0.30 \times 42.7 = 12.8 \text{ kips}$$

Centrifugal forces and radial wind forces on live load induce torques in the superstructure because they are assumed to act 6 ft above the roadway surface. For this structure, the effects of the wind and centrifugal forces are small enough to be neglected. But for illustrative purposes, they will be calculated.

Because of the sharp curvature, design speed is taken as 30 mi/h. Then, the centrifugal forces equal the following percentages of a truck load in lanes 1 and 2:

$$C = \frac{6.68S^2}{R} = \frac{6.68(30)^2}{295} = 20.4\% \qquad C = \frac{6.68(30)^2}{284} = 21.2\%$$

Application of these percentages to the axle load per lane permits use of the results of previous calculations for moments and shears. Thus, the horizontal force per axle is

$$H = 32 \times 0.204 + 32 \times 0.212 = 13.3 \text{ kips}$$

This force is assumed to act 6 ft above the roadway surface or about 8 ft above the centroidal axis of the girders. Thus, it causes a torque

$$T = 8 \times 13.3 = 106.4 \text{ ft} \cdot \text{kips}$$

This is resisted by a couple comprising a downward vertical force on G_1 and an upward vertical force on G_3:

$$P = \frac{106.4}{22} = 4.83 \text{ kips}$$

By proportion, the maximum moment M_C in G_1 due to the centrifugal forces can be obtained from the maximum moment M_{p1} previously computed for a truck load in lane 1, 1340 ft·kips.

$$M_C = \frac{1340 \times 4.83}{32} = 202 \text{ ft} \cdot \text{kips}$$

Similarly, the maximum shear in G_1 due to the centrifugal forces is

$$V_C = \frac{35.5 \times 4.83}{32 \times 0.55} = 9.7 \text{ kips}$$

AASHTO specifications require a wind load on the live load of at least 0.1 kip/ft. This would cause a torque of $0.1 \times 8 = 0.8$ ft·kip per ft and a downward vertical force on G_1 of $0.8/22 = 0.0364$ kip/ft. Hence, the maximum shear in G_1 due to wind on live load is

$$V_{WL} = \tfrac{1}{2} \times 0.0364 \times 90 = 1.6 \text{ kips}$$

The maximum moment in G_1 due to this load is

$$M_{WL} = \frac{0.0364(90)^2}{8} = 36 \text{ ft} \cdot \text{kips}$$

Combined, centrifugal forces and wind induce in G_1 a maximum shear

$$V_C + V_{WL} = 9.7 + 1.6 = 11.3 \text{ kips}$$

Similarly, the combined maximum moment in G_1 is

TABLE 12.29 Mid-span Bending Moments in Girders, ft·kips

(*a*) Unfactored mid-span bending moments in girders

	M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_C	M_{WL}
Girder G_1	1767	838	1308	202	36
Girder G_2	1287	571	1760	—	—
Girder G_3	659	323	664	—	—

(*b*) Factored mid-span bending moments in girder G_1

$1.3\,M_{DL}$	$1.3\,M_{SDL}$	$2.17(M_{LL} + M_I)$	$1.3\,M_C$
2297	1090	2839	263

$$M_C + M_{WL} = 202 + 36 = 238 \text{ ft}\cdot\text{kips}$$

The maximum unfactored moments and shears for design therefore are as given in Tables 12.29*a* and 12.30*a*.

AASHTO requires, in the design of curved girders, the use of the strength limit state, which considers the stability of each element or plastic state (ultimate moment) of compact I-girders. Load combinations for the strength limit state would include the factored loads as given in the AASHTO standard specifications:

$$\text{Group I loading combination} = 1.3[DL + {}^5\!/_3\,(LL + I) + CF] \qquad \text{for bending}$$
$$= 1.41[DL + {}^5\!/_3\,(LL + I) + CF] \qquad \text{for shear}$$

Factored moments and shears for girder G_1 are given in Tables 12.29*b* and 12.30*b*.

Properties of Composite Section. Design of the girders follows the procedures indicated for plate-girder stringers in Art. 12.4, except that the lateral bending stress in the flanges due to curvature must be taken into account.

For illustrative purposes, girder G_1 will be designed. The effective width of the concrete slab, governed by its 7-in effective thickness, is 84 in. A trial section for the plate girder is selected with the aid of Eq. (12.16).

Assume that the girder web will be 60 in deep. This satisfies the requirements that the depth-span ratio for girder plus slab exceed 1:25 and for girder alone 1:30.

From Eq. (12.18), the thickness of the stiffened web is required to be at least the following:

$$t_w = \frac{D_c\sqrt{F_y}}{577} = \frac{35.71\sqrt{36}}{577} = 0.371 \text{ in}$$

TABLE 12.30 End Shears in Girder G_1, kips

(*a*) Unfactored end shears in girder G_1

V_{DL}	V_{SDL}	$V_{LL} + V_I$	V_C	V_{WL}
73.4	34.9	55.5	9.7	1.6

(*b*) Factored end shears in girder G_1

$1.41V_{DL}$	$1.41V_{SDL}$	$2.35(V_{LL} + V_I)$	$1.41V_C$	Total V
103.5	49.2	130.5	13.7	297

Use a web plate 60 × $^7/_{16}$ in with a cross-sectional area $A_w = 26.25$ in^2.

The web will be subjected to a maximum factored shear of $V = 297$ kips (Table 12.30*b*). Determine shear capacity following Art. 12.4.8.

$$V_p = 0.58F_yDt_w = 0.58 \times 36 \times 60 \times 0.438 = 548 \text{ kips}$$

Try a transverse stiffener spacing of 5 ft, or 60 in.

$$\frac{d_0}{D} = \frac{60}{60} = 1 < 3 \qquad \text{OK}$$

$$\frac{67{,}600}{(60/0.438)^2} = 3.6 > \frac{d_0}{D} \qquad \text{OK}$$

The coefficient C is computed from Eq. (12.24).

$$C = \frac{45{,}000k}{(D/t_w)^2 F_y}$$

where $k = 5[1 + 1/(d_0/D)^2] = 5(1 + 1/1^2) = 10$ and $D/t_w = 60/0.438 = 137$.

$$C = \frac{45{,}000 \times 10}{137^2 \times 36} = 0.665$$

From Eq. (12.23), the shear capacity of the girder is

$$V_u = 548\left[0.665 + \frac{0.87(1-0.665)}{\sqrt{1+1^2}}\right] = 327 \text{ kips}$$

Since the total factored end shear $V_{\max} = 297$ kips < $V_u = 327$ kips, the section is adequate for shear.

From Eq. (12.16), the required bottom-flange area is estimated as

$$A_{sb} = \frac{12(2297 + 1090 + 2839 + 263)}{36 \times 60} = 36.05 \text{ in}^2$$

Try 22- × 2-in bottom flange, area = 44 in^2. Considering the effect of lateral bending, estimate the required area of the steel top flange as $A_{st}/2 = 43.8/2 = 21.9$ in^2. Try a 18 × 1$^1/_2$-in top flange, area = 27 in^2.

The trial section is shown in Fig. 12.21. Its neutral axis can be located by taking moments of web and flange areas about mid-depth of the web. This computation and that for the section moduli S_{st} and S_{sb} of the plate girder alone are conveniently tabulated in Table 12.31.

In computation of the properties of the composite section, the concrete slab, ignoring the haunch area, is transformed into an equivalent steel area, with $n = 24$ for superimposed dead load and $n = 8$ for live loads. The computations of neutral-axis location and section moduli for the composite section are tabulated in Table 12.32. To locate the neutral axis, moments of the areas are taken about middepth of the girder web.

Stresses in Composite Section. Since the girders will not be shored when the concrete is cast and cured, the stresses in the steel section for load *DL* are determined with the section moduli of the steel section alone (Table 12.31). Stresses for load *SDL* are computed with the section moduli of the composite section when $n = 24$ (Table 12.32*a*). And stresses in the steel for live loads and impact are calculated with section moduli of the composite section when $n = 8$ (Table 12.32*b*). Lateral bending stresses in the bottom flange are superimposed on other stresses. Lateral bending moment in the top flange should be computed for load *DL*. For composite beams, lateral bending in the top flange under *SDL* and live load can be ignored.

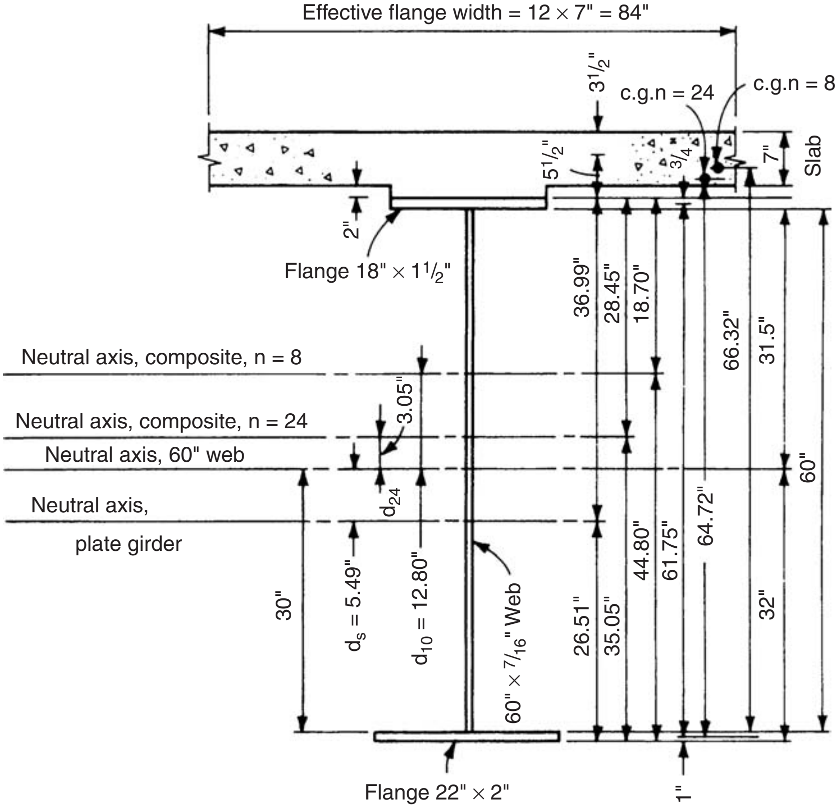


FIGURE 12.21 Cross section of composite plate girder at mid-span.

TABLE 12.31 Steel Section for G_1 for Maximum Moment

Material	A	d	Ad	Ad^2	I_o	I
Top flange 18 × 1 1/2	27.0	30.75	830	25,500		25,500
Web 60 × 7/16	26.3				7,900	7,900
Bottom flange 22 × 2	44.0	−31.0	−1,364	42,300		42,300
	97.3		−534			75,700
$d_s = -534/97.3 = -5.49$ in						−5.49 × 534 = −2,900
						$I_{NA} = 72{,}800$

Distance from neutral axis of steel section to:

Top of steel = 30 + 1.50 + 5.49 = 36.99 in

Bottom of steel = 30 + 2.00 − 5.49 = 26.51 in

Section moduli

Top of steel	Bottom of steel
$S_{st} = 72{,}800/36.99 = 1{,}970\ \text{in}^3$	$S_{sb} = 72{,}800/26.51 = 2{,}750\ \text{in}^3$

TABLE 12.32 Composite Section for G_1 for Maximum Moment

(a) For dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	97.3		−534			75,700
Concrete 84 × 7/24	24.5	37.0	906	33,500	100	33,600
	121.8		372			109,300
$d_{s0} = 372/121.8 = 3.05$ in					−3.05 × 372 =	−1,100
					I_{NA} =	108,200

Distance from neutral axis of composite section to:

Top of steel = 31.50 − 3.05 = 28.45 in

Bottom of steel = 32.00 + 3.05 = 35.05 in

Top of concrete = 28.45 + 2 + 7 = 37.45 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
S_{st} = 108,200/28.45 = 3,800 in^3	S_{sb} = 108,200/35.05 = 3,100 in^3	S_c = 108,200/37.45 = 2,900 in^3

(b) For live loads, $n = 8$

Materials	A	d	Ad	Ad^2	I_o	I
Steel section	97.3		−534			75,700
Concrete 84 × 7/10	73.5	37.0	2,720	100,600	300	100,900
	170.8		2,186			176,600
$d_{10} = 2{,}186/170.8 = 12.80$					−12.80 × 2,186 =	−28,000
					I_{NA} =	148,600

Distance from neutral axis of composite section to:

Top of steel = 31.50 − 12.80 = 18.70 in

Bottom of steel = 32.00 + 12.80 = 44.80 in

Top of concrete = 18.70 + 2 + 7 = 27.70 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
S_{st} = 148,600/18.70 = 7,900 in^3	S_{sb} = 148,600/44.80 = 3,300 in^3	S_c = 148,600/27.70 = 5,400 in^3

The moments causing the lateral bending stresses can be computed from Eq. (12.36). For use in this calculation (Table 12.33), the centroid of the compression flange is located by taking the moment of the transformed area of the concrete slab about the centroid of the steel top flange. Thus, for the steel section alone, the distance between flange centroids is

$$h = 60 + \frac{2}{2} + \frac{1.5}{2} = 61.75 \text{ in}$$

TABLE 12.33 Maximum Lateral Bending Moments, ft·kips

DL: $M_L = -0.1 \times 12 \times 2297(15)^2/(300 \times 61.75) = -34$

SDL: $M_L = -0.1 \times 12 \times 1090(15)^2/(300 \times 64.72) = -15$

$LL + I + CF$: $M_L = -0.1 \times 12 \times 3102(15)^2/(300 \times 66.32) = -42$

Total: −91

TABLE 12.34 Steel Stresses in G_1, ksi

Top of steel (compression)	Bottom of steel (tension)
DL: $f_b = 2297 \times 12/1970 = 13.99$	$f_b = 2297 \times 12/2750 = 10.02$
SDL: $f_b = 1090 \times 12/3800 = 3.44$	$f_b = 1090 \times 12/3100 = 4.22$
$LL + I + CF$: $f_b = 3102 \times 12/7900 = 4.71$	$f_b = 3102 \times 12/3300 = 11.28$
L: $f_b = 34 \times {}^{12}/_{81} = 5.04$	$f_b = 91 \times 12/161.5 = 6.76$
Total: $20.18 < 36$	$32.28 < 36$

For the composite section, $n = 24$,

$$h = 61.75 + \frac{6.25 \times 24.5}{27.0 + 24.5} = 61.75 + 2.97 = 64.72 \text{ in}$$

For the composite section, $n = 8$,

$$h = 61.75 + \frac{6.25 \times 73.5}{27.0 + 73.5} = 61.75 + 4.57 = 66.32 \text{ in}$$

The section moduli of the top and bottom flanges about their central vertical axes are

$$S_{\text{ft}} = \frac{1.5(18)^2}{6} = 81 \text{ in}^3 \qquad S_{\text{fb}} = \frac{2(22)^2}{6} = 161.5 \text{ in}^3$$

TABLE 12.35 Stresses in G_1 at Top of Concrete, ksi

SDL: $f_c = 1090 \times 12/(2900 \times 24) = 0.19$
$LL + I + CF$: $f_c = 3102 \times 12/(5400 \times 8) = 0.86$
Total: $1.05 < 4.0$

Calculations of the steel stresses in G_1 are given in Table 12.34. The trial section is satisfactory. By inspection, other group loadings will not be critical.

Stresses in the concrete slab are determined with the section moduli of the composite section with $n = 24$ for SDL (Table 12.32*a*) and $n = 8$ for $LL + I + CF$ (Table 12.32*b*). The calculation is given in Table 12.35.

Therefore, the composite section for G_1 is satisfactory. Use for G_1 in the region of maximum moment the section shown in Fig. 12.21.

The procedure is the same for design of other sections and for the other stringers. For design of other elements, see Arts. 12.2 and 12.4. Fatigue design is similar to that for straight girders. In addition, the erection sequence should be investigated for constructibility, checking deflections, stress, and concrete crack control.

12.7 DECK PLATE-GIRDER BRIDGES WITH FLOORBEAMS

For long spans, use of fewer but deeper girders to span the long distance between supports becomes more efficient. With appropriately spaced stringers between the main girders of highway bridges, depth of concrete roadway slab can be kept to the minimum permitted, thus avoiding increase in dead load from the deck. Spans of the longitudinal stringers are kept short by supporting them on transverse floorbeams spanning between the girders. If spacing of the floorbeams is 25 ft or less, additional diaphragms or cross frames between the girders are not required.

This type of construction can be used with deck or through girders. Through girders carry the roadway between them. Their use generally is limited to locations where vertical clearances below the bridge are critical. Deck girders carry the roadway on the top flange. They generally are preferred for highway bridges where vertical clearances are not severely restricted, because the girders, being below the deck, do not obstruct the view from the deck. Structurally, deck girders have the advantage that the concrete deck is available for bracing the top flange of the girders and for composite action. Bracing of the bottom flange is accomplished with horizontal lateral bracing.

The design procedure for through plate girders with floorbeams is described in Art. 12.9. In general, design of the stringers is much like that for a stringer bridge (Art. 12.2). In the following example, however, the stringers and girders are not designed for composite action. See also Art. 12.3.

12.8 EXAMPLE—ALLOWABLE STRESS DESIGN OF DECK PLATE-GIRDER BRIDGE WITH FLOORBEAMS

Two simply supported, welded, deck plate girders carry the four lanes of a highway bridge on a 137.5-ft span. The girders are spaced 35 ft c to c. Loads are distributed to the girders by longitudinal stringers and floorbeams (Fig. 12.22). The typical cross section in Fig. 12.23 shows a 48-ft roadway flanked by 3-ft-wide safety walks. Grade 50 steel is to be used for the girders and Grade 36 for stringers, floorbeams, and other components. Concrete to be used for the deck is class A, with 28-day strength $f'_c = 4000$ psi and allowable compressive stress $f_c = 1400$ psi. Appropriate design criteria given in Chap. 10 will be used for this structure.

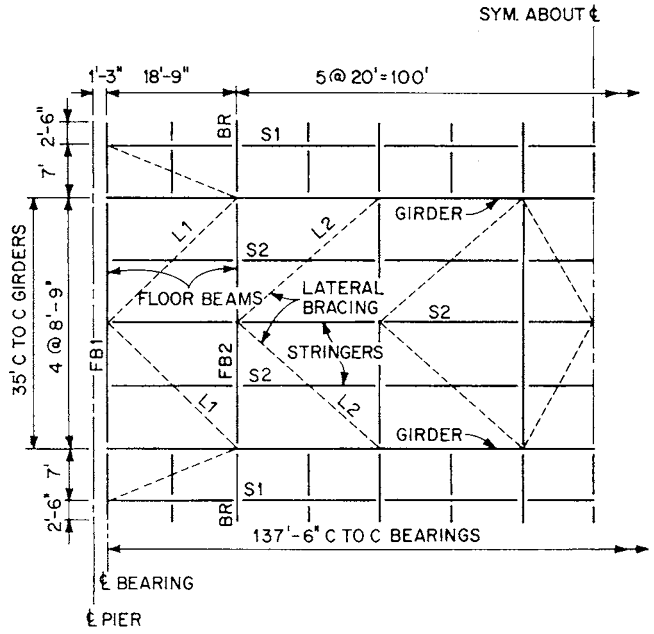


FIGURE 12.22 Framing plan for four-lane highway bridge with deck plate girders.

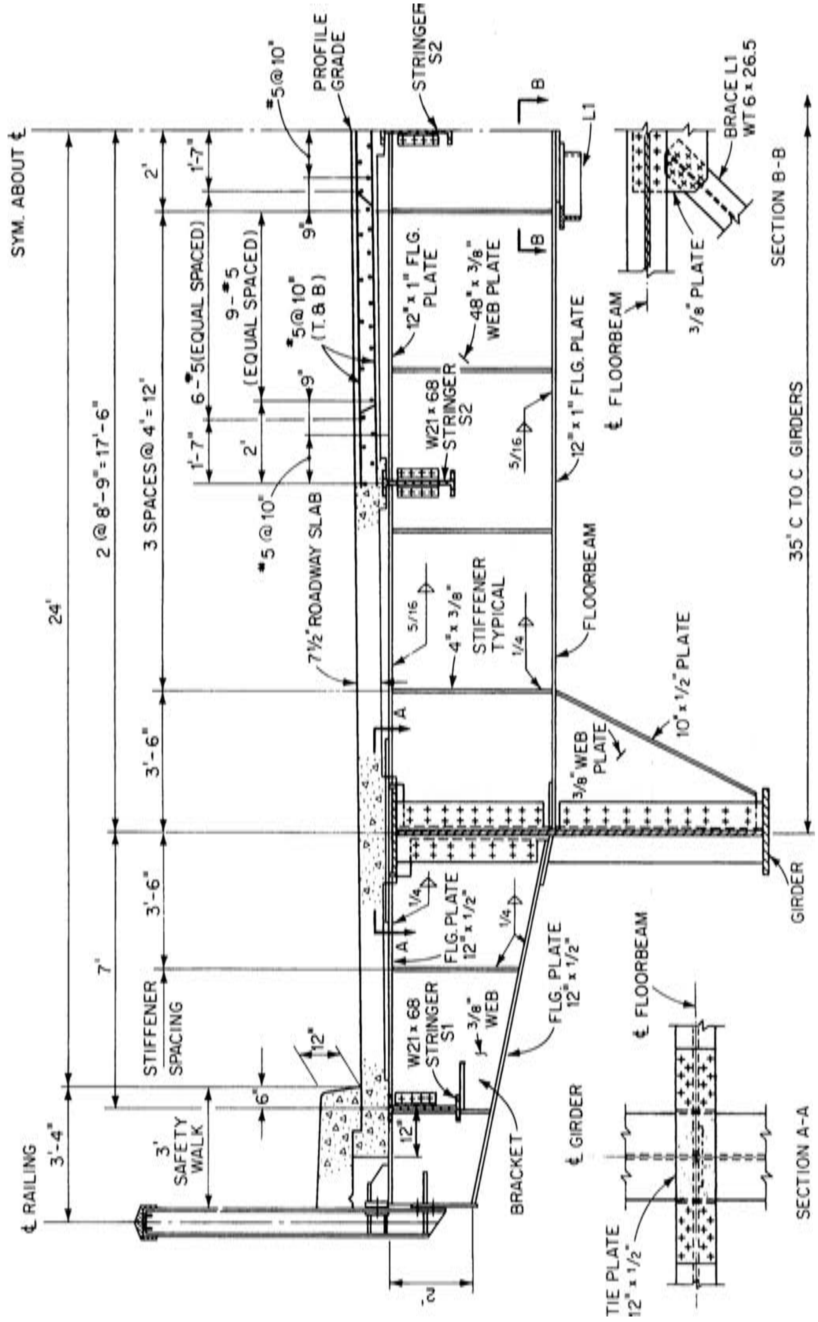


FIGURE 12.23 Typical cross section of deck-girder bridge at a floorbeam.

12.8.1 Design of Concrete Slab

The slab is designed to span transversely between stringers, in the same way as for rolled-beam stringers (Art. 12.2). A 7.5-in-thick concrete slab will be used.

12.8.2 Design of Interior Stringer

Spacing of interior stringers c to c is 8.75 ft. Simply supported, a typical stringer S2 spans 20 ft. Table 12.36 lists the dead loads on S2. Maximum dead-load moment occurs at mid-span and equals

TABLE 12.36 Dead Load on S2, kips/ft

Slab: $0.150 \times 8.75 \times 7.5/12 =$	0.820
Haunch—assume:	0.035
Stringer—assume:	0.068
DL per stringer:	0.923

$$M_{DL} = \frac{0.923(20)^2}{8} = 46.1 \text{ ft} \cdot \text{kips}$$

Maximum dead-load shear occurs at the supports and equals

$$V_{DL} = \frac{0.923 \times 20}{2} = 9.2 \text{ kips}$$

The live load distributed to the stringer with spacing $S = 8.75$ ft is

$$\frac{S}{5.5} = \frac{8.75}{5.5} = 1.59 \text{ wheel loads} = 0.795 \text{ axle loads}$$

Maximum moment induced in a 20-ft span by a standard HS20 truck is 160 ft·kips. Hence, the maximum live-load moment in a stringer is

$$M_{LL} = 0.795 \times 160 = 127.2 \text{ ft} \cdot \text{kips}$$

Maximum shear caused by the truck is 41.6 kips. Consequently, maximum live-load shear in the stringer is

$$V_{LL} = 0.795 \times 41.6 = 33.0 \text{ kips}$$

Impact is taken as 30% of live-load stress, because

$$I = \frac{50}{L+125} = \frac{50}{20+125} = 0.35 > 0.30$$

So the maximum moment due to impact is

$$M_I = 0.30 \times 127.2 = 38.1 \text{ ft} \cdot \text{kips}$$

and the maximum shear due to impact is

$$V_I = 0.30 \times 33.0 = 9.9 \text{ kips}$$

Maximum moments and shears in S2 are summarized in Table 12.37.

With an allowable bending stress $F_b = 20$ ksi for a stringer of Grade 36 steel, the section modulus required is

TABLE 12.37 Maximum Moments and Shears in S2

	DL	*LL*	*I*	Total
Moments, ft·kips	46.1	127.2	38.1	211.4
Shears, kips	9.2	33.0	9.9	52.1

$$S = \frac{M}{F_b} = \frac{211.4 \times 12}{20} = 127 \text{ in}^3$$

With an allowable shear stress F_v = 12 ksi, the web area required is

$$A_w = \frac{52.1}{12} = 4.33 \text{ in}^2$$

Use a W21 × 68. It provides a section modulus of 139.9 in^3 and a web area of $0.43 \times 21.13 = 9.1$ in^2.

12.8.3 Design of an Exterior Stringer

Exterior stringer S1 is simply supported and spans 20 ft. It carries sidewalk as well as truck loads (Fig. 12.22). Dead loads are apportioned between S1 and the girder, 7 ft away, by treating the slab as simply supported at the girder. Table 12.38 lists the dead loads on S1.

Maximum dead-load moment occurs at mid-span and equals

$$M_{DL} = \frac{1.15(20)^2}{8} = 57.5 \text{ ft} \cdot \text{kips}$$

Maximum dead-load shear occurs at the supports and equals

$$V_{DL} = \frac{1.15 \times 20}{2} = 11.5 \text{ kips}$$

The live load from the roadway distributed to the exterior stringer with spacing S = 7 ft from the girder is

$$\frac{S}{4.0 + 0.25S} = \frac{7}{4.0 + 0.25 \times 7} = 1.22 \text{ wheel loads} = 0.61 \text{ axle loads}$$

Maximum moment induced in a 20-ft span by a standard HS20 truck load is 160 ft·kips. Hence, the maximum live-load moment in S1 is

$$M_{LL} = 0.61 \times 160 = 97.6 \text{ ft} \cdot \text{kips}$$

Maximum shear caused by the truck is 41.6 kips. Therefore, maximum live-load shear in S1 is

$$V_{LL} = 0.61 \times 41.6 = 25.4 \text{ kips}$$

Impact for a 20-ft span is 30% of live-load stress. Hence, maximum moment due to impact is

$$M_I = 97.6 \times 0.3 = 29.3 \text{ ft} \cdot \text{kips}$$

and maximum shear due to impact is

$$V_I = 25.4 \times 0.3 = 7.6 \text{ kips}$$

TABLE 12.38 Dead Load on S1, kips/ft

Railing: 0.070 × 9.83/7	= 0.098
Sidewalk: 0.150 × 1 × 3 × 8/7	= 0.514
Slab: 0.150 × 8 × 7.5/12 × 4/7	= 0.428
Stringers, brackets, framing details—assume:	0.110
DL per stringer:	1.150

TABLE 12.39 Maximum Moments and Shears in S1

	DL	*LL*	*I*	Total
Moments, ft · kips	57.5	97.7	29.3	184.5
Shears, kips	11.5	25.4	7.6	44.5

Sidewalk loading at 85 lb/ft^2 on the 3-ft-wide sidewalk imposes a uniformly distributed load w_{SLL} on the stringer. With the slab assumed simply supported at the girder,

$$w_{SLL} = \frac{0.085 \times 3 \times 8}{7} = 0.29 \text{ kip/ft}$$

This causes a maximum moment of

$$M_{SLL} = \frac{0.29(20)^2}{8} = 14.5 \text{ ft} \cdot \text{kips}$$

and a maximum shear of

$$V_{SLL} = \frac{0.29 \times 20}{2} = 2.9 \text{ kips}$$

Maximum moments and shears in S1 are summarized in Table 12.39.

If the exterior stringer has at least the capacity of the interior stringers, the allowable stress may be increased 25% when the effects of sidewalk live load are combined with those from dead load, traffic live load, and impact. In this case, the moments and shears due to sidewalk live load are less than 25% of the moments and shears without that load. Hence, they may be ignored.

With an allowable bending stress $F_b = 20$ ksi for Grade 36 steel, the section modulus required for S1 is

$$S = \frac{M}{F_b} = \frac{184.4 \times 12}{20} = 111 \text{ in}^3$$

With an allowable shear stress $F_v = 12$ ksi, the web area required is

$$A_w = \frac{44.5}{12} = 3.7 \text{ in}^2$$

Use a W21 × 68, as for S1.

12.8.4 Design of an Interior Floorbeam

Floorbeam FB2 is considered to be a simply supported beam with 35-ft span and symmetrical 9.5-ft brackets, or overhangs (Fig. 12.23). It carries a uniformly distributed dead load due to its own weight and that of a concrete haunch, assumed at 0.21 kip/ft. Also, FB2 carries a concentrated load from S1 of 2 × 11.5 = 23.0 kips and a concentrated load from each of three interior stringers S2 of 2 × 9.2 = 18.4 kips (Fig. 12.24).

Moments and Shears in Main Span. Because of the brackets, negative moments occur and reach a maximum at the supports. The maximum negative dead-load moment is

$$M_{DL} = -0.21\left(\frac{9.5}{2}\right)^2 - 23.0 \times 7 = -171 \text{ ft} \cdot \text{kips}$$

The reaction at either support under the symmetrical dead load is

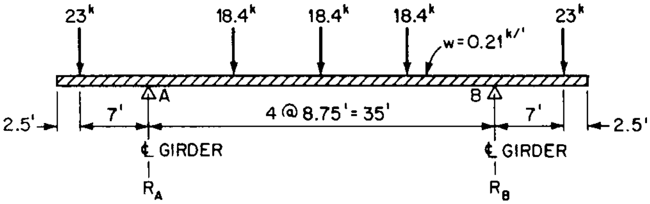


FIGURE 12.24 Dead loads on a floorbeam of the deck-girder bridge.

$$R_{DL} = \frac{3\times18.4}{2} + 23 + \frac{0.21\times54}{2} = 56.3 \text{ kips}$$

Maximum dead-load shear in the overhang is

$$V_{DL} = 23 + 0.21 \times 9.5 = 25.0 \text{ kips}$$

Hence, the maximum shear between girders is

$$V_{DL} = 56.3 - 25.0 = 31.3 \text{ kips}$$

Maximum positive dead-load moment occurs at mid-span and equals

$$M_{DL} = 31.3\times17.5 - 18.4\times8.75 - \frac{0.21(17.5)^2}{2} - 171 = 184 \text{ ft}\cdot\text{kips}$$

Maximum live-load stresses in the floorbeam occur when the center truck wheels pass over it (Fig. 12.25). In that position, the wheels impose on FB2 a load of

$$W = 16 + \frac{16\times6}{20} + \frac{4\times6}{20} = 22 \text{ kips}$$

For maximum positive moment, trucks should be placed in the two central lanes, as close to mid-span as permissible (Fig. 12.26). Then, the maximum moment is

$$M_{LL} = 2 \times 22 \times 15.5 - 22 \times 6 = 550 \text{ ft}\cdot\text{kips}$$

Maximum negative moment occurs at a support with a truck in the outside lane with a wheel 2 ft from the curb (Fig. 12.27). This moment equals

$$M_{LL} = -22 \times 4.5 = -99 \text{ ft}\cdot\text{kips}$$

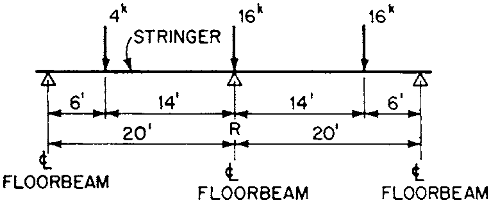


FIGURE 12.25 Positions of loads on a stringer for maximum live load on a floorbeam.

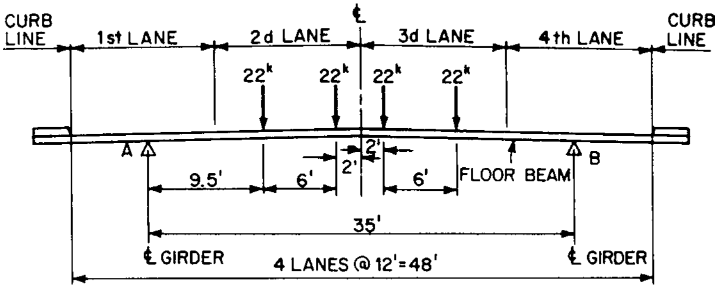


FIGURE 12.26 Positions of loads for maximum positive moment in a floorbeam.

Maximum live-load shear between girders occurs at support *A* with three lanes closest to that support loaded, as indicated in Fig. 12.28. Because three lanes are loaded, the floorbeam needs to be designed for only 90% of the resulting shear. The reaction at *A* is

$$R_{LL} = \frac{0.90 \times 22(39.5+33.5+27.5+21.5+15.5+9.5)}{35} = 83.2 \text{ kips}$$

Subtraction of the shear in the bracket for this loading gives the maximum live-load shear between girders of

$$V_{LL} = 83.2 - 0.9 \times 22 = 63.4 \text{ kips}$$

The maximum live-load shear in the overhang is produced by the loading in Fig. 12.27 and is V_{LL} in 22 kips.

Impact is taken as 30% of live-load stress, because

$$I = \frac{50}{L+125} = \frac{50}{35+125} = 0.31 > 0.30$$

Sidewalk loading transmitted by exterior stringers S1 to the floorbeam equals 2 × 2.9 = 5.8 kips. This induces a shear in the overhang $V_{SLL} = 5.8$ kips. Also, it causes a reaction

$$R_{SLL} = \frac{5.8 \times 42}{35} = 7.0 \text{ kips}$$

Subtraction of the overhang shear gives the maximum shear between girders:

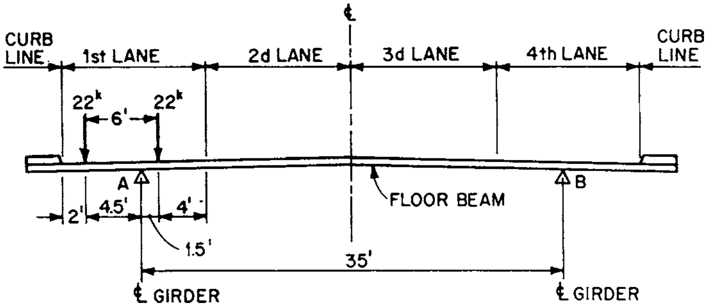


FIGURE 12.27 Positions of loads for maximum negative moment and maximum shear in the overhang of a floorbeam.

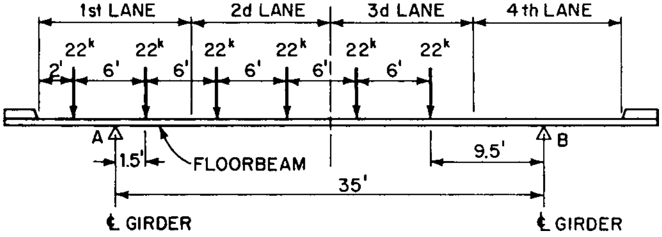


FIGURE 12.28 Positions of loads for maximum shear at *A* in main span of floorbeam.

$$V_{SLL} = 7.0 - 5.8 = 1.2 \text{ kips}$$

Maximum negative moment due to the sidewalk live load is

$$M_{SLL} = -5.8 \times 7 = 41 \text{ ft}\cdot\text{kips}$$

Results of the preceding calculations are summarized in Table 12.40.

Main-Span Section. FB2 will be designed as a plate girder of Grade 36 steel. Assume a 48-in-deep web. If the floorbeam is not stiffened longitudinally, web thickness must be at least $t = D/170 = 48/170 = 0.283$ in. To satisfy the allowable shear stress of 12 ksi, with a maximum shear from Table 12.40 of 114.9 kips, web thickness should be at least $t = 114.9/(12 \times 48) = 0.20$ in. These requirements could be met with a $^5/_{16}$-in web, the minimum thickness required. But fewer stiffeners will be needed if a slightly thicker plate is selected. So use a 48 × $^3/_8$-in web.

Assume that the tension and compression flanges will be the same size and that each flange will have two holes for $^7/_8$-in-diameter high-strength bolts. To satisfy the allowable bending stress of 20 ksi, with a maximum moment of 899 ft·kips from Table 12.40, flange area should be about

$$A_f = \frac{899 \times 12}{20(48+1)} = 11 \text{ in}^2$$

With an allowance for the bolt holes, assume for each flange a plate 12 × 1 in. Width–thickness ratio of 12:1 for the compression flange is less than the 24:1 maximum and is satisfactory.

The trial section assumed is shown in Fig. 12.29. Moment of inertia and section modulus of the net section are calculated as shown in Table 12.41. Distance from neutral axis to top or bottom of the floorbeam is 25 in. Hence, the section modulus provided is

$$S_{\text{net}} = \frac{15{,}456}{25} = 618 \text{ in}^3$$

Maximum bending stress therefore is

TABLE 12.40 Maximum Moments, Shears, and Reactions in Floorbeam FB2

	DL	*LL*	*I*	*SLL*	Total
Negative moments, ft·kips	–171	–99	–30	–41	–341
Positive moments, ft·kips	184	550	165	. . .	899
Shear in main span, kips	31.3	63.4	19.0	1.2	114.9
Shear in overhang, kips	25.0	22.0	6.6	5.8	59.4
Reaction, kips	56.3	83.2	25.0	7.0	171.5

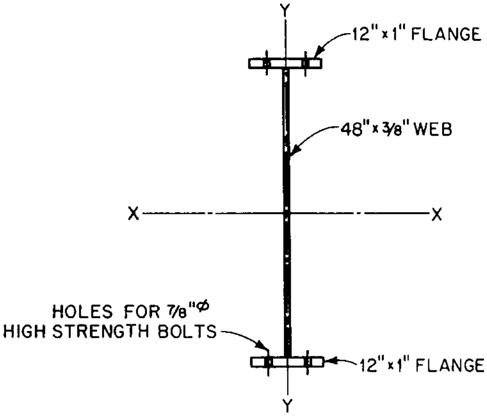


FIGURE 12.29 Cross section of floorbeam in main span.

$$f_b = \frac{899 \times 12}{618} = 17.5 < 20 \text{ ksi}$$

The section is satisfactory.

A check of the weight of the floorbeam is desirable to verify the assumptions made in dead-load calculations. Weight of slab haunch, beam, and details was assumed at 0.21 kip/ft. Average weight of haunch will be about 0.05 kip/ft. Thus, the assumed weight of floorbeam and details was about 0.16 kip/ft. If 8% of the weight is assumed in details, actual weight is $1.08(2 \times 40.8 + 61.2) = 154 <$ 160 lb/ft assumed.

Flange-to-Web Welds. Each flange will be connected to the web by fillet welds on opposite sides of the web. These welds must resist the horizontal shear between flange and web. The minimum size of weld permissible for the thickest plate at the connection usually determines the size of weld. In some cases, however, the size of weld may be determined by the maximum shear. In this example, shear does not govern, but the calculations are presented to illustrate the procedure.

The gross moment of inertia, 17,856 in^4, is used in computing the shear v, kips/in, between flange and web. From Table 12.40, maximum shear is 114.9 kips. Still needed is the static moment Q of the flange area:

$$Q = 12 \times 1 \times 24.5 = 294 \text{ in}^3$$

Then, the shear is

TABLE 12.41 Moment of Inertia of Floorbeam FB2 at Mid-span

Material	A	d	Ad^2 or I_o
2 flanges 12 × 1	24	24.5	14,400
Web 48 × $^3/_8$	18		3,456
			I_g = 17,856
4 holes 1 × 1	4	24.5	−2,400
			I_{net} = 15,456 in^4

$$v = \frac{VQ}{I} = \frac{114.9 \times 294}{17{,}856} = 1.89 \text{ kips/in}$$

The allowable stress on the weld is not determined by fatigue. It is sufficient and necessary that the base metal in the flange be investigated for fatigue and the weld metal be checked for maximum shear stress. For fatigue, the stress category is B. On the assumption that the bridge is a nonredundant-load-path structure, the allowable stress range in FB2 for 500,000 cycles of loading is 23 ksi. The stress range due to live load plus impact moments is 12[550 − (−99) + 165 − (−30)]/618 = 16.4 ksi < 23 ksi. The base metal is satisfactory for fatigue.

The allowable shear stress is $F_v = 0.27\ F_u = 0.27 \times 58 = 15.6$ ksi. Hence, the allowable load per weld is 15.6 × 0.707 = 11.03 kips per in, and for two welds, 22.06 kips/in. So the weld size required to resist the shear is 1.89/22.06 = 0.09 in. The minimum size of weld permitted with the 1-in-thick flange plate, however, is $^5/_{16}$ in. Use two $^5/_{16}$-in welds at each flange.

Connection to Girder. Connection of the floorbeam to the girder is made with 18 A325 high-strength bolts. Each has a capacity in a slip-critical connection with Class A surface of 9.3 kips. For the maximum shear of 114.9 kips, the number of bolts required is 114.9/9.3 = 13. The 18 provided are satisfactory.

Main-Span Stiffeners. Bearing stiffeners are not needed, because the web is braced at the supports by the connections with the girders. Whether intermediate transverse stiffeners are needed can be determined from Art. 10.7. The compressive bending stress at the support is

$$f_b = \frac{341 \times 12 \times 25}{17{,}856} = 5.73 \text{ ksi}$$

For a girder web with transverse stiffeners, the depth–thickness ratio should not exceed

$$\frac{D}{t} = \frac{730}{\sqrt{5.73}} = 304 > 170$$

Hence, web thickness should be at least 48/170 = 0.28 < 0.375 in. Actual $D/t_w = 48/0.375 = 128 < 170$. The average shear stress at the support is

$$f_v = \frac{114.9}{18} = 6.38 \text{ ksi}$$

The limiting shear stress for the girder web without stiffeners is, from Art. 10.7,

$$F_v = \left(\frac{270}{128}\right)^2 = 4.45 \text{ ksi} < 6.38 \text{ ksi}$$

Hence, web thickness for shear should be at least

$$t_w = \frac{48}{270}\sqrt{6.38} = 0.45 \text{ in}$$

This is larger than the $^3/_8$-in web thickness assumed. Therefore, intermediate transverse stiffeners are required. (A change in web thickness from $^3/_8$ to $^7/_{16}$ in would eliminate the need for the stiffeners.)

Stiffener spacing is determined by the shear stress computed from Eq. (10.25a). Assume that the stiffener spacing $d_o = 48$ in = the web depth D. Hence, $d_o/D = 1$. From Eq. (10.24d), $k = 5(1 + 1^2) = 10$ and $\sqrt{k/F_y} = \sqrt{10/36} = 0.527$. Since $D/t_w = 128$, C in Eq. (10.25a) is determined by the parameter 128/0.527 = 243 > 237. Hence, C is given by Eq. (10.24c):

$$C = \frac{45{,}000k}{(D/t_w)^2 F_y} = \frac{45{,}000 \times 10}{128^2 \times 36} = 0.763$$

From Eq. (10.25a), the maximum allowable shear for $d_o = 48$ in is

$$F_v' = F_v\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]$$

$$= 12\left[0.763 + \frac{0.87(1-0.763)}{\sqrt{1+1^2}}\right] = 10.9 > 6.38 \text{ ksi}$$

Since the allowable stress is larger than the computed stress, the stiffeners can be spaced 48 in apart. (Because of the brackets, the floorbeam can be considered continuous at the supports. Thus, the stiffener spacing need not be half the calculated spacing, as would be required for the first two stiffeners at simple supports.) Stiffener locations are shown in Fig. 12.23.

Stiffeners may be placed in pairs and welded on each side of the web with two $^1/_4$-in welds. Moment of inertia provided by each pair must satisfy Eq. (10.21), with J as given by Eq. (10.22):

$$J = 2.5\left(\frac{48}{48}\right)^2 - 2 = 0.5$$

$$I = 48(^3/_8)^3 0.5 = 1.27 \text{ in}^4$$

Use 4 × $^3/_8$-in stiffeners. They satisfy minimum requirements for thickness of projection from the web and provide a moment of inertia

$$I = 0.375(2 \times 4 + 0.375)^3/12 = 18.36 > 1.27 \text{ in}^4$$

12.8.5 Design of Floorbeam Bracket

The floorbeam brackets are designed next. They can be tapered, because the rapid decrease in bending stress from the girder outward permits a corresponding reduction in web depth. To ensure adequate section throughout, the brackets are tapered from the 48-in depth of the floorbeam main span to 2 ft at the outer end (Fig. 12.23).

Splice at Girder. Bracket flanges are made the same size as the plate required for the moment splice to the main span. This plate is assumed to carry the full maximum negative moment of −341 ft·kips. With an allowable bending stress of 20 ksi, the splice plates then should have an area of at least

$$A_f = \frac{341 \times 12}{48.5 \times 20} = 4.2 \text{ in}^2$$

Use a 12 × $^1/_2$-in plate, with gross area of 6 in^2. After deduction of two holes for $^7/_8$-in-diameter bolts, it provides a net area of 6 − 2 × 1 × $^1/_2$ = 5 in^2. Hence, the bracket flanges also are 12 × $^1/_2$-in plates. Use minimum-size $^1/_4$-in flange-to-web fillet welds.

The number of bolts required in the splice is determined by whichever is larger, 75% of the strength of the splice plate or the average of the calculated stress and the strength of the plate. The calculated stress is $20 \times 4.2/5 = 16.8$ ksi. The average stress is $(20 + 16.8)/2 = 18.4$ ksi. This governs, because 75% of 20 ksi is 15 ksi < 18.4. For A325 $^7/_8$-in bolts with a capacity of 9.3 kips (slip-critical, Class A surface), the number of bolts needed is

$$n = \frac{18.4 \times 5}{9.3} = 10$$

Use 12 bolts.

Connection to Girder. The connection of each bracket to a girder must carry a shear of 59.4 kips. The number of bolts required is

$$n = \frac{59.4}{9.3} = 7$$

Use at least 8 bolts.

Bracket Stiffeners. Stiffener spacing on the brackets generally should not exceed the web depth. Locations of the stiffeners are shown in Fig. 12.23. Use pairs of 4 × $^3/_8$-in plates, as in the main span, with $^1/_4$-in fillet welds.

Check of Bracket Section. Bending and shear stresses at an intermediate point on the bracket should be checked to ensure that, because of the reduction in depth, allowable stresses are not exceeded. For the purpose, a section midway between stringer S1 and the girder is selected. Depth of web there is 48 – 3.5 (48 – 24)/9.5 = 39.15 in. The dead load consists of 0.16 kip/ft from weight of bracket, 0.05 kip/ft from weight of concrete haunch, and 23 kips from the stringers. Thus, the dead-load moment is

$$M_{DL} = -\frac{0.16(6)^2}{2} - \frac{0.05(4.5)^2}{2} - 23 \times 3.5 = -83.9 \text{ ft} \cdot \text{kips}$$

The dead-load shear is

$$V_{DL} = 0.16 \times 6 + 0.05 \times 4.5 + 23 = 24.2 \text{ kips}$$

Live load is 22 kips at 1 ft from the section. Hence, the live-load and impact moments are

$$M_{LL} = -22 \times 1 = -22 \text{ ft} \cdot \text{kips} \qquad M_I = -0.3 \times 22 = -6.6 \text{ ft} \cdot \text{kips}$$

Live-load and impact shears are

$$V_{LL} = 22 \text{ kips} \qquad V_I = 0.3 \times 22 = 6.6 \text{ kips}$$

Moments and shears due to sidewalk live load are

$$M_{SLL} = -5.8 \times 3.5 = -20.3 \text{ ft} \cdot \text{kips} \qquad V_{SLL} = 5.8 \text{ kips}$$

Hence, the total moments and shears at the section are

$$M = -132.8 \text{ ft} \cdot \text{kips} \qquad V = 58.6 \text{ kips}$$

Shear stress in the web is

$$f_v = \frac{58.6}{39.15 \times 0.375} = 4.0 < 12 \text{ ksi}$$

The moment of inertia of the section is

$$I = 2 \times 6 \left(\frac{40.15}{2} \right)^2 + \frac{0.375(39.15)^3}{12} = 6720 \text{ in}^4$$

and the section modulus is

$$S = \frac{6720}{20.08} = 334 \text{ in}^3$$

So the maximum bending stress at the section is

$$f_b = \frac{132.8 \times 12}{334} = 4.8 < 20 \text{ ksi}$$

Therefore, the bracket section is satisfactory.

12.8.6 Design of a Girder Supporting Floorbeams

The girders will be made of Grade 50 steel. Simply supported, they span 137.5 ft, but have a loaded length of 140 ft. They will be made identical.

Loading. Most of the load carried by each girder is transmitted to it by the floorbeams as concentrated loads. Computations are simpler, however, if the floorbeams are ignored and the girder treated as if it received loads only from the slab. Moments and shears computed with this assumption are sufficiently accurate for design purpose because of the relatively close spacing of the floorbeams. Thus, the dead load on the girders may be considered uniformly distributed (Table 12.42).

TABLE 12.42 Dead Load on Girder, kips/ft

Railing:	0.07
Sidewalk: $0.150 \times 1 \times 3$	= 0.45
Slab: $0.150 \times 27 \times 7.5/12$	= 2.53
Floorbeams and stringers:	0.40
Girder—assume:	0.60
Lateral bracing—assume:	0.10
Utilities and miscellaneous:	0.10
DL per girder:	4.25

Sidewalk live load, because the span exceeds 100 ft, is determined from a formula, with loaded length of sidewalk $L = 140$ ft and sidewalk width $W = 3$ ft:

$$p = \frac{(0.03 + 3/L)(55 - W)}{50} = \frac{(0.03 + 3/140)(55 - 3)}{50}$$

$$= 0.0535 \text{ kip/ft}^2$$

Thus, the live load from the 3-ft sidewalk is

$$w_{SLL} = 0.0535 \times 3 = 0.160 \text{ kip/ft}$$

Live load, for maximum effect on a girder, should be placed as indicated in Fig. 12.30. Because of load reductions permitted in accordance with number of lanes of traffic loaded, the number of lanes to be loaded is determined by trial. Let W = wheel load, kips. Then, if two lanes are loaded, with no reduction permitted, the load P, kips, distributed to the girder is

$$P_2 = \frac{(39.5 + 33.5 + 27.5 + 21.5)W}{35} = \frac{122W}{35} = 3.48W$$

If three lanes are loaded, with 10% reduction,

$$P_3 = \frac{0.9(122 + 15.5 + 9.5)W}{35} = \frac{132.3W}{35} = 3.78W > P_2$$

And if all four lanes are loaded, with 25% reduction,

$$P_4 = \frac{0.75(147 + 3.5 - 2.5)W}{35} = \frac{111W}{35} = 3.17W < P_3$$

Therefore, loading in three lanes governs. The girder receives 3.78 wheel loads, or 1.89 axle loads.

Impact for loading over the whole span is taken as the following fraction of live-load stress:

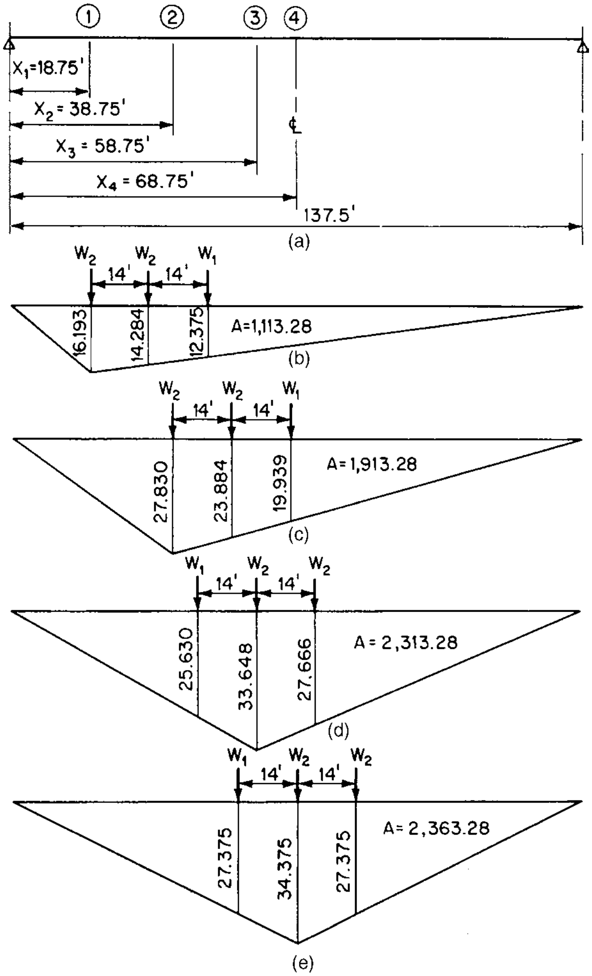


FIGURE 12.30 Moment influence lines for deck girder. (*a*) Location of four points on the girder for which influence diagrams are drawn. (*b*) Diagram for point 1. (*c*) Diagram for 2. (*d*) Diagram for 3. (*e*) Diagram for 4.

$$I = \frac{50}{L+125} = \frac{50}{137.5+125} = 0.19$$

Moments. Curves for maximum moments at points along the span will be drawn by plotting maximum moments at mid-span and at each floorbeam (points 1 to 4 in Fig. 12.30*a*). These moments are calculated with the aid of influence lines drawn for moment at these points (Fig. 12.30*b* to *e*).

Dead-load moments are obtained by multiplying the uniform load w_{DL} = 4.25 kips/ft by the area A of the appropriate influence diagram. Moments due to sidewalk live loading are similarly calculated with uniform load w_{SLL} = 0.16 kip/ft. Dead-load moments are summarized in Table 12.43.

TABLE 12.43 Dead-Load Moments and Sidewalk Live-Load Moments, ft·kips

	Distance from support, ft			
	18.75	38.75	58.75	68.75
Influence area	1,113.3	1,913.3	2,313.3	2,363.3
$M_{DL} = Aw_{DL}$	4,732	8,132	9,832	10,044
$M_{SLL} = Aw_{SLL}$	178	306	370	378

Maximum live-load moments are produced by truck loading on a 137.5-ft span. Since the girder receives 1.89 axle loads, it is subjected at 14-ft intervals to moving concentrated loads:

$$\text{Two } W_2 = 1.89 \times 32 = 60.48 \text{ kips} \qquad W_1 = 1.89 \times 8 = 15.12 \text{ kips}$$

For maximum moment at a point along the span, one load W_2 is placed at the point (Fig. 12.30*b* to *e*). The maximum moment then is the sum of the products of each load by the corresponding ordinate of the applicable influence diagram. Impact moments are 19% of the live-load moments. Table 12.44 summarizes maximum live-load and impact moments.

Total maximum moments are given and the curve of maximum moments (moment envelope) is plotted in Fig. 12.31.

Reaction. Maximum reaction occurs with full load over the entire span. For dead load, with w_{DL} = 4.25 kips/ft,

$$R_{DL} = \frac{4.25 \times 140}{2} = 297.5 \text{ kips}$$

For sidewalk live load, with $w_{SLL} = 0.16$ kip/ft,

$$R_{SLL} = \frac{0.16 \times 140}{2} = 11.2 \text{ kips}$$

Lane loading governs for live load. For maximum reaction and shear, the uniform load of 0.64 kip/ft should cover the entire span and the 26-kip concentrated load should be placed at the support, in each design lane.

$$R_{LL} = 1.89\left(26 + \frac{0.64 \times 140}{2}\right) = 134 \text{ kips}$$

$$R_I = 0.19 \times 134 = 25.4 \text{ kips}$$

The total maximum reaction is $R = 468.1$ kips, say 470 kips.

Shears. Maximum live-load shears at floorbeam locations occur with truck loading between the beam and the far support. A heavy wheel should be at the beam in each design lane. The shears are

TABLE 12.44 Maximum Live-Load and Impact Moments, ft·kips

	Distance from support, ft			
	18.75	38.75	58.75	68.75
M_{LL}	2030	3429	4096	4149
M_I	386	651	778	788

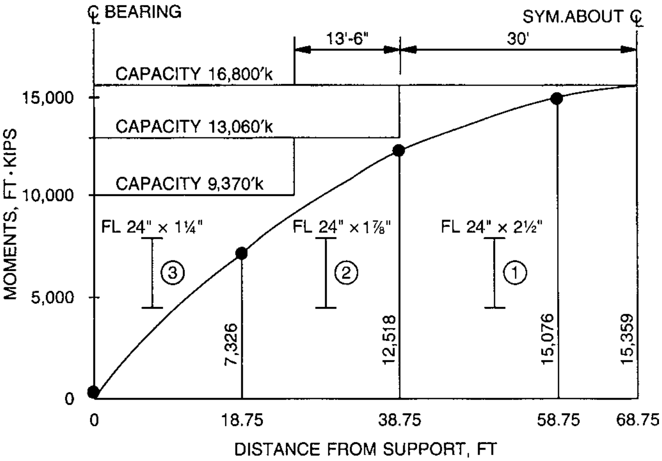


FIGURE 12.31 Moment diagram for deck girder and capacities of various sections.

readily computed with influence diagrams. For example, the influence line for shear at point 1 (Fig. 12.32*a*) is shown in Fig. 12.32*b*. Dead-load shear is obtained as the product of the uniform dead load $w_{DL} = 4.25$ kips/ft by the area of the complete influence diagram.

$$V_{DL} = 4.25(51.276 - 1.279) = 213 \text{ kips}$$

Sidewalk live-load shear is the product of the load $w_{SLL} = 0.16$ kip/ft and the larger of the positive or negative areas of the influence diagram.

$$V_{SLL} = 0.16 \times 51.276 = 8 \text{ kips}$$

Maximum live-load shear is the sum of the products of each load by the corresponding ordinate of the influence diagram (Fig. 12.32*b*).

$$V_{LL} = 60.48 \times 0.8636 + 60.48 \times 0.7618 + 15.12 \times 0.6600 = 108 \text{ kips}$$

The loaded length for impact is 137.5 – 18.75 = 118.75 ft.

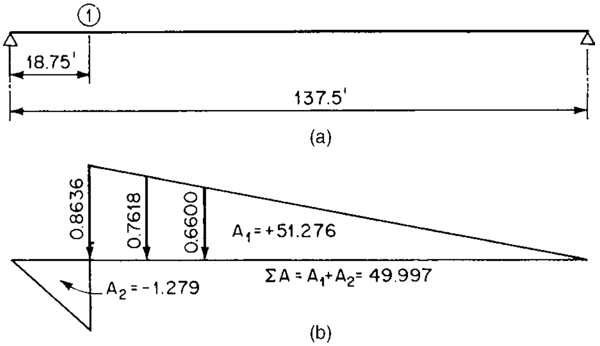


FIGURE 12.32 (*a*) Location of point 1 on girder. (*b*) Influence diagram for shear at point 1.

TABLE 12.45 Maximum Shear, kips

	Distance from support, ft			
	0	18.75	38.75	58.75
Dead load	298	213	127	42
Sidewalk live load	11	8	6	4
Live load	134	108	89	69
Impact	(At 0.19) 26	(At 0.205) 22	(At 0.223) 20	(At 0.245) 17
Total	470	351	242	132

$$V_I = \frac{50}{118.75 + 125} \times 108 = 0.205 \times 108 = 22 \text{ kips}$$

Total maximum shear V_1 = 351 kips (Table 12.45).

Shears at other points are computed in the same way and are listed in Table 12.45.

Web Size. Minimum depth-span ratio for a girder is 1:25. Greater economy and a stiffer member are obtained, however, with a deeper member when clearances permit. In this example, the web is made 110 in deep, $^1/_{15}$ of the span. With an allowable stress of 17 ksi for thin Grade 50 steel, the web thickness required for shear is

$$t = \frac{470}{17 \times 110} = 0.25 \text{ in}$$

Without a longitudinal stiffener, according to Table 10.26 thickness must be at least

$$t = \frac{110\sqrt{50}}{990} = 0.8 \text{ in, say } {}^{13}/_{16} \text{ in}$$

Even with a longitudinal stiffener, however, to prevent buckling, web thickness, from Table 10.26, must be at least

$$t = \frac{110\sqrt{50}}{1980} = 0.393 \text{ in, say } {}^{7}/_{16} \text{ in}$$

though transverse stiffeners also are provided.

If the web were made $^{13}/_{16}$ in thick, it would weigh 304 lb/ft. If it were $^7/_{16}$ in thick, it would weigh 164 lb/ft, 140 lb/ft less. Since the longitudinal stiffener may weigh less than 10 lb/ft, economy favors the thinner web. Use a 110 × $^7/_{16}$-in web with a longitudinal stiffener.

Flange Size at Mid-span. For Grade 50 steel 4 in thick or less, F_y = 50 ksi and the allowable bending stress is 27 ksi. With a maximum moment at mid-span from Fig. 12.31, of 15,359 ft·kips, and distance between flange centroids of about 113 in, the required area of one flange is about

$$A_f = \frac{15{,}359 \times 12}{113 \times 27} = 60.4 \text{ in}^2$$

Assume a 24 × 2$^1/_2$-in plate for each flange. It provides an area of 60 in^2 and has a width–thickness ratio

$$\frac{b}{t} = \frac{24}{2.50} = 9.6$$

which is less than 20 permitted.

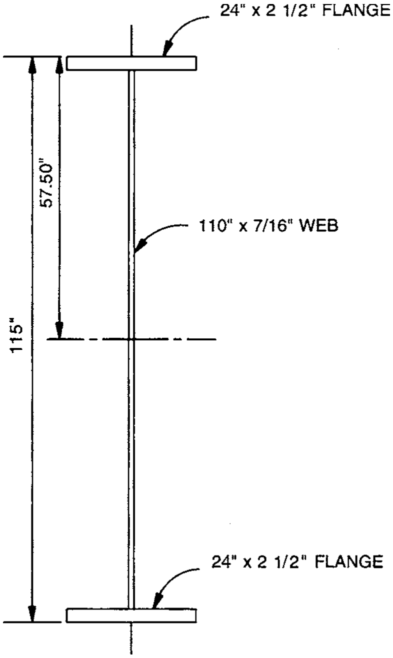


FIGURE 12.33 Cross section of deck girder at mid-span.

The trial section is shown in Fig. 12.33. Moment of inertia is calculated in Table 12.46. Distance from neutral axis to top or bottom of the girder is 57.50 in. Hence, the section modulus is

$$S = \frac{428,200}{57.50} = 7,447$$

Maximum bending stress therefore is

$$f_b = \frac{15,359 \times 12}{7,447} = 24.7 < 27 \text{ ksi}$$

The section is satisfactory. Moment capacity supplied is

$$M_C = \frac{27 \times 7,447}{12} = 16,760 \text{ ft} \cdot \text{kips}$$

The flange thickness will be reduced between mid-span and the supports, and the flange width will remain 24 in. Splices at the changes in thickness will be made with complete-penetration groove welds. For fatigue, the stress category at these splices is B. On the assumption that the structure supports a major highway with an ADTT less than 2500, the number of stress cycles of truck loading is 500,000. Since the bridge is supported by two simple-span girders and floorbeams, it is a non-redundant-path structure, and the allowable stress range is therefore 23 ksi at the splices.

Changes in Flange Size. At a sufficient distance from mid-span, the bending moment decreases sufficiently to permit reducing the thickness of the flange plates to $1^7/_8$ in. The moment of inertia of the section reduces to 330,150, and the section modulus to 5805. Thus, with 24 × $1^7/_8$-in flange plates, the section has a moment capacity of

$$M_C = \frac{5,805 \times 23}{12} = 11,100 \text{ ft} \cdot \text{kips} > 5,315 \text{ ft} \cdot \text{kips}$$

and

$$M_C = \frac{5,805 \times 27}{12} = 13,000 > 12,518 \text{ ft} \cdot \text{kips}$$

TABLE 12.46 Moment of Inertia of Girder

Material	A	d	Ad^2 or I_o
2 flanges 24 × $2^1/_2$	120.0	56.25	379,700
Web 110 × $^7/_{16}$	48.1		48,500
			$I = 428,200 \text{ in}^4$

When this capacity is plotted in Fig. 12.31, the horizontal line representing it stays above the moment envelope until within 35 ft of mid-span. Hence, flange size can be decreased before that point. Length of the $2^1/_2$-in plate then is 75 ft. (See Fig. 12.34.)

At a greater distance from mid-span, thickness of the flange plates can be reduced to a minimum of $1^1/_4$ in because 24/20 = 1.20 in. The moment of inertia drops to 234,200, and the section modulus to 4164. Consequently, with 24 × $1^1/_4$-in plates, the section has a moment capacity of

$$M_C = \frac{4164 \times 23}{12} = 7980 \text{ ft} \cdot \text{kips}$$

and

$$M_C = \frac{4164 \times 27}{12} = 9370 \text{ ft} \cdot \text{kips}$$

When this capacity is plotted in Fig. 12.31, the horizontal line representing it stays above the moment envelope until within 49.5 ft of mid-span. Economy should also be considered while determining a change in flange size. Total cost of a flange splice includes material and labor costs. Labor costs are a function of design, purchasing, and shop practices. For an economical splice, savings in material should exceed the labor associated with it. As a point of reference, an average of approximately 700-lb savings in flange material generally justifies the introduction of a shop splice in a flange. Using this as a guide, the length of 24-in × $1^7/_8$-in flange plates can be determined as follows:

$$L_1 = \frac{700}{(2^1/_2 - 1^7/_8) \times 24 \times 490/144} = 13.7 \text{ ft} \qquad (\text{say, } L_1 = 15 \text{ ft})$$

Then 75/2 + 15 = 52.50 ft > 49.5 ft. The length of 24 × $1^1/_4$-in plate which extends to the end of the girder is therefore (137.50 + 2 × 0.75 − 75.00 − 2 × 15)/2 = 17 ft. (Fig. 12.34).

Flange-to-Web Welds. Each flange will be connected to the web by fillet welds on opposite sides of the web. These welds must resist the horizontal shear between flange and web. At the end section of the girder, for determination of the shear, the static moment is

$$Q = 24 \times 1.25 \times 55.63 = 1669 \text{ in}^3$$

The shear stress then is

$$v = \frac{VQ}{I} = \frac{470 \times 1669}{234,200} = 3.35 \text{ kips/in}$$

The minimum size of fillet weld permissible, governed by the thickest plate at the section, is $^5/_{16}$ in. With an allowable shear stress $F_v = 0.27F_u = 0.27 = 65 \times 17.6$ ksi, the allowable load per weld 17.6 × 0.707 = 12.44 kips/in, and for two welds, 24.89 kips/in. Hence, the capacity of two $^5/_{16}$-in fillet welds is 24.89 × $^5/_{16}$ = 7.78 kips/in > 3.35 kips/in. Use two $^5/_{16}$-in welds. (See also the design of fillet welds in Art. 12.8.4.)

Intermediate Transverse Stiffeners. Where required, a pair of transverse stiffeners of Grade 36 steel will be welded to the girder web. Minimum width of stiffener is 24/4 = 6.0 in > (2 + 110/30 = 5.7 in). Use a $7^1/_2$-in-wide plate. Minimum thickness required is $^7/_{16}$ in. Try a pair of $7^1/_2$ × $^7/_{16}$-in stiffeners.

Maximum spacing of the transverse stiffeners can be computed from Eq. (10.25a). For the 110 × $^7/_{16}$-in girder web and a maximum shear at the support of 470 kips, the average shear stress is 470/48.1 = 9.77 ksi. The web depth–thickness ratio $D/t_w = 110/(^7/_{16}) = 251$. Maximum spacing of stiffeners is limited to $110(260/251)^2 = 118$ in. Try a stiffener spacing $d_o = 80$ in. This provides a depth-spacing ratio $D/d_o = 110/80 = 1.375$. From Eq. (10.24d), for use in Eq. (10.25a),

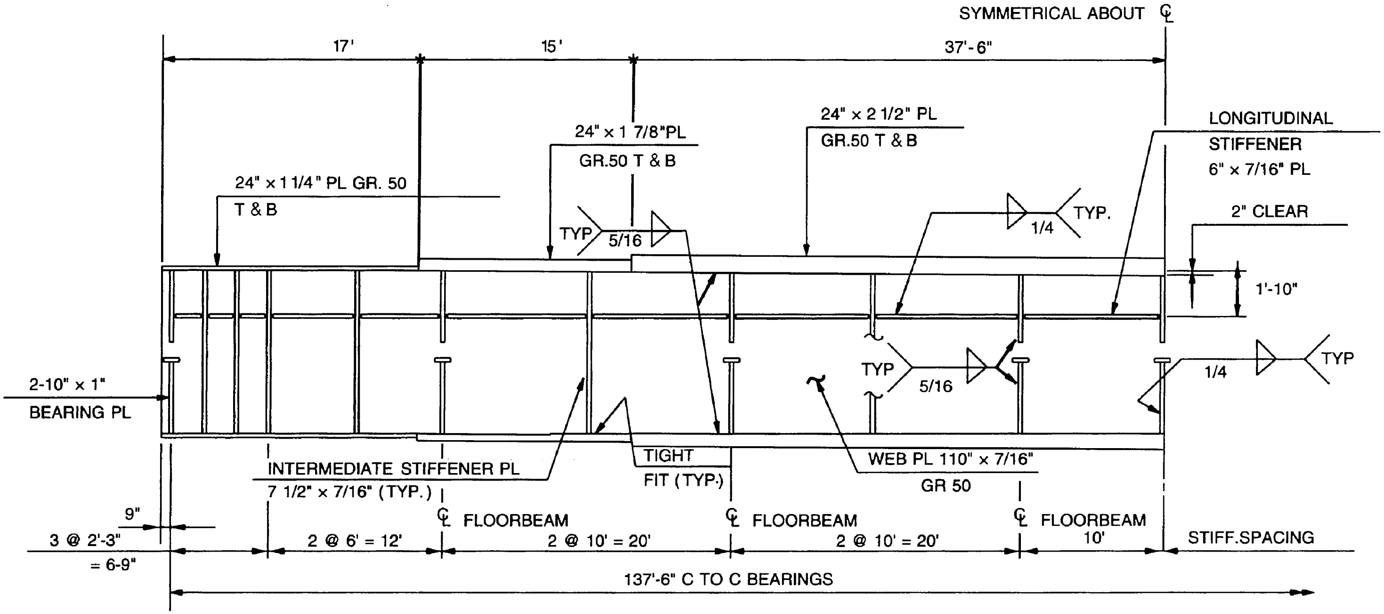


FIGURE 12.34 Details of deck plate girder for four-lane highway bridge.

$k = 5[1 + (1.375)^2] = 14.45$ and $\sqrt{k/F_y} = \sqrt{14.45/50} = 0.537$. Since $D/t_w = 251$, C in Eq. (10.25a) is determined by the parameter $251/0.537 = 467 > 237$. Hence, C is given by Eq. (10.24c):

$$C = \frac{45{,}000k}{(D/t_w)^2 F_y} = \frac{45{,}000 \times 14.45}{251^2 \times 50} = 0.206$$

From Eq. (10.25a), the maximum allowable shear for $d_o = 80$ in is

$$F_v' = F_v\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]$$

$$= \frac{50}{3}\left[0.206 + \frac{0.87(1-0.206)}{\sqrt{1+(80/110)^2}}\right] = 12.74 \text{ ksi} > 9.77 \text{ ksi}$$

Since the allowable stress is larger than the computed stress, the stiffeners may be spaced 80 in apart. The location of floorbeams, however, may make closer spacing preferable.

The AASHTO standard specifications limit the spacing of the first intermediate transverse stiffener to the smaller of $1.5D = 1.5 \times 110 = 165$ and the spacing for which the allowable shear stress in the end panel does not exceed

$$F_v = \frac{CF_y}{3} = 0.206 \times \frac{50}{3} = 3.43 \text{ ksi} < 9.77 \text{ ksi}$$

Much closer spacing than 80 in is required near the supports. Try $d_o = 27$ in, for which $k = 88$ and $C = 1$. Hence, $F_v = 50/3 = 17$ ksi > 9.62 ksi. Spacing selected for intermediate transverse stiffeners between the supports and the first floorbeam is shown in Fig. 12.36.

At that beam, the shear stress is $f_v = 351/48.1 = 7.28$ ksi. Try a stiffener spacing $d_o = 10$ ft = 120 in, which is less than the 127-in limit. This provides $D/d_o = 0.917$, $k = 9.20$, and $C = 0.131$. The allowable shear for this spacing then is

$$F_v = \frac{50}{3}\left[0.131 + \frac{0.87(1-0.131)}{\sqrt{1+(120/110)^2}}\right] = 10.69 \text{ ksi} > 7.28 \text{ ksi}$$

The 10-ft spacing is satisfactory. Actual spacing throughout the span is shown in Fig. 12.36.

The moment of inertia provided by each pair of stiffeners must satisfy Eq. (10.21), with J as given by Eq. (10.22):

$$J = 2.5\left(\frac{110}{27}\right)^2 - 2 = 39.5 > 0.5$$

$$I = 27(^7/_{16})^3 39.5 = 89.3 \text{ in}^4$$

The moment of inertia furnished by a pair of $7^1/_2$-in-wide stiffeners is

$$I = \frac{(^7/_{16})(15.437)^3}{12} = 134 > 89.3 \text{ in}^4$$

Hence, the pair of $7^1/_2 \times {}^7/_{16}$-in stiffeners is satisfactory.

Longitudinal Stiffener. One longitudinal stiffener of Grade 36 steel will be welded to the web. It should be placed with its centerline at a distance 110/5 = 22 in below the bottom surface of the compression flange for a symmetrical girder (Fig. 12.36).

Assume a 6-in-wide stiffener. Then, by Eq. (10.28b), the thickness required is

$$t = \frac{6\sqrt{36}}{82.22} = 437 \text{ in} \qquad (\text{say } {}^{7}\!/_{16} \text{ in})$$

Moment of inertia furnished with respect to the edge in contact with the web is

$$I = \frac{0.437(6)^3}{3} = 31.5 \text{ in}^4$$

With transverse stiffeners spaced 120 in apart, the moment of inertia required by Eq. (10.28a), is

$$I_{\min} = 110(0.437)^3 \left[2.4\left(\frac{120}{110}\right)^2 - 0.13 \right] = 25.1 < 31.5 \text{ in}^4$$

Therefore, use a 6 × $^{7}/_{16}$-in plate for the longitudinal stiffener. A 6 × $^{3}/_{8}$-in plate would also check.

Bearing Stiffeners. A pair of bearing stiffeners of Grade 50 steel is provided at each support. They are designed to transmit the 470-kip end reaction between bearing and girder.

Try 10 × 1-in plates. With provision for clearing the flange-to-web fillet weld, the effective width of each plate is 10 – 1.0 = 9.00 in. The effective bearing area is 2 × 1 × 9.00 = 18.0 in^2. Allowable bearing stress is 40 ksi. Actual bearing stress is

$$f_p = \frac{470}{18.0} = 26.1 < 40 \text{ ksi}$$

The width–thickness ratio of the assumed plate $b/t = 10/1 = 10$ satisfies Eq. (10.20), with $F_y = 50$ ksi:

$$\frac{b}{t} = \frac{69}{\sqrt{50}} = 9.75 < 10$$

The pair of stiffeners is designed as a column acting with a length of web equal to 18 times the web thickness, or 7.88 in. Area of the column is

$$2 \times 10 \times 1 + 7.88 \times \frac{7}{16} = 23.44 \text{ in}^2$$

Buckling is prevented by the floorbeam connecting to the stiffeners. Hence, the stress in the stiffeners must be less than the allowable compressive stress of 27 ksi and need not satisfy the column formulas. For the 470-kip reaction, the compressive stress is

$$f_a = \frac{470}{23.44} = 20.1 < 27 \text{ ksi}$$

Therefore, the pair of 10 × 1-in bearing stiffeners is satisfactory.

Stiffener–web welds must be capable of developing the entire reaction. With fillet welds on opposite sides of each stiffener, four welds are used. They extend the length of the stiffeners, from the bottom of the 48-in-deep floorbeam to the girder tension flange. Thus, total length of the welds is 4(100 – 48 – $^{7}/_{16}$ – 2.5) = 236 in. Average shear on the welds is

$$v = \frac{470}{236} = 1.99 \text{ kips/in}$$

Weld size required to carry this shear is, with allowable stress $F_v = 0.27F_o = 17.6$ ksi,

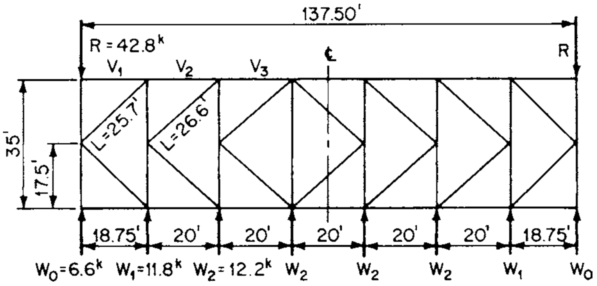


FIGURE 12.35 Lateral bracing system for deck-girder bridge.

$$\frac{1.99}{0.707 \times 17.6} = 0.16 \text{ in}$$

This, however, is less than the $^5/_{16}$-in minimum size of weld required for a 1-in-thick plate. Therefore, use $^5/_{16}$-in fillet welds.

12.8.7 Design of Horizontal Lateral Bracing

Each girder flange is subjected to half the transverse wind load. The top flange is assisted by the concrete deck in resisting the load and requires no lateral bracing. The following illustrates design of lateral bracing for the bottom.

Figure 12.35 shows the layout of the lateral truss system, which lies in a plane at the bottom of the floorbeams. The girders comprise the chords of the truss, and the floorbeams the transverse members, or posts. The truss must be designed to resist a wind load of 50 lb/ft^2, but not less than 300 lb/lin ft, on the exposed area. The wind is considered a uniformly distributed, moving load acting perpendicular to the girders and reversible in direction.

The uniform load on the girder for an exposed depth of 12.14 ft (Table 12.47) is

TABLE 12.47 Exposed Area, ft^2/lin ft

Railing	0.91
Slab	1.83
Girder	9.40
Total	12.14

$$w = 0.050 \times 12.14 = 0.61 \text{ kip/ft}$$

It is resolved into a concentrated load at each panel point (Fig. 12.35):

$$W_2 = 0.61 \times 20 = 12.2 \text{ kips}$$

$$W_1 = \frac{0.61(20 + 18.75)}{2} = 11.8 \text{ kips}$$

$$W_0 = 0.61\left(\frac{18.75}{2} + 1.5\right) = 6.6 \text{ kips}$$

The reaction at each support is

$$R = 2 \times 12.2 + 11.8 + 6.6 = 42.8 \text{ kips}$$

With the wind considered a moving load, maximum shear in each panel is

$$V_1 = 42.8 - 6.6 = 36.2 \text{ kips}$$

$$V_2 = 36.2 - 11.8 \times \frac{118.75}{137.5} = 25.9 \text{ kips}$$

$$V_3 = 25.9 - 12.2 \times \frac{98.75}{137.5} = 17.1 \text{ kips}$$

$$V_4 = 17.1 - 12.2 \times \frac{78.75}{137.5} = 10.1 \text{ kips}$$

The shear is assumed to be shared equally by the two diagonals in each panel. Since the direction of the wind is reversible, the stress in each diagonal may be tension or compression. Design of the members is governed by compression.

The diagonals, being secondary compression members, are permitted a slenderness ratio L/r up to 140. (The effective length factor K is taken conservatively as unity.) For the end panel, the length c to c of connections is

$$L = 25.7 - 3 = 22.7 \text{ ft}$$

Hence, the radius of gyration should be at least $r = 22.7 \times 12/140 = 1.95$ in. Similarly, for interior panels, minimum $r = 23.6 \times 12/140 = 2.02$ in.

Assume for the diagonals a WT6 × 26.5 (Fig. 12.36). It has the following properties:

$$S_x = 3.54 \text{ in}^3 \qquad r_x = 1.51 \text{ in} \qquad r_y = 2.48 \text{ in} \qquad A = 7.80 \text{ in}^2 \qquad y = 1.02 \text{ in}$$

To permit the slenderness ratio about the vertical axis to govern the design, provide a vertical brace at mid-length of each diagonal. The minimum slenderness ratio then is

$$\frac{L}{r_y} = \frac{22.7 \times 12}{2.48} = 110 < 140$$

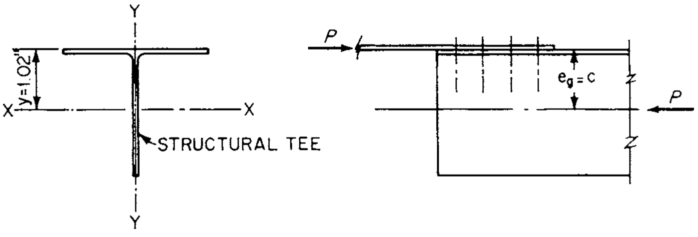
Horizontal Buckling. For a column of Grade 36 steel with this slenderness ratio and with bolted ends, the allowable compressive stress is

$$F_a = 16.98 - 0.00053\left(\frac{L}{r}\right)^2 = 16.98 - 0.00053(110)^2 = 10.57 \text{ ksi}$$

Maximum stress occurs in the end panel where wind shear is a maximum, 36.2 kips. Each diagonal is assumed to carry half this, 18.1 kips. Thus, it is subjected to an axial force of

$$F = \frac{18.1 \times 25.7}{17.5} = 26.6 \text{ kips}$$

This causes an average compressive stress in the diagonal of



(a) (b)

FIGURE 12.36 Diagonal brace. (*a*) Cross section. (*b*) Eccentric loading on end connection of the diagonal.

$$f_a = \frac{26.6}{7.80} = 3.4 < 10.57 \text{ ksi}$$

Hence, the WT6 × 26.5 is adequate for resisting buckling in the horizontal direction.

Vertical Buckling. Because of the T shape of the WT, its end connections load it eccentrically. Therefore, the diagonal should be checked for combined axial plus bending stresses and buckling in the vertical direction. The eccentricity and c distance from the neutral axis to the top of the compression flange is 1.02 in (Fig. 12.36). The slenderness ratio for buckling in the vertical direction, with a conservative value of $K = 1.0$ and provision for a mid-length brace, is

$$\frac{L}{r_x} = \frac{12 \times 22.7/2}{1.51} = 90.2$$

Members subjected to combined axial compression and bending must satisfy

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex})F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey})F_{by}} \leq 1.0$$

where $F'_e = \dfrac{\pi^2 E}{FS(K_b L_b/r_b)^2}$

$FS = 2.12$

C_m = coefficient defined in Art. 5.7 (1.0 is conservative)

The axial stress f_a is 3.4 ksi and the allowable stress is

$$F_a = 16.98 - 0.00053\left(\frac{KL}{r_x}\right)^2 = 16.98 - 0.00053(90.2)^2 = 12.7 \text{ ksi}$$

The bending stress f_b is 26.6 × 1.02/3.54 = 7.66 ksi. The allowable bending stress for Grade 36 steel in this case is $F_b = 20.0$ ksi.

$$F'_e = \frac{\pi^2(29{,}000)}{2.12(90.2)^2} = 16.6 \text{ ksi}$$

Substitution in the interaction equation gives

$$\frac{3.4}{12.7} + \frac{1.0 \times 7.66}{[1 - (3.4/16.6)]20.0} = 0.27 + 0.48 = 0.75 < 1.0 \qquad \text{OK}$$

Use the WT6 × 26.5 for all the diagonals.

Bracing Connections. End connections of the laterals are to be made with A325 $^7/_8$-in-diameter high-strength bolts. These have a capacity of 9.3 kips in slip-critical connections with Class A surfaces. The number of bolts required is determined by whichever is larger, 75% of the strength of the diagonal or the average of the calculated stress and strength of the diagonal.

In the computation of the tensile strength of the T section, the effective area should be taken as the net area of the connected flange plus half the area of the outstanding web (Table 12.48). With an allowable stress of 20 ksi, the tensile capacity is

$$T = 5.71 \times 20 = 114 \text{ kips}$$

TABLE 12.48 New Area of Diagonal, in^2

Gross area:	7.80
Half web area: $-5.45 \times 0.345/2$	$= -0.94$
Two holes: $-2 \times 1 \times 0.576$	$= -1.15$
Net area:	5.71

Compressive capacity with $F_a = 8.5$ ksi on the gross area is

$$C = 7.80 \times 8.5 = 66 < 114 \text{ kips}$$

Tensile capacity governs. Hence, the number of bolts required is determined by

$$0.75 \times 114 = 86 \text{ kips} > \left(\frac{26.6 + 114}{2} = 70 \text{ kips}\right)$$

and equals 86/9.3 = 9.2. Use ten $^7/_8$-in high-strength bolts.

12.9 THROUGH-PLATE–GIRDER BRIDGES WITH FLOORBEAMS

For long or heavily loaded bridge spans, restrictions on depth of structural system imposed by vertical clearances under a bridge generally favor use of through construction. Through girders support the deck near their bottom flange. Such spans preferably should contain only two main girders, with the railway or roadway between them (Fig. 12.37). In contrast, deck girders support the deck on the top flange (Art. 12.7).

The projection of the girders above the deck in through bridges may be objectionable for highway structures, because they obstruct the view from the bridge of pedestrians or drivers. But they may offer the advantage of eliminating the need for railings and parapets. For railroad bridges over highways, streets, or other facilities from which the bridges are highly visible to the general public, through girders provide a more attractive structure than through trusses.

The projection of the girders above the deck also has the disadvantage of requiring special provisions for bracing the compression flange of the girders. Deck girders usually require no special provision for this purpose, because when a rigid deck is used, it provides the needed lateral support. Through girders should be laterally braced with gusset plates or knee braces with solid webs connected to the stiffeners.

In railroad bridges, spacing of the through girders should be at least $^1/_{20}$ of the span, or should be adequate to ensure that the girders and other structural components provide required clearances for trains, whichever is greater.

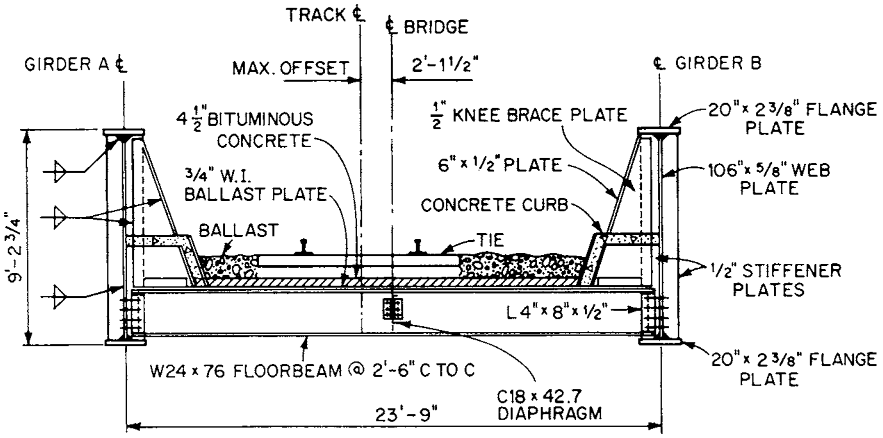


FIGURE 12.37 Cross section of through-girder railroad bridge.

Article 12.10 presents an example to indicate the design procedure for a through-girder bridge with floorbeams. Because the example in Art. 12.8 dealt with highway loading, additional information is provided by designing a railroad bridge in the following example. Also, a curved alignment is selected, whereas the girders are kept straight, to illustrate the application of centrifugal forces to the structure. Note that because the girders are straight, the centerline of the track is offset from the centerline of the bridge. Design procedures not discussed in the example generally are the same as for deck girders (Art. 12.8) or plate-girder stringers. See also Chap. 11.

12.10 EXAMPLE—ALLOWABLE STRESS DESIGN OF THROUGH-PLATE–GIRDER BRIDGE

Two simply supported, welded, through-plate girders carry the single track of a railroad bridge on an 86-ft span (Fig. 12.37). The girders are spaced 23.75 ft c to c. The track is on an 8° curve, for which the maximum design speed is 30 mi/h. Maximum offset of centerline of track for centerline of bridge is 2.12 ft. Live loads from the trains are distributed by ties, ballast, and a Grade 50W steel ballast plate to rolled-steel floorbeams spaced 2.5 ft c to c. These beams transmit the loads to the girders. Steel to be used is Grade 36. Loading is Cooper E65.

12.10.1 Design of Floorbeams

For convenience in computing maximum moment, the dead load on a floorbeam may be considered to consist of three parts: weight of track and load-distributing material, spread over about 18.5 ft; weight of floorbeam and connections, distributed over the span, which is taken as 23.5 ft; and weight of concrete curb, which is treated as a concentrated load (Table 12.49). This loading produces a reaction

TABLE 12.49 Dead Load on Floorbeam, kips/ft

Track: 0.200 × 2.5/18.5	= 0.027
Tie: 0.160/18.5	= 0.009
Ballast: 0.120 × 1 × 2.5	= 0.300
Bituminous concrete: 0.150 × 2.5 × 4.5/12	= 0.140
$^3/_4$-in ballast plate: 0.0306 × 2.5	= 0.077
Load over 18.5 ft:	0.553
Beam—assume:	0.080
Concrete curb: 0.150 × 2.5 × 2.5 × 2 = 1.9 kips	

$$R_{DL} = \frac{0.553 \times 18.5}{2} + \frac{0.080 \times 23.5}{2} + 1.9 = 7.9 \text{ kips}$$

Maximum bending moment occurs at mid-span and equals

$$\begin{aligned} M_{DL} &= 7.9 \times 11.75 - 1.9 \times 10.50 - 0.553 \times 9.25 \times 4.63 - 0.080 \times 11.75 \times 5.88 \\ &= 44 \text{ ft} \cdot \text{kips} \end{aligned}$$

The live load P, kips, carried by the floorbeam can be computed from

$$P = \frac{1.15AD}{S} \qquad S \geq d \tag{12.42}$$

where A = axle load, kips
S = axle spacing, ft
D = effective beam spacing, ft
d = actual beam spacing, ft

with D taken equal to d. The axle load A = 65 kips, and the axle spacing S = 5 ft.

$$P = \frac{1.15 \times 65 \times 2.5}{5} = 37.4 \text{ kips}$$

$P/2$ = 18.7 kips is applied as a concentrated load at each rail. Then loads cause a reaction at maximum offset of

$$R_{LL} = \frac{18.7(11.98 + 7.27)}{23.5} = 15.3 \text{ kips}$$

Maximum moment occurs under a rail and equals

$$M_{LL} = 15.3 \times 11.52 = 176.5 \text{ ft} \cdot \text{kips}$$

Impact for this example is taken as 36% of wheel live-load stresses. (See Art. 10.5.2 for current requirements.) Impact moment then is

$$M_I = 0.36 \times 176.5 = 63.5 \text{ ft} \cdot \text{kips}$$

The total moment is 284 ft·kips. This requires a section modulus,

$$S = \frac{284 \times 12}{20} = 170 \text{ in}^3$$

Use a W24 × 76, with S = 176 in^3.

The centrifugal force as a percentage of live load, as calculated in the next article, is 8.43%. When it is resolved into a couple, it causes an increase in vertical wheel load on the outside rail equivalent to (6/4.71) × 0.0843 = 0.11. The centrifugal force moment is

$$M_{CF} = 163.8 \times 0.11 = 18 \text{ ft} \cdot \text{kips}$$

The reduction on the inside rail will be ignored.

Maximum live-load floorbeam reaction is 37.4 − 15.3 = 22.1 kips. The maximum floorbeam reaction is

$$R = 7.9 + 22.1 + 22.1 \times 0.36 + 22.1 \times 0.11 = 40.4 \text{ kips}$$

12.10.2 Design of Girders

The girders will be made of Grade 36 steel. Simply supported, they span 86 ft. They will be made identical.

Dead Load. Most of the load carried by each girder is transmitted to it by the floorbeams as concentrated loads. Computations are simpler, however, if the floorbeams are ignored and the girder is treated as if it received load from the ballast plate. Moments and shears computed with this assumption are sufficiently accurate for design purposes because of the relatively close spacing of the floorbeams. Thus, the dead load on the girder may be considered uniformly distributed. It is computed to be 3.765 kips/ft.

Maximum dead-load moment occurs at mid-span and equals

$$M_{DL} = \frac{3.765(86)^2}{8} = 3{,}500 \text{ ft} \cdot \text{kips}$$

Dead-load moments along the span are listed in Table 12.50. Maximum dead-load shear and the reaction is

$$R_{DL} = \frac{3.765 \times 86}{2} = 162 \text{ kips}$$

TABLE 12.50 Moments in Girders, ft·kips

Distance from supports, ft							
L_1	L_2	Dead load M_{DL}	Equivalent E10 loading q	$3.83q$	Live load M_{LL}, $3.83qL_1L_2/2$	Impact M_r, $0.175 \times 6.5qL_1L_2/2$	Centrifugal force C
15	71	2010	1.40	5.37	2860	850	180
30	56	3170	1.33	5.09	4280	1270	280
40	46	3470	1.34	5.12	4720	1400	310
43	43	3500	1.33	5.09	4720	1400	310

Live Load. Computation of live-load moments, shears, and reactions is simplified with the aid of tables or charts. (See, for example, D. B. Steinman, "Locomotive Loadings for Railway Bridges," *ASCE Transactions*, vol. 86, pp. 606–723, and T. A. Ostram and S. L. Mellon, "Bridge Engineering," *Standard Handbook for Civil Engineers*, 5th ed., McGraw-Hill, New York.) If figures are available for any magnitude of Cooper E loading, those for any other magnitude can be obtained by proportion.

Since the tracks are not centered between the girders, one girder will be more heavily loaded than the other. The amount of live load transmitted to the more heavily loaded girder may be obtained by taking moments about the other girder. Let q be the equivalent uniform live load for E10 loading, kips/ft. Then, $6.5q$ is the equivalent load for E65, and the girder receives $6.5q \times 140.0/23.75 = 3.83q$. Live-load moments along the span are listed in Table 12.50.

Maximum reaction and shear under E10 loading is 66.1 kips. Hence, the maximum reaction for E65 loading is

$$R_{LL} = 66.1 \times 3.83 = 254 \text{ kips}$$

Impact is taken as 17.5% of the axle live-load stresses. Therefore, the impact moment is

$$M_I = 0.175 \times 6.5 \times 1.33 \times 43^2/2 = 1400 \text{ ft}\cdot\text{kips}$$

and the impact reaction is

$$R_I = 0.175 \times 6.5 \times 1.33 \times \frac{86}{2} = 65 \text{ kips}$$

Centrifugal Force. This is computed as a percentage of live load, with speed S = 30 mi/hr and degree of curve $D = 8°$.

$$C = 0.00117S^2D = 0.00117(30)^2 8 = 8.43\%$$

Application of this percentage to the equivalent load producing maximum moment yields the equivalent centrifugal force,

$$C_e = 0.0843 \times 1.33 \times 6.5 = 0.73 \text{ kip/ft}$$

This force acts 6 ft above top of rail, or 10.8 ft above bottom of girder. It is resisted by a couple consisting of a vertical force at each girder equivalent to

$$q_C = \frac{0.73 \times 10.8}{23.75} = 0.332 \text{ kip/ft}$$

The maximum moment produced by these forces can be obtained by proportion from the maximum live-load moment:

$$M_C = \frac{4,720 \times 0.332}{5.09} = 310 \text{ ft} \cdot \text{kips}$$

Similarly, the maximum shear and reaction equal

$$R_C = \frac{254 \times 0.332}{5.09} = 17 \text{ kips}$$

Longitudinal Force. The longitudinal force for E-80 loading is given as the larger of the force due to braking or force due to traction.

Braking force = 45 + 1.2L = 45 + 1.2 × 86 = 148 kips, acting 8 ft above top of rail

Traction force = $25\sqrt{L} = 25\sqrt{86} = 232$ kips, acting 3 ft above top of rail

Longitudinal braking force governs the design, causing a reaction of

$$R_L = \left(148 \times \frac{12.8}{86}\right)\left(\frac{14}{23.75}\right)\left(\frac{65}{80}\right) = 11 \text{ kips}$$

at the bottom of the girder at each support.

Wind Transverse to Bridge. The wind may act on live load and structure in any horizontal direction. The wind load on the train should be taken as a moving load of 0.3 kip/ft, acting 8 ft above top of rail. Wind load on the structure should be taken as 0.03 kip/ft^2, acting on 1.5 times the vertical projection of the span.

Transverse to the bridge, wind on the live load, acting 12.8 ft above the bottom of the girder, imposes vertical forces on the girder of 0.3 × 12.8/23.75 = 0.162 kip/ft. This causes a mid-span bending moment of

$$M_{WLL} \frac{0.162(86)^2}{8} = 150 \text{ ft} \cdot \text{kips}$$

Maximum shear and reaction equal

$$R_{WLL} = \frac{0.162 \times 86}{2} = 7 \text{ kips}$$

In addition, acting at each of the four girder supports is a transverse horizontal force,

$$H_{WLL} = \frac{0.3 \times 86}{4} = 6.5 \text{ kips}$$

Transverse wind on a projection of 9.3 ft of structure imposes a load of

$$0.030 \times 9.3 \times 1.5 = 0.420 \text{ kip/ft}$$

It acts about 4.7 ft above the bottom of the girder. The resulting overturning moment causes vertical forces in the girders of 0.420 × 4.7/23.75 = 0.083 kip/ft. These forces produce a mid-span bending moment

$$M_W = \frac{0.083(86)^2}{8} = 77 \text{ ft} \cdot \text{kips}$$

The reaction is

$$R_W = \frac{0.083 \times 86}{2} = 3.6 \text{ kips}$$

Also, a transverse horizontal force acts at each of the four girder supports:

$$H_W = \frac{0.42 \times 86}{4} = 9 \text{ kips}$$

Longitudinal Wind. For girder spans, longitudinal wind on the live load transmitted to the girder equals 25% of the lateral wind force, 0.25 × 0.42/2 = 0.05 kip/ft. Acting 12.8 ft above the bottom of the girder, it imposes vertical and horizontal longitudinal forces at the supports:

$$R_{WL} = \frac{0.05 \times 86 \times 12.8}{86} = 0.7 \text{ kip}$$

$$H_{WL} = 0.05 \times 86 = 4.3 \text{ kips}$$

Wind on Unloaded Bridge. The structure also should be investigated for a transverse wind load of 50 lb/ft^2 on 1.5 times the vertical projection of the span. Moments and shears caused can be obtained by proportion from those previously computed.

$$M_W = \frac{77 \times 50}{30} = 128 \text{ ft} \cdot \text{kips}$$

$$R_W = \frac{3.6 \times 50}{30} = 6 \text{ kips}$$

Loading Combinations. Three loading combinations are investigated:

Case I: *DL* + *LL* + *I* + *C* at full basic allowable stresses (Table 12.51)

Case II: Case I + wind on loaded bridge + longitudinal force at 125% of basic allowable stresses (Table 12.52)

Case III: Dead load + wind on unloaded bridge at 125% of basic allowable stresses (Table 12.53)

Moments and shears for case I are larger than those for case III and, when allowance is made for a 25% increase in allowable stresses for case II, also larger than those for case II. Hence, case I at basic allowable stresses governs the design.

The curve of maximum moments at various points of the span, or moment envelope (Fig. 12.38), can now be plotted for case I.

TABLE 12.51 Loading Case I—Maximum Moments and Shears

Type of loading	Moment, ft·kips	Shear, kips
DL	3500	162
LL	4720	254
I	1400	65
C	310	17
Total	9930	498

TABLE 12.52 Loading Case II—Maximum Moments and Shears

Type of loading	Moment, ft·kips	Shear, kips
Case I	9,930	498
Wind	230	11
LF	…	11
Total	10,160	520

TABLE 12.53 Loading Case III—Moments and Shears

Type of loading	Moment, ft·kips	Shear, kips
DL	3500	162
Wind on bridge	128	6
Total	3628	168

Web Size. While depth of web has no effect on vertical clearances under through-girder bridges, it has several effects on economy. The deeper the girders, the less flange material is required and the stiffer the members. But web thickness and number of stiffeners required usually increase. Also, girder spacing may have to be increased, because of wider gussets or knee braces needed for lateral bracing.

In this example, a web depth of 106 in is assumed. With an allowable stress of 12.5 ksi, the web thickness required for shear is

$$t = \frac{504}{12.5 \times 106} = 0.380 \text{ in}$$

To prevent buckling, however, even with transverse stiffeners, thickness should be at least $^1/_{160}$ of the clear distance between flanges.

$$t = \frac{106}{160} = 0.663 \text{ in} \qquad (\text{say } ^{11}/_{16} \text{ in})$$

Use a 106 × $^{11}/_{16}$-in web.

Flange Size at Mid-span. To select a trial size for the flange, assume an allowable bending stress $F_b = 20$ ksi and distance between flange centroids of about 110 in. Then, for a maximum moment of 9930 ft·kips, the required area of one flange is about

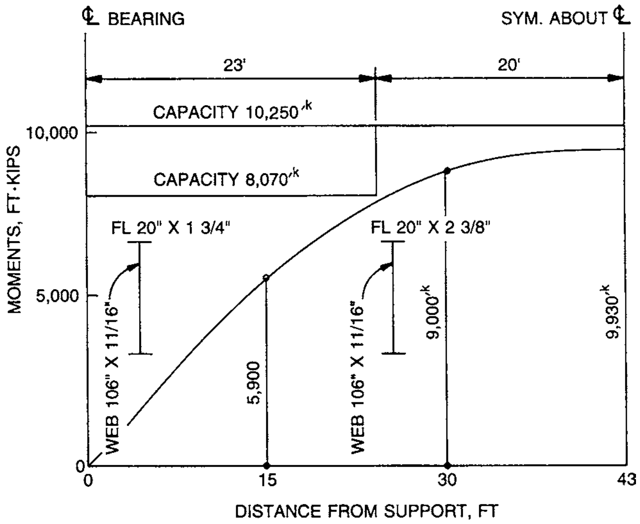


FIGURE 12.38 Moment envelope for through girder and capacities of cross sections.

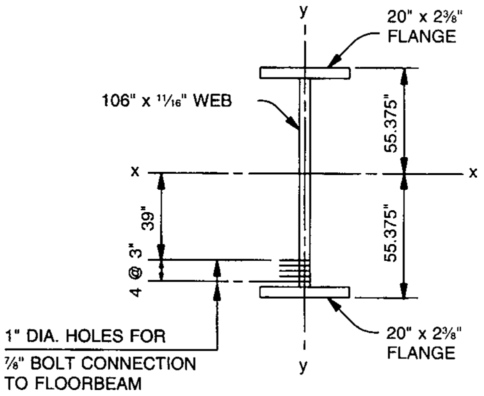


FIGURE 12.39 Cross section of through girder at mid-span.

$$A_f = \frac{9930 \times 12}{110 \times 20} = 54 \text{ in}^2$$

Assume a 20 × $2^3/_8$-in plate for each flange, with an area of 47.5 in^2. Width–thickness ratio of outstanding portion is 10/2.38 = 4.2, which is less than 12 permitted. The trial section is shown in Fig. 12.39. Moment of inertia of the section about the *X* axis is calculated in Table 12.54. Distance from neutral axis to top or bottom of the girder is 55.4 in. Hence, the gross and net section moduli are

$$S_g = \frac{347{,}100}{55.4} = 6{,}270 \text{ in}^3 \qquad S_{\text{net}} = \frac{340{,}100}{55.4} = 6{,}150 \text{ in}^3$$

The allowable tensile bending stress is 20 ksi. The actual tensile stress is

$$f_b = \frac{9930 \times 12}{6150} = 19.4 < 20 \text{ ksi}$$

The allowable compressive bending stress is a function of *l*, the distance, in, between points of lateral support of the compression flange, and r_y, the radius of gyration, in, of the compression flange and that portion of the web area on the compression side of the axis of bending, about the axis in the plane of the web. For a rectangular section, $r = 0.289d$, where d = depth of section perpendicular to axis. Hence, for the compression flange,

$$r_y = 0.289 \times 20 = 5.78 \text{ in}$$

TABLE 12.54 Moment of Inertia of Through-Plate Girder at Mid-span

Material	A	d	Ad^2 or I_o
2 flanges 20 × $2^3/_8$	95.0	54.19	278,900
Web 106 × $^{11}/_{16}$			68,200
			I_g = 347,100 in^4
4 holes: $-1(^{11}/_{16})(39^2 + 42^2 + 45^2 + 48^2 + 51^2)$			= −7,000
			I_{net} = 340,100 in^4

AREMA specifications limit the spacing of lateral supports for the compression flange to a maximum of 12 ft for through girders. Since the knee braces are placed at floorbeam locations, which are 30 in apart, space the knee braces 10 ft = 120 in c to c. Then, the allowable compressive stress is the larger of the following:

$$F_b = 20 - 0.0004\left(\frac{I}{r_y}\right)^2 = 20 - 0.0004\left(\frac{120}{5.78}\right)^2 = 19.83 \text{ ksi}$$

$$F_b = \frac{10{,}500A_f}{ld} = \frac{10{,}500 \times 47.5}{120 \times 110.75} = 37.5 \text{ ksi}$$

but not to exceed 20 ksi. The actual compressive stress is

$$f_b = \frac{9930 \times 12}{6270} = 19.0 < 19.83 \text{ ksi}$$

The section is satisfactory. Moment capacity supplied is

$$M_C = \frac{20 \times 6{,}150}{12} = 10{,}250 \text{ ft} \cdot \text{kips}$$

Intermediate Transverse Stiffeners. For the web, the depth–thickness ratio $d/t = 106/(^{11}/_{16}) = 154$. This exceeds the AREMA limit of 60 for an unstiffened web. Transverse stiffeners are required and spacing should not exceed $d = 332t/\sqrt{f_v} \le 72$ in, where f_v is the shear stress, ksi.

$$f_v = \frac{498}{72.9} = 6.83 \text{ ksi}$$

For this shear, $d = 332(^{11}/_{16})/\sqrt{6.83} = 87 \text{ in} > 72$ in. Use a stiffener spacing of 60 in. Try a pair of plates at each location, with the width equal at least to $D/30 + 2 = {}^{106}/_{30} + 2 = 5.5 \text{ in} < 16$ in. Use 6 × $^3/_8$-in plates welded to the web for the intermediate stiffeners.

Change in Flange Size. At a sufficient distance from mid-span, the bending moment decreases sufficiently to permit reducing the thickness of the flange plates to $1^3/_4$ in. The net moment of inertia reduces to 265,000 in^4 and the section modulus to 4840 in^3. Thus, with 20 × $1^3/_4$-in flange plates, the section has a moment capacity of

$$M_C = \frac{20 \times 4840}{12} = 8070 \text{ ft} \cdot \text{kips}$$

When this is plotted in Fig. 12.38, the horizontal line representing it stays above the moment envelope until within 20 ft of mid-span. Hence, flange size can be decreased at that point. Length of the $2^3/_8$-in plate then is 40 ft and of the $1^3/_4$-in plates, which extend to the end of the girder, 23 ft.

The flange plates will be spliced with complete-penetration groove welds. For calculation of fatigue stresses, the welded connection is Stress Category B′ and for this span of less than 100 ft should be designed for 2,000,000 cycles of loading. The allowable stress range is 14.5 ksi. The actual stress range for live loads plus impact is estimated to be

$$f_r = \frac{5200 \times 12}{4840} = 12.9 \text{ ksi} < 14.5 \text{ ksi} \qquad \text{OK}$$

Flange-to-Web Welds. AREMA specifications require that the flange plates be connected to the web with continuous, full-penetration groove welds.

Knee Braces. A knee brace with solid web (Fig. 12.40*a*) braces the compression flange of the girder at 10-ft intervals. Attached with bolts to the top of the floorbeam and welded to a girder stiffener, the

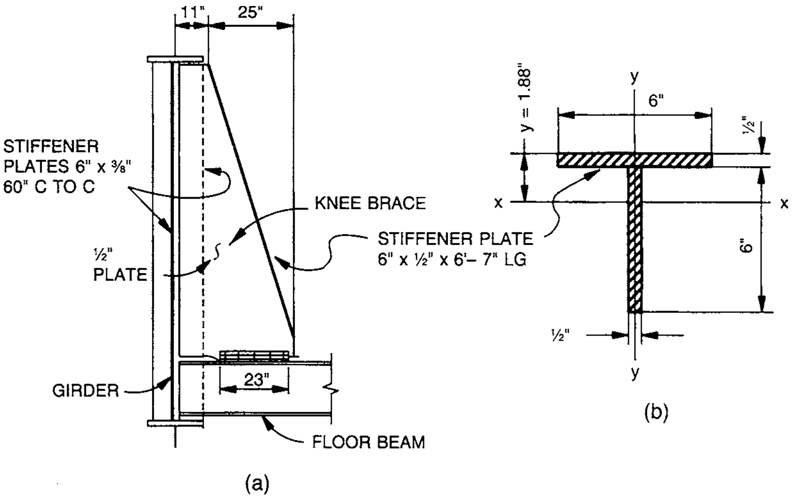


FIGURE 12.40 Knee brace for compression flange of through girder. (*a*) Elevation. (*b*) Cross section assumed effective.

brace extends from the floorbeam to the top flange of the girder, and from the web of the girder outward a maximum of 36 in. The outer edge is cut to a slope of 3-on-1. (Some railroads prefer a maximum slope of 2.8-on-1.) The length of this edge is 75 in.

Assume a $^1/_2$-in-thick plate for the web. Since the 75-in length of the edge exceeds $60 \times {}^1/_2 = 30$ in, the edge is stiffened with a $6 \times {}^1/_2$-in plate. This plate is considered to act with 6 in of the web in transmitting the buckling force to the floorbeam (Fig. 12.40*b*). This force is assumed horizontal and equal to 2.5% of the force in the $20 \times 2^3/_8$-in compression flange. With a compressive stress in the flange of 19.0 ksi, the force to be resisted is

$$F = 0.025 \times 20 \times 2.375 \times 19.0 = 23 \text{ kips}$$

The T section in Fig. 12.40*b* therefore is subjected, because of the 3-on-1 slope, to a force

$$P = 23 \times {}^{79}/_{25} = 72.7 \text{ kips}$$

Area of the T is $2 \times 6 \times {}^1/_2 = 6 \text{ in}^2$. Distance of the neutral axis from the outer surface of the flange is

$$y = \frac{3 \times 0.25 + 3 \times 3.5}{6} = 1.88 \text{ in}$$

Moments of inertia are computed to be $I_x = 24.83 \text{ in}^4$ and $I_y = 9.00 \text{ in}^4$. The latter governs. Thus, the least radius of gyration is

$$r_y = \sqrt{\frac{9.0}{6}} = 1.227 \text{ in}$$

The slenderness ratio then is 79/1.227 = 65. Hence, treated as a column, the T section has an allowable compressive stress of

$$F_a = 21.5 - \frac{0.1kL}{r} = 21.5 - 0.1 \times 65 = 15.0 \text{ ksi}$$

And the brace has a capacity of

$$P = 15.0 \times 6 = 90.0 > 72.7 \text{ kips}$$

Therefore, the knee brace is satisfactory.

The number of $^7/_8$-in-diameter high-strength bolts required to transmit the 23-kip horizontal force to the floorbeam, with a capacity of 9.3 kips per bolt, is 23/(2 × 9.3) = 2. For sealing, however, the maximum bolt spacing is $4 + 4t = 4 + 4 \times {}^1/_2 = 6$ in. Sealing controls. Use five $^7/_8$-in bolts 5 in c to c and two angles $4 \times 4 \times {}^1/_2$ in by 23 in long.

Other Details. Stiffeners are designed and located in the same way as for deck plate girders (Art. 12.8). They should be placed at floorbeams, but need not be at every beam. Other details also are treated in the same way as for plate girders.

12.11 COMPOSITE BOX-GIRDER BRIDGES

Box girders have several favorable characteristics that make their use desirable for spans of about 120 ft and up. Structural steel is employed at high efficiency, because a high percentage can be placed in wide flanges where the metal is very effective in resisting bending. Corrosion resistance is higher than in plate-girder and rolled-beam bridges. For, with more than half the steel surface inside the box, less steel, especially corners, which are highly susceptible, is exposed to corrosive influences. Also, the box shape is more effective in resisting torsion than the I shape used for plate girders and rolled beams. In addition, box girders offer an attractive appearance.

The high torsional rigidity of box girders makes this type of construction preferable for bridges with curved girders. Also, the high rigidity assists the deck in distributing loads transversely. This is illustrated in Fig. 12.41. A single load placed off-center on a bridge with single-web girders is carried mainly by nearby girders. But similarly placed on a box-girder bridge, the load is supported nearly equally by all the girders. The effect of the deck is ignored in this illustration.

Depending on its width, a bridge may be supported on one or more box girders. Each girder may comprise one or more cells. For economy in long-span construction, the cells may be made wide and deep. Width, for example, may be 12 ft or more. Usual thickness of the concrete deck, however,

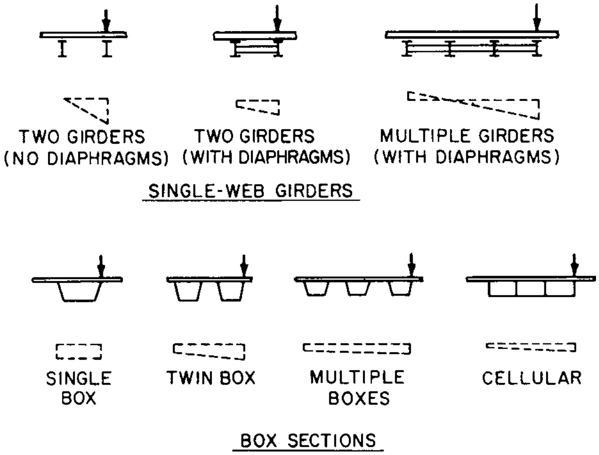


FIGURE 12.41 Comparison of lateral load distribution for single-web girders and box girders.

generally limits spacing of the girder webs to about 10 ft and cantilevers to about 5 ft. Consequently, thicker slabs are justified to take advantage of the economies accruing from wider girder cells.

Some designers have found it advantageous to use an alternative scheme with narrow box girders. They place a pair of boxes near the roadway edges and distribute the loads to these girders through longitudinal stringers and transverse floorbeams, as is done in plate-girder construction (Art. 12.8).

Box girders may be simply supported or continuous. Since they generally are used principally in long spans, continuity is highly desirable for economy and increased stiffness. Also, use of high-strength steels is advantageous in the longer spans.

Box girders are adaptable to composite and orthotropic-plate construction. With composite contraction, only a narrow top flange is needed with each web. The flanges usually need be only wide enough for load distribution to the web and to provide required clearances and edge distances for welded shear connectors. Figures 12.42 and 12.43 show several types of box-girder bridges that have been constructed with and without composite construction.

Boxes may be rectangular or trapezoidal. (Triangular boxes with apex down have been used, but they have several disadvantages. They usually have to be deeper than rectangular or trapezoidal boxes. Also, because of smaller area, triangular boxes have less torsional resistance. Furthermore,

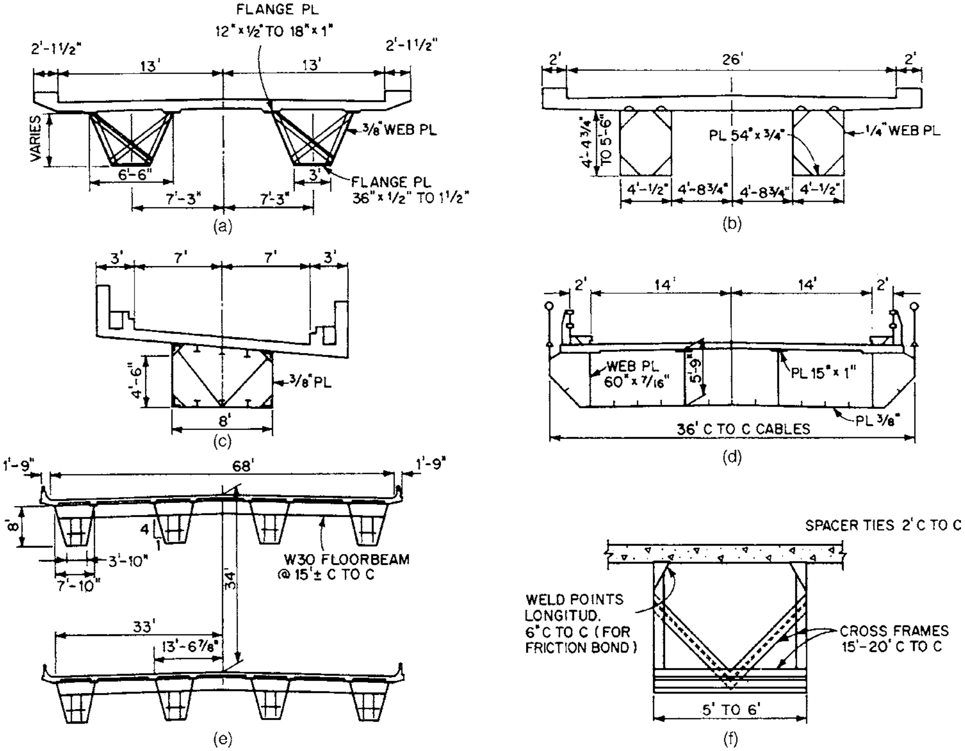


FIGURE 12.42 Examples of cross sections of composite box-girder highway bridges. (*a*) Rigid-frame construction with inclined legs, over Stillaguamish River. Spans are 50–160–85 ft and 200 ft c to c of leg pins. (*b*) Boxes with corners trussed for rigidity, in 110-ft span. King County, Wash. (*c*) Ramp with minimum horizontal radius of 67 ft and continuous spans of 58.5–52–73 ft, in Port Authority of New York and New Jersey Bus Terminal. (*d*) Suspension-bridge spans of 170–430–170 ft over Klamath River, Orleans, Calif. (*e*) Double-deck approach spans of 170–170 ft, Fremont Bridge, Portland, Ore. (*f*) Box UV girder proposed by Homer Hadley. V-shaped troughs formed by corner plates atop the webs are filled with concrete to secure composite action with concrete deck.

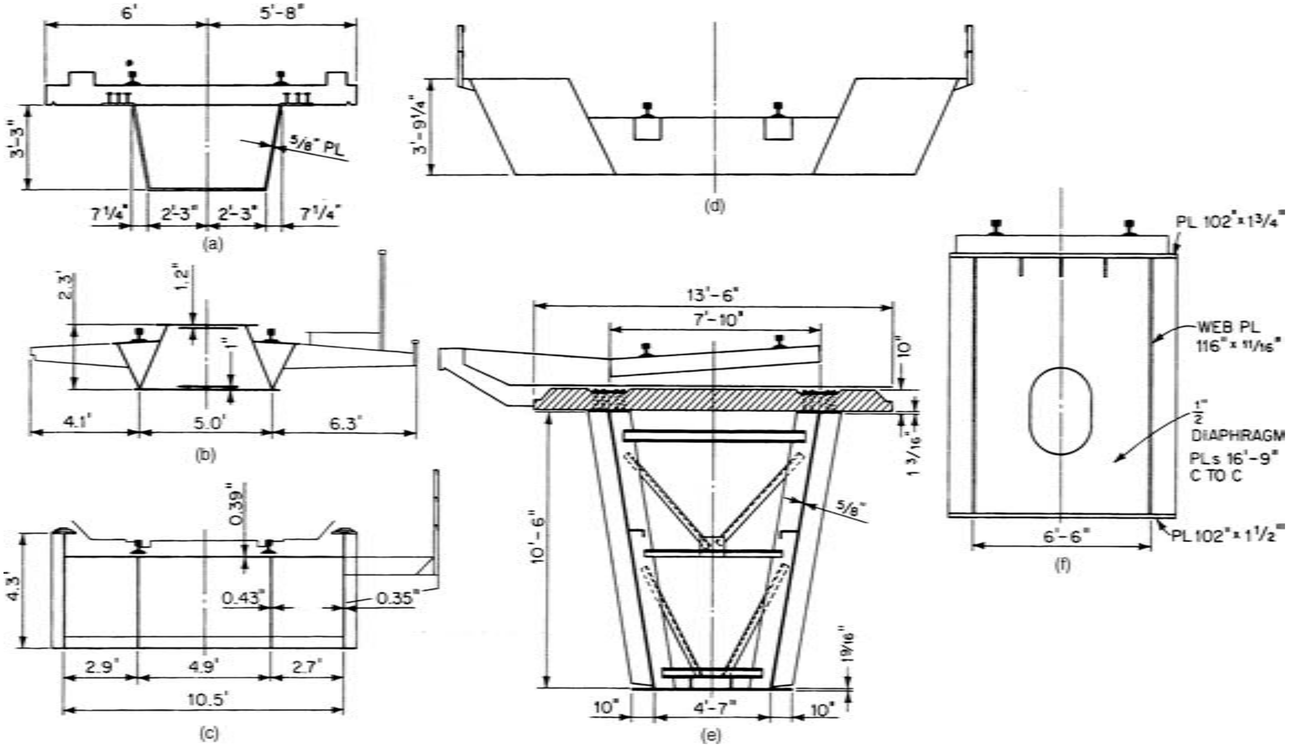


FIGURE 12.43 Examples of cross sections of railroad box-girder bridges. (*a*) Composite girder with 85-ft span for elevated rapid-transit system, San Francisco, Calif. (*b*) Rails bear directly on steel box of 56-ft span over Autobahn, Kirchweyhe, Bremen, Germany. (*c*) Three-cell box over Alsstrasse, M-Gladbach, Germany, with 80-ft span. (*d*) Rigid-frame construction with 117-ft span carries track with radius of 1780 ft, near Frankfurt, Germany. (*e*) Precast-concrete deck bolted to girder for composite action in 152.5-ft span, Czech Republic. (*f*) Typical section of four 122-ft spans in Chester, Pa.

the bottom flange often has to be a heavy built-up section, complicated by bent plates for connecting to the webs.) With trapezoidal boxes, fewer girders may be required, but a thicker bottom plate or thicker concrete slab may be needed than for rectangular boxes. Fabrication costs for either shape are about the same.

Construction costs for box-girder bridges often are kept down by shop fabrication of the boxes. Thus, designers should bear in mind the limitations placed by shipping clearances on the width of box girders as well as on length and depths. If the girders are to be transported by highway, and single box girders with widths exceeding about 12 ft are required, use of more but narrower girders may be more economical.

12.12 EXAMPLE—ALLOWABLE STRESS DESIGN OF COMPOSITE BOX-GIRDER BRIDGE

Following is an example to indicate the design procedure for a bridge with box girders composite with a concrete deck. The procedure does not differ greatly from that for a single-web plate girder with composite deck. The example incorporates the major differences.

A two-lane highway bridge with simply supported, composite box girders will be designed. The deck is carried by two trapezoidal girders (Fig. 12.44). Top width of each box is 8 ft 6 in, as is the distance c to c of adjacent top flanges of the girders. Bottom width is 5 ft 10 in. Thus, the webs have a slope of 4-on-1. The girders span 120 ft. Structural steel to be used is Grade 36. Loading is HS20-44. Appropriate design criteria given in Chap. 10 will be used for this structure.

Concrete Slabs. The general design procedure outlined in Art. 12.2 for slabs on rolled beams also holds for slabs on box girders. A 7.5-in-thick concrete slab will be used with the box girders.

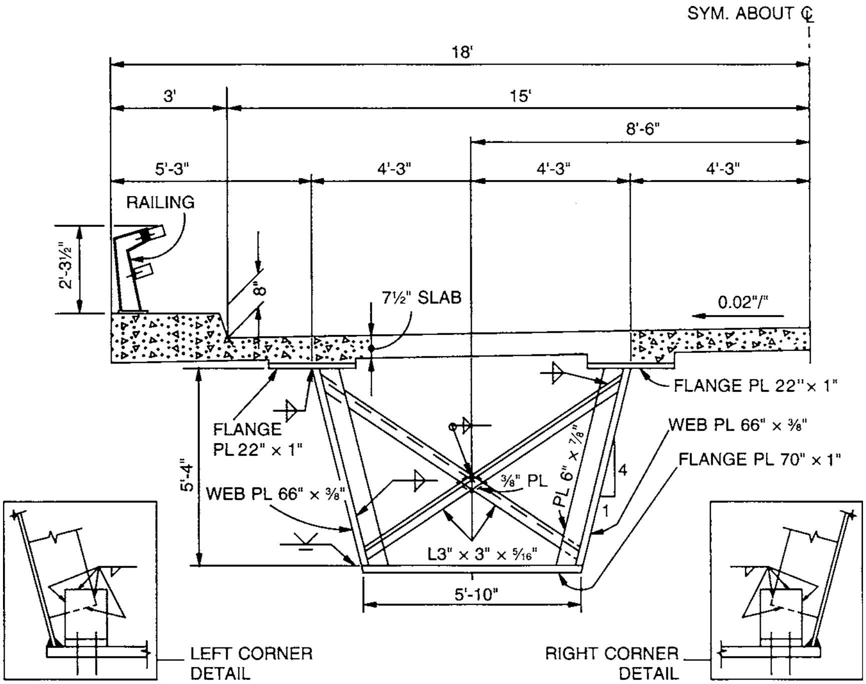


FIGURE 12.44 Half cross section of composite box girder with 120-ft span.

TABLE 12.55 Dead Load on Box Girder, kips/ft

(*a*) Dead load on steel box		(*b*) Dead load on composite section	
Slab: 0.150 × 18 × 7.5/12	= 1.69	Future overlay: 0.025 × 15	= 0.38
Haunches: 2 × 0.150 × 2 × $^1/_{12}$	= 0.05	Railing:	0.02
Girder and framing details—assume:	0.60	Safety walk: 0.150 × 3 × 8.38/12	= 0.31
DL per girder:	2.34	*SDL* per girder:	0.71

Design Criteria. AASHTO "Standard Specifications for Highway Bridges" apply to single-cell box girders where width c to c between top steel flanges is approximately equal to the distance c to c of adjacent top steel flanges of adjacent box girders. (The distance c to c of flanges of adjacent boxes should be between 0.8 and 1.2 times the distance c to c of the flanges of each box.) In this example, both the width and spacing equal 8 ft 6 in. Also, the deck overhang must not exceed 6 ft or 60% of the spacing. In this example, the overhang of 5.25 ft is nearly equal to 0.60 × 8.5 = 5.1 ft. Hence, AASHTO specifications for composite box girders may be used.

Loads, Moments, and Shears. Assume that the girders will not be shored during casting of the concrete slab. Hence, the dead load on each girder includes the weight of the 18-ft-wide half of the deck as well as weights of steel girders and framing details. This dead load will be referred to as *DL* (Table 12.55*a*). Maximum moment occurs at the center of the 120-ft span and equals

$$M_{DL} = \frac{2.34(120)^2}{8} = 4210 \text{ ft} \cdot \text{kips}$$

Maximum shear occurs at the supports and equals

$$V_{DL} = \frac{2.34 \times 120}{2} = 140.4 \text{ kips}$$

Railings and safety walks will be placed after the concrete slab has cured. This superimposed dead load will be designated *SDL* (Table 12.55*b*). Maximum moment occurs at mid-span and equals

$$M_{SDL} = \frac{0.71(120)^2}{8} = 1280 \text{ ft} \cdot \text{kips}$$

Maximum shear occurs at supports and equals

$$V_{SDL} = \frac{0.71 \times 120}{2} = 42.6 \text{ kips}$$

The HS20-44 live load imposed may be a truck load or lane load. But for this span, truck loading governs. The center of gravity of the three axles lies between the two heavier loads and is 4.66 ft from the center load. Maximum moment occurs under the center axle load when its distance from mid-span is the same as the distance of the center of gravity of the loads from mid-span, or 4.66/2 = 2.33 ft. Thus, the center load should be placed $^{120}/_2 - 2.33 = 57.67$ ft from a support. Then, the maximum moment is

$$M_T = \frac{72(120/2 + 2.33)^2}{120} - 32 \times 14 = 1880 \text{ ft} \cdot \text{kips}$$

Under AASHTO specifications, the live-load bending moment for each girder is determined by applying to the girder the fraction W_L of a wheel load (both front and rear) as given by

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} \tag{12.43}$$

where $R = N_w/N$, with $0.5 \le R \le 1.5$
$N_w = W_c/12$, reduced to nearest whole number
N = number of box girders
W_c = roadway width, ft, between curbs or between barriers if curbs are not used

In this example, $W_c = 30$, $N_w = {}^{30}/_{12} = 2$, $N = 2$, and $R = {}^{2}/_{2} = 1$. Therefore,

$$W_L = 0.1 + 1.7 \times 1 + \frac{0.85}{2} = 2.225 \text{ wheels} = 1.113 \text{ axles}$$

AASHTO standard specifications do not allow reduction of load intensity where W_L is obtained using the preceding equation. Therefore, the maximum live-load moment is

$$M_{LL} = 1.113 \times 1880 = 2100 \text{ ft} \cdot \text{kips}$$

Though this moment does not occur at mid-span as do the maximum dead-load moments, stresses due to M_{LL} may be combined with those from M_{DL} and M_{SDL} to produce the maximum stress, for all practical purposes.

For maximum shear with the truck load, the outer 32-kip load should be placed at the support. Then, the shear is

$$V_T = \frac{72(120 - 14 + 4.66)}{120} = 66.4 \text{ kips}$$

On the assumption that the live-load distribution is the same as for bending moment, the maximum live-load shear is

$$V_{LL} = 1.113 \times 66.4 = 73.8 \text{ kips}$$

Impact is taken as the following fraction of live-load stress:

$$I = \frac{50}{L + 125} = \frac{50}{120 + 125} = 0.204$$

Hence, the maximum moment due to impact is

$$M_1 = 0.204 \times 2100 = 430 \text{ ft} \cdot \text{kips}$$

and the maximum shear due to impact is

$$V_1 = 0.204 \times 73.8 = 15.1 \text{ kips}$$

Mid-span bending moments, ft·kips			End shear, kips			
M_{DL}	M_{SDL}	$M_{LL} + M_I$	V_{DL}	V_{SDL}	$V_{LL} + V_I$	Total V
4210	1280	2530	140.4	42.6	88.9	271.9

Properties of Composite Section. The 7.5-in-thick roadway slab includes an allowance of 0.5 in for a wearing surface. Hence, the effective thickness of the concrete slab for composite action is 7 in. Half the width of the deck, 18 ft = 216 in, is considered to participate in the composite action with each box girder.

A trial section for a girder is assumed as shown in Fig. 12.45. Its neutral axis can be located by taking moments of web and flange areas about a horizontal axis at mid-depth of the web. This computation and those for the section moduli S_{st} and S_{sb} of the steel section alone are conveniently tabulated in Table 12.56. The moment of inertia of each inclined web I_x may be computed from

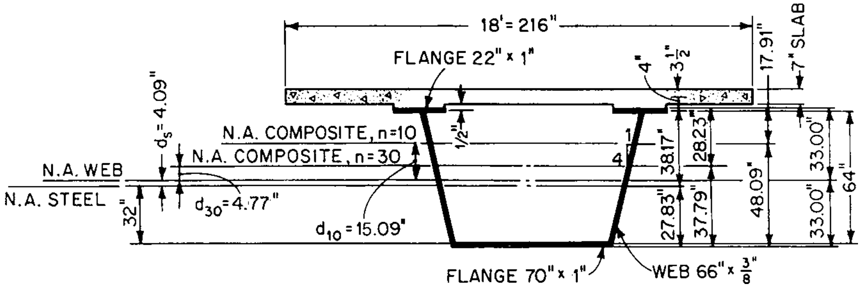


FIGURE 12.45 Locations of neutral axes of steel box alone and of composite box section.

$$I_x = \frac{s^2}{s^2+1} I \tag{12.44}$$

where s = slope of web with respect to horizontal axis
I = moment of inertia of web with respect to axis at middepth normal to the web = $ht^3/12$
h = depth of web in its plane
t = web thickness normal to its plane

In computation of the properties of the composite section, the concrete slab, ignoring the haunch area, is transformed into an equivalent steel area. For the purpose, for this bridge, the concrete area is divided by the modular ratio $n = 10$ for short-time loading, such as live loads and impact. For long-time loading, such as dead loads, the divisor is $3n = 30$, to account for the effects of creep. The computations of neutral-axis location and section moduli for the composite section are tabulated in Table 12.57. To locate the neutral axis, moments are taken about middepth of the girder webs.

Stress in Composite Section. Since the girders will not be shored when the concrete is cast and cured, the stresses in the steel section for load *DL* are determined with the section moduli of the steel section alone (Table 12.56). Stresses for load *SDL* are computed with the section moduli of the composite section when $n = 30$ (Table 12.57*a*), And stresses in the steel for live loads and impact are calculated with section moduli of the composite section when $n = 10$ (Table 12.57*b*). Calculations for the stresses are given in Table 12.58.

TABLE 12.56 Steel Section for Maximum Moment in Box Girder

Material	A	d	Ad	Ad^2	I_o	I
Two top flanges 22 × 1	44.0	32.50	1,430	46,500		46,500
Two webs 66 × $^3/_8$	49.5				16,900	16,900
Bottom flange 70 × 1	70.0	−32.50	−2,275	73,900		73,900
	163.5		−845			137,300
$d_s = -845/163.5 = -5.17$ in					−5.17 × 845 =	−4,300
						I_{NA} = 134,000

Distance from neutral axis of steel section to:

Top of steel = 32 + 1.00 + 5.17 = 38.17 in

Bottom of steel = 32 + 1.00 − 5.17 = 27.83 in

Section moduli

Top of steel	Bottom of steel
$S_{st} = 133{,}000/38.17 = 3{,}480 \text{ in}^3$	$S_{sb} = 133{,}000/27.83 = 4{,}780 \text{ in}^3$

TABLE 12.57 Composite Section for Maximum Moment in Box Girder

(a) For dead loads, $n = 30$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	163.5		−845			133,000
Concrete 216 × 7/30	50.4	37.0	1,865	69,000	200	69,200
	213.9		1,020			202,200
$d_{30} = 1{,}020/213.9 = 4.77$ in						$-4.77 \times 1{,}020 = -4{,}900$
						$I_{NA} = 197{,}300$

Distance from neutral axis of composite section to:

Top of steel = 33.00 − 4.77 = 28.23 in

Bottom of steel = 33.00 + 4.77 = 37.77 in

Top of concrete = 28.23 + 0.50 + 7 = 35.73 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 197{,}300/28.23$ $= 7{,}000\ \text{in}^3$	$S_{sb} = 197{,}300/37.77$ $= 5{,}220\ \text{in}^3$	$S_c = 197{,}300/35.73$ $= 5{,}520\ \text{in}^3$

(b) For live loads, $n = 10$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	163.5		−845			133,000
Concrete 216 × 7/10	151.2	37.0	5,594	207,000	600	207,600
	314.7		4,749			340,600
$d_{10} = 4{,}749/314.7 = 15.09$ in						$-15.09 \times 4{,}749 = -71{,}600$
						$I_{NA} = 269{,}000$

Distance from neutral axis of composite section to:

Top of steel = 33.00 − 15.09 = 17.91 in

Bottom of steel = 33.00 + 15.09 = 48.09 in

Top of concrete = 17.91 + 0.50 + 7 = 25.41 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 269{,}000/17.91$ $= 15{,}000\ \text{in}^3$	$S_{sb} = 269{,}000/48.09$ $= 5{,}600\ \text{in}^3$	$S_c = 269{,}000/25.41$ $= 10{,}600\ \text{in}^3$

TABLE 12.58 Stresses in Composite Box Girder, ksi

(a) Steel stresses

Top of steel (compression)		Bottom of steel (tension)	
DL: $f_b = 4{,}210 \times 12/3{,}580$	= 14.52	$f_b = 4{,}210 \times 12/4{,}780 =$	10.57
SDL: $f_b = 1{,}280 \times 12/7{,}000$	= 2.19	$f_b = 1{,}280 \times 12/5{,}220 =$	2.94
$LL + I$: $f_b = 2{,}530 \times 12/15{,}000$	= 2.02	$f_b = 2{,}530 \times 12/5{,}600 =$	5.42
Total:	18.73 < 20		18.93 < 20

(b) Stresses at top of concrete

SDL: $f_c = 1{,}280 \times 12/(5{,}520 \times 30)$	= 0.09
$LL + I$: $f_c = 2{,}530 \times 12/(10{,}600 \times 10)$	= 0.29
Total:	0.38 < 1.6

The width–thickness ratio of the unstiffened compression flanges now can be checked, as for an I shape, by the general formula applicable for any stress level:

$$\frac{b}{t} = \frac{194}{\sqrt{F_y}} = \frac{194}{\sqrt{36}} = 32 > \frac{22}{1}$$

Hence, the trial section is satisfactory.

Stresses in the concrete are determined with the section moduli for the composite section with $n = 30$ for *SDL* (Table 12.57*a*) and $n = 10$ for $LL + I$ (Table 12.57*b*). Since the inclination of web plates to a plane normal to the bottom flange is not greater than 1-to-4, and the width of the bottom flange is not greater than 20% of the span [70 in $< 0.20\,(120)\,(12) = 288$ in], secondary stresses (transverse bending stresses) resulting from distortion of the span, and from distortion of the girder cross section, and from vibrations of the bottom plate need not be considered. Therefore, the composite section is satisfactory. With the thickness specified for maximum moment, no changes in flange thicknesses are desirable. Use the section shown in Fig. 12.45 throughout the span.

Check of Web. The 64-in vertical projection of the webs satisfies the requirements that the depth–span ratio for girder plus slab exceed 1:25 and for girder alone 1:30. The depth–thickness ratio of each web is 66/0.375 = 176. This is close enough to the AASHTO specifications limiting requirement of $D/t \leq 170$ to be acceptable without longitudinal stiffeners. For the maximum compressive bending stress of 18.73 ksi, the maximum depth–thickness ratio permitted with transverse stiffeners but without longitudinal stiffeners is

$$\frac{D}{t} = \frac{727}{\sqrt{f_b}} = \frac{727}{\sqrt{18.73}} = 168$$

The design shear for the inclined web V_w equals the vertical shear V_v divided by the cosine of the angle of inclination θ of the web plate to the vertical. For a maximum shear of 271.9 kips and a slope of 4 on 1 ($\cos\theta = 0.97$), the design shear is

$$V_w = \frac{V_v}{\cos\theta} = \frac{271.9}{0.97} = 280 \text{ kips}$$

With a cross-sectional area of 49.5 in^2, the web will be subjected to shearing stress considerably below the 12 ksi permitted.

$$f_v = \frac{280}{49.5} = 5.7 < 12 \text{ ksi}$$

Maximum shears at sections along the span are given in Table 12.59.

Flange-to-Web Welds. Fillet welds placed on opposite sides of each girder web to connect it to each flange must resist the horizontal shear between flange and web. In this example, as is usually the case, the minimum size of weld permissible for the thickest plate at the connection determines the size of weld. For both the 1-in bottom flange and the 1-in top flanges, the minimum size of weld permitted is $^5/_{16}$ in. Therefore, use a $^5/_{16}$-in fillet weld on opposite sides of each web at each flange.

Intermediate Transverse Stiffeners. To determine if transverse stiffeners are required, the allowable shear stress F_v will be computed and compared with the average shear stress $f_v = 5.03$ ksi at the support.

TABLE 12.59 Maximum Shear in Composite Box Girder

	Distance from support, ft						
	0	10	20	30	40	50	60
DL, kips	183	153	124	93	62	31	0
LL + *I*, kips	89	81	73	65	57	49	41
Total, kips	272	234	197	158	119	80	41
f_v, ksi	5.49	4.73	3.98	3.19	2.40	1.62	0.83

$$F_v = \left(\frac{270}{D/t}\right)^2 = \left(\frac{270}{168}\right)^2 = 2.60 \text{ ksi} < 5.03 \text{ ksi}$$

Therefore, transverse intermediate stiffeners are required.

Maximum spacing of stiffeners may not exceed 3 × 64 = 192 in or $D[260/(D/t)]^2 = 64(260/168)^2 =$ 153 in. Try a stiffener spacing $d_o = 90$ in. This provides a depth–spacing ratio $D/d_o = 64/90 = 0.711$. From Eq. (10.24d), for use in Eq. (10.25a), $k = 5[1 + (0.711)^2] = 7.53$ and $\sqrt{k/F_y} = \sqrt{7.53/36} = 0.457$. Since $D/t = 168$, C in Eq. (10.25a) is determined by the parameter 168/0.457 = 368 > 237. Hence, C is given by

$$C = \frac{45{,}000k}{(D/t)^2 F_y} = \frac{45{,}000 \times 7.53}{168^2 \times 36} = 0.333$$

From Eq. (10.25a), the maximum allowable shear for $d_o = 90$ in is

$$F_v' = \frac{F_y}{3}\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]$$

$$= \frac{36}{3}\left[0.333 + \frac{0.87(1-0.333)}{\sqrt{1+(90/64)^2}}\right] = 8.03 \text{ ksi} > 5.03 \text{ ksi}$$

Since the allowable stress is larger than the computed stress, the stiffeners may be spaced 90 in apart.

The AASHTO standard specifications limit the spacing of the first intermediate stiffener to the smaller of $1.5D = 1.5 \times 64 = 96$ in and the spacing for which the allowable shear stress in the end panel does not exceed

$$F_v = CF_y/3 = 0.333 \times 36/3 = 4.0 \text{ ksi} < 5.03 \text{ ksi}$$

Therefore, closer spacing is needed near the supports. Try $d_o = 45$ in, for which $k = 15.08$, $C = 0.667$, and $F_v = CF_y/3 = 0.667 \times {}^{36}\!/_3 = 8.0$ ksi > 5.03. Therefore, 45-in spacing will be used near the supports and 90-in spacing in the next 22.5 ft of girder, as shown in Fig. 12.46. Transverse stiffeners are omitted from the central 60 ft of girder, except at mid-span.

Where required, a single plate stiffener of Grade 36 steel will be welded inside the box girder to each web. Minimum width of stiffeners is one-fourth the flange width, or 21/4 = 5.25 > 2 + 66/30 = 4.2 in. Use a 6-in-wide plate. Minimum thickness required is 6/16 = $^3/_8$ in. Try 6 × $^3/_8$-in stiffeners.

The moment of inertia provided by each stiffener must satisfy Eq. (10.21), with J as given by Eq. (10.22).

$$J = 2.5\left(\frac{64}{90}\right)^2 - 2 = -0.73 \qquad \text{Use } 0.5$$

$$I = 90(^3/_8)^3 0.5 = 2.37$$

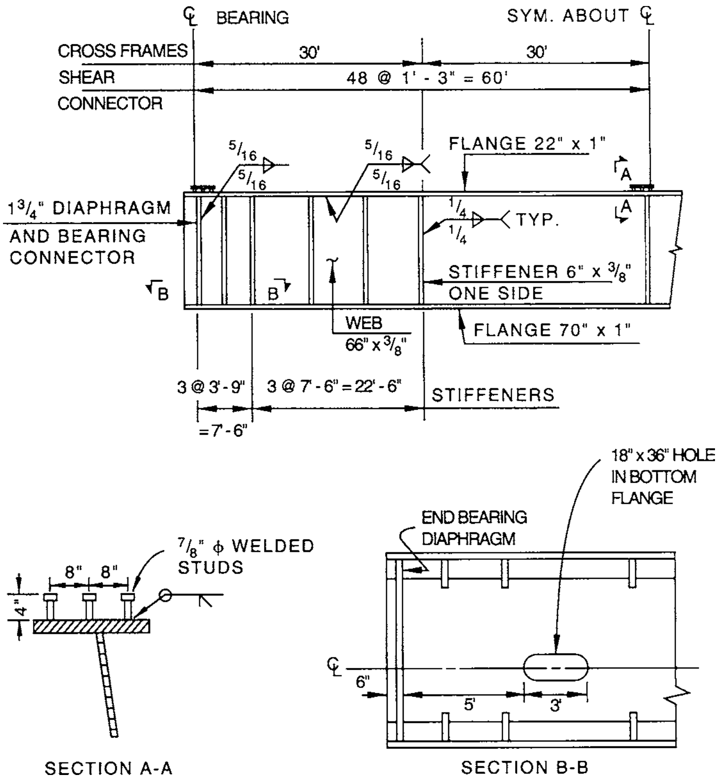


FIGURE 12.46 Locations of stiffeners, cross frames, and shear connectors for composite box girder.

The moment of inertia furnished is

$$I = \frac{(^3/_8)6^3}{3} = 27 > 2.37 \text{ in}^4$$

Hence, the $6 \times {}^3/_8$-in stiffeners are satisfactory. Weld them to the webs with a pair of ${}^1/_4$-in fillet welds.

Bearing Stiffeners. Instead of narrow-plate stiffeners and a cross frame over the bearings, a plate diaphragm extending between the webs is specified. The plate diaphragm has superior resistance to rotation, displacement, and distortion of the box girder. Assume for the diaphragm a bearing length of 20 in at each web, or a total of 40 in. The allowable bearing stress is 29 ksi. Then, the thickness required for bearing is

$$t = \frac{271.9}{40 \times 29} = 0.23 \text{ in}$$

But the thickness of a bearing stiffener also is required to be at least

$$t = \frac{b'}{12}\sqrt{\frac{F_y}{33}} = \frac{20}{12}\sqrt{\frac{36}{33}} = 1.74 \text{ in}$$

Therefore, use a plate 64 × 1³/₄ in extending between the webs at the supports, with a 30-in-square access hole.

The welds to the webs must be capable of developing the entire 271.9-kip reaction. Minimum-size fillet weld for the 1³/₄-in diaphragm is ⁵/₁₆ in. With two such welds at each web, their required length, with an allowable stress of 15.7 ksi, is

$$\frac{271.9}{4(^5/_{16})0.707 \times 15.7} = 19.6 \text{ in}$$

Weld the full 66-in depth of web.

Shear Connectors. To ensure composite action of concrete deck and box girders, shear connectors welded to the top flanges of the girders must be embedded in the concrete (Art. 11.16). For this structure, ⁷/₈-in-diameter welded studs are selected. They are to be installed in groups of three at specified locations to resist the horizontal shear between the steel section and the concrete slab (Fig. 12.46). With height $H = 4$ in, they satisfy the requirement $H/d \geq 4$, where d = stud diameter, in.

With $f'_c = 2800$ psi for the concrete, the ultimate strength of a ⁷/₈-in welded stud is, from Eq. (12.26),

$$S_u = 0.4d^2\sqrt{f'_c E_c} = 0.4(^7/_8)^2\sqrt{2.8 \times 2900} = 27.6 \text{ kips}$$

This value is needed for determining the number of shear connectors required to develop the strength of the steel girder or the concrete slab, whichever is smaller. With an area $A_s = 163.5$ in^2, the strength of the girder is

$$P_1 = A_s F_y = 163.5 \times 36 = 5890 \text{ kips}$$

The compressive strength of the concrete slab is

$$P_2 = 0.85 f'_c bt = 0.85 \times 2.8 \times 216 \times 7 = 3600 < 5890 \text{ kips}$$

Concrete strength governs. Hence, from Eq. (12.25), the number of studs provided between mid-span and each support must be at least

$$N_1 = \frac{P_1}{\phi S_u} = \frac{3600}{0.85 \times 27.6} = 153$$

With the studs placed in groups of three on each top flange, there should be at least 153/6 = 26 groups on each half of the girder.

Pitch is determined by fatigue requirements. The allowable load range, kips per stud, may be computed from Eq. (12.4). With $\alpha = 10.6$ for 500,000 cycles of load (AASHTO specifications),

$$Z_r = 10.6(0.875)^2 = 8.12 \text{ kips/stud}$$

At the supports, the shear range $V_r = 89$ kips, the shear produced by live load plus impact. Consequently, with $n = 10$ for the concrete, and the transformed concrete area equal to 151.2 in^2, and $I = 269{,}000$ in^4 from Table 12.57*b*, the range of horizontal shear stress is

$$S_r = \frac{V_r Q}{I} = \frac{89 \times 151.2 \times 21.91}{269{,}000} = 1.10 \text{ kips/in}$$

Hence, the pitch required for stud groups near the supports is

$$p = \frac{6Z_r}{S_r} = \frac{6 \times 8.12}{1.10} = 44 \text{ in}$$

Use a pitch of 15 in to satisfy both this requirement and that for 26 groups of studs between mid-span and each support (Fig. 12.46).

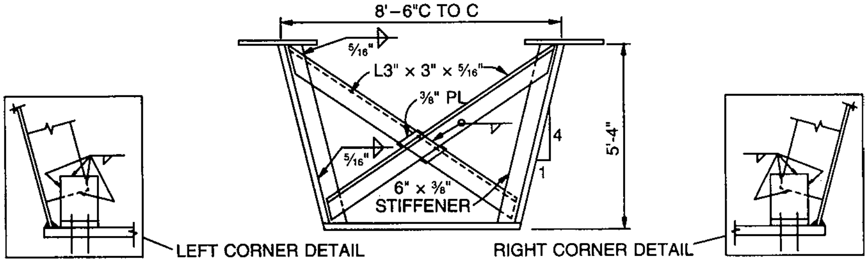


FIGURE 12.47 Intermediate cross frame.

Intermediate Cross Frames. Though intermediate cross frames or diaphragms are not required by standard specifications, it is considered good practice by many designers to specify such interior bracing in box girders to help maintain the shape under torsional loading. So, in addition to the end bearing diaphragm, cross frames will be installed at 30-ft intervals. Minimum-size angles can be used (Fig. 12.47).

Camber. The girders should be cambered to compensate for dead-load deflections under *DL* and *SDL*. For computation of deflections for *DL*, the moment of inertia I of the steel section alone should be used. For *SDL*, I should apply to the composite section with $n = 30$ (Table 12.57*a*). Both deflections can be computed from Eq. (12.5) with $w_{DL} = 2.34$ kips/ft and $w_{SDL} = 0.33$ kip/ft.

DL: $\delta = 22.5 \times 2.34(120)^4/(29{,}000 \times 133{,}000) =$	2.83 in
SDL: $\delta = 22.5 \times 0.33(120)^4/(29{,}000 \times 197{,}300) =$	0.27 in
Total:	3.10 in

Live-Load Deflection. Maximum live-load deflection should be checked to ensure that it does not exceed 12*L*/800. This deflection may be obtained with acceptable accuracy from Eq. (12.6), with

$$P_T = 8 \times 1.113 + 0.204 \times 8 \times 1.113 = 10.73 \text{ kips}$$

From Table 12.57*b*, for $n = 10$, $I = 269{,}000$ in^4. Therefore,

$$\delta = \frac{324 \times 10.73}{29{,}000 \times 269{,}000}(120^3 - 555 \times 120 + 4780) = 0.74 \text{ in}$$

And the deflection–span ratio is

$$\frac{0.74}{120 \times 12} = \frac{1}{1940} < \frac{1}{800}$$

Thus, the live-load deflection is acceptable.

Other Details. These may be treated in the same way as for I-shaped plate girders.

12.13 CONTINUOUS-BEAM BRIDGES

Articles 12.1 and 12.3 recommended use of continuity for multispan bridges. Advantages over simply supported spans include less weight, greater stiffness, smaller deflections, and fewer bearings and expansion joints. Disadvantages include more complex fabrication and erection and often the costs of additional field splices.

Continuous structures also offer greater overload capacity. Failure does not necessarily occur if overloads cause yielding at one point in a span or at supports. Bending moments are redistributed to parts of the span that are not overstressed. This usually can take place in bridges because maximum positive moments and maximum negative moments occur with loads in different positions on the spans. Also, because of moment redistribution due to yielding, small settlements of supports have no significant effects on the ultimate strength of continuous spans. If, however, foundation conditions are such that large settlements could occur, simple-span construction is advisable.

While analysis of continuous structures is more complicated than that for simple spans, design differs in only a few respects. In simple spans, maximum dead-load moment occurs at mid-span and is positive. In continuous spans, however, maximum dead-load moment occurs at the supports and is negative. Decreasing rapidly with distance from the support, the negative moment becomes zero at an inflection point near a quarter point of the span. Between the two dead-load inflection points in each interior span, the dead-load moment is positive, with a maximum about half the negative moment at the supports.

As for simple spans, live loads are placed on continuous spans to create maximum stresses at each section. Whereas in simple spans maximum moments at each section are always positive, maximum live-load moments at a section in continuous spans may be positive or negative. Because of the stress reversal, fatigue stresses should be investigated, especially in the region of dead-load inflection points. At interior supports, however, design usually is governed by the maximum negative moment, and in the mid-span region, by maximum positive moment. The sum of the dead-load and live-load moments usually is greater at supports than at mid-span. Usually also, this maximum is considerably less than the maximum moment in a simple beam with the same span. Furthermore, the maximum negative moment decreases rapidly with distance from the support.

The impact fraction for continuous spans depends on the length L, ft, of the portion of the span loaded to produce maximum stress. For positive moment, use the actual loaded length. For negative moment, use the average of two adjacent loaded spans.

Ends of continuous beams usually are simply supported. Consequently, moments in three-span and four-span continuous beams are significantly affected by the relative lengths of interior and exterior spans. Selection of a suitable span ratio can nearly equalize maximum positive moments in those spans and thus permit duplication of sections. The most advantageous ratio, however, depends on the ratio of dead load to live load, which, in turn, is a function of span length. Approximately, the most advantageous ratio for length of interior to exterior span is 1.33 for interior spans less than about 60 ft, 1.30 for interior spans between about 60 to 110 ft, and about 1.25 for longer spans.

When composite construction is advantageous (see Art. 12.1), it may be used either in the positive-moment regions or throughout a continuous span. Design of a section in the positive-moment region in either case is similar to that for a simple beam. Design of a section in the negative-moment regions differs in that the concrete slab, as part of the top flange, cannot resist tension. Consequently, steel reinforcement must be added to the slab to resist the tensile stresses imposed by composite action.

Additionally, for continuous spans with a cast-in-place concrete deck, the sequence of concrete pavement is an important design consideration. Bending moments, bracing requirements, and uplift forces must be carefully evaluated.

12.14 ALLOWABLE STRESS DESIGN OF BRIDGE WITH CONTINUOUS, COMPOSITE STRINGERS

The structure is a two-lane highway bridge with overall length of 298 ft. Site conditions require a central span of 125 ft. End spans, therefore, are each 86.5 ft (Fig. 12.48*a*). The typical cross section in Fig. 12.48*b* shows a 30-ft roadway, flanked on one side by a 21-in-wide barrier curb and on the other by a 6-ft-wide sidewalk. The deck is supported by six rolled-beam, continuous stringers of Grade 36 steel. Concrete to be used for the deck is Class A, with 28-day strength $f'_c = 4000$ psi and

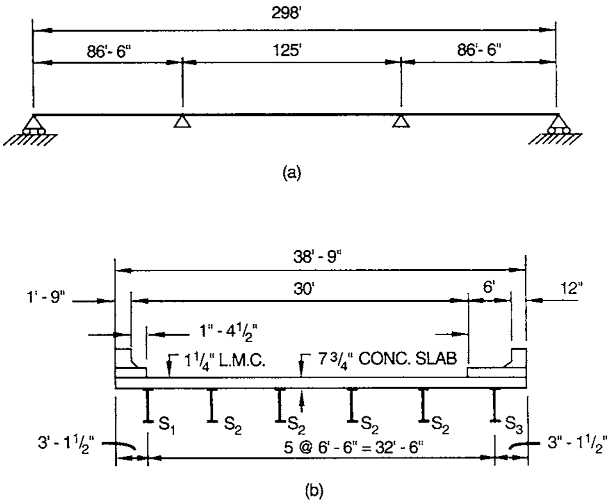


FIGURE 12.48 (*a*) Spans of a continuous highway bridge. (*b*) Typical cross section of bridge.

allowable compressive stress $f_c = 1600$ psi. Loading is HS20-44. Appropriate design criteria given in Chap. 10 will be used for this structure.

Concrete Slab. The slab is designed to span transversely between stringers, as in Art. 12.2. A 9-in-thick, two-course slab will be used. No provision will be made for a future 2-in wearing course.

Stringer Loads. Assume that the stringers will not be shored during casting of the concrete slab. Then, the dead load on each stringer includes the weight of a strip of concrete slab plus the weights of steel shape, cover plates, and framing details. This dead load will be referred to as *DL* and is summarized in Table 12.60.

Sidewalks, parapets, and barrier curbs will be placed after the concrete slab has cured. Their weights may be equally distributed to all stringers. Some designers, however, prefer to calculate the heavier load imposed on outer stringers by the cantilevers by taking moments of the cantilever loads about the edge of curb, as shown in Table 12.61. In addition, the six composite beams must carry the weight, 0.016 kip/ft^2, of the 30-ft-wide latex-modified concrete (LMC) wearing course. The total superimposed dead load will be designated *SDL*.

The HS20-44 live load imposed may be a truck load or lane load. For these spans, truck loading governs. With stringer spacing $S = 6.5$ ft, the live load taken by outer stringers S_1 and S_3 is

TABLE 12.60 Dead Load on Continuous Steel Beams, kips/ft

	Stringers S_1 and S_3	Stringers S_2
Slab	0.618	0.630
Haunch and SIP forms:	0.102	0.047
Rolled beam and details—assume:	0.320	0.320
DL per stringer	1.040	0.997

TABLE 12.61 Dead Load on Composite Stringers, kips/ft

	SDL	*x*	Moment
Barrier curb: 0.530/6	0.088	1.33	0.117
Sidewalk: 0.510/6	0.085	3.50	0.298
Parapet: 0.338/6	0.056	6.50	0.364
Railing: 0.015/6	0.002	6.50	0.013
	0.231		0.675
$1\frac{1}{4}$-in LMC course	0.078		
SDL for S_2:	0.309		

Eccentricity for $S_1 = 0.117/0.088 + 6.5 + 1.38 = 9.21$ ft
Eccentricity for $S_3 = 0.675/0.143 + 6.5 - 3.88 = 7.34$ ft
SDL for $S_1 = 0.309 \times 9.21/6.5 = 0.438$
SDL for $S_3 = 0.309 \times 7.34/6.5 = 0.349$

$$\frac{S}{4+0.25S} = \frac{6.5}{4+0.25\times 6.5} = 1.115 \text{ wheels} = 0.578 \text{ axle}$$

The live load taken by S_2 is

$$\frac{S}{5.5} = \frac{6.5}{5.5} = 1.182 \text{ wheels} = 0.591 \text{ axle}$$

Sidewalk live load (*SLL*) on each stringer is

$$w_{SLL} = \frac{0.060\times 6}{6} = 0.060 \text{ kips/ft}$$

The impact factor for positive moment in the 86.5-ft end spans is

$$I = \frac{50}{L+125} = \frac{50}{86.5+125} = 0.237$$

For positive moment in the 125-ft center span,

$$I = \frac{50}{125+125} = 0.200$$

And for negative moments at the interior supports, with an average loaded span $L = (86.5 + 125)/2 = 105.8$ ft,

$$I = \frac{50}{105.8+125} = 0.217$$

Stringer Moments. The steel stringers will each consist of a single rolled beam of Grade 36 steel, composite with the concrete slab only in regions of positive moment. To resist negative moments, top and bottom cover plates will be attached in the region of the interior supports. To resist maximum positive moments in the center span, a cover plate will be added to the bottom flange of the composite section. In the end spans, the composite section with the rolled beam alone must carry the positive moments.

For a precise determination of bending moments and shears, these variations in moments of inertia of the stringer cross sections should be taken into account. But this requires that the cross sections be known in advance or assumed, and the analysis without a computer is tedious. Instead, for a preliminary analysis, to determine the cross sections at critical points, the moment of inertia may

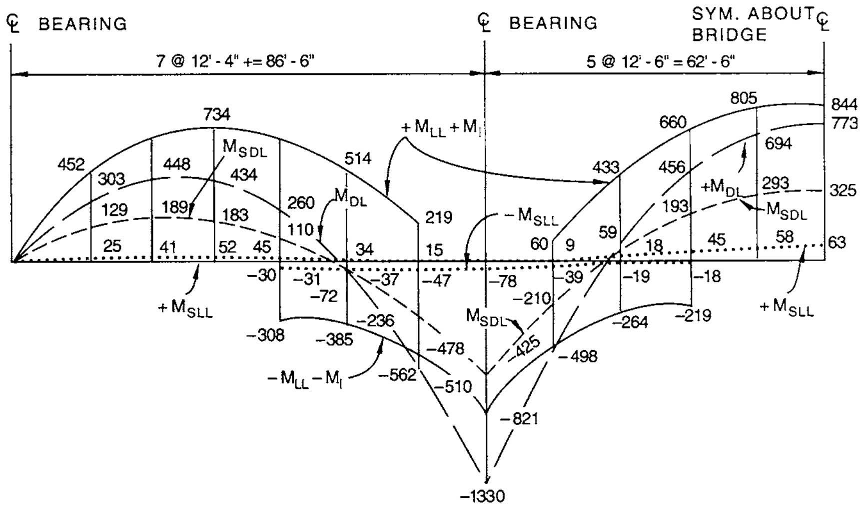


FIGURE 12.49 Maximum moments in outer stringer S_1.

be assumed constant and the same in each span. This assumption considerably simplifies the analysis and permits use of tables of influence coefficients. (See, for example, "Moments, Shears, and Reactions for Continuous Highway Bridges," *American Institute of Steel Construction.*) The resulting design also often is sufficiently accurate to serve as the final design. In this example, dead-load negative moment at the supports, computed for constant moment of inertia, will be increased 10% to compensate for the variations in moment of inertia.

Curves of maximum moment (moment envelopes) are plotted in Figs. 12.49 and 12.50 for S_1 and S_2, respectively. Because total maximum moments at critical points are nearly equal for S_1, S_2, and S_3, the design selected for S_1 will be used for all stringers. (In some cases, there may be some cost savings in using shorter cover plates for the stringers with smaller moments.)

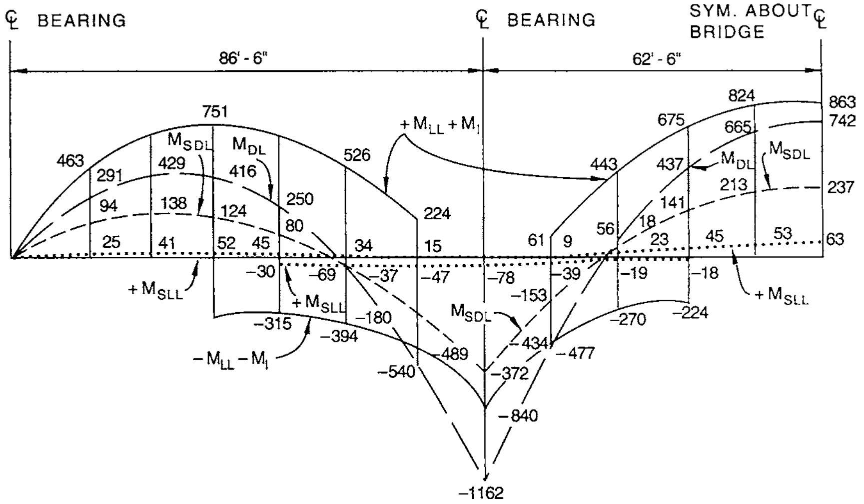


FIGURE 12.50 Maximum moments in interior stringer S_2.

Properties of Negative-Moment Section. The largest bending moment occurs at the interior supports, where the section consists of a rolled beam and top and bottom cover plates. With the dead load at the supports as indicated in Fig. 12.49 increased 10% to compensate for the variable moment of inertia, the moments in stringer S_1 at the supports are as follows:

S_1 MOMENTS AT INTERIOR SUPPORTS, FT · KIPS

M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_{SLL}	Total M
−1331	−510	−821	−78	−2740

For computing the minimum depth–span ratio, the distance between center-span inflection points can be taken approximately as 0.7 × 125 = 87.5 > 86.5 ft. In accordance with AASHTO specifications, the depth of the steel beam alone should be at least 87.5 × 12/30 = 35 in. Select a 36-in wide-flange beam. With an effective depth of 8.5 in for the concrete slab, allowing $^1/_2$ in for wear, overall depth of the composite section is 44.5 in. Required depth is 87.5 × 12/25 = 42 < 44.5 in.

With an allowable bending stress of 20 ksi, the cover-plated beam must provide a section modulus of at least

$$S = \frac{2740 \times 12}{20} = 1644 \text{ in}^3$$

Try a W36 × 280. It provides a moment of inertia of 18,900 in^4 and a section modulus of 1,030 in^3 with a depth of 36.50 in. The cover plates must increase this section modulus by at least 1644 − 1030 = 614 in^3. Hence, for an assumed distance between plates of 37 in, area of each plate should be about 614/37 = 16.6 in^2. Try top and bottom plates 14 × 1$^3/_8$ in (area = 19.25 in^2). The 16.6-in flange width provides at least 1 in on both sides of the cover plates for fillet-welding the plates to the flange.

The assumed section provides a moment of inertia of

$$I = 18{,}900 + 2 \times 19.25(18.94)^2 = 32{,}700 \text{ in}^4$$

Hence, the section modulus provided is

$$S = \frac{32{,}700}{19.63} = 1{,}666 > 1{,}644 \text{ in}^3$$

Use a W36 × 280 with top and bottom cover plates 14 × 1$^3/_8$ in. Weld plates to flanges with $^5/_{16}$-in fillet welds, minimum size permitted for the flange thickness.

Allowable Compressive Stress Near Supports. Because the bottom flange of the beam is in compression near the supports and is unbraced, the allowable compressive stress may have to be reduced to preclude buckling failure. AASHTO specifications, however, permit a 20% increase in the reduced stress for negative moments near interior supports. The unbraced length should be taken as the distance between diaphragms or the distance from interior support to the dead-load inflection point, whichever is smaller. In this example, if distance between diaphragms is assumed not to exceed about 22 ft, the allowable bending stress for a flange width of 16.6 in is computed as follows.

Allowable compressive stress F_b, ksi, on extreme fibers of rolled beams and built-up sections subject to bending, when the compression flange is partly supported, is determined from

$$F_b = \frac{50{,}000}{S_{xc}} C_b \left(\frac{I_{yc}}{l}\right) \sqrt{\frac{0.772J}{I_{yc}} + 9.87\left(\frac{d}{l}\right)^2} \leq 0.55F_y \tag{12.45}$$

where $C_b = 1.75 + 1.05(M_1/M_2) + 0.3\ (M_1/M_2)^2 \leq 2.3$
S_{xc} = section modulus with respect to the compression flange, in^3
= 1666 in^3
I_{yc} = moment of inertia of compression flange about vertical axis in plane of web, in^4
= 1.57 × 16.6^3/12 = 598 in^4

l = length of unsupported flange between lateral connections, knee braces, or other points of support, in

= 22 × 12 = 264 in

J = torsional constant, in^4

$= \frac{1}{3}(bt_{fc}^3 + bt_{ft}^3 + dt_w^3)$

$= \frac{1}{3}[16.6(1.57)^3 + 16.6(1.57)^3 + 36.52(0.89)^3] = 51 \text{ in}^4$

d = depth of girder, in = 36.52 in

M_1 = smaller end moment in the unbraced length of the stringer

= −121 − 52 − 394 − 38 = −605 ft·kip

M_2 = larger end moment in the unbraced length of the stringer

= −1331 − 510 − 821 − 78 = −2740 ft·kip

$C_b = 1.75 + 1.05(605/2740) + 0.3(605/2740)^2 = 2.00$

Substitution of the above values in Eq. (12.45) yields

$$F_b = \frac{50{,}000 \times 2.0}{1{,}666}\left(\frac{598}{264}\right)\sqrt{0.772\left(\frac{51}{598}\right) + 9.87\left(\frac{36.52}{264}\right)^2}$$

$$= 68.62 \text{ ksi} > (0.55 \times 36 = 19.8 \text{ ksi})$$

Use F_b = 19.8 ksi.

Cutoffs of Negative-Moment Cover Plates. Because of the decrease in moments with distance from an interior support, the top and bottom cover plates can be terminated where the rolled beam alone has sufficient capacity to carry the bending moment. The actual cutoff points, however, may be determined by allowable fatigue stresses for the base metal adjacent to the fillet welds between flanges and ends of the cover plates. The number of cycles of load to be resisted for HS20–44 loading is 500,000 for a major highway. For Grade 36 steel and these conditions, the allowable fatigue stress range for this redundant-load-path structure and the Stress Category E′ connection is F_r = 9.2 ksi.

Resisting moment of the W36 × 280 alone with F_r = 9.2 ksi is

$$M = \frac{9.2 \times 1030}{12} = 790 \text{ ft} \cdot \text{kips}$$

This equals the live-load bending-moment range in the end span about 12 ft from the interior support. Minimum terminal distance for the 14-in cover plate is 1.5 × 14 = 21 in. Try an actual cutoff point 14 ft 6 in from the support. At the theoretical cutoff point, the moment range is 219 − (−562) = 781 ft·kips. Thus, the stress range is

$$F_r = \frac{781 \times 12}{1030} = 9.1 \text{ ksi} < 9.2 \text{ ksi}$$

Fatigue does not govern. Use a cutoff 14 ft 6 in from the interior support in the end span.

In the center span, the resisting moment of the W36 equals the bending moment about 8 ft 4 in from the interior support. With allowance for the terminal distance, the plates may be cut off 10 ft 6 in from the support. Fatigue does not govern there.

Properties of End-Span Composite Section. The 9-in-thick roadway slab includes an allowance of 0.5 in for wear. Hence, the effective thickness of the concrete slab for composite action is 8.5 in.

The effective width of the slab as part of the top flange of the T beam is the smaller of the following:

$\frac{1}{4}$ span = $\frac{1}{4}$ × 86.5 × 12 = 260 in

Overhang + half the spacing of stringers = 37.5 + 78/2 = 76.5 in

12 × slab thickness = 12 × 8.5 = 102 in

Hence the effective width is 76.5 in (Fig. 12.51).

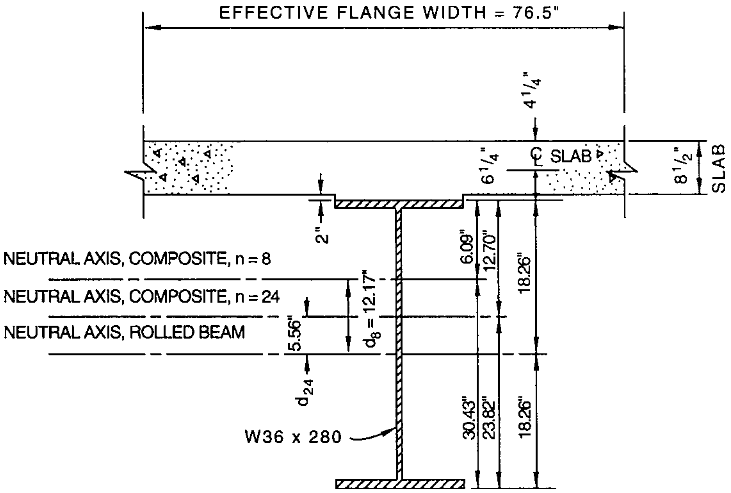


FIGURE 12.51 Composite section for end span of continuous girder.

To resist maximum positive moments in the end span, the W36 × 280 will be made composite with the concrete slab. As in Art. 12.2, the properties of the end-span composite section are computed with the concrete slab, ignoring the haunch area, transformed into an equivalent steel area. The computations for neutral-axis locations and section moduli for the composite section are tabulated in Table 12.62. To locate the neutral axes of $n = 24$ and $n = 8$, moments are taken about the neutral axis of the rolled beam.

Stresses in End-Span Composite Section. Since the stringers will not be shored when the concrete is cast and cured, the stresses in the steel section for load *DL* are determined with the section moduli of the steel section alone. Stresses for load *SDL* are computed with section moduli of the composite section when $n = 24$. And stresses in the steel for live loads and impact are calculated with section moduli of the composite section when $n = 8$. See Table 12.62. Maximum positive bending moments in the end span are estimated from Fig. 12.49:

Maximum Positive Moments
in End Span, ft · kips

M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_{SLL}
434	183	734	52

Stresses in the concrete are determined with the section moduli of the composite section with $n = 24$ for *SDL* from Table 12.62*a* and $n = 8$ for $LL + I$ from Table 12.62*b* (Table 12.63).

Since the bending stresses in steel and concrete are less than the allowable, the assumed steel section is satisfactory for the end span.

Properties of Center-Span Section for Maximum Positive Moment. For maximum positive moment in the middle portion of the center span, the rolled beam will be made composite with the concrete slab and a cover plate will be added to the bottom flange. Area of cover plate required A_{sb} will be estimated from Eq. (12.1*a*) with $d_{cg} = 35$ in and $t = 8.5$ in.

TABLE 12.62 End-Span Composite Section

(*a*) For dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	82.4				18,900	18,900
Concrete 76.5 × 7.75/24	24.7	24.14	596	14,400	120	14,520
	107.1		596			33,420
$d_{24} = 596/107.1 = 5.56$ in					$-5.56 \times 596 =$	$-3,320$
					$I_{NA} =$	30,100

Distance from neutral axis of composite section to:

Top of steel = 18.26 − 5.56 = 12.70 in

Bottom of steel = 18.26 + 5.56 = 23.82 in

Top of concrete = 12.70 + 2 + 7.75 = 22.45 in

Steel moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 30,100/12.70$ $= 2,370 \text{ in}^3$	$S_{sb} = 30,100/23.82$ $=1,264 \text{ in}^3$	$S_c = 30,100/22.45$ $= 1,341 \text{ in}^3$

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	82.4				18,900	18,900
Concrete 76.5 × 8.5/8	81.3	24.51	1,993	48,840	490	49,330
	163.7		1,993			68,230
$d_8 = 1,993/163.7 = 12.17$ in					$-12.17 \times 1,993 =$	$-24,260$
					$I_{NA} =$	43,970

Distance from neutral axis of composite section to:

Top of steel = 18.26 − 12.17 = 6.09 in

Bottom of steel = 18.26 + 12.17 = 30.43 in

Top of concrete = 6.09 + 2 + 8.5 = 16.59 in

Steel moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 43,970/6.09$ $= 7,220 \text{ in}^3$	$S_{sb} = 43,970/30.43$ $= 1,445 \text{ in}^3$	$S_c = 43,970/16.59$ $= 2,650 \text{ in}^3$

TABLE 12.63 Stresses in End Span for Maximum Positive Moment, ksi

(*a*) Steel stresses

Top of steel (compression)	Bottom of steel (tension)
DL: $f_b = 434 \times 12/1030 = 5.06$	$f_b = 434 \times 12/1030 = 5.06$
SDL: $f_b = 183 \times 12/2370 = 0.93$	$f_b = 183 \times 12/1264 = 1.74$
$LL + I$: $f_b = 786 \times 12/7220 = 1.31$	$f_b = 786 \times 12/1445 = 6.53$
Total: 7.30 < 20	13.33 < 20

(*b*) Stresses at top of concrete

SDL: $f_c = 183 \times 12/(1341 \times 24) = 0.07$

$LL + I$: $f_c = 786 \times 12/(2650 \times 8) = 0.44$

Total: 0.51 < 1.6

MAXIMUM POSITIVE MOMENTS
IN CENTER SPAN, FT·KIPS

M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_{SLL}
773	325	844	63

$$A_{sb} = \frac{12}{20}\left(\frac{773}{35} + \frac{325 + 844 + 63}{35 + 8.5}\right) = 30.2 \text{ in}^2$$

The bottom flange of the W36 × 280 provides an area of 26.0 in^2. Hence, the cover plate should supply an area of about 30.2 − 26.0 = 4.2 in^2. Try to 10 × $^1/_2$-in plate, area = 5.0 in^2.

The trial section is shown in Fig. 12.52. Properties of the cover-plated steel section alone are computed in Table 12.64. In determination of the properties of the composite section, use is made of the computations for the end-span composite section in Table 12.63. Calculations for the center-span section are given in Table 12.65. In all cases, the neutral axes are located by taking moments about the neutral axis of the rolled beam.

Mid-span Stresses in Center Span. Stresses caused by maximum positive moments in the center span are computed in the same way as for the end-span composite section (Table 12.66*a*). Stresses in the concrete are computed with the section moduli of the composite section with $n = 24$ for *SDL* and $n = 8$ for *LL* + *I* (Table 12.66*b*).

Since the bending stresses in steel and concrete are less than the allowable, the assumed steel section is satisfactory. Use the W36 × 280 with 10 × $^1/_2$-in cover plate on the bottom flange. Weld to flange with $^3/_8$-in fillet welds, minimum size permitted for the flange thickness.

Cutoffs of Positive-Moment Cover Plate. Bending moments decrease almost parabolically with distance from mid-span. At some point on either side of mid-span, therefore, the bottom cover plate is not needed for carrying bending moment. After the plate is cut off, the remaining section of the stringer is the same as the composite section in the end span. Properties of this section can be obtained from Table 12.62. Try a theoretical cutoff point 12.5 ft on both sides of mid-span.

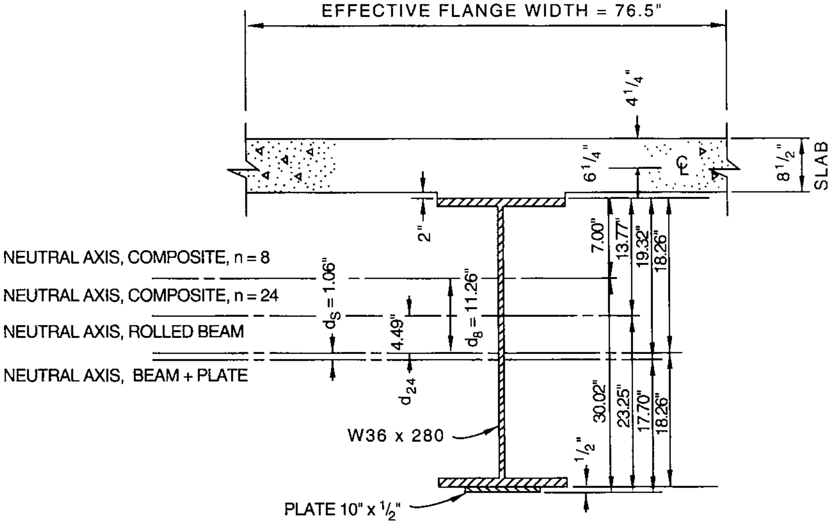


FIGURE 12.52 Composite section for center span of continuous girder.

TABLE 12.64 Rolled Beam with Cover Plate

Material	A	d	Ad	Ad^2	I_o	I
W36 × 280	82.4				18,900	18,900
Cover plate 10 × $^1/_2$	5.0	−18.51	−93	1,710		1,710
	87.4		−93			20,610
$d_s = -93/87.4 = -1.06$ in					$-1.06 \times 93 =$	−100
					$I_{NA} =$	20,510

Distance from neutral axis of steel section to:

Top of steel = 18.26 + 1.06 = 19.32 in

Bottom of steel = 18.26 + 0.50 − 1.06 = 17.70 in

Section moduli

Top of steel	Bottom of steel
$S_{st} = 20{,}510/19.32 = 1{,}062$ in^3	$S_{sb} = 20{,}510/17.70 = 1{,}159$ in^3

TABLE 12.65 Center-Span Composite Section for Maximum Positive Moment

(*a*) For dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I
End-span composite section	107.1		596		33,420
Cover plate 10 × $^1/_2$	5.0	−18.51	−93	1,710	1,710
	112.1		503		35,130
$d_{24} = 503/112.1 = 4.49$ in					$-4.49 \times 503 = -2{,}260$
					$I_{NA} = 32{,}870$

Distance from neutral axis of composite section to:

Top of steel = 18.26 − 4.49 = 13.77 in

Bottom of steel = 18.26 + 0.50 + 4.49 = 23.25 in

Top of concrete = 13.77 + 2 + 7.75 = 23.52 in

Steel moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 32{,}870/13.77$ $= 2{,}387$ in^3	$S_{sb} = 32{,}870/23.25$ $= 1{,}414$ in^3	$S_c = 32{,}870/23.52$ $= 1{,}398$ in^3

(*b*) For live loads, $n = 5$

Material	A	d	Ad	Ad^2	I
End-span composite section	163.7		1,993		68,230
Cover plate 10 × $^1/_2$	5.0	−18.51	−93	1,710	1,710
	168.7		1,900		69,940
$d_s = 1{,}900/168.7 = 11.26$ in					$-11.26 \times 1{,}900 = -21{,}390$
					$I_{NA} =$ 48,550

Distance from neutral axis of composite section to:

Top of steel = 18.26 − 11.26 = 7.00 in

Bottom of steel = 18.26 + 0.50 + 11.26 = 30.02 in

Top of concrete = 7.00 + 2 + 8.5 = 17.50 in

Section moduli

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 48{,}550/7.00$ $= 6{,}936$ in^3	$S_{sb} = 48{,}550/30.02$ $= 1{,}617$ in^3	$S_c = 48{,}550/17.50$ $= 2{,}774$ in^3

TABLE 12.66 Stresses in Center Span for Maximum Positive Moment, ksi

(*a*) Steel stresses			
Top of steel (compression)		Bottom of steel (tension)	
DL: $f_b = 773 \times 12/1062 =$	8.73	$f_b = 773 \times 12/1159 =$	8.00
SDL: $f_b = 325 \times 12/2387 =$	1.63	$f_b = 325 \times 12/1414 =$	2.76
$LL + I$: $f_b = 907 \times 12/6936 =$	1.56	$f_b = 907 \times 12/1617 =$	6.70
Total:	$11.92 < 20$		$17.46 < 20$

(*b*) Stresses at top of concrete	
SDL: $f_c = 325 \times 12/(1398 \times 24) =$	0.12
$LL + I$: $f_c = 907 \times 12/(2774 \times 8) =$	0.49
Total:	$0.61 < 1.6$

CENTER-SPAN MOMENTS, FT · KIPS,
12.5 FT FROM MID-SPAN

M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_{SLL}
694	293	805	58

Calculations for the stresses at the theoretical cutoff point are given in Table 12.67. The composite section without cover plate is adequate at the theoretical cutoff point. With an allowance of 1.5 × 10 × 15 in for the terminal distance, actual cutoff would be about 14 ft from mid-span. Since there is no stress reversal, fatigue does not govern there. Use a cover plate 10 × ½ in by 28 ft long.

Stringer design as determined so far is illustrated in Fig. 12.53.

TABLE 12.67 Tensile Stresses 12.5 ft from Mid-span, ksi

DL: $f_b = 694 \times 12/1030 =$	8.09
SDL: $f_b = 293 \times 12/1264 =$	2.78
$LL + I$: $f_b = 863 \times 12/1445 =$	7.17
Total:	$18.04 < 20$

Bolted Field Splice. The 298-ft overall length of the stringer is too long for shipment in one piece. Hence, field splices are necessary. They should be made where bending stresses are small. Suitable locations are in the center span near the dead-load inflection points. Provide a bolted field splice in the center span 20 ft from each support. Use A325 ⅞-in-diameter high-strength bolts in slip-critical connections with Class A surfaces.

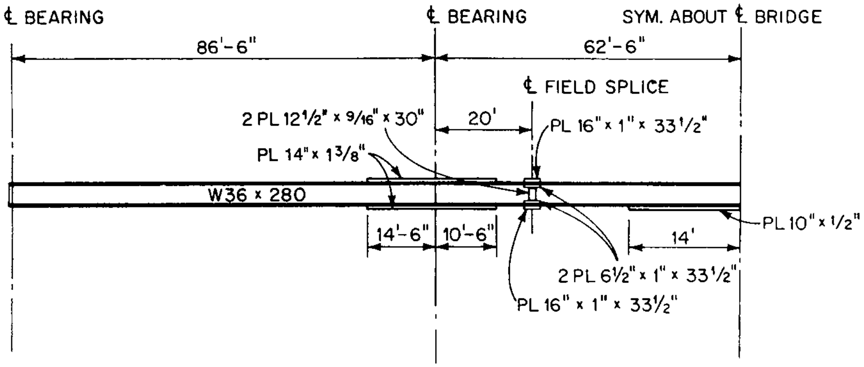


FIGURE 12.53 Cover plates and field splice for typical girder.

Bending moments at each splice location are identical because of symmetry. They are obtained from Fig. 12.49.

MOMENTS AT FIELD SPLICE, FT·KIPS

	M_{DL}	M_{SDL}	$M_{LL} + M_I$	M_{SLL}	Total M
Positive	–80	–50	280	10	160
Negative	–80	–50	–330	–20	–480

Because of stress reversal, a slip-critical connection must be used. Also, fatigue stresses in the base metal adjacent to the bolts must be taken into account for 500,000 cycles of loading. The allowable fatigue stress range, ksi, in the base metal for tension or stress reversal for the Stress Category B connection and the redundant-load-path structure, is 29 ksi. The allowable shear stress for bolts in a slip-critical connection is 15.5 ksi.

The web splice is designed to carry the shear on the section. Since the stresses are small, the splice capacity is made 75% of the web strength. For web strength 0.885 × 36.5 = 32.3 and $F_v = 12$ ksi,

$$V = 0.75 \times 32.3 \times 12 = 291 \text{ kips}$$

Each bolt has a capacity in double shear of 2 × 0.601 × 15.5 = 18.6 kips. Hence, the number of bolts required is 291/18.6 = 16. Use two rows of bolts on each side of the splice, each row with 10 bolts and 3-in pitch. Also, use on each side of the web a 30 × $^9/_{16}$-in splice plate, total area = 33.7 > 32.3 in^2.

The flange splice is designed to carry the moment on the section. With the allowable bending stress of 20 ksi, the W36 × 280 has a resisting moment of

$$M = \frac{1030 \times 20}{12} = 1720 > 480 \text{ ft} \cdot \text{kips}$$

The average of the resisting and calculated moment is 1100 ft·kips, which is less than 0.75 × 1720 = 1290 ft·kips. Therefore, the splice should be designed for a moment of 1290 ft·kips. With a moment arm of 35 in, force in each flange is

$$P = \frac{1290 \times 12}{35} = 442 \text{ kips}$$

Then, the number of bolts in double shear required is 442/18.6 = 24. Use on each side of the splice four rows of bolts, each row with six bolts. But to increase the net section of the flange splice plates, the bolts in inner and outer rows should be staggered $1^1/_2$ in.

The flange splice plates should provide a net area of 442/20 = 22.1 in^2. Try a 16 × 1-in plate on the outer face of each flange and a $6^1/_2$ × 1-in plate on the inner face on both sides of the web. Table 12.68 presents the calculations for the net area of the splice plates. The plates can be considered satisfactory. See Fig. 12.53.

Shear in Web. Maximum shear in the stringer totals 270 kips. Average shear stress in the web, which has an area of 32.3 in^2, is

$$f_v = \frac{270}{32.3} = 8.35 \text{ ksi}$$

TABLE 12.68 Net Area of Splice Plates, in^2

Plate	Gross area	Hole area	$S^2/4g$	Net area
16 × 1	16	–4	2(2.25/12 + 2.25/26.4)	12.55
2—$6^1/_2$ × 1	13	–4	2(2.25/12)	9.38
Total	29			21.93

With an allowable stress in shear of 12 ksi, the web has ample capacity. Furthermore, since the shear stress is less than 0.75 × 12 = 9 ksi, bearing stiffeners are not required.

Shear Connectors. To ensure composite action of concrete slab and steel stringer, shear connectors welded to the top flange of the stringer must be embedded in the concrete (Art. 12.4.10). For this structure, $^3/_4$-in-diameter welded studs are selected. They are to be installed at specified locations in the positive-moment regions of the stringer in groups of three (Fig. 12.54*b*) to resist the horizontal shear at the top of the steel stringer. With height $H = 6$ in, they satisfy the requirement $H/d \geq 4$, where d = stud diameter, in.

Computation of number of welded studs required and pitch is similar to that for the simply supported stringer designed in Art. 12.2. With $f'_c = 4000$ psi for the concrete, the ultimate strength of each stud is $S_u = 27$ kips. By Eq. (12.4), the allowable load range, kips/stud, for fatigue resistance is, for 500,000 cycles of load, $Z_r = 5.97$.

In the end-span positive-moment region, the strength of the rolled beam is

$$P_1 = A_s F_y = 82.4 \times 36 = 2970 \text{ kips}$$

The compressive strength of the concrete slab is

$$P_2 = 0.85 f'_c bt = 0.85 \times 4.0 \times 76.5 \times 8.5 = 2210 < 2970 \text{ kips}$$

Concrete strength governs. Therefore, the number of studs to be provided between the point of maximum moment and both the support and the dead-load inflection point must be at least

$$N = \frac{P_2}{0.85 S_u} = \frac{2210}{0.85 \times 27} = 96$$

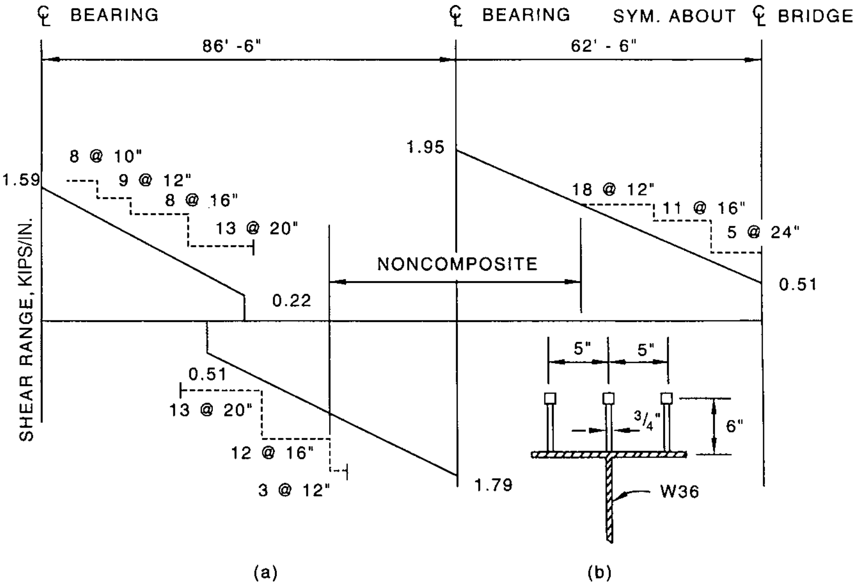


FIGURE 12.54 Variation of shear range (solid lines) and pitch selected for shear connectors (dashed lines) for continuous girder.

Since the point of maximum moment is about 37 ft from the support, and the studs are placed in groups of three, there should be at least 32 groups within that distance. Similarly, there should be at least 32 groups in the 23 ft from the point of maximum moment and the dead-load inflection point.

Pitch is determined by fatigue requirements. The sloping lines in Fig. 12.54*a* represent the range of horizontal shear stress, kips per in, $S_r = V_r Q/I$, where V_r is the shear range, or change in shear caused by live loads, Q is the moment about the neutral axis of the transformed concrete area ($n = 8$), and I is the moment of inertia of the composition section. Shear resistance provided, kips/in, equals $3Z_r/p = 17.91/p$, where p is the pitch.

Spacing, in	10	12	14	16	20	24
Shear resistance, kips/in	1.79	1.49	1.28	1.12	0.90	0.75

The shear-connector spacings selected to meet the preceding requirements are indicated in Fig. 12.54*a*.

Additional connectors are required at the dead-load inflection point in the end span over a distance of one-third the effective slab width. The number depends on A_r the total area, in^2, of longitudinal slab reinforcement for the stringer over the interior support. AASHTO specifications require that in the negative-moment regions of continuous spans, the minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1% of the cross-sectional area of the concrete section. Therefore,

$$A_r = 0.01 \times 76.5 \times 8.5 = 6.50 \text{ in}^2$$

The range of stress in the reinforcement may be taken as $f_r = 10$ ksi. Then, the additional connectors needed total

$$N_c = \frac{A_r f_r}{Z_r} = \frac{6.50 \times 10}{5.97} = 10.9 \qquad \text{say } 12$$

These are indicated for the noncomposite region at the inflection point in Fig. 12.54*a*.

In the center span also, concrete strength also determines the number of shear connectors required between mid-span and each dead-load inflection point. As in the end span, at least 18 groups of connectors should be provided. In addition, at least three groups are required at the inflection points within a distance of $^{75}/_3 = 25$ in. The pitch is determined by the shear range as for the end span. Figure 12.54*a* indicates the pitch selected.

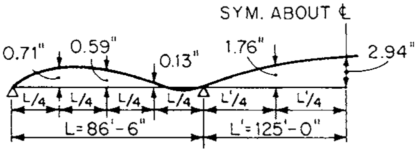


FIGURE 12.55 Camber of girder to offset dead-load deflections.

Bearings. Fixed and expansion bearings for the continuous stringers are the same as for simply supported stringers. However, some bearings may require uplift restraint.

Camber. Dead-load deflections can be computed by elastic methods for the actual moments of inertia along the stringer. The camber to offset these deflections is indicated in Fig. 12.55.

Live-Load Deflection. Maximum live-load deflection occurs at the middle of the center span and equals 1.39 in. The deflection–span ratio is 1.39/(125 × 12) = 1/1080. This is less than 1/1000, the maximum for bridges in urban areas, and is satisfactory.

12.15 EXAMPLE—LOAD AND RESISTANCE FACTOR DESIGN (LRFD) OF COMPOSITE PLATE-GIRDER BRIDGE

As discussed in Chap. 10, the AASHTO LRFD Bridge Design Specifications represent a major step forward in improved highway bridge design and are intended to replace the AASHTO standard specifications. It is anticipated that bridges designed using LRFD should exhibit superior serviceability, enhanced long-term maintainability, and a more uniform level of safety.

To illustrate LRFD design, calculations are presented for simply supported, composite, plate-girder stringers for a two-lane highway bridge (Fig. 12.13) similar to that considered in Art. 12.4 by the LFD method. The span length is 100 ft and the girder spacing is 8 ft 4 in. The HL-93 live load for LRFD will be used (Art. 10.5).

12.15.1 Stringer Design Procedure

The design procedure for LRFD in most cases resembles that discussed in Art. 12.4.1 for LFD design, but the detailed design criteria differ as discussed in the following articles.

12.15.2 Concrete Slab

The slab is designed to span transversely between stringers using a procedure similar to that for LFD (Art. 12.4.2). The same 9-in-thick, one-course concrete slab is used for this example. AASHTO LRFD Bridge Design Specifications allow use of approximate elastic methods or refined analysis methods for design of decks. Also allowed, for monolithic concrete bridge decks satisfying specific conditions, is the use of an empirical design method that does not require analysis. The AASHTO LRFD Bridge Design Specifications provide deck-slab design tables that can be used to determine design live-load moments for different girder spacings. Consideration should be given to the assumptions and limitations used in developing those tables: (1) concrete slabs are supported on parallel girders continuous over at least three girders; (2) distance between the centerlines of fascina girders is 14 ft or more; (3) equivalent strip method is used to calculate the moments; (4) tabulated values are for live-load HL-93; (5) effects of multiple presence factors and dynamic load allowance are included. For other limitations, refer to AASHTO LRFD specifications.

A table in the AASHTO LRFD Bridge Design Specifications provides maximum live-load moments per unit width for different girder spacings at various transverse locations from the girder centerline. This feature allows the user to determine the negative moments at a section that corresponds to the effective slab span length between girders.

The negative live-load moments for a center-to-center distance between girders of 8 ft 4 in can be obtained by interpolation from the AASHTO table values listed below as follows.

$$\text{8 ft 3 in c/c spacing, } M_{LL} = -5.74 \text{ ft}\cdot\text{kips/ft, 3 in from CL}$$

$$M_{LL} = -4.90 \text{ ft}\cdot\text{kips/ft, 6 in from CL}$$

$$\text{8 ft 6 in c/c spacing, } M_{LL} = -5.82 \text{ ft}\cdot\text{kips/ft, 3 in from CL}$$

$$M_{LL} = -4.98 \text{ ft}\cdot\text{kips/ft, 6 in from CL}$$

For concrete slab on steel beams, the effective span length will be the distance between quarter points of the top flange plate, which is 16 in/4 = 4 in. By interpolation, 4 in from centerline of girder and for 8 ft 4 in c/c spacing, the negative slab moment is $M_{LL} = -5.49$ ft·kips/ft, which includes the effects of multiple presence factors and dynamic load allowance.

Maximum positive live-load moment is located midway between the girders. AASHTO table values are as follows:

$$\text{8 ft 3 in c/c beam spacing, } M_{LL} = 5.83 \text{ ft}\cdot\text{kips/ft}$$

$$\text{8 ft 6 in c/c beam spacing, } M_{LL} = 5.99 \text{ ft}\cdot\text{kips/ft}$$

By interpolation, for 8 ft 4 in spacing, the positive slab moment is $M_{LL} = 5.88$ ft·kips/ft.

For computation of permanent dead loads,

$$\text{Weight of concrete slab: } 0.150 \times {}^{9}/_{12} = 0.113$$

$$^{3}/_{8}\text{-in extra concrete in stay-in-place forms: } 0.150(^{3}/_{8})/12 = \underline{0.005}$$

$$\text{Total component dead loads, } DC = 0.12 \text{ kips/ft}$$

$$\text{For dead load of wearing surfaces and utilities, } DW = 0.025 \text{ kips/ft}$$

Assuming a factor of 0.8 is applied to account for continuity of slab over more than three stringers, the maximum dead-load bending moments are

$$+M_{DC} = -M_{DC} = \frac{w_{DC}S^2}{10} = \frac{0.12(8.33)^2}{10} = 0.83\ \text{ft}\cdot\text{kips/ft}$$

$$+M_{DW} = -M_{DW} = \frac{w_{DW}S^2}{10} = \frac{0.025(8.33)^2}{10} = 0.17\ \text{ft}\cdot\text{kips/ft}$$

To determine the negative dead-load moments at the same section as the live-load moments,

$$+R_{DC} = \frac{w_{DC}S}{2} = \frac{0.12(8.33)}{2} = 0.50\ \text{kips/ft}$$

$$+R_{DW} = \frac{w_{DW}S}{2} = \frac{0.025(8.33)}{2} = 0.10\ \text{kips/ft}$$

and 4 in from the centerline of the girder,

$$-M_{DC} = 0.83 - 0.50(0.33) + \frac{0.12(0.33)^2}{2} = 0.68\ \text{ft}\cdot\text{kips/ft}$$

$$-M_{DW} = 0.17 - 0.10(0.33) + \frac{0.025(0.33)^2}{2} = 0.15\ \text{ft}\cdot\text{kips/ft}$$

Load Combinations. Fatigue need not be investigated for concrete deck slabs in multigirder applications. By inspection, strength I limit state will govern the deck-slab design. The effect of factored loads is expressed as

$$Q = \eta \sum \gamma_i q_i \tag{12.46}$$

where

$$\eta = \eta_D \eta_R \eta_I \geq 0.95 \tag{12.47}$$

and

$\eta_D = 1.0$ for conventional designs
$\eta_R = 1.0$ for conventional levels of redundancy
$\eta_I = 1.0$ for typical bridges

Thus, for this design,

$$\eta = 1.0 \geq 0.95$$

From Eq. (12.46), since $\gamma_p = 1.25$ for load *DC* and $\gamma_p = 1.50$ for *DW*, the positive moment is

$$Q = 1.0(1.25 \times 0.83 + 1.50 \times 0.17 + 1.75 \times 5.88) = 11.58\ \text{ft}\cdot\text{kips/ft}$$

And the negative moment is

$$Q = 1.0(1.25 \times 0.68 + 1.50 \times 0.15 + 1.75 \times 5.49) = -10.68\ \text{ft}\cdot\text{kips/ft}$$

The required factored resistance, M_r, must be no greater than the nominal resistance, M_n, times the factor ϕ. Thus, $M_r \leq \phi M_n$. For rectangular sections, the nominal resistance reduces to

$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right) \tag{12.48}$$

where A_s = area, in^2, of steel reinforcement
f_y = yield point = 60.0 ksi for Grade 60 rebar
d_s = effective depth of steel reinforcement
= 9 in − $2^1/_2$ in (concrete cover) − $^1/_2(^6/_8)$ (assuming No. 6 rebar) = 6.13 in

From Eq. (12.8), for f'_c = 4.0 ksi and b = 12-in strip, depth of compressive stress block is

$$a = \frac{A_s f_y}{0.85\, f'_c b} = \frac{60 A_s}{0.85 \times 4 \times 12} = 1.47 A_s$$

For flexure of reinforced concrete, ϕ = 0.90 and negative moment governs the design. Equate the negative moment ϕM_n, where M_n is given by Eq. (12.48), and solve for the required steel area as follows:

$$10.68(12) = 0.90 \times 60 A_s \left(6.13 - \frac{1.47 A_s}{2}\right)$$

which leads to

$$A_s^2 - 8.34 A_s + 3.23 = 0$$

Thus, A_s = 0.41 in^2/ft. Try No. 5 bars @ 8 in: A_s = 0.47 in^2/ft > 0.41 in^2/ft—OK.

Check minimum reinforcement as follows. AASHTO requires that the amount of tensile reinforcement at any section of a flexural component be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of 1.2M_{cr} or 1.33 M_{fT}, where M_{cr} is the critical moment as defined below and M_{fT} is the factored transverse slab moment.

$$M_{cr} = f_r S \tag{12.49}$$

where f_r is modulus of rupture and S is section modulus. For normal-weight concrete,

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{4.0} = 0.48 \text{ ksi}$$

$$S = \frac{bh^2}{6} = \frac{12(8.5)^2}{6} = 144.5 \text{ in}^3$$

Thus,

$$M_r \geq 1.2\left[144.5 \times \frac{0.48}{12}\right] = 6.94 \text{ ft} \cdot \text{kips/ft} \qquad \text{(Governs)}$$

$$M_r \geq 1.33(11.58) = 15.40 \text{ ft} \cdot \text{kips/ft}$$

Since the reinforcement provided for flexure in the deck slab for the factored moment of M = 11.58 ft·kips/ft is greater than 6.94 ft·kips/ft, the minimum reinforcement requirement of AASHTO is met.

Alternatively, for nonprestressed components, the minimum reinforcement provision of AASHTO will be considered satisfied if the following minimum reinforcement ratio is provided:

$$\rho_{\min} \geq 0.03 \frac{f'_c}{f_y} \tag{12.50}$$

In this case, the criterion becomes $\rho_{\min} \geq 0.03(4.0/60.0) = 0.002$. For the steel area provided,

$$\rho = \frac{A_s}{12d_s} = \frac{0.47}{6.13 \times 12} = 0.0064 > 0.002.$$

Therefore the minimum reinforcement requirement is met.

Check maximum reinforcement as follows. AASHTO requires that

$$\frac{c}{d_e} \leq 0.42 \tag{12.51}$$

where $d_e = d_s$ in a nonprestressed concrete section, and c is the distance from the extreme compression fiber to the neutral axis defined as

$$c = \frac{a}{\beta_1} \tag{12.52}$$

where the factor $\beta_1 = 0.85$ for $f_c' = 4.0$-ksi concrete. Thus, for the steel area provided,

$$c = \frac{1.47 \times 0.47}{0.85} = 0.81 \qquad \text{and} \qquad \frac{c}{d_e} = \frac{0.81}{6.13} = 0.13 < 0.42 \qquad \text{OK}$$

The maximum reinforcement requirement is also met. The section is termed “underreinforced” and sufficient ductility is provided.

Check distribution of reinforcement for control of cracking. AASHTO imposes the following stress limitation on the tensile stress in the reinforcement at the service limit state:

$$f_{sa} \leq \frac{Z}{(d_c A)^{1/3}} \leq 0.6 f_y \tag{12.53}$$

where $Z = 170$ ksi for moderate exposure conditions
d_c = depth of reinforcement bars
= 2.0 in (max. conc. cover) + $^1/_2(^5/_8$ in) = 2.31 in for top bars
d_c = 1.0 in (concrete cover) + $^1/_2(^5/_8$ in) = 1.31 in for bottom bars
A = area of concrete around rebar
$A_{top} = 2 \times 2.31 \times 8$ in (bar spacing) = 36.96 in^2
$A_{bot} = 2 \times 1.31 \times 8$ in (bar spacing) = 20.96 in^2

Substitution in Eq. (12.53) yields $f_{sa} = 38.61$ ksi for top bars, but it is limited to $f_{sa} = 0.6 \times 60 = 36.0$ ksi for Grade 60 rebars. There is no need to calculate f_{sa} for bottom rebars because it will not be critical.

Next determine the tensile stress in the reinforcement at the service limit state, because AASHTO requires service I limit state to be used for crack control. The negative moment is calculated as previously but with $\gamma_p = 1.00$:

$$Q = 1.0(1.00 \times 0.68 + 1.00 \times 0.15 + 1.00 \times 5.49) = 6.32 \text{ ft} \cdot \text{kips/ft}$$
$$a = 1.47A_s = 1.47 \times 0.47 = 0.69 \text{ in}$$

The stress in the reinforcement is determined by substituting in $M_r = \phi M_n$:

$$6.32 \times 12 = 0.9 \times 0.47 f_s \left(6.13 - \frac{0.69}{2}\right)$$
$$f_s = 30.99 < 0.6 f_y = 36.0 \text{ ksi} \qquad \text{OK}$$

Therefore, the AASHTO requirement for distribution of reinforcement for control of cracking is met.

For main reinforcement at top and bottom of the deck slab, use No. 5 bars @ 8 in.

12.15.3 Loads, Moments, and Shears

There are fewer load combinations specified for LRFD than for LFD, and some of the load combinations apply only to concrete superstructures. The following strength I limit state load combination will govern in this example problem:

$$\text{Strength I limit state:} \qquad \gamma_p(\text{dead load}) + 1.75(LL + IM)$$

Different load factors are applied to different types of dead loads. In addition, AASHTO specifies minimum and maximum values for these loads factors, and the most unfavorable load factor must be used in the design.

$$\lambda_p = 1.25 \text{ for component and attachments, } DC$$
$$\lambda_p = 1.50 \text{ for wearing surfaces and utilities, } DW$$

Then, strength I limit state load combination becomes:

$$1.25DC + 1.50DW + 1.75(LL + IM)$$

and loads are calculated as follows.

PERMANENT LOAD OF MEMBER COMPONENTS, *DC*

Slab—$0.150 \times 8.33 \times {}^{9}/_{12}$	= 0.938
Haunch—16 × 2 in: $0.150 \times 1.33 \times 0.167$	= 0.034
Steel stringer and framing details—assume:	= 0.323
Stay-in-place forms and additional concrete in forms:	= 0.090
Two barrier curbs $2 \times 0.530/4$ stringers	= 0.265
DC per stringer	= 1.650 kips/ft

PERMANENT LOAD OF WEARING SURFACES AND UTILITIES, *DW*

Future wearing surface $0.025 \times 8.33 = 0.210$

DW per stringer = 0.210 kips/ft

AASHTO states that vehicular live loading on bridges, designated HL-93, shall consist of a combination of the design truck or tandem, and design lane load. The configuration of the design truck is similar to an HS-20 truck as specified in AASHTO standard specifications, design tandem to alternate military load, and design lane load to lane load (without the concentrated load). (See Art. 10.5.)

The multiple presence factor, *m*, will be taken as 1.0 for two lanes for a 26-foot-wide bridge. Dynamics load allowance, *IM*, to be applied to the static load, to account for wheel load impact from moving vehicles, will be taken as (1 + *IM*/100), or 1.33, since *IM* is given in the AASHTO LRFD specifications as 33% for all limit states except for fatigue and fracture limit states.

Distribution of live loads per lane for moment in interior steel beams with concrete decks when two or more design lanes are loaded may be obtained from Eq. (10.12) As

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

where S = beam spacing, 8.33 ft
L = span of beam, 100 ft
t_s = deck slab thickness, 8.5 in (${}^{1}/_{2}$ in allowance for wearing surface)
K_g = longitudinal stiffness parameter defined as

$$K_g = n\left(I + Ae_g^2\right) = 1{,}457{,}000 \text{ in}^4$$

where $n = E_B/E_D$ ($n = 8$ for $f'_c = 4.0$ ksi concrete)
I = moment of inertia of steel beam only, 42,780 in^4
e_g = distance between c.g. of basic beam and c.g. of deck, 38.92 in + 2 in + 4.25 in = 45.17 in
A = area of beam, 68.3 in^2

Substituting the above values,

$$DF = 0.075 + 0.924 \times 0.608 \times 1.070 = 0.676$$

Distribution of live loads per lane for shear for this condition may be obtained from

$$DF = 0.2 + S/12 - (S/35)^2 \qquad (12.54)$$

where S is the spacing of girders, ft. Substitution of $S = 8.33$ ft gives $DF = 0.838$.

AASHTO LRFD specifications have provisions for reduction of live-load distribution factors for moment in longitudinal beams on skewed supports and correction factors for live-load distribution factors for end shear at the obtuse corner, but since the bridge in this example has no skew, no adjustments to the distribution factors will be made.

Maximum dead-load moments occur at mid-span:

$$M_{DC} = {}^1/_8(1.65)(100)^2 = 2063 \text{ ft} \cdot \text{kips}$$
$$M_{DW} = {}^1/_8(0.21)(100)^2 = 263 \text{ ft} \cdot \text{kips}$$

Maximum dead-load shears at supports are

$$V_{DC} = {}^1/_2(1.65)(100) = 82.5 \text{ kips}$$
$$V_{DW} = {}^1/_2(0.21)(100) = 10.5 \text{ kips}$$

Maximum moment due to the design truck occurs when the center axle is at 47.67 ft from a support (see Fig. 12.14*a*). Then, the maximum live-load moment per lane is

$$M(\text{truck}) = \frac{72(100/2 + 2.33)^2}{100} - 32 \times 14 = 1524 \text{ ft} \cdot \text{kips}$$

Similarly, maximum live-load moment due to design tandem occurs when the front (or rear) axle is 49 ft from a support.

$$M(\text{tandem}) = \frac{50(100/2 + 1.00)^2}{100} - 25 \times 4 = 1201 \text{ ft} \cdot \text{kips}$$
$$M(\text{lane}) = {}^1/_8(0.64)(100)^2 = 800 \text{ ft} \cdot \text{kips}$$

Maximum live-load shears due to truck, tandem, and lane loads are as follows:

$$V(\text{truck}) = \frac{72(100 - 14 + 4.66)}{100} = 65.3 \text{ kips}$$
$$V(\text{tandem}) = \frac{50(100 - 4 + 2.00)}{100} = 49.0 \text{ kips}$$
$$V(\text{lane}) = {}^1/_2(0.64)(100) = 32.0 \text{ kips}$$

The governing live-load combination is design truck and design lane load.

$$M_{LL} = (1524 + 800)0.676 = 1571 \text{ ft} \cdot \text{kips}$$
$$M_{IM} = 1524 \times 0.676 \times 0.33 = 340 \text{ ft} \cdot \text{kips}$$

since dynamic load allowance is not applied to design lane load according to AASHTO.

$$M_{LL+IM} = 1911 \cdot \text{ft} \cdot \text{kips}$$

Similarly,

$$V_{LL} = (65.3 + 32.0)0.838 = 81.5 \text{ kips}$$
$$V_{IM} = 65.3 \times 0.838 \times 0.33 = 18.1 \text{ kips}$$
$$V_{LL+IM} = 99.6 \text{ kips}$$

The total factored moment and shear values for strength I limit state are

$$M_{fT} = 1.25 \times 2063 + 1.50 \times 263 + 1.75 \times 1911 = 6318 \text{ ft} \cdot \text{kips}$$
$$V_{ft} = 1.25 \times 82.5 + 1.50 \times 10.5 + 1.75 \times 99.6 = 293.2 \text{ kips}$$

The mid-span factored bending moment, ft · kips, due to the various loads and the total are as follows:

M_{fDC}	M_{fDW}	$M_{fLL} + M_{fIM}$	M_{fT}
2579	395	3344	6318

The corresponding factored end shears, kips, are as follows:

V_{fDC}	V_{fDW}	$V_{fLL} + V_{fIM}$	V_{fT}
103.1	15.8	174.3	293.2

The strength I limit state shear diagram due to factored loads is shown in Fig. 12.56.

Fatigue Limit State. The fatigue limit state is defined as a fatigue and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck having the weights and spacing of axles as shown in Fig. 12.57. For this limit state the rear-axle spacing remains constant. A dynamic load allowance of 15% will be applied to the fatigue load.

Distribution of live loads per lane for moment in interior steel beams with concrete decks when one design lane is loaded may be obtained from Eq. (10.11) as

$$D = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$
$$= 0.06 + 0.812 \times 0.474 \times 1.070 = 0.472$$

Distribution of live loads per lane for shear for this condition may be obtained from

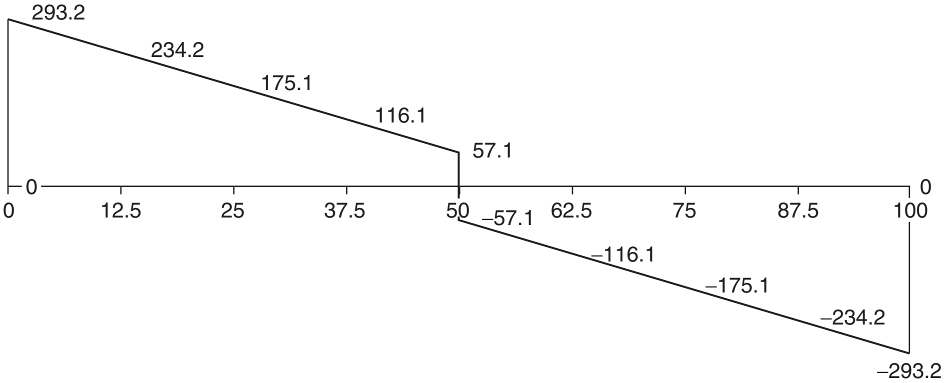


FIGURE 12.56 Plot of factored shear, kips, versus length along span, ft.

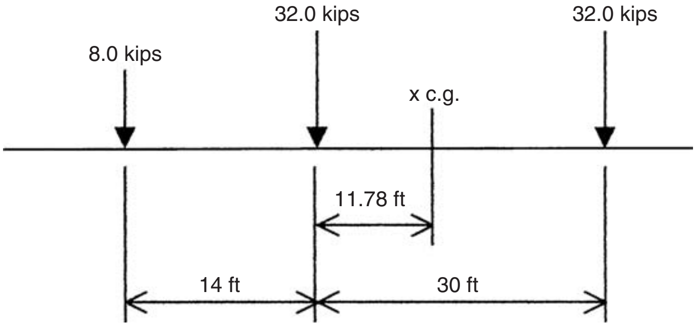


FIGURE 12.57 Axle load configuration of fatigue design truck.

$$DF = 0.36 + \frac{S}{25} \tag{12.55}$$

where S is the spacing of girders, ft. Substitution of $S = 8.33$ ft gives $DF = 0.693$.

The multiple presence factor for live load is not to be applied to the fatigue limit state for which a single design truck is used without regard to the number of design lanes on the bridge. Since the distribution factor is obtained using the single-lane approximate method, but not by the statistical method or level rule, the multiple presence factor, $m = 1.20$, will be removed from the distribution factors for fatigue investigation.

Total load for the three axles of the fatigue truck is 72 kips, and maximum moment occurs when the center axle is $^1/_2(100 - 11.78) = 44.11$ ft from one support, 55.39 ft from the other. Finding the end reaction and taking moments gives

$$M_{\text{Fatigue}} = \frac{72(55.89^2)}{100} - 32 \times 30 = 1289 \text{ ft} \cdot \text{kips}$$

Similarly, the maximum shear due to the fatigue truck is

$$V_{\text{Fatigue}} = \frac{72(100 - 30 + 11.78)}{100} = 58.9 \text{ kips}$$

The factored fatigue bending moments at 5.89 ft from mid-span are as follows:

$$M_{fLL} = 0.75 \times 1289 \times \frac{0.472}{1.20} = 380 \text{ ft} \cdot \text{kips}$$
$$M_{fIM} = 0.15 \times 380 = 57 \text{ ft} \cdot \text{kips}$$
$$M_{f(LL+IM)} = 437 \text{ ft} \cdot \text{kips}$$

The factored fatigue end shears are

$$V_{fLL} = 0.75 \times 58.9 \times \frac{0.693}{1.20} = 25.5 \text{ kips}$$
$$V_{fIM} = 0.15 \times 25.5 = 3.8 \text{ kips}$$
$$V_{f(LL+IM)} = 29.3 \text{ kips}$$

12.15.4 Trial Girder Section

Flexural components will be proportioned such that

$$0.1 \le I_{yc}/I_{yt} \le 1.0 \tag{12.56}$$

where I_{yt} = moment of inertia of the tension flange about the vertical axis, in^4
I_{yc} = moment of inertia of the compression flange about the vertical axis, in^4

A trial section with a web plate 60 × $^7/_{16}$ in, top flange plate 16 × $^3/_4$ in, and bottom flange plate 20 × 1$^1/_2$ in will be assumed in this example. Steel is Grade 36. Assumed cross section of the plate girder for the maximum factored moment is illustrated in Fig. 12.58.

Compression and tension flanges should satisfy the following proportioning requirements:

$b_f/2t_f \le 12.0$	Compression flange: 16/(2 × 0.75) = 10.7 < 12.0—OK
	Tension flange: 20/(2 × 1.5) = 6.7 = 12.0—OK
$b_f \ge D/6$	16 > 60/6 = 10—OK
$t_f \ge 1.1t_w$	0.75 > 1.1($^7/_{16}$) = 0.48 in—OK

$$I_{yc} = \frac{(^3/_4)16^3}{12} = 256 \text{ in}^4$$

$$I_{yt} = \frac{(1^1/_2)20^3}{12} = 1000 \text{ in}^4$$

$$0.1 < \left(\frac{256}{1000} = 0.26\right) < 10 \qquad \text{OK}$$

Webs without longitudinal stiffeners should be proportioned such that

$$\frac{D}{t_w} \le 150 \tag{12.57}$$

where D is depth of the web between flanges, in, and t_w is web thickness, in.

$$\frac{D}{t_w} = \frac{60}{^7/_{16}} = 137 < 150 \qquad \text{OK}$$

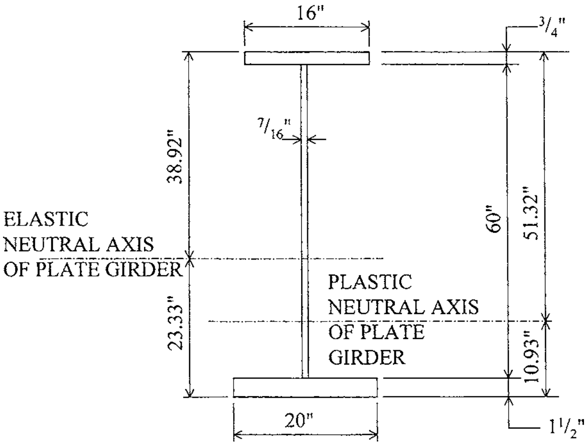


FIGURE 12.58 Cross section assumed for plate girder for LRFD example.

The concrete section for the interior stringer, not including the concrete haunch, is 8 ft 4 in wide (c to c stringers) and $8\frac{1}{2}$ in deep ($\frac{1}{2}$ in of slab is deducted as the wearing course). Elastic section properties are tabulated for the trial steel section and composite section in Tables 12.69 and 12.70.

The plastic moment capacity of the composite section will be determined by force equilibrium. Concrete haunch and deck reinforcement will be neglected. Assume plastic neutral axis (PNA) is at top of top flange:

$$P_t + P_w + P_c \geq P_s$$

$$A_{bf}F_y + A_wF_y + F_{tf}F_y \geq 0.85 f'_c\, b_{\text{eff}}t_s$$

$$30(36) + 26.3(36) + 12(36) \geq 0.85(4.0)(100)(8\tfrac{1}{2})$$

$$1080 + 947 + 432 = 2459 < 2890 \text{ kips}$$

Therefore, PNA is in deck slab.

$$\bar{y} = \frac{2890 - 2459}{0.85 \times 4.0 \times 100} = 1.27 \text{ in from bottom of slab}$$

Compute the plastic moment capacity, M_p, by summing moments about the PNA.

TABLE 12.69 Elastic Section Properties and Stresses for the Trial Section

(*a*) Properties of steel section for maximum factored moment

Material	A	d	Ad	Ad^2	I_o	I
Top flange $16 \times \frac{3}{4}$	12.0	30.38	365	11,090		11,090
Web $60 \times \frac{7}{16}$	26.3				7,880	7,880
Bottom flange $20 \times 1\frac{1}{2}$	30.0	−30.75	−923	28,370		28,370
	68.3		−558			47,340
$d_s = 558/68.3 = -8.17$ in						$-8.17 \times 558 = -4{,}560$
						$I_{NA} = 42{,}780$

Distance from neutral axis of steel section to:

Top of steel = 30 + 0.75 + 8.17 = 38.92 in

Bottom of steel = 30 + 1.50 − 8.17 = 23.33 in

Section modulus, top of steel	Section modulus, bottom of steel
$S_{st} = 42{,}780/38.92$ $= 1{,}100 \text{ in}^3$	$S_{sb} = 42{,}780/23.33$ $= 1{,}830 \text{ in}^3$

(*b*) Service limit state stresses

Steel stresses, ksi

Top of steel (compression)		Bottom of steel (tension)	
$DC = 2{,}063 \times 12/1{,}100 =$	22.51	$DC = 2{,}063 \times 12/1{,}830 =$	13.53
$DW = 263 \times 12/3{,}850 =$	0.82	$DW = 263 \times 12/2{,}340 =$	1.35
$LL + IM = 1{,}911 \times 12/11{,}230 =$	2.04	$LL + IM = 1{,}911 \times 12/2{,}520 = 2{,}839$	9.10
Total	25.37		23.98

(*c*) Strength I limit state stresses

Steel stresses, ksi

Top of steel (compression)		Bottom of steel (tension)	
$DC = 2{,}579 \times 12/1{,}100 =$	28.13	$DC = 2{,}579 \times 12/1{,}830 =$	16.91
$DW = 395 \times 12/3{,}850 =$	1.23	$DW = 395 \times 12/2{,}340 =$	2.03
$LL + IM = 3{,}344 \times 12/11{,}230 =$	3.57	$LL + IM = 3{,}344 \times 12/2{,}520 = 2{,}839$	15.92
Total	32.93		34.86

TABLE 12.70 Properties of Composite Section for Maximum Factored Moment

(*a*) For superimposed dead loads, $n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	68.3		−558			47,340
Concrete 100 × 8.5/24	35.4	37.00	1,310	48,470	210	48,680
	103.7		752			96,020
$d_{24} = 752/103.7 = 7.25$ in						$-7.25 \times 752 = -5{,}450$
						$I_{NA} = 90{,}570$

Distance from neutral axis of composite section to:

Top of steel = 30.75 − 7.25 = 23.50 in

Bottom of steel = 31.50 + 7.25 = 38.75 in

Top of concrete = 23.50 + 2 + 8.50 = 34.00 in

Section modulus

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 90{,}570/23.50$ $= 3{,}850\ \text{in}^3$	$S_{sb} = 90{,}570/38.75$ $= 2{,}340\ \text{in}^3$	$S_c = 90{,}570/34.00$ $= 2{,}660\ \text{in}^3$

(*b*) For live loads, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	68.3		−558			47,340
Concrete 100 × 8.5/8	106.3	37.00	3,933	145,520	640	146,160
	174.6		3,375			193,500
$d_8 = 3{,}375/174.6 = 19.33$ in						$-19.33 \times 3{,}375 = -65{,}240$
						$I_{NA} = 128{,}260$

Distance from neutral axis of composite section to:

Top of steel = 30.75 − 19.33 = 11.42 in

Bottom of steel = 31.50 + 19.33 = 50.83 in

Top of concrete = 11.42 + 2 + 8.50 = 21.92 in

Section modulus

Top of steel	Bottom of steel	Top of concrete
$S_{st} = 128{,}260/11.42$ $= 11{,}230\ \text{in}^3$	$S_{sb} = 128{,}260/50.83$ $= 2{,}520\ \text{in}^3$	$S_c = 128{,}260/21.92$ $= 5{,}850\ \text{in}^3$

$$
\begin{aligned}
M_p &= P_s d_s + P_c d_c + P_w d_w + P_t d_t \\
&= \frac{2{,}459(8.50-1.27)}{2} + 1{,}080(61.50+2+1.27) \\
&\quad + 947(30.75+2+1.27) + 432(0.38+2+1.27) \\
&= 8{,}890 + 69{,}950 + 32{,}220 + 1{,}580 \\
M_p &= 9{,}380\ \text{ft} \cdot \text{kips}
\end{aligned}
$$

Flexural members should be designed for the strength limit state flexural resistance, the service limit state control of permanent deflection, the fatigue and fracture limit state for details and the fatigue requirements of webs, the strength limit state shear resistance, and for constructibility. The following design process will be adopted in this example using simplified methods given in AASHTO LRFD specifications.

Constructibility check for
- Flexure
- Shear

Service limit state check for
- Elastic deformations
- Permanent deformations

Fatigue and fracture state check for
- Fatigue
- Fracture
- Special fatigue requirement for webs

Strength limit state check for

Flexure
- Determine if the section is compact.
- Check whether ductility requirement is met.

Shear
- Check whether web stiffeners are required.
- Transverse stiffener design
- Flange-to-web welds
- Bearing stiffeners
- Shear connectors for composite sections

12.15.5 Service Limit State Check

Elastic Deformations. Deflection check in AASHTO LRFD specifications is optional except for orthotropic decks and lightweight decks such as filled or partially filled grid decks. Some owners may choose to invoke the deflection control for their bridges. In such cases, some or all of the following principles may apply:

- All design lanes are loaded.
- Service I live loads plus *IM* are considered.
- All girders deflect equally.
- Deflection limit for steel and/or concrete construction = span/1000 unless the owner specifies differently.

Maximum live-load deflection for service I load combination can be calculated using Eq. (12.6) for the design truck portion of the governing live loading including dynamic load allowance, and $5wL^4/384EI$ for the uniformly distributed design lane loading portion without the dynamic load allowance. Terms are as defined for Eqs. (12.5) and (12.6). Governing deflection will be the greater of the deflection due to the design truck loading alone or deflection due to design lane loading plus 25% of design truck loading.

Design truck loading deflection:

$$P_{T+IM} = 8 \times 0.676(1 + 0.33) = 7.2 \text{ kips}$$

$$\delta_{T+IM} = \left[324 \times \frac{7.2}{(29{,}000 \times 128{,}260)}\right](100^3 - 555 \times 100 + 4{,}780) = 0.6 \text{ in}$$

Design lane loading deflection:

$$w = 0.64 \times 0.676 = 0.43 \text{ kip/ft, or } 0.036 \text{ kip/in}$$

$$\delta_L = 5 \times 0.036 \times \frac{1{,}200^4}{(384 \times 29{,}000 \times 128{,}260)} = 0.26 \text{ in}$$

Design truck loading deflection condition governs, and since $\delta_{LL+IM} = 0.6$ in $< L/1000 = 1.2$ in, the section satisfies the elastic deformations requirement.

Permanent Deformations. Flange of the composite section should satisfy the following conditions for service II loading combination using the short-term and long-term composite section properties.

Top flange, $f_f \le 0.95 R_h F_{yf}$:

$$f_f = 22.51 + 0.82 + 1.3 \times 2.04 = 25.98 \text{ ksi} < 0.95 \times 1.0 \times 36 = 34.2 \text{ ksi}$$

using stresses already calculated in Table 12.69*b*.

Bottom flange, $f_f + f_l/2 \le 0.95 R_h F_{yf}$:

$$f_f = 13.53 + 1.35 + 1.3 \times 9.10 = 26.71 \text{ ksi} < 0.95 \times 1.0 \times 36 = 34.2 \text{ ksi}$$

taking $f_l = 0$ for an interior girder.

Therefore, this section satisfies the service limit state checks.

12.15.6 Special Fatigue Requirement for Webs

AASHTO LRFD specifications require the interior panels of webs with or without longitudinal stiffeners to satisfy the following:

$$V_u \le V_{cr} \tag{12.58}$$

where V_u = shear in the web at the section under consideration due to unfactored permanent loads plus factored fatigue load, kips

V_{cr} = shear-buckling resistance determined from $V_n = V_{cr} = CV_p$, kips

V_p is the plastic shear force (kips) determined from

$$V_p = 0.58 F_{yw} D t_w \tag{12.59}$$

where F_{yw} is the specified minimum yield stress of the web, ksi; D is the web depth, in; and t_w is the web thickness, in. C is the ratio of shear-buckling resistance to shear yield strength determined from

$$C = \left[\frac{1.57}{(D/t_w)^2}\right]\left(\frac{Ek}{F_{yw}}\right) \tag{12.60}$$

when $D/t_w > 1.40\sqrt{Ek/F_{yw}}$.

Shear buckling coefficient, k, is calculated from

$$k = 5 + \frac{5}{(d_0/D)^2} \tag{12.61}$$

where d_0 is transverse stiffener spacing and D is web depth.

In this example, $k = 5 + 5/(150/60)^2 = 5.8$, assuming 150-in spacing for transverse stiffeners. $D/t_w = 137 > 95$. Thus,

$$C = \left(\frac{1.57}{137^2}\right)\left(29{,}000 \times \frac{5.8}{36}\right) = 0.39$$

and

$$V_n = 0.39 \times 0.58 \times 36 \times 60 \times {}^7/_{16} = 213 \text{ kips}$$

At the first interior panel, 12.5 ft from bearing, unfactored permanent load shears are

$$V_{DC} = 82.5 - 12.5 \times 1.65 = 61.9 \text{ kips}$$

$$V_{DW} = 10.5 - 12.5 \times 0.21 = 7.9 \text{ kips}$$

Similarly,

$$V_{\text{Fatigue}} = \left[\frac{72(87.5 - 30 + 11.78)}{100}\right]\left(\frac{0.693}{1.20}\right) = 28.8 \text{ kips}$$

AASHTO requires that, for the purposes of this check, the fatigue load be taken as twice that calculated from the fatigue load combination. Then,

$$V_u = 61.9 + 7.9 + (0.75 \times 1.15 \times 28.8)\, 2 = 119.5 \text{ kips} < V_{cr} = 213 \text{ kips} \qquad \text{OK}$$

The web will not be subjected to significant elastic flexing, and it is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking.

12.15.7 Strength Limit State Check

Compactness Check. If the composite section satisfies the following requirements, it will qualify as a compact section:

- Yield strength of flanges, $F_y = 36$ ksi < 70 ksi—OK
- Web proportion, $D/t_w = 60/({}^7/_{16}) = 137 < 150$—OK
- Web slenderness, $2D_{cp}/t_w \le 3.76\sqrt{E/F_{yc}}$, where D_{cp} is the web depth in compression

Previously it was determined that the plastic neutral axis was in the deck slab. As a result, D_{cp} will be taken as zero, and the compact section web slenderness requirement of AASHTO is considered to be satisfied. The section is compact provided that the following flexure requirement is satisfied.

Flexural Resistance. At the strength limit state,

$$M_u + {}^1/_3 f_l S_{xt} \le \phi_f M_n \tag{12.62}$$

where M_u = factored bending moment about the major axis, ft·kips
f_l = lateral bending stress in flange, ksi
S_{xt} = elastic section modulus taken as M_{yt}/F_{yt}, in^3
M_{yt} = yield moment with respect to the tension flange
F_{yt} = yield strength of tension flange, ksi
ϕ_f = resistance factor for flexure = 1.00
M_n = nominal flexural resistance of the section, ft·kips

In this example, $M_u = 6318$ ft·kips.

Lateral bending moment, M_l, due to eccentric loadings, such as loading from cantilever brackets of the deck forms, loading from deck concrete-pouring sequence, or wind loading on exterior girders between the brace points, may be calculated from $M_l = F_l\, L_b^2/12$, or $M_l = P_l L_b/8$, depending on the assumption of the load application. Since this example is for an interior girder, it will be assumed that $M_l = 0$ and $f_l = 0$.

The nominal flexural resistance, M_n, of the composite section in the positive flexural region of a simple span will be taken as $M_n = M_p$ when

$$D_p \le 0.1 D_t \tag{12.63}$$

where D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment and D_t is the total depth of the composite section.

$$D_p = 8.50 - 1.27 = 7.23 \text{ in}$$

$$D_t = 8.50 + 2 + 0.75 + 60 + 1.50 = 72.75 \text{ in}$$

$$0.1D_t = 0.1 \times 72.75 \text{ in} = 7.28 \text{ in}$$

Thus, $D_p \leq 0.1D_t$ and therefore $M_n = M_p = 9380$ ft·kips, as determined before. Since $M_u = 6318$ ft · kips < $M_n = 9380$ ft·kips, the section satisfies the strength limit state for flexure.

Ductility Requirement. The section must also be checked to see if it satisfies the ductility requirement to ensure that the concrete slab is protected from premature crushing and spalling when the composite section approaches the plastic moment. The following ratio for compact and noncompact sections ensures significant yielding of the bottom flange when the crushing strain is reached at the top of the concrete deck:

$$D_p \leq 0.42D_t \tag{12.64}$$

In this design, 7.23 in < 0.42 × 72.75 in = 30.56 in—OK.

12.15.8 Shear Design

Next, the nominal shear resistance of an unstiffened web will be determined from $V_n = V_{cr} = CV_p$ kips, with C and V_p as defined in Art. 12.15.7. In this design, $D/t_w = 60/({}^7\!/_{16}) = 137$, which is greater than $1.40\sqrt{Ek/F_{yw}} = 89$, assuming the shear buckling coefficient as $k = 5$ for an unstiffened web. From Eq. (12.60),

$$C = \left(\frac{1.57}{137^2}\right)(29{,}000 \times {}^5\!/_{36}) = 0.337$$

Thus, with V_p from Eq. (12.59), the nominal shear resistance is

$$V_n = CV_p = 0.337 \times 0.58 \times 36 \times 60 \times ({}^7\!/_{16}) = 184.7 \text{ kips}$$

This is less than $V_{ft} = 293.2$ kips, the factored shear load for the strength I limit state. Therefore, intermediate transverse stiffeners are required for the range where $V_{ft} > 184.7$ kips.

Transverse Stiffener Design. Transverse stiffeners may consist of plates or angles welded or bolted to the web on one side or both sides. The width, b_t, of each projecting stiffener element should satisfy both of the following conditions to prevent local buckling of the transverse stiffener:

$$2 + \frac{d}{30} \leq b_t \tag{12.65}$$

$$0.25b_f \leq b_t \leq 16t_p \tag{12.66}$$

where d = total depth of the steel section, in
t_p = thickness of projecting element, in
b_f = full width of wider flange at a section, in.

For Eq. (12.65),

$$2 + \frac{62.25}{30} = 4.08 \qquad \text{so } b_t \text{ must be at least 4.08 in}$$

For Eq. (12.66),

$$0.25(20) \le b_t \le 16t_p \qquad \text{or} \qquad 5.0 \le b_t \le 16t_p$$

For $b_t = 6$ in, $t_p \ge 6/16 = 0.375$ in.

Try a pair of transverse stiffener plates of 6 × ½ in for the end panels. Check to see if the moment of inertia of the transverse stiffeners satisfies the following condition:

$$I_t \ge d_0 t_w^3 J \tag{12.67}$$

where

$$J = 2.5\left(\frac{D_p}{d_0}\right)^2 - 2.0 \ge 0.5 \tag{12.68}$$

In the above, I_t = moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners, or taken about the mid-thickness of the web for stiffener pairs, in^4; D_p = web depth for webs without longitudinal stiffeners or maximum subpanel depth for webs with longitudinal stiffeners, in; and d_0 = stiffener spacing, in. The maximum stiffener spacing in end panels is limited to 1.5D, which in this case is 1.5 × 60 = 90 in. From Eq. (12.68), for $d_0 = 90$ and $D_p = 60$, find $J = 0.5$. Then, from Eq. (12.67), $I_t = 90(^7/_{16})^3(0.5) = 3.77$ in^4.

Next, the nominal shear resistance of interior web panels of compact sections, V_n, can be determined from Eq. (12.23) if $M_u \le 0.5\phi_f M_p$, where M_u is the maximum moment in the panel under consideration, ϕ_f is the resistance factor for flexure, 1.00, and M_p is the plastic moment resistance, 9380 ft·kips in this case.

The first intermediate diaphragm is located 25 ft or 300 in from centerline of end bearing and stiffeners are usually spaced equally between the diaphragms. For a first trial, check stiffener spacing of $d_0 = 300/2 = 150$ in. Check to see that the panel proportions satisfies the following:

$$\frac{2Dt_w}{b_{fc}t_{fc} + b_{ft}t_{ft}} \le 2.5 \tag{12.69}$$

$$\frac{2 \times 60 \times {}^7/_{16}}{(16 \times {}^3/_4) + (20 \times 1.5)} = 1.25 < 2.5 \qquad \text{OK}$$

From the LFD example of Art. 12.4, $V_n = 322$ kips, which is greater than $V_{ft} = 293.2$ kips and is OK. Therefore, continue with the design of transverse stiffeners for the 150-in spacing. Note that this does not exceed the maximum spacing permitted, $3D = 180$ in.

Assuming a pair of 6 × ½ in stiffeners, substitute in Eqs. (12.67) and (12.68):

$$J = 2.5\left(\frac{60}{150}\right)^2 - 2.0 = -1.6 < 0.5 \qquad \text{Use } J = 0.5.$$

$$I_t \ge 150(^7/_{16})^3 \times 0.5 = 6.3 \text{ in}^4$$

Then,

$$I_t = A\bar{y}^2 + I_0 = 2[6 \times {}^1/_2 \times (3 + {}^7/_{32})^2 + {}^1/_2(6)^3/12] = 80.2 \text{ in}^4 > 6.3 \text{ in}^4 \qquad \text{OK}$$

Transverse stiffeners are required to have sufficient area to carry the vertical component of forces imposed by tension field action of the web:

$$A_s \ge [0.15B(D/t_w)(1.0 - C)\left(\frac{V_u}{\phi_v V_n}\right) - 18.0]\, t_w^2 \left(\frac{F_{yw}}{F_{ys}}\right) \tag{12.70a}$$

where B = 1.0 for stiffeners on both sides of web
C = ratio of shear buckling stress to shear yield strength
V_u = shear due to factored loads at the strength limit state, kips

For this design, $C = 0.39$ (from Art. 12.4), $V_u = (293.2 + 234.2)/2 = 263.7$ kips, and $A_s = 2 \times 6 \times {}^1\!/_2 = 6.0$ in^2.

$$F_{crs} = \frac{0.31E}{(b_t/t_p)^2} \leq F_{ys} \tag{12.70b}$$

$F_{crs} = 0.31 \times 29{,}000/(6/0.5)^2 = 62.4$ ksi $> F_{ys} = 36$ ksi. Thus, from Eq. (12.70a),

$$6.0 \geq [0.15 \times 1.0 \times (60/{}^7\!/_{16}) \times (1.0 - 0.39)(263.7/322) - 18.0] \times ({}^7\!/_{16})^2 \times 1$$

$$6.0 \geq -1.48$$

The negative result indicates that the web alone is sufficient to carry the vertical component of the tension field. Therefore, the assumed transverse stiffeners satisfy all requirements. Use 6 in × ¹⁄₂ in transverse stiffener plates on both sides of the web of the interior girders.

Flange-to-Web Welds. Each flange will be connected to the web by a fillet web on each side of the web. The horizontal shear between the web and the flange must be resisted by the weld. Fillet welded connections subjected to shear on the effective area will be designed for the lesser of either the factored resistance of the connected material or the factored resistance of the weld metal.

The factored resistance of the weld metal, R_r, is

$$R_r = 0.6\phi_{e2}F_{exx} \tag{12.71}$$

where ϕ_{e2} = resistance factor for shear in throat of weld metal = 0.80
F_{exx} = classification strength of weld metal = 70.0 ksi (for E70XX weld metal)

Thus, $R_r = 0.6 \times 0.80 \times 70.0 = 33.60$ ksi.

The gross moment of inertia of the steel section, $I = 42{,}780$ in^4, will be used in computing the shear, v, kips/in, between flange and web. The static moment of the flange area is $Q = 1.5 \times 20.0 \times (23.33 - 1.5/2) = 677$ in^3, and the factored maximum shear is 293.2 kips. Thus, $v = VQ/I = 293.2 \times 677/42{,}780 = 4.64$ kips/in. The weld size required to resist the shear is $v/(R_r \times 0.707 \times 2$ sides of web$) = 4.64 / (33.60 \times 0.707 \times 2) = 0.10$ in. The minimum-size fillet weld required for a base metal thicker than ³⁄₄ in is ⁵⁄₁₆ in. Therefore, use two ⁵⁄₁₆-in continuous fillet welds to connect the web to the bottom flange.

Bearing Stiffener Design. Bearing stiffeners should be provided at all bearing locations or at other points of concentrated loads if the limit states of web local yielding or web crippling conditions are not satisfied.

For web local yielding,

$$R_u \leq \phi_b R_n \tag{12.72}$$

where R_u = factored concentrated load or bearing reaction
ϕ_b = resistance factor for bearing = 1.0 for bearing on milled surfaces
R_n = nominal resistance to the concentrated loading, kips

For end reactions or for concentrated loads acting within d from the end of the member,

$$R_n = (2.5k + N)F_{yw}t_w \tag{12.73}$$

where k is the distance from the outer face of the flange to the web toe of the fillet, in, and N is the length of the bearing, in. In this example, assuming a bearing length of 16 in, $R_n = [2.5({}^5\!/_{16}) + 16]36({}^7\!/_{16}) = 264$ kips $< R_u = 293.2$ kips. Therefore, bearing stiffeners are required to prevent localized yielding of web.

For web crippling,

$$R_u \le \phi_w R_n \tag{12.74}$$

where ϕ_w = resistance factor for web crippling = 0.80.
R_n = nominal resistance to the concentrated loading, kips

When $N/d > 0.2$,

$$R_n = 0.4t_w^2 \left[1+\left(\frac{4N}{d-0.2}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{EF_{yw}t_f/t_w} \tag{12.75}$$

In this example, $N/d = 16/60 = 0.27$ and

$$\begin{aligned} R_n &= 0.4(^7/_{16})^2\left[1+\left(\frac{4\times 16}{60-0.2}\right)\left(\frac{^7/_{16}}{1.50}\right)^{1.5}\right]\sqrt{29{,}000\times 36\times 1.50/(^7/_{16})} \\ &= 164 \text{ kips} < R_u = 293.2 \text{ kips} \end{aligned}$$

Thus, the web would be subjected to crippling if bearing stiffeners were not provided.

Bearing stiffeners should be welded or bolted on both sides of the web of rolled beams and plate girders, should extend the full depth of the web and, as closely as practical, to the outer edges of flanges. The width, b_t, of each projecting element should satisfy

$$b_t \le 0.48t_p\sqrt{\left(\frac{E}{F_{ys}}\right)} \tag{12.76}$$

where t_p = thickness of projecting element, in
F_{ys} = yield stress of stiffener, ksi

For the design of the stiffeners at the end bearings, try two stiffener plates, 9 in wide, welded to each side of the web. Calculate minimum required thickness from the following form of Eq. (12.76):

$$\begin{aligned} t_p &\ge \frac{b_t}{0.48\sqrt{E/F_{ys}}} \\ &\ge \frac{9}{0.48(28.38)} = 0.66 \text{ in} \end{aligned}$$

Try $t_p = {}^3/_4$ in. The factored bearing resistance, $(R_{sb})_r$, of the stiffeners is calculated from

$$(R_{sb})_r = \phi_b(R_{sb})_n = \phi_b 1.4\, A_{pn}F_{ys} \tag{12.77}$$

Assuming a 1-in-long clip at the stiffener base to clear the web-to-flange fillet weld,

$$\begin{aligned} A_{pn} &= 2(9.0-1.0)\times {}^3/_4 = 12.0 \text{ in}^2 \\ (R_{sb})_r &= 0.8\times 1.4\times 12.0\times 36.0 = 483 \text{ kips} > 293.2 \text{ kips} \qquad \text{OK} \end{aligned}$$

The factored axial resistance, P_r, is determined from

$$P_r = \phi_c P_n \tag{12.78}$$

where ϕ_c is the resistance factor for compression, 0.90, and P_n is the nominal compressive resistance, equal to the axial resistance of an equivalent column section that consists of the stiffener pair plus a centrally located strip of web extending $9t_w$ on each side of the stiffeners as shown in Fig. 12.59. The gross cross-sectional area of the equivalent column is $A_s = 2[(^7/_{16})3.94 + (^3/_4)9] = 16.94 \text{ in}^2$. The

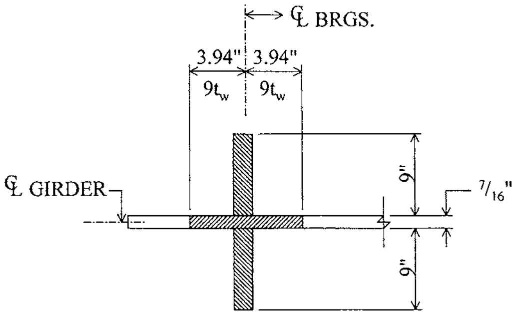


FIGURE 12.59 Equivalent column for bearing stiffener design.

moment of inertia of the equivalent column about centerline of web is $I_s = (^3\!/_4)(2 \times 9 + {}^7\!/_{16})^3/12 = 392$ in^4 and the radius of gyration is $r_s = \sqrt{(392/16.94)} = 4.8$ in.

The slenderness ratio must satisfy $KL/r < 120$, where the effective length factor, K, is 0.75 and $L = D$. Thus, $KL/r_s = 0.75(60)/4.8 = 9.4 < 120$—OK.

The nominal compressive resistance is as follows:

When $\lambda \le 2.25$,

$$P_n = 0.66^{\lambda} F_y A_s \tag{12.79}$$

When $\lambda > 2.25$,

$$P_n = \frac{0.88 F_y A_s}{\lambda} \tag{12.80}$$

where

$$\lambda = \left(\frac{KL}{r_s \pi}\right)^2 \frac{F_y}{E} \tag{12.81}$$

Thus, $\lambda = (9.4/\pi)^2(36/29{,}000) = 0.011 < 2.25$. From Eq. (12.79), $P_n = 0.66^{0.011}$ (36)(16.94) = 607 kips, and from Eq. (12.78), $P_r = 0.90(607) = 546$ kips > 293.2 kips—OK. Use two $^3\!/_4 \times 9$ in plates for bearing stiffeners.

The welds to the web must be capable of developing the entire reaction. Minimum size fillet weld for the $^3\!/_4$-in bearing stiffener is $^1\!/_4$ in as specified in AASHTO for base-metal thickness of thicker part joined ≥ $^3\!/_4$ in. Subtracting for top and bottom clips, the length available for each weld is $60 - 2 \times 2^1\!/_2 = 55$ in. For two stiffeners, there are $2 \times 2 = 4$ lines of weld. Total shear resistance that can be developed in welds is $R = 4 \times {}^1\!/_4 \times 0.707 \times 55 \times 33.60 = 1306$ kips > 293.2 kips—OK. Use $^1\!/_4$-in full-height fillet welds to connect bearing stiffeners to web.

Shear Connector Design. For shear connectors, as in the example of Art. 12.4, $^7\!/_8$-in-diameter by 6-in-long welded studs will be used to satisfy the AASHTO requirement that the ratio of the height to the diameter of a stud connector not be less than 4.0.

The fatigue resistance of an individual shear connector, Z_r, is

$$Z_r = \alpha d^2 \ge \frac{5.5d^2}{2} \tag{12.82}$$

where $\alpha = 34.5 - 4.28 \log N$
d = diameter of stud, in
N = number of cycles

The number of cycles is determined as follows (see Art. 10.9). The bridge is located on a major highway with an average daily truck traffic (ADTT) in one direction less than 2500. ADTT will be adjusted for single-lane truck traffic as specified in AASHTO, since no specific site information is available, as follows:

$$\text{ADTT}_{SL} = p \times \text{ADTT} \tag{12.83}$$

where p = fraction of truck traffic in a single lane = 0.85 for a bridge with two lanes available to trucks. Thus, $\text{ADTT}_{SL} = 0.85 \times 2500 = 2125$. Calculate N from

$$N = (365 \text{ days/year})(75 \text{ years})(n)\text{ADTT}_{SL} \tag{12.84}$$

where n = number of stress range cycles per truck passage = 1.0 for simple-span girders with $L >$ 40 ft. In this case, $N = 365 \times 75 \times 1.0 \times 2125 = 5.8 \times 10^7$ cycles. Then, calculate α as $\alpha = 34.5 - 4.28 \log(5.8 \times 10^7) = 1.27$. From Eq. (12.82), since $1.27 < 5.5/2$, $Z_r = 5.5d^2/2 = 5.5(^7/_8)^2/2 = 2.1$ kips/stud.

Determine the pitch, p (in), of shear connectors from

$$p \leq \frac{nZ_rI}{V_{sr}Q} \tag{12.85}$$

where n = number of shear connectors in a cross section
I = moment of inertia of composite section for short-term loads, in^4
Q = first moment of the transformed area of the slab about the neutral axis of the short-term composite section, in^3
V_{sr} = shear-force range under $LL + IM$ determined for the fatigue limit state, kips

The pitch should also satisfy

$$6d < p < 24 \text{ in} \tag{12.86}$$

which, for $d = ^7/_8$ in, becomes 5.25 in $< p <$ 24 in.

Assume 3 studs per row and substitute the following values in Eq. (12.85): $V_{sr} = 35.2$ kips as calculated before, $I = 128{,}260 \text{ in}^4$, and $Q = 106.3(21.92 - 8.5/2) = 1{,}878 \text{ in}^3$.

$$p \leq \frac{3 \times 2.1 \times 128{,}260}{35.2 \times 1{,}878}$$

$$\leq 12.2 \text{ in}$$

Thus, to satisfy the fatigue and fracture limit state, the longitudinal stud spacing must not exceed 12.2 in.

Check the transverse spacing of studs. The minimum transverse spacing of stud shear connectors is 4 stud diameters center-to-center and minimum clear distance between the edge of the top flange and the nearest shear stud is 1.0 in as required by AASHTO. Assuming 3 studs per row and setting the edge distance as 1.0 in, transverse stud spacing is calculated as $[16.0 - 2(1.0) - ^7/_8]/2 = 6^9/_{16}$ in $> 4.0(^7/_8) = 3^1/_2$ in—OK.

Next check the strength limit state. The factored resistance of shear connectors, Q_r, is

$$Q_r = \phi_{sc}Q_n \tag{12.87}$$

where ϕ_{sc} = resistance factor for shear connectors, 0.85, and Q_n is the nominal shear resistance of a single shear connector embedded in a concrete slab.

$$Q_n = 0.5A_{sc}\sqrt{(f'_cE_c)} \leq A_{sc}F_u \tag{12.88}$$

where A_{sc} = cross-sectional area of a stud shear connector, in^2
f'_c = minimum specified compressive strength of concrete, ksi
F_u = specified minimum tensile strength of a stud shear connector, 60 ksi
E_c = modulus of elasticity of concrete, ksi

Substitution of values gives $Q_n = 0.5 \times 0.60\ \sqrt{4.0 \times 3600}\ = 36.0 \leq 0.60 \times 60 = 36.0$ kips and $Q_r = 0.85 \times 36.0 = 30.6$ kips.

The number of shear connectors provided between the maximum positive-moment section and the zero-moment section (in a simple span) must not be less than

$$n = \frac{V_h}{Q_r} \tag{12.89}$$

The nominal horizontal shear force, V_h, is the lesser of the following:

$$V_h = 0.85\, f'_c b t_s \qquad \text{(concrete deck limit)} \tag{12.90}$$

$$V_h = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad \text{(steel girder limit)} \tag{12.91}$$

From Eq. (12.90), $V_h = 0.85 \times 4.0 \times 100 \times 8.5 = 2890$ kips, and from Eq. (12.91), $V_h = 36(60 \times {}^7\!/_{16} + 16 \times {}^3\!/_4 + 20 \times 1^1\!/_2) = 2457$ kips. Thus, $V_h = 2457$ kips. From Eq. (12.89), $n = 2457/30.6 = 81$ studs. With the studs placed in groups of three, there should be at least 27 rows of studs in each half of the girder. Then the average pitch is $p = 50$ ft/27 = 1.85 ft or 22.2 in. This is greater than the value of 12.2 in previously determined from the fatigue requirements and does not control.

Consequently, use three ${}^7\!/_8$-in-diameter by 6-in-long shear studs per row, spaced at 12 in.

12.15.9 Constructibility Checks

AASHTO LRFD does not allow reliance on postbuckling resistance for main load-carrying members during construction. At each critical construction stage, the section should be checked for flexure and shear.

Flexure Check. During construction, the compression flanges of composite sections are not usually continuously braced prior to the placement of the deck slab. Discretely braced compression flanges should satisfy conditions for flange nominal yielding, flexural resistance, and web bend buckling, as given by the following:

$$f_{bu} + f_l \le \phi_f R_h F_{yc} \tag{12.92}$$

$$f_{bu} + {}^1\!/_3 f_l \le \phi_f F_{nc} \tag{12.93}$$

$$f_{bu} \le \phi_f F_{crw} \tag{12.94}$$

Similarly, for discretely braced tension flanges, tension flange nominal yielding should be checked.

$$f_{bu} + f_l \le \phi_f R_h F_{yt} \tag{12.95}$$

Longitudinal tensile stresses in continuous composite deck due to factored loads should not exceed ϕf_r.

Terms are defined as follows, with values shown for this example:

f_{bu} = calculated stress in flange due to factored loads = 28.13 ksi (Table 12.69*c*)
f_l = lateral bending stress in flange, ksi = 0
ϕ_f = resistance factor for flexure = 1.00
F_{nc} = nominal flexure resistance of the flange = 36 ksi
F_{crw} = nominal bend-buckling resistance for webs, ksi
R_h = hybrid factor = 1.0 for homogenous built-up sections
F_{yt} = yield strength of tension flange = 36 ksi

Flange Nominal Yielding:

$$28.13 \text{ ksi} + 0 < 1.00 \times 1.0 \times 36 \text{ ksi} \qquad \text{or} \qquad 28.13 \text{ ksi} < 36 \text{ ksi} \qquad \text{OK}$$

Flexural Resistance. The flexural resistance is controlled by the local buckling resistance or the lateral torsional buckling resistance of the compression flange, whichever is smaller.

The local buckling resistance of the compression flange, F_{nc}, is given by

$$F_{nc} = R_b R_h F_{yc} \qquad \text{when } \lambda_f \le \lambda_{pf} \tag{12.96}$$

or, otherwise,

$$F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{fr} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} \tag{12.97}$$

where λ_f = compression flange slenderness ratio, $b_{fc}/2t_{fc}$
λ_{pf} = limiting slenderness ratio for compression flange, $0.38\sqrt{E/F_{yc}}$

For this example,

$$\lambda_f = \frac{16}{2 \times 0.75} = 10.7 < \lambda_{pf} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.8$$

Then, $F_{nc} = 1.0 \times 1.0 \times 36 = 36$ ksi, since the web load-shedding factor $R_b = 1.0$ for constructibility.

Determine the lateral torsional buckling resistance of the compression flange, F_{nc}, from the following. For $L_p < L_b \le L_r$,

$$F_{nc} = C_b\left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc} \tag{12.98}$$

where L_b = unbraced length, in, for this example 12.5 ft or 300 in
L_p = limiting unbraced length, in, $1.0 r_t \sqrt{E/F_{yc}}$
$L_r = \pi r_t \sqrt{E/F_{yc}}$, in
r_t = effective radius of gyration for lateral torsional buckling, in

$$r_t = \frac{b_{fc}}{\sqrt{12(1 + {}^1/_3(D_c t_w / b_{fc} t_{fc})}} \tag{12.99}$$

D_c = web depth in compression, in

$$D_c = \left(\frac{-f_c}{|f_c| + f_t}\right) d - t_{fc} \ge 0 \tag{12.100}$$

Using service limit state stresses for elastic range,

$$D_c = \left(\frac{25.37}{25.37 + 23.98}\right) 62.25 - 0.75 = 31.25 \text{ in}$$

$$r_t = \frac{16}{\sqrt{12(1 + {}^1/_3\ (31.25 \times {}^7/_{16})/(16 \times 0.75)}} = 3.93$$

$$L_p = 1.0 \times 3.93\sqrt{\frac{29{,}000}{36}} = 111.5 \text{ in}$$

$L_r = \pi r_t \sqrt{E/F_{yc}}$, where F_{yc} is the smaller of $0.7F_{yc} = 0.7 \times 36 = 25.2$ ksi or $F_{yw} = 36$ ksi, but not less than $0.5F_{yc} = 0.5 \times 36 = 18$ ksi.

$$L_r = \pi 3.93\sqrt{\frac{29{,}000}{25.2}} = 418 \text{ in}$$

Substituting these values in Eq. (12.98), and conservatively taking $C_p = 1.0F_{nc} = 29.4$ ksi, then 28.13 ksi + 0 < 1.00 × 29.4 ksi or 28.13 ksi < 29.4 ksi—OK (Eq. 12.93).

Web Bend Buckling. For webs without longitudinal stiffeners,

$$F_{crw} = \frac{0.9Ek}{(D/t_w)^2} \leq R_h F_{yc} \text{ or } \leq \frac{F_{yw}}{0.7}, \text{whichever is smaller} \tag{12.101}$$

where $k = 9/(D_c/D)^2$ and D_c is the depth of the web in compression in elastic range, in.

In this example, $k = 9/(38.17/60)^2 = 22.24$ and $D_c = 38.17$ in from Fig. 12.58. Thus, from Eq. (12.94), $F_{crw} = 0.9 \times 29{,}000 \times 22.24/137^2 = 30.9$ ksi ≤ 1.00 × 36 ksi, and 28.13 ksi < 30.9 ksi—OK.

Shear Check. For interior panels of the transversely stiffened web during the deck-construction sequence, the following shear requirement should be met:

$$V_u \leq \phi_f V_{cr} = \phi_f C V_p \tag{12.102}$$

where ϕ_v = resistance factor for shear = 1.00
V_u = shear due to factored permanent loads and factored construction loads on the noncomposite section, kips
V_{cr} = shear buckling resistance, ksi
C = ratio of shear buckling resistance to shear yield strength = 0.39 as determined in Art. 12.15.6
V_p = plastic shear force, kips, from $0.58F_{yw}Dt_w = 0.58 \times 36 \times 60 \times {}^7/_{16} = 548$ kips

Then $V_u = 1.00 \times 0.39 \times 548 = 213.7$ kips.

$$V_u = 103.1 \text{ kips} < 213.7 \text{ kips} \qquad \text{OK}$$

The trial section has satisfied the flexure, shear, fatigue and fracture, and constructibility requirements of the AASHTO LRFD specifications. For completeness, the need for temporary wind bracing during construction should also be investigated.

CHAPTER 13

TRUSS BRIDGES*

John M. Kulicki, Ph.D., P.E.
President, CEO, and Chief Engineer
Modjeski and Masters, Inc.
Harrisburg, Pennsylvania

Joseph E. Prickett, P.E.
Former Senior Associate
Modjeski and Masters, Inc.
Harrisburg, Pennsylvania

David H. LeRoy, P.E.
Former Vice President
Modjeski and Masters, Inc.
Harrisburg, Pennsylvania

A truss is a structure that acts like a beam but with major components, or members, subjected primarily to axial stresses. The members are arranged in triangular patterns. Ideally, the end of each member at a joint is free to rotate independently of the other members at the joint. If this does not occur, secondary stresses are induced in the members. Also if loads occur other than at panel points, or joints, bending stresses are produced in the members.

Though trusses were used by the ancient Romans, the modern truss concept seems to have been originated by Andrea Palladio, a sixteenth-century Italian architect. From his time to the present, truss bridges have taken many forms.

Early trusses might be considered variations of an arch. They applied horizontal thrusts at the abutments, as well as vertical reactions. In 1820, Ithiel Town patented a truss that can be considered the forerunner of the modern truss. Under vertical loading, the Town truss exerted only vertical forces at the abutments. But unlike modern trusses, the diagonals, or web systems, were of wood-lattice construction and chords were composed of two or more timber planks.

In 1830, Colonel Long of the U.S. Army Corps of Engineers patented a wood truss with a simpler web system. In each panel, the diagonals formed an X. The next major step came in 1840, when William Howe patented a truss in which he used wrought-iron tie rods for vertical web members, with X wood diagonals. This was followed by the patenting in 1844 of the Pratt truss with wrought-iron X diagonals and timber verticals.

The Howe and Pratt trusses were the immediate forerunners of numerous iron bridges. In a book published in 1847, Squire Whipple pointed out the logic of using cast iron in compression and wrought iron in tension. He constructed bowstring trusses with cast-iron verticals and wrought-iron X diagonals.

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*Revised and updated from "Truss Bridges" by Jack P. Shedd, Sec. 12 in the First Edition.

These trusses were statically indeterminate. Stress analysis was difficult. Later, simpler web systems were adopted, thus eliminating the need for tedious and exacting design procedures.

To eliminate secondary stresses due to rigid joints, early U.S. engineers constructed pin-connected trusses. European engineers used primarily rigid joints. Properly proportioned, the rigid trusses gave satisfactory service and eliminated the possibility of frozen pins, which induce stresses not usually considered in design. Experience indicated that rigid and pin-connected trusses were nearly equal in cost, except for long spans. Hence, modern design favors rigid joints.

Many early truss designs were entirely functional, with little consideration given to appearance. Truss members and other components seemed to lie in all possible directions and to have a variety of sizes, thus giving the impression of complete disorder. Yet the appearance of a bridge often can be improved with very little increase in construction cost. By the 1970s, many speculated that the cable-stayed bridge would entirely supplant the truss, except on railroads. But improved design techniques, including load factor design, and streamlined detailing have kept the truss viable. For example, some designs utilize Warren trusses without verticals. In some cases, sway frames are eliminated and truss-type portals are replaced with beam portals, resulting in an open appearance.

Because of the large number of older trusses still in the transportation system, some historical information in this chapter applies to those older bridges in an evaluation or rehabilitation context.

(H. J. Hopkins, “A Span of Bridges,” Praeger Publishers, New York; S. P. Timoshenko, “History of Strength of Materials,” McGraw-Hill, New York.)

13.1 SPECIFICATIONS

The design of truss bridges usually follows the specifications of the American Association of State Highway and Transportation Officials (AASHTO) or the Manual of the American Railway Engineering and Maintenance-of-Way Association (AREMA). (See Chaps. 10 and 11.) A transition in AASHTO specifications is currently being made from the “Standard Specifications for Highway Bridges,” Sixteenth Edition, to the “LRFD Specifications for Highway Bridges,” Second Edition. The “Standard Specification” covers service load design for truss bridges, and in addition, the “Guide Specification for the Strength Design of Truss Bridges” covers extension of the load factor design process permitted for girder bridges in the “Standard Specifications” to truss bridges. Where the “Guide Specification” is silent, applicable provisions of the “Standard Specification” apply.

To clearly identify which of the three AASHTO specifications are being referred to in this chapter, the following system will be adopted. If the provision under discussion applies to all the specifications, reference will simply be made to the “AASHTO Specifications.” Otherwise, the following notation will be observed:

“AASHTO SLD Specifications” refers to the service load provisions of “Standard Specifications for Highway Bridges.”

“AASHTO LFD Specifications” refers to “Guide Specification for the Strength Design of Truss Bridges.”

“AASHTO LRFD Specifications” refers to “LRFD Specifications for Highway Bridges.”

13.2 TRUSS COMPONENTS

Principal parts of a highway truss bridge are indicated in Fig. 13.1; those of a railroad truss are shown in Fig. 13.2.

Joints are intersections of truss members. Joints along upper and lower chords often are referred to as panel points. To minimize bending stresses in truss members, live loads generally are transmitted through floor framing to the panel points of either chord in older, shorter-span trusses. Bending stresses in members due to their own weight was often ignored in the past. In modern trusses, bending due to the weight of the members should be considered.

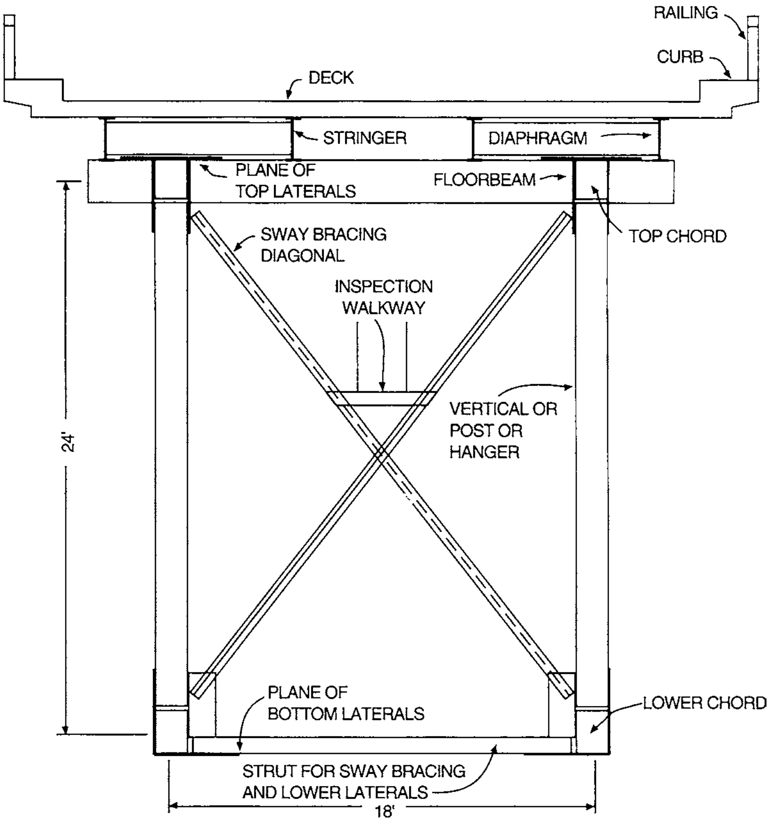


FIGURE 13.1 Cross section shows principal parts of a deck-truss highway bridge.

Chords are top and bottom members that act like the flanges of a beam. They resist the tensile and compressive forces induced by bending. In a constant-depth truss, chords are essentially parallel. They may, however, range in profile from nearly horizontal in a moderately variable-depth truss to nearly parabolic in a bowstring truss. Variable depth often improves economy by reducing stresses where chords are more highly loaded, around mid-span in simple-span trusses and in the vicinity of the supports in continuous trusses.

Web members consist of diagonals and also often of verticals. Where the chords are essentially parallel, diagonals provide the required shear capacity. Verticals carry shear, provide additional panel points for introduction of loads, and reduce the span of the chords under dead-load bending. When subjected to compression, verticals often are called posts, and when subjected to tension, hangers. Usually, deck loads are transmitted to the trusses through end connections of floorbeams to the verticals.

Counters, which are found on many older truss bridges still in service, are a pair of diagonals placed in a truss panel, in the form of an X, where a single diagonal would be subjected to stress reversals. Counters were common in the past in short-span trusses. Such short-span trusses are no longer economical and have been virtually totally supplanted by beam and girder spans. X pairs are still used in lateral trusses, sway frames, and portals, but are seldom designed to act as true counters, on the assumption that only one counter acts at a time and carries the maximum panel shear in tension. This implies that the companion counter takes little load because it buckles. In modern design, counters are seldom used in the primary trusses. Even in lateral trusses, sway frames, and portals, X-shaped trusses are usually comprised of rigid members, that is, members that will not buckle. If adjustable counters are used, only one may be placed in each truss panel, and it should have open

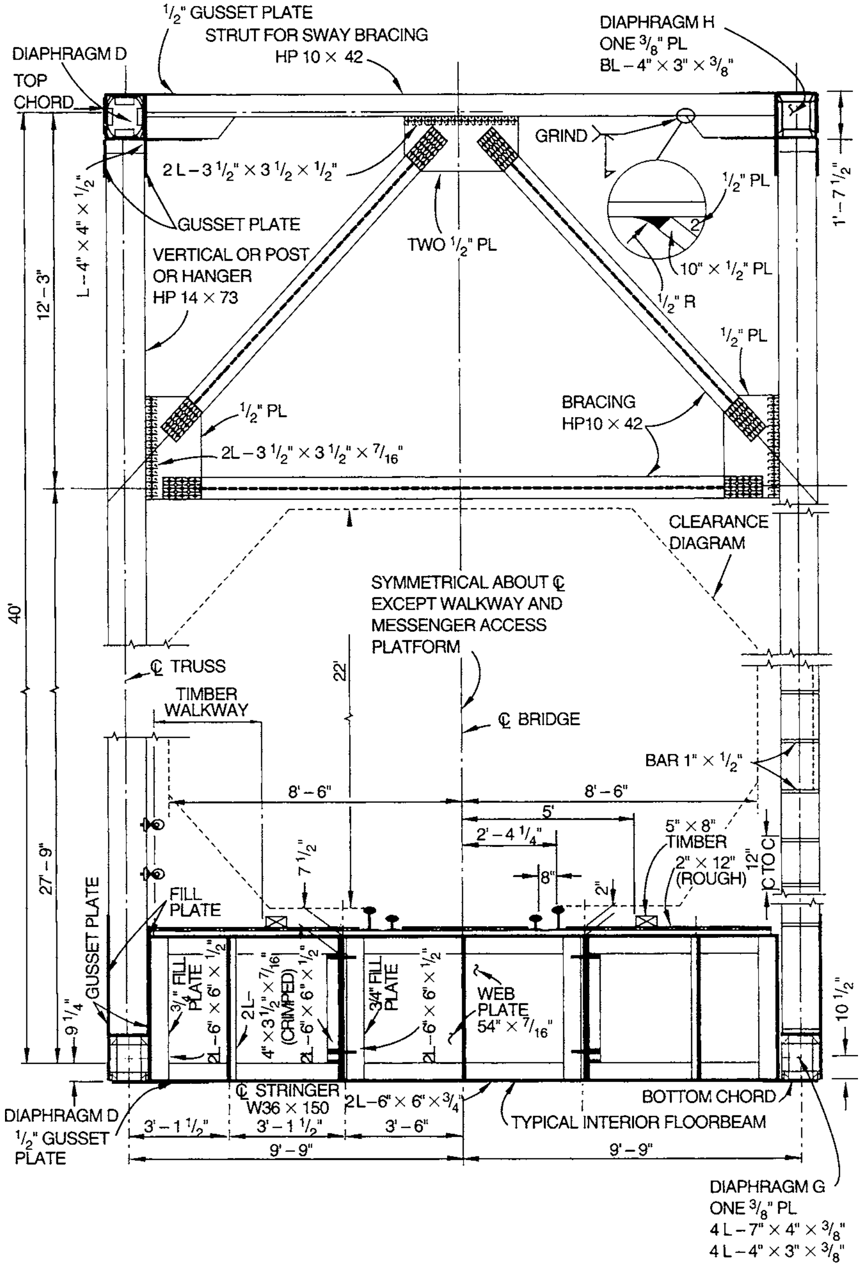


FIGURE 13.2 Cross section shows principal parts of a through-truss railway bridge.

turnbuckles. AASHTO LRFD specifies that counters should be avoided. The commentary to that provision contains reference to the historical initial force requirement of 10 kips. Design of such members by AASHTO SLD or LFD specifications should include an allowance of 10 kips for initial stress. Sleeve nuts and loop bars should not be used.

End posts are compression members at supports of simple-span trusses. Wherever practical, trusses should have inclined end posts. Laterally unsupported hip joints should not be used.

Working lines are straight lines between intersections of truss members. To avoid bending stresses due to eccentricity, the gravity axes of truss members should lie on working lines. Some eccentricity may be permitted, however, to counteract dead-load bending stresses. Furthermore, at joints, gravity axes should intersect at a point. If an eccentric connection is unavoidable, the additional bending caused by the eccentricity should be included in the design of the members utilizing appropriate interaction equations.

AASHTO Specifications require that members be symmetrical about the central plane of a truss. They should be proportioned so that the gravity axis of each section lies as nearly as practicable in its center.

Connections may be made with welds or high-strength bolts. AREMA practice, however, excludes field welding, except for minor connections that do not support live load.

The deck is the structural element providing direct support for vehicular loads. Where the deck is located near the bottom chords (through spans), it should be supported by only two trusses.

Floorbeams should be set normal or transverse to the direction of traffic. They and their connections should be designed to transmit the deck loads to the trusses.

Stringers are longitudinal beams, set parallel to the direction of traffic. They are used to transmit the deck loads to the floorbeams. If stringers are not used, the deck must be designed to transmit vehicular loads to the floorbeams.

Lateral bracing should extend between top chords and between bottom chords of the two trusses. This bracing normally consists of trusses placed in the planes of the chords to provide stability and lateral resistance to wind. Trusses should be spaced sufficiently far apart to preclude overturning by design lateral forces.

Sway bracing may be inserted between truss verticals to provide lateral resistance in vertical planes. Where the deck is located near the bottom chords, such bracing, placed between truss tops, must be kept shallow enough to provide adequate clearance for passage of traffic below it. Where the deck is located near the top chords, sway bracing should extend in full-depth of the trusses.

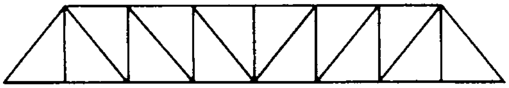
Portal bracing is sway bracing placed in the plane of end posts. In addition to serving the normal function of sway bracing, portal bracing also transmits loads in the top lateral bracing to the end posts (Art. 13.6).

Skewed bridges are structures supported on piers that are not perpendicular to the planes of the trusses. The **skew angle** is the angle between the transverse centerline of bearings and a line perpendicular to the longitudinal centerline of the bridge.

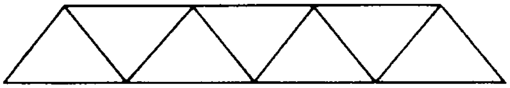
13.3 TYPES OF TRUSSES

Figure 13.3 shows some of the common trusses used for bridges. **Pratt trusses** have diagonals sloping downward toward the center and parallel chords (Fig. 13.3*a*). **Warren trusses,** with parallel chords and alternating diagonals, are generally, but not always, constructed with verticals (Fig. 13.3*c*) to reduce panel size. When rigid joints are used, such trusses are favored because they provide an efficient web system. Most modern bridges are of some type of Warren configuration.

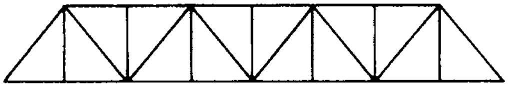
Parker trusses (Fig. 13.3*d*) resemble Pratt trusses but have variable depth. As in other types of trusses, the chords provide a couple that resists bending moment. With long spans, economy is improved by creating the required couple with less force by spacing the chords farther apart. The Parker truss, when simply supported, is designed to have its greatest depth at mid-span, where moment is a maximum. For greatest chord economy, the top-chord profile should approximate a parabola. Such a curve, however, provides too great a change in slope of diagonals, with some loss of economy in weights of diagonals. In practice, therefore, the top-chord profile should be set for the



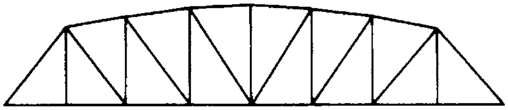
(a) PRATT



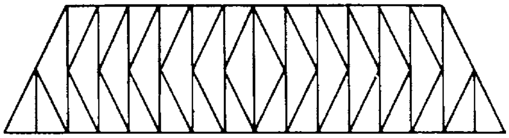
(b) WARREN-WITHOUT VERTICALS



(c) WARREN-WITH VERTICALS



(d) PARKER



(e) K TRUSS

FIGURE 13.3 Types of simple-span truss bridges.

greatest change in truss depth commensurate with reasonable diagonal slopes, for example, between 40° and 60° with the horizontal.

K trusses (Fig. 13.3*e*) permit deep trusses with short panels to have diagonals with acceptable slopes. Two diagonals generally are placed in each panel to intersect at midheight of a vertical. Thus, for each diagonal, the slope is half as large as it would be if a single diagonal were used in the panel. The short panels keep down the cost of the floor system. This cost would rise rapidly if panel width were to increase considerably with increase in span. Thus, K trusses may be economical for long spans, for which deep trusses and narrow panels are desirable. These trusses may have constant or variable depth.

Bridges also are classified as highway or railroad, depending on the type of loading the bridge is to carry. Because highway loading is much lighter than railroad loading, highway trusses generally are built of much lighter sections. Usually, highways are wider than railways, thus requiring wider spacing of trusses.

Trusses are also classified as to location of deck: deck, through, or half-through trusses. **Deck trusses** locate the deck near the top chord so that vehicles are carried above the chord. **Through trusses** place the deck near the bottom chord so that vehicles pass between the trusses. **Half-through trusses** carry the deck so high above the bottom chord that lateral and sway bracing cannot be placed between the top chords. The choice of deck or through construction normally is dictated by the economics of approach construction.

The absence of top bracing in half-through trusses calls for special provisions to resist lateral forces. AASHTO Specifications require that truss verticals, floorbeams, and their end connections be proportioned to resist a lateral force of at least 0.30 kip/lin ft, applied at the top-chord panel points of each truss. The top chord of a half-through truss should be designed as a column with elastic lateral supports at panel points. The critical buckling force of the column, so determined, should be at least 50% larger than the maximum force induced in any panel of the top chord by dead and live loads plus impact. Thus, the verticals have to be designed as cantilevers, with a concentrated load at top-chord level and rigid connection to a floorbeam. This system offers elastic restraint to buckling of the top chord. The analysis of elastically restrained compression members is covered in T. V. Galambos, "Guide to Stability Design Criteria for Metal Structures," Structural Stability Research Council, 1998.

13.4 BRIDGE LAYOUT

Trusses, offering relatively large-depth, open-web construction, and members subjected primarily to axial stress, provide large carrying capacity for comparatively small amounts of steel. For maximum economy in truss design, the area of metal furnished for members should be varied as often as required by the loads. To accomplish this, designers usually have to specify built-up sections that require considerable fabrication, which tend to offset some of the savings in steel.

Truss Spans. Truss bridges are generally comparatively easy to erect, because light equipment often can be used. Assembly of mechanically fastened joints in the field is relatively labor-intensive, which may also offset some of the savings in steel. Consequently, trusses seldom can be economical for highway bridges with spans less than about 450 ft.

Railroad bridges, however, involve different factors because of the heavier loading. Trusses generally are economical for railroad bridges with spans greater than 150 ft.

The current practical limit for simple-span trusses is about 800 ft for highway bridges and about 750 ft for railroad bridges. Some extension of these limits should be possible with improvements in materials and analysis, but as span requirements increase, cantilever or continuous trusses are more efficient. The North American span record for cantilever construction is 1600 ft for highway bridges and 1800 ft for railroad bridges.

For a bridge with several truss spans, the most economical pier spacing can be determined after preliminary designs have been completed for both substructure and superstructure. One guideline provides that the cost of one pier should equal the cost of one superstructure span, excluding the floor system. In trial calculations, the number of piers initially assumed may be increased or decreased by one, decreasing or increasing the truss spans. Cost of truss spans rises rapidly with increase in span. A few trial calculations should yield a satisfactory picture of the economics of the bridge layout. Such an analysis, however, is more suitable for approach spans than for main spans. In most cases, the navigation or hydraulic requirement is apt to unbalance costs in the direction of increased superstructure cost. Furthermore, girder construction is currently used for span lengths that would have required approach trusses in the past.

Panel Dimensions. To start economic studies, it is necessary to arrive at economic proportions of trusses so that fair comparisons can be made among alternatives. Panel lengths will be influenced by type of truss being designed. They should permit slope of the diagonals between 40° and 60° with the horizontal for economic design. If panels become too long, the cost of the floor system increases substantially and heavier dead loads are transmitted to the trusses. A subdivided truss becomes more economical under these conditions.

For simple-span trusses, experience has shown that a depth–span ratio of 1:5 to 1:8 yields economical designs. Some design specifications limit this ratio, with 1:10 a common historical limit. For continuous trusses with reasonable balance of spans, a depth–span ratio of 1:12 should be satisfactory. Because of the lighter live loads for highways, somewhat shallower depths of trusses may be used for highway bridges than for railway bridges.

Designers, however, do not have complete freedom in selection of truss depth. Certain physical limitations may dictate the depth to be used. For through-truss highway bridges, for example, it is impractical to provide a depth of less than 24 ft, because of the necessity of including suitable sway frames. Similarly, for through railway trusses, a depth of at least 30 ft is required. The trend toward double-stack cars encourages even greater minimum depths.

Once a starting depth and panel spacing have been determined, permutation of primary geometric variables can be studied efficiently by computer-aided design methods. In fact, preliminary studies have been carried out in which every primary truss member is designed for each choice of depth and panel spacing, resulting in a very accurate choice of those parameters.

Bridge Cross Sections. Selection of a proper bridge cross section is an important determination by designers. In spite of the large number of varying cross sections observed in truss bridges, actual selection of a cross section for a given site is not a large task. For instance, if a through highway truss were to be designed, the roadway width would determine the transverse spacing of trusses. The span and consequent economical depth of trusses would determine the floorbeam spacing, because the floorbeams are located at the panel points. Selection of the number of stringers and decisions as to whether to make the stringers simple spans between floorbeams or continuous over the floorbeams, and whether the stringers and floorbeams should be composite with the deck, complete the determination of the cross section.

Good design of framing of floor system members requires attention to details. In the past, many points of **stress relief** were provided in floor systems. Due to corrosion and wear resulting from use of these points of movement, however, experience with them has not always been good. Additionally, the relative movement that tends to occur between the deck and the trusses may lead to out-of-plane bending of floor system members and possible fatigue damage. Hence, modern detailing practice strives to eliminate small unconnected gaps between stiffeners and plates, rapid change in stiffness due to excessive flange coping, and other distortion fatigue sites. Ideally, the whole structure is made to act as a unit, thus eliminating distortion fatigue.

Deck trusses for highway bridges present a few more variables in selection of cross section. Decisions have to be made regarding the transverse spacing of trusses and whether the top chords of the trusses should provide direct support for the deck. Transverse spacing of the trusses has to be large enough to provide lateral stability for the structure. Narrower truss spacings, however, permit smaller piers, which will help the overall economy of the bridge.

Cross sections of railway bridges are similarly determined by physical requirements of the bridge site. Deck trusses are less common for railway bridges because of the extra length of approach grades often needed to reach the elevation of the deck. Also, use of through trusses offers an advantage if open-deck construction is to be used. With through trusses, only the lower chords are vulnerable to corrosion caused by salt and debris passing through the deck.

After preliminary selection of truss type, depth, panel lengths, member sizes, lateral systems, and other bracing, designers should review the appearance of the entire bridge. Esthetics can often be improved with little economic penalty.

13.5 DECK DESIGN

For most truss members, the percentage of total stress attributable to dead load increases as span increases. Because trusses are normally used for long spans, and a sizable portion of the dead load (particularly on highway bridges) comes from the weight of the deck, a lightweight deck is advantageous. It should be no thicker than actually required to support the design loading.

In the preliminary study of a truss, consideration should be given to the cost, durability, maintainability, inspectability, and replaceability of various deck systems, including transverse, longitudinal, and four-way reinforced concrete decks, orthotropic-plate decks, and concrete-filled or overlaid steel grids. Open-grid deck floors will seldom be acceptable for new fixed truss bridges but may be advantageous in rehabilitation of bridges and for movable bridges.

The design procedure for railroad bridge decks is almost entirely dictated by the proposed cross section. Designers usually have little leeway with the deck, because they are required to use standard railroad deck details wherever possible.

Deck design for a highway bridge is somewhat more flexible. Most highway bridges have a reinforced-concrete slab deck, with or without an asphalt wearing surface. Reinforced concrete decks may be transverse, longitudinal, or four-way slabs.

- Transverse slabs are supported on stringers spaced close enough so that all the bending in the slabs is in a transverse direction.
- Longitudinal slabs are carried by floorbeams spaced close enough so that all the bending in the slabs is in a longitudinal direction. Longitudinal concrete slabs are practical for short-span trusses where floorbeam spacing does not exceed about 20 ft. For larger spacing, the slab thickness becomes so large that the resultant dead load leads to an uneconomical truss design. Hence, longitudinal slabs are seldom used for modern trusses.
- Four-way slabs are supported directly on longitudinal stringers and transverse floorbeams. Reinforcement is placed in both directions. The most economical design has a spacing of stringers about equal to the spacing of floorbeams. This restricts use of this type of floor system to trusses with floorbeam spacing of about 20 ft. As for floor systems with a longitudinal slab, four-way slabs are generally uneconomical for modern bridges.

13.6 LATERAL BRACING, PORTALS, AND SWAY FRAMES

Lateral bracing should be designed to resist the following: (1) lateral forces due to wind pressure on the exposed surface of the truss and on the vertical projection of the live load; (2) seismic forces; (3) lateral forces due to centrifugal forces when the track or roadway is curved; (4) for railroad bridges, lateral forces due to the nosing action of locomotives caused by unbalanced conditions in the mechanism and also forces due to the lurching movement of cars against the rails because of the play between wheels and rails. Adequate bracing is one of the most important requirements for a good design.

Since the loadings given in design specifications only approximate actual loadings, it follows that refined assumptions are not warranted for calculation of panel loads on lateral trusses. The lateral forces may be applied to the windward truss only and divided between the top and bottom chords according to the area tributary to each. A lateral bracing truss is placed between the top chords or the bottom chords, or both, of a pair of trusses to carry these forces to the ends of the trusses.

Besides its use to resist lateral forces, other purposes of lateral bracing are to provide stability, stiffen structures, and prevent unwarranted lateral vibration. In deck-truss bridges, however, the floor system is much stiffer than the lateral bracing. Here, the major purpose of lateral bracing is to true-up the bridges and to resist wind load during erection.

The portal usually is a sway frame extending between a pair of trusses whose purpose also is to transfer the reactions from a lateral-bracing truss to the end posts of the trusses, and, thus, to the foundation. This action depends on the ability of the frame to resist transverse forces.

The portal is normally a statically indeterminate frame. Because the design loadings are approximate, an exact analysis is seldom warranted. It is normally satisfactory to make simplifying assumptions. For example, a plane of contraflexure may be assumed halfway between the bottom of the portal knee brace and the bottom of the post. The shear on the plane may be assumed divided equally between the two end posts.

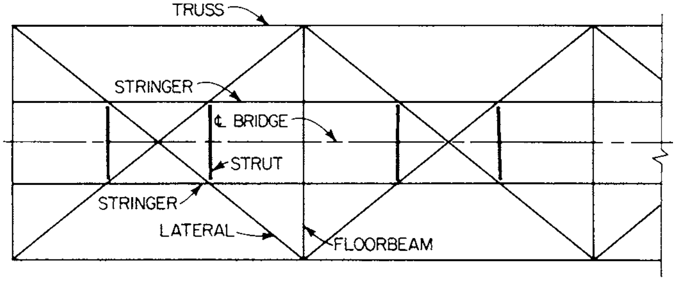
Sway frames are placed between trusses, usually in vertical planes, to stiffen the structure (Figs. 13.1 and 13.2). They should extend the full depth of deck trusses and should be made as deep as possible in through trusses. The AASHTO SLD Specifications require sway frames in every panel. But many bridges are serving successfully with sway frames in every other panel, even lift bridges whose alignment is critical. Some designs even eliminate sway frames entirely. The AASHTO LRFD Specifications makes the use and number of sway frames a matter of design concept as expressed in the analysis of the structural system.

Diagonals of sway frames should be proportioned for slenderness ratio as compression members. With an X system of bracing, any shear load may be divided equally between the diagonals. An approximate check of possible loads in the sway frame should be made to ensure that stresses are within allowable limits.

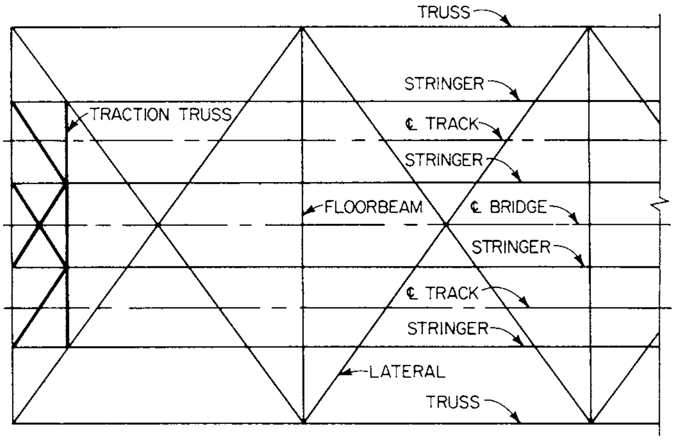
13.7 RESISTANCE TO LONGITUDINAL FORCES

Acceleration and braking of vehicular loads, and longitudinal wind, apply longitudinal loads to bridges. In highway bridges, the magnitudes of these forces are generally small enough that the design of main truss members is not affected. In railroad bridges, however, chords that support the floor system might have to be increased in section to resist tractive forces. In all truss bridges, longitudinal forces are of importance in design of truss bearings and piers.

In railway bridges, longitudinal forces resulting from accelerating and braking may induce severe bending stresses in the flanges of floorbeams, at right angles to the plane of the web, unless such forces are diverted to the main trusses by traction frames. In single-track bridges, a transverse strut may be provided between the points where the main truss laterals cross the stringers and are connected to them (Fig. 13.4*a*). In double-track bridges, it may be necessary to add a traction truss (Fig. 13.4*b*).



(a) SINGLE-TRACK BRIDGE



(b) DOUBLE-TRACK BRIDGE

FIGURE 13.4 Lateral bracing and traction trusses for resisting longitudinal forces on a truss bridge.

When the floorbeams in a double-track bridge are so deep that the bottoms of the stringers are a considerable distance above the bottoms of the floorbeams, it may be necessary to raise the plane of the main truss laterals from the bottom of the floorbeams to the bottom of the stringers. If this cannot be done, a complete and separate traction frame may be provided either in the plane of the tops of the stringers or in the plane of their bottom flanges.

The forces for which the traction frames are designed are applied along the stringers. The magnitudes of these forces are determined by the number of panels of tractive or braking force that are resisted by the frames. When one frame is designed to provide for several panels, the forces may become large, resulting in uneconomical members and connections.

13.8 TRUSS DESIGN PROCEDURE

The following sequence may serve as a guide to the design of truss bridges:

- Select span and general proportions of the bridge, including a tentative cross section.
- Design the roadway or deck, including stringers and floorbeams.
- Design upper and lower lateral systems.
- Design portals and sway frames.
- Design posts and hangers that carry little stress or loads that can be computed without a complete stress analysis of the entire truss.
- Compute preliminary moments, shears, and stresses in the truss members.
- Design the upper-chord members, starting with the most heavily stressed member.
- Design the lower-chord members.
- Design the web members.
- Recalculate the dead load of the truss and compute final moments and stresses in truss members.
- Design joints, connections, and details.
- Compute dead-load and live-load deflections.
- Check secondary stresses in members carrying direct loads and loads due to wind.
- Review design for structural integrity, esthetics, erection, and future maintenance and inspection requirements.

13.8.1 Analysis for Vertical Loads

Determination of member forces using conventional analysis based on frictionless joints is often adequate when the following conditions are met:

1. The plane of each truss of a bridge, the planes through the top chords, and the planes through the bottom chords are fully triangulated.
2. The working lines of intersecting truss members meet at a point.
3. Cross frames and other bracing prevent significant distortions of the box shape formed by the planes of the truss described above.
4. Lateral and other bracing members are not cambered; i.e., their lengths are based on the final dead-load position of the truss.
5. Primary members are cambered by making them either short or long by amounts equal to, and opposite in sign to, the axial compression or extension, respectively, resulting from dead-load stress. Camber for trusses can be considered as a correction for dead-load deflection. (If the original design provided excess vertical clearance and the engineers did not object to the sag, then

trusses could be constructed without camber. Most people, however, object to sag in bridges.) The cambering of the members results in the truss being out of vertical alignment until all the dead loads are applied to the structure (geometric condition).

When the preceding conditions are met and are rigorously modeled, three-dimensional computer analysis yields about the same dead-load axial forces in the members as the conventional pin-connected analogy and small secondary moments resulting from the self-weight bending of the member. Application of loads other than those constituting the geometric condition, such as live load and wind, will result in sag due to stressing of both primary and secondary members in the truss.

Rigorous three-dimensional analysis has shown that virtually all the bracing members participate in live-load stresses. As a result, total stresses in the primary members are reduced below those calculated by the conventional two-dimensional pin-connected truss analogy. Since trusses are usually used on relatively long-span structures, the dead-load stress constitutes a very large part of the total stress in many of the truss members. Hence, the savings from use of three-dimensional analysis of the live-load effects will usually be relatively small. This holds particularly for through trusses where the eccentricity of the live load and, therefore, forces distributed in the truss by torsion are smaller than for deck trusses.

The largest secondary stresses are those due to moments produced in the members by the resistance of the joints to rotation. Thus, the secondary stresses in a pin-connected truss are theoretically less significant than those in a truss with mechanically fastened or welded joints. In practice, however, pinned joints always offer frictional resistance to rotation, even when new. If pin-connected joints freeze because of dirt, or rust, secondary stresses might become higher than those in a truss with rigid connections. Three-dimensional analysis will, however, quantify secondary stresses, if joints and framing of members are accurately modeled. If the secondary stress exceeds 4 ksi for tension members or 3 ksi for compression members, both the AASHTO SLD and LFD Specifications require that excess be treated as a primary stress. The AASHTO LRFD Specifications take a different approach, including:

- A requirement to detail the truss so as to make secondary force effects as small as practical
- A requirement to include the bending caused by member self-weight, as well as moments resulting from eccentricities of joint or working lines
- Relief from including both secondary force effects from joint rotation and floorbeam deflection if the component being designed is more than 10 times as long as it is wide in the plane of bending

When the working lines through the centroids of intersecting members do not intersect at the joint, or where sway frames and portals are eliminated for economic or esthetic purposes, the state of bending in the truss members, as well as the rigidity of the entire system, should be evaluated by a more rigorous analysis than the conventional.

The attachment of floorbeams to truss verticals produces out-of-plane stresses, which should be investigated in highway bridges and must be accounted for in railroad bridges, due to the relatively heavier live load in that type of bridge. An analysis of a frame composed of a floorbeam and all the truss members present in the cross section containing the floorbeam is usually adequate to quantify this effect.

Deflection of trusses occurs whenever there are changes in length of the truss members. These changes may be due to strains resulting from loads on the truss, temperature variations, or fabrication effects or errors. Methods of computing deflections are similar in all three cases. Prior to the introduction of computers, calculation of deflections in trusses was a laborious procedure and was usually determined by energy or virtual work methods or by graphical or semigraphical methods, such as the Williot-Mohr diagram. With the widespread availability of matrix structural analysis packages, the calculation of deflections and analysis of indeterminant trusses are speedily executed.

13.8.2 Analysis for Wind Loads

The areas of trusses exposed to wind normal to their longitudinal axis are computed by multiplying widths of members as seen in elevation by the lengths center to center of intersections. The overlapping areas at intersections are assumed to provide enough surplus to allow for the added areas of gussets. The AREMA Manual specifies that for railway bridges this truss area be multiplied by the number of trusses, on the assumption that the wind strikes each truss fully (except where the leeward trusses are shielded by the floor system). The AASHTO Specifications require that the area of the trusses and floor as seen in elevation be multiplied by a wind pressure that accounts for 1½ times this area being loaded by wind.

The area of the floor should be taken as that seen in elevation, including stringers, deck, railing, and railing pickets.

AREMA specifies that when there is no live load on the structure, the wind pressure should be taken as at least 50 lb/ft^2, which is equivalent to a wind velocity of about 125 mi/h. When live load is on the structure, reduced wind pressures are specified for the trusses plus full wind load on the live load: 30 lb/ft^2 on the bridge, which is equivalent to a 97-mi/h wind, and 300 lb/lin ft on the live load on one track applied 8 ft above the top of the rail.

AASHTO SLD Specifications require a wind pressure on the structure of 75 lb/ft^2. Total force, lb/lin ft, in the plane of the windward chords should be taken as at least 300 and, in the plane of the leeward chords, at least 150. When live load is on the structure, these wind pressures can be reduced 70% and combined with a wind force of 100 lb/lin ft on the live load applied 6 ft above the roadway. The AASHTO LFD Specifications do not expressly address wind loads, so the SLD Specifications pertain by default.

Article 3.8 of the AASHTO LRFD Specifications establishes wind loads consistent with the format and presentation currently used in meteorology. Wind pressures are related to a base wind velocity V_B of 100 mi/h, as was common in past specifications. If no better information is available, the wind velocity at 30 ft above the ground, V_{30}, may be taken as equal to the base wind V_B. The height of 30 ft was selected to exclude ground effects in open terrain. Alternatively, the base wind speed may be taken from basic wind-speed charts available in the literature, or site-specific wind surveys may be used to establish V_{30}.

At heights above 30 ft, the design wind velocity V_{DZ}, mi/h, on a structure at a height Z, ft, may be calculated based on characteristic meteorology quantities related to the terrain over which the winds approach as follows. Select the friction velocity V_0 and friction length Z_0 from Table 13.1. Then calculate the velocity from

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right) \tag{13.1}$$

If V_{30} is taken equal to the base wind velocity V_B then V_{30}/V_B is taken as unity. The correction for structure elevation included in Eq. (13.1), which is based on current meteorological data, replaces the ¹⁄₇ power rule used in the past.

For design, Table 13.2 gives the base pressure P_B, kips/ft^2, acting on various structural components for a base wind velocity of 100 mi/h. The design wind pressure P_D, kips/ft^2, for the design wind velocity V_{DZ}, mi/h, is calculated from

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 \tag{13.2}$$

Additionally, minimum design wind pressures, comparable to those in the AASHTO SLD Specification, are given in the LRFD Specifications.

AASHTO Specifications also require that wind pressure be applied to vehicular live load.

TABLE 13.1 Basic Wind Parameters

	Terrain		
	Open country	Suburban	City
V_0, mi/h	8.20	10.9	12.0
Z_0, ft	0.23	3.28	8.20

TABLE 13.2 Base Pressures, P_B, for Base Wind Velocity, V_B, of 100 mi/h

Structural component	Windward load, kips/ft^2	Leeward load, kips/ft^2
Trusses, columns, and arches	0.050	0.025
Beams	0.050	NA
Large flat surfaces	0.040	NA

Wind Analysis. Wind analysis is typically carried out with the aid of computers with a space truss and some frame members as a model. It is helpful, and instructive, to employ a simplified, noncomputer method of analysis to compare with the computer solution to expose major modeling errors that are possible with space models. Such a simplified method is presented in the following.

Idealized Wind-Stress Analysis of a Through Truss with Inclined End Posts. The wind loads computed as indicated above are applied as concentrated loads at the panel points.

A through truss with parallel chords may be considered as having reactions to the top lateral bracing system only at the main portals. The effect of intermediate sway frames, therefore, is ignored. The analysis is applied to the bracing and to the truss members.

The lateral bracing members in each panel are designed for the maximum shear in the panel resulting from treating the wind load as a moving load; that is, as many panels are loaded as necessary to produce maximum shear in that panel. In design of the top-chord bracing members, the wind load, without live load, usually governs. The span for top-chord bracing is from hip joint to hip joint. For the bottom-chord members, the reduced wind pressure usually governs because of the considerable additional force that usually results from wind on the live load.

For large trusses, wind stress in the trusses should be computed for both the maximum wind pressure without live load and for the reduced wind pressure with live load and full wind on the live load. Because wind on the live load introduces an effect of "transfer," as described later, the following discussion is for the more general case of a truss with the reduced wind pressure on the structure and with wind on the live load applied 8 ft above the top of rail, or 6 ft above the deck.

The effect of wind on the trusses may be considered to consist of three additive parts:

1. **Chord stresses** in the fully loaded top and bottom lateral trusses.
2. **Horizontal component,** which is a uniform force of tension in one truss bottom chord and compression in the other bottom chord, resulting from transfer of the top lateral end reactions down the end portals. This may be taken as the top lateral end reaction times the horizontal distance from the hip joint to the point of contraflexure divided by the spacing between main trusses. It is often conservatively assumed that this point of contraflexure is at the end of span, and, thus, the top lateral end reaction is multiplied by the panel length, divided by the spacing between main trusses. Note that this convenient assumption does not apply to the design of portals themselves.
3. **Transfer stresses** created by the moment of wind on the live load and wind on the floor. This moment is taken about the plane of the bottom lateral system. The wind force on live load and wind force on the floor in a panel length is multiplied by the height of application above the bracing plane and divided by the distance center to center of trusses to arrive at a total vertical panel load. This load is applied downward at each panel point of the leeward truss and upward at each panel point of the windward truss. The resulting stresses in the main vertical trusses are then computed.

The total wind stress in any main truss member is arrived at by adding all three effects: chord stresses in the lateral systems, horizontal component, and transfer stresses.

Although this discussion applies to a parallel-chord truss, the same method may be applied with only slight error to a truss with curved top chord by considering the top chord to lie in a horizontal plane between hip joints, as shown in Fig. 13.5. The nature of this error will be described in the following.

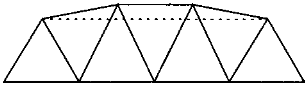


FIGURE 13.5 Top chord in a horizontal plane approximates a curved top chord.

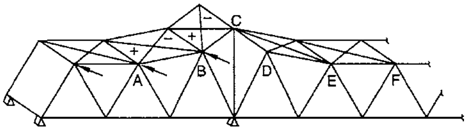


FIGURE 13.6 Wind on a cantilever truss with curved top chord is resisted by the top lateral system.

Wind Stress Analysis of Curved-Chord Cantilever Truss. The additional effects that should be considered in curved-chord trusses are those of the vertical components of the inclined bracing members. These effects may be illustrated by the behavior of a typical cantilever bridge, several panels of which are shown in Fig. 13.6.

As transverse forces are applied to the curved top lateral system, the transverse shear creates stresses in the top lateral bracing members. The longitudinal and vertical components of these bracing stresses create wind stresses in the top chords and other member of the main trusses. The effects of these numerous components of the lateral members may be determined by the following simple method:

- Apply the lateral panel loads to the **horizontal projection** of the top-chord lateral system and compute all **horizontal components** of the chord stresses. The stresses in the inclined chords may readily be computed from these horizontal components.
- Determine at every point of slope change in the top chord all the vertical forces acting on the point from both bracing diagonals and bracing chords. Compute the truss stresses in the vertical main trusses from those forces.
- The final truss stresses are the sum of the two contributions above and also of any transfer stress, and of any horizontal component delivered by the portals to the bottom chords.

13.8.3 Computer Determination of Wind Stresses

For computer analysis, the structural model is a three-dimensional framework composed of all the load-carrying members. Floorbeams are included if they are part of the bracing system or are essential for the stability of the structural model.

All wind-load concentrations are applied to the framework at braced points. Because the wind loads on the floor system and on the live load do not lie in a place of bracing, these loads must be “transferred” to a plane of bracing. The accompanying vertical required for equilibrium also should be applied to the framework.

Inasmuch as significant wind moments are produced in open-framed portal members of the truss, flexural rigidity of the main-truss members in the portal is essential for stability. Unless the other framework members are released for moment, the computer analysis will report small moments in most members of the truss.

With cantilever trusses, it is a common practice to analyze the suspended span by itself and then apply the reactions to a second analysis of the anchor and cantilever arms.

Some consideration of the rotational stiffness of piers about their vertical axis is warranted for those piers that support bearings that are fixed against longitudinal translation. Such piers will be subjected to a moment resulting from the longitudinal forces induced by lateral loads. If the stiffness (or flexibility) of the piers is not taken into account, the sense and magnitude of chord forces may be incorrectly determined.

13.8.4 Wind-Induced Vibration of Truss Members

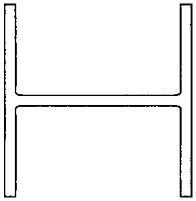
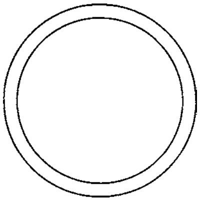
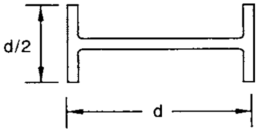
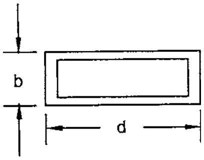
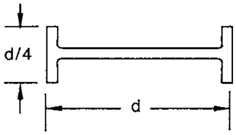
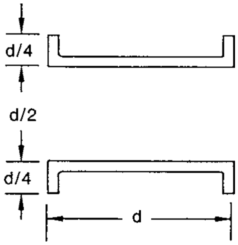
When a steady wind passes by an obstruction, the pressure gradient along the obstruction causes eddies or vortices to form in the wind stream. These occur at stagnation points located on opposite sides of the obstruction. As a vortex grows, it eventually reaches a size that cannot be tolerated by the wind stream and is torn loose and carried along in the wind stream. The vortex at the opposite stagnation point then grows until it is shed. The result is a pattern of essentially equally spaced (for small distances downwind of the obstruction) and alternating vortices called the "vortex street' or "von Karman trail." This vortex street is indicative of a pulsating periodic pressure change applied to the obstruction. The frequency of the vortex shedding and, hence, the frequency of the pulsating pressure, is given by

$$f = \frac{VS}{D} \tag{13.3}$$

where V is the wind speed, ft/s, D is a characteristic dimension, ft, and S is the Strouhal number, the ratio of velocity of vibration of the obstruction to the wind velocity (Table 13.3).

When the obstruction is a member of a truss, self-exciting oscillations of the member in the direction perpendicular to the wind stream may result when the frequency of vortex shedding coincides with a natural frequency of the member. Thus, determination of the torsional frequency and bending frequency in the plane perpendicular to the wind and substitution of those frequencies into Eq. (13.3) leads to an estimate of wind speeds at which resonance may occur. Such vibration has led to fatigue cracking of some truss and arch members, particularly cable hangers and I-shaped members. The preceding proposed use of Eq. (13.3) is oriented toward guiding designers in providing sufficient

TABLE 13.3 Strouhal Number for Various Sections*

Wind direction	Profile	Strouhal number S	Profile	Strouhal number S
→		0.120		0.200
↓		0.137		
↓		0.144		
↓		0.145	b/d	
			2.5	0.060
			2.0	0.080
			1.5	0.103
			1.0	0.133
↓		0.147	0.7	0.136
			0.5	0.138

*As given in "Wind Forces on Structures," *Transactions*, vol. 126, part II, p. 1180, American Society of Civil Engineers.

stiffness to reasonably preclude vibrations. It does not directly compute the amplitude of vibration and, hence, it does not lead directly to determination of vibratory stresses. Solutions for amplitude are available in the literature. See, for example, M. Paz, *Structural Dynamics: Theory and Computation*, John Wiley & Sons, New York, 1979; R. J. Melosh and H. A. Smith, "New Formulation for Vibration Analysis," *ASCE Journal of Engineering Mechanics,* vol. 115, no. 3, March 1989.

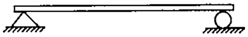
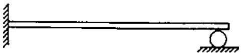
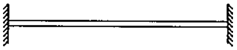
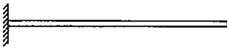
C. C. Ulstrup, in "Natural Frequencies of Axially Loaded Bridge Members," *ASCE Journal of the Structural Division*, vol. 104, pp. 357–364, 1978, proposed the following approximate formula for estimating bending and torsional frequencies for members whose shear center and centroid coincide:

$$f_n = \frac{a}{2\pi}\left(\frac{k_n L}{L}\right)^2 \left[1 + \epsilon_p \left(\frac{KL}{\pi}\right)^2\right]^{1/2} \tag{13.4}$$

where f_n = natural frequency of member for each mode corresponding to $n = 1, 2, 3, \ldots$
$k_n L$ = eigenvalue for each mode (see Table 13.4)
K = effective length factor (see Table 13.4)
L = length of the member, in
I = moment of inertia, in^4, of the member cross section
a = coefficient dependent on the physical properties of the member
$= \sqrt{EIg/\gamma A}$ for bending
$= \sqrt{EC_w g/\gamma I_p}$ for torsion
ϵ_p = coefficient dependent on the physical properties of the member
$= P/EI$ for bending
$= (GJA + PI_p)/AEC_w$ for torsion
E = Young's modulus of elasticity, psi
G = shear modulus of elasticity, psi
γ = weight density of member, lb/in^3
g = gravitational acceleration, in/s^2
P = axial force (tension is positive), lb
A = area of member cross section, in^2
C_w = warping constant
J = torsion constant
I_p = polar moment of inertia, in^4

In design of a truss member, the frequency of vortex shedding for the section is set equal to the bending and torsional frequency and the resulting equation is solved for the wind speed V. This is the wind speed at which resonance occurs. The design should be such that V exceeds by a reasonable margin the velocity at which the wind is expected to occur uniformly.

TABLE 13.4 Eigenvalue $k_n L$ and Effective Length Factor K

	$k_n L$			K		
Support condition	$n = 1$	$n = 2$	$n = 3$	$n = 1$	$n = 2$	$n = 3$
	π	2π	3π	1.000	0.500	0.333
	3.927	7.069	10.210	0.700	0.412	0.292
	4.730	7.853	10.996	0.500	0.350	0.259
	1.875	4.694	7.855	2.000	0.667	0.400

13.8.5 Fracture Criticality

Conventional wisdom says that failure of one primary member will cause collapse of a truss. This derives from the assumption of pin-connected joints and two-dimensional design approaches. However, a well-detailed truss is really a tubular structure. All bracing members, the floor system, and moment continuity provided by modern joint detailing can be sources of load-carrying capacity in the event of failure of one member. The anecdotal history of truss bridges contains many illustrations of survival despite damaged or destroyed members.

One way to address the question of fracture criticality is through rigorous three-dimensional analysis. The criteria for performing a refined analysis to demonstrate that part of a structure is not fracture-critical has not yet been fully codified. Therefore, the loading cases to be studied, location of potential damage, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of models and choice of element type should all be agreed on by the owner and the engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered, and the choice of software should be mutually agreed on by the owner and the engineer. Relief from the full factored loads associated with the conventional design-load combinations should be considered, as should the number of loaded design lanes versus the number of striped traffic lanes.

While difficult to quantify, the use of high-performance steel and the associated welding techniques can add further robustness to truss bridges.

13.9 TRUSS MEMBER DETAILS

The following shapes for truss members are typically considered:

H sections, made with two side segments (composed of angles or plates) with solid web, perforated web, or web of stay plates and lacing. Modern bridges almost exclusively use H sections made of three plates welded together.

Channel sections, made with two angle segments, with solid web, perforated web, or web of stay plates and lacing. These are seldom used on modern bridges.

Single box sections, made with side channels, beams, angles and plates, or side segments of plates only. The side elements may be connected top and bottom with solid plates, perforated plates, or stay plates and lacing. Alternatively, they may be connected at the top with solid cover plates and at the bottom with perforated plates, or stay plates and lacing. Modern bridges use primarily four-plate welded box members. The cover plates are usually solid, except for access holes for bolting joints.

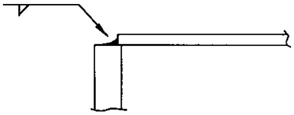
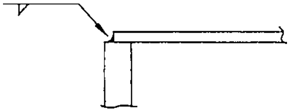
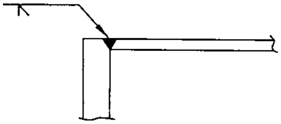
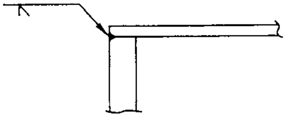
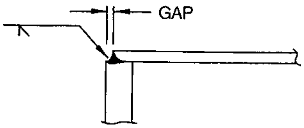
Double box sections, made with side channels, beams, angles and plates, or side segments of plates only. The side elements may be connected together with top and bottom perforated cover plates, or stay plates and lacing.

To obtain economy in member design, it is important to vary the area of steel in accordance with variations in total loads on the members. The variation in cross section plus the use of appropriate-strength grades of steel permit designers to use essentially the weight of steel actually required for the load on each panel, thus assuring an economical design.

With respect to shop fabrication of welded members, the H shape usually is the most economical section. It requires four fillet welds and no expensive edge preparation. Requirements for elimination of vortex shedding, however, may offset some of the inherent economy of this shape.

Box shapes generally offer greater resistance to vibration due to wind, to buckling in compression, and to torsion, but require greater care in selection of welding details. For example, various types of welded cover-plate details for boxes considered in design of the second Greater New Orleans Bridge and reviewed with several fabricators resulted in the observations in Table 13.5.

TABLE 13.5 Various Welded Cover-Plate Designs for Second Greater New Orleans Bridge

	Conventional detail. Has been used extensively in the past. It may be susceptible to lamellar tearing under lateral or torsional loads.
	Overlap increases for thicker web plate. Cover plate tends to curve up after welding.
	Very difficult to hold out-to-out dimension of webs, due to thickness tolerance of the web plates. Groove weld is expensive, but easier to develop cover plate within the connection to gusset plate.
	The detail requires a wide cover plate and tight tolerance of the cover-plate width. With a large overlap, the cover may curve up after welding. Groove weld is expensive, but easier to develop cover plate within the connection to the gusset plate.
	Same as above, except the fabrication tolerance, which will be better with this detail.

Additional welds placed inside a box member for development of the cover plate within the connection to the gusset plate are classified as AASHTO Category E at the termination of the inside welds and should be not be used. For development of the cover plate within the gusset-plate connection, groove welds, large fillet welds, large gusset plates, or a combination of the last two should be used.

Tension Members. Where practical, tension members should be arranged so that there will be no bending in the members from eccentricity of the connections. If this is possible, then the total stress can be considered uniform across the entire net area of the member. At a joint, the greatest practical proportion of the member surface area should be connected to the gusset or other splice material.

Designers have a choice of a large variety of sections suitable for tension members, although box and H-shaped members are typically used. The choice will be influenced by the proposed type of fabrication and range of areas required for tension members. The design should be adjusted to take full advantage of the selected type. For example, welded plates are economical for tubular or box-shaped members. Structural tubing is available with almost 22 in^2 of cross-sectional area and might be advantageous in welded trusses of moderate spans. For longer spans, box-shape members can be shop-fabricated with almost unlimited areas.

Tension members for bolted trusses involve additional considerations. For example, only 50% of the unconnected leg of an angle or tee is commonly considered effective, because of the eccentricity of the connection to the gusset plate at each end.

To minimize the loss of section for fastener holes and to connect into as large a proportion of the member surface area as practical, it is desirable to use a staggered fastener pattern. In Fig. 13.7, which shows a plate with staggered holes, the net width along chain 1-1 equals plate width W, minus three hole diameters. The net width along chain 2-2 equals W, minus five hole diameters, plus the quantity $S^2/4g$ for each of four gages, where S is the pitch and g is the gage.

Compression Members. Compression members should be arranged to avoid bending in the member from eccentricity of connections. Though the members may contain fastener holes, the gross area

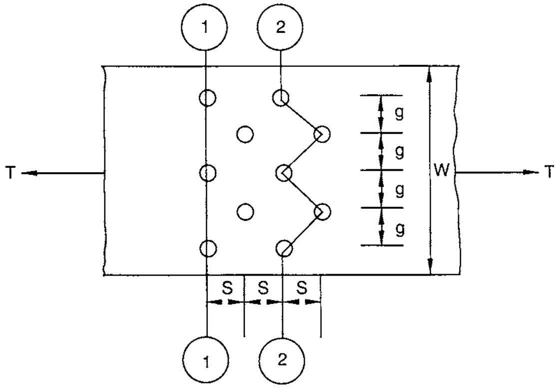


FIGURE 13.7 Chains of bolt holes used for determining the net section of a tension member.

may be used in design of such columns, on the assumption that the body of the fastener fills the hole. Welded box and H-shaped members are typically used for compression members in trusses.

Compression members should be so designed that the main elements of the section are connected directly to gusset plates, pins, or other members. It is desirable that member components be connected by solid webs. Care should be taken to ensure that the criteria for slenderness ratios, plate buckling, and fastener spacing are satisfied.

Posts and Hangers. Posts and hangers are the vertical members in truss bridges. A post in a Warren deck truss delivers the load from the floorbeam to the lower chord. A hanger in a Warren through truss delivers the floorbeam load to the upper chord.

Posts are designed as compression members. The posts in a single-truss span are generally made identical. At joints, overall dimensions of posts have to be compatible with those of the top and bottom chords to make a proper connection at the joint.

Hangers are designed as tension members. Although wire ropes or steel rods could be used, they would be objectionable for esthetic reasons. Furthermore, to provide a slenderness ratio small enough to maintain wind vibration within acceptable limits will generally require rope or rod area larger than that needed for strength.

Truss-Member Connections. Main truss members should be connected with gusset plates and other splice material, although pinned joints may be used where the size of a bolted joint would be prohibitive. To avoid eccentricity, fasteners connecting each member should be symmetrical about the axis of the member. It is desirable that fasteners develop the full capacity of each element of the member. Thickness of a gusset plate should be adequate for resisting shear, direct stress, and flexure at critical sections where these stresses are maximum. Reentrant cuts should be avoided; however, curves made for appearance are permissible.

13.10 MEMBER AND JOINT DESIGN EXAMPLES—LFD AND SLD

Design of a truss member by the AASHTO LFD and SLD Specifications is illustrated in the following examples, The design includes a connection in a Warren truss in which splicing of a truss chord occurs within a joint. Some designers prefer to have the chord run continuously through the joint and be spliced adjacent to the joint. Satisfactory designs can be produced using either approach. Chords of trusses that do not have a diagonal framing into each joint, such as a Warren truss, are usually continuous through joints with a post or hanger. Thus, many of the chord members are usually two panels long. Because of limitations on plate size and length for shipping, handling, or fabrication, it is sometimes necessary, however, to splice the plates within the length of a member. Where this is necessary, common practice is to offset the splices in the plates so that only one plate is spliced at any cross section.

13.10.1 Load Factor Design of Truss Chord

A chord of a truss is to be designed to withstand a factored compression load of 7878 kips and a factored tensile load of 1748 kips. Corresponding service loads are 4422 kips compression and 391 kips tension. The structural steel is to have a specified minimum yield stress of 36 ksi. The member is 46 ft long and the slenderness factor K is to be taken as unity. A preliminary design yields the cross section shown in Fig. 13.8. The section has the following properties:

A_g = gross area = 281 in^2
I_{gx} = gross moment of inertia with respect to x axis = 97,770 in^4
I_{gy} = gross moment of inertia with respect to y axis = 69,520 in^4
w = weight per linear foot = 0.98 kip

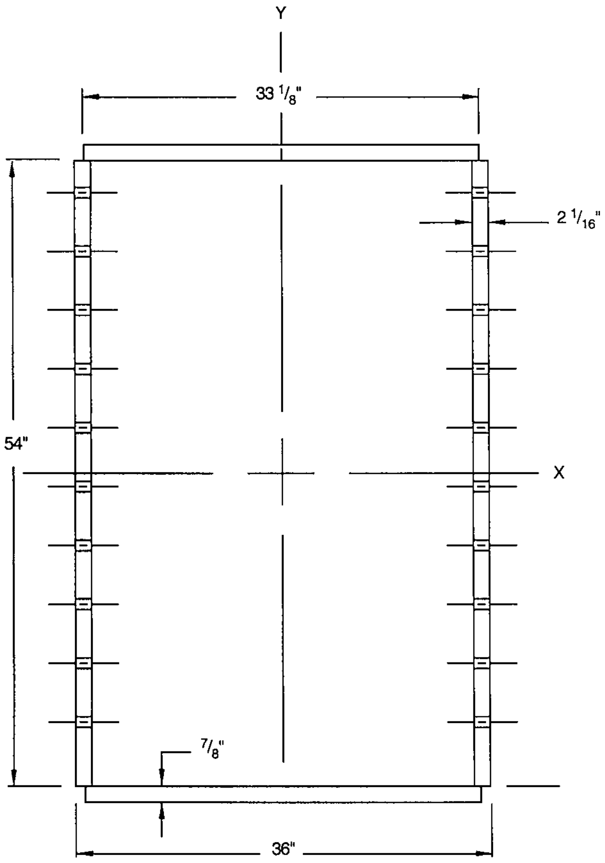


FIGURE 13.8 Cross section of a truss chord with a box section.

Ten $1\frac{1}{4}$-in-diameter bolt holes are provided in each web at the section for the connections at joints. The welds joining the cover plates and webs are minimum size, $\frac{3}{8}$ in, and are classified as AASHTO Fatigue Category B.

Compression in Chord from Factored Loads. The uniform stress on the section is

$$f_c = \frac{7878}{281} = 28.04 \text{ ksi}$$

The radius of gyration with respect to the weak axis is

$$r_y = \sqrt{\frac{I_{gy}}{A_g}} = \sqrt{\frac{69{,}520}{281}} = 15.73 \text{ in}$$

and the slenderness ratio with respect to that axis is

$$\frac{KL}{r_y} = \frac{1 \times 46 \times 12}{15.73} = 35 < \left(\sqrt{\frac{2\pi^2 E}{F_y}} = 126\right)$$

where E = modulus of elasticity of the steel = 29,000 ksi. The critical buckling stress in compression is

$$\begin{aligned} F_{cr} &= F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r_y}\right)^2\right] \\ &= 36\left[1 - \frac{36}{4\pi^2 E}(35)^2\right] = 34.6 \text{ ksi} \end{aligned} \tag{13.5}$$

The maximum strength of a concentrically loaded column is $P_u = A_g f_{cr}$ and

$$f_{cr} = 0.85F_{cr} = 0.85 \times 34.6 = 29.42 \text{ ksi}$$

For computation of the bending strength, the sum of the depth–thickness ratios for the web and cover plates is

$$\sum \frac{s}{t} = 2 \times \frac{54}{2.0625} + 2 \times \frac{36 - 2.0625}{0.875} = 129.9$$

The area enclosed by the centerlines of the plates is

$$A = 54.875(36 - 2.0625) = 1{,}862 \text{ in}^2$$

Then, the design bending stress is given by

$$\begin{aligned} F_a &= F_y\left[1 - \frac{0.0641 F_y S_g L\sqrt{\Sigma(s/t)}}{EA\sqrt{I_y}}\right] \\ &= 36\left[1 - \frac{0.0641 \times 36 \times 3{,}507 \times 46 \times 12\sqrt{129.9}}{29{,}000 \times 1{,}862\sqrt{69{,}520}}\right] \\ &= 35.9 \text{ ksi} \end{aligned} \tag{13.6}$$

For the dead load of 0.98 kip/ft, the dead-load factor of 1.30, the 46-ft span, and a factor of 1/10 for continuity in bending, the dead-long bending moment is

$$M_{DL} = 0.98(46)^2 \times 12 \times \frac{1.30}{10} = 3235 \text{ kip} \cdot \text{in}$$

The section modulus is

$$S_g = \frac{I_{gx}}{c} = \frac{97,770}{(54/2) + 0.875} = 3507 \text{ in}^3$$

Hence, the maximum compressive bending stress is

$$f_b = \frac{M_{DL}}{S_g} = \frac{3235}{3507} = 0.92 \text{ ksi}$$

The plastic section modulus is

$$Z_g = 2(33.125 \times 0.875\left(\frac{54}{2} + \frac{0.875}{2}\right) + 2 \times 2 \times 2.0625 \times \frac{54}{2} \times \frac{54}{4} = 4598 \text{ in}^4$$

The ratio of the plastic section modulus to the elastic section modulus is $Z_g/S_g = 4598/3507 = 1.31$.

For combined axial load and bending, the axial force P and moment M must satisfy the following equations:

$$\frac{P}{0.85 A_g F_{cr}} + \frac{MC}{M_u(1 - P/A_g F_e)} \leq 1.0 \qquad (13.7a)$$

$$\frac{P}{0.85 A_g F_y} + \frac{M}{M_P} \leq 1.0 \qquad (13.8a)$$

where M_u = maximum strength, kip·in, in bending alone
= $S_g f_a$
M_p = full plastic moment, kip·in, of the section
= ZF_y
Z = plastic modulus = $1.31 S_g$
C = equivalent moment factor, taken as 0.85 in this case
F_e = Euler buckling stress, ksi, with 0.85 factor = $0.85 E\pi^2/(KL/r_x)^2$

The effective length factor K is taken equal to unity and the radius of gyration r_x with respect to the x axis, the axis of bending, is

$$r_x = \sqrt{\frac{I_g}{A_g}} = \sqrt{\frac{97,770}{281}} = 18.65 \text{ in}$$

The slenderness ratio KL/r_x then is 46 × 12/18.65 = 29.60.

$$F_e = 0.85 \times \frac{29,000\pi^2}{29.60^2} = 278 \text{ ksi}$$

For convenience of calculation, Eq. (13.7a) can be rewritten, for $P = A_gF_c$, $0.85F_{cr} = f_{cr}$, $M = S_gf_b$, and $M_u = S_gF_a$, as

$$\frac{f_c}{f_{cr}} + \frac{f_b}{F_a} \cdot \frac{C}{1 - P/A_gF_e} \leq 1.0 \tag{13.7b}$$

Substitution of previously calculated stress values in Eq. (13.7b) yields

$$\frac{28.04}{29.42} + \frac{0.92}{35.9} \cdot \frac{0.85}{1 - 7878/(281 \times 278)} = 0.953 + 0.026$$
$$= 0.979 \leq 1.0$$

Similarly, Eq. (13.8a) can be rewritten as

$$\frac{f_c}{0.85F_y} + \frac{f_b}{F_yZ/S_g} \leq 1.0 \tag{13.8b}$$

Substitution of previously calculated stress values in Eq. (13.8b) yields

$$\frac{28.04}{0.85 \times 36} + \frac{0.92}{36 \times 1.31} = 0.916 + 0.020 = 0.936 \leq 1.0$$

The sum of the ratios, 0.981, governs (stability) and is satisfactory. The section is satisfactory for compression.

Local Buckling. For LFD design, and stress in ksi, the AASHTO Specifications limit the depth–thickness ratio of the webs to a maximum of

$$\frac{d}{t} = \frac{180}{\sqrt{f_c}} = \frac{180}{\sqrt{28.04}} = 34.0$$

The actual d/t is 54/2.0625 = 26.2 < 34.0 OK

Maximum permissible width–thickness ratio for the cover plates is

$$\frac{b}{t} = \frac{213.4}{\sqrt{f_c}} = \frac{213.4}{\sqrt{28.04}} = 40.3$$

The actual b/t is 33.125/0.875 = 37.9 < 40.3 OK

Tension in Chord from Factored Loads. The following treatment is based on a composite of AASHTO SLD Specifications for the capacity of tension members, and other aspects from the AASHTO LFD Specifications. This is done because the AASHTO LFD Specifications have not been updated. Clearly, this is not in complete compliance with the AASHTO LFD Specifications. Based on the above, the tensile capacity will be the lesser of the yield strength times the design gross area, or 90% of the tensile strength times the net area. Both areas are defined below. For determinations of the design strength of the section, the effect of the bolt holes must be taken into account by deducting the area of the holes from the gross section area to obtain the net section area. Furthermore, the full gross area should not be used if the holes occupy more than 15% of the gross area. When they do, the excess above 15% of the holes not greater than $1^1/_4$ inches in diameter, and all of the areas of larger holes, should be deducted from the gross area to obtain the design gross area. The holes occupy 10 × 1.25 = 12.50 in of web-plate length, and 15% of the 54-in plate is 8.10 in. The excess is 4.40 in. Hence, the net area is $A_n = 281 - 12.50 \times 2.0625 = 255$ in^2 and the design gross area, $A_{DG} = 281 - 2 \times 4.40 \times 2.0625 = 263$ in^2. The tensile capacity is the lesser of $0.90 \times 255 \times 58 = 13{,}311$ kips or $263 \times 36 = 9{,}468$ kips. Thus, the design gross section capacity controls and the tensile capacity is 9,468 kips.

For computation of design gross moment of inertia, assume that the excess is due to 4 bolts, located 7 and 14 in on both sides of the neutral axis in bending about the x axis. Equivalent diameter of each hole is 4.40/4 = 1.10 in. The deduction from the gross moment of inertia $I_g = 97{,}770$ in^4 then is

$$I_d = 2 \times 2 \times 1.10 \times 2.0625(7^2 + 14^2) = 2220 \text{ in}^4$$

Hence, the design gross moment of inertia I_{DG} is $97{,}770 - 2{,}220 = 95{,}550$ in^4, and the design gross elastic section modulus is

$$S_{DG} = \frac{95{,}550}{54/2 + 0.875} = 3428 \text{ in}^3$$

The stress on the design gross section for the axial tension load of 1748 kips alone is

$$f_t = \frac{1748}{263} = 6.65 \text{ ksi}$$

The bending stress due to $M_{DL} = 3235$ kip·in, computed previously, is

$$f_b = \frac{3235}{3428} = 0.94 \text{ ksi}$$

For combined axial tension and bending, the sum of the ratios of required strength to design strength is

$$\frac{P}{A_n F_y} + \frac{M}{S_n F_y Z} = \frac{f_t}{F_y} + \frac{f_b}{F_y Z} = \frac{6.65}{36} + \frac{0.94}{36 \times 1.31} = 0.205 < 1 \qquad \text{OK}$$

The section is satisfactory for tension.

Fatigue at Welds. Fatigue is to be investigated for the truss as a nonredundant path structure subjected to 500,000 cycles of loading. The Category B welds between web plates and cover plates have an allowable stress range of 23 ksi. Maximum service loads on the chord are 391 kips tension and 4422 kips compression. The stress range then is

$$f_{\text{sr}} = \frac{391 - (-4422)}{281} = 17.1 \text{ ksi} < 23 \text{ ksi}$$

The section is satisfactory for fatigue.

13.10.2 Service-Load Design of Truss Chord

The truss chord designed in Art. 13.10.1 by load factor design and with the cross section shown in Fig. 13.8 is designed for service loads in the following, for illustrative purposes. Properties of the section are given in Art 13.10.1.

Compression in Chord for Service Loads. The uniform stress in the section for the 4422-kip load on the gross area $A_g = 281$ in^2 is

$$f_c = \frac{4422}{281} = 15.74 \text{ ksi}$$

The AASHTO standard specifications give the following formula for the allowable axial stress for $F_y = 36$ ksi:

$$F_a = 16.98 - 0.00053\left(\frac{KL}{r_y}\right)^2 \tag{13.9}$$

For the slenderness ratio $KL/r_y = 35$, determined in Art. 13.10.1, the allowable stress then is

$$F_a = 16.98 - 0.00053(35)^2 = 16.33 \text{ ksi} > 15.74 \text{ ksi} \qquad \text{OK}$$

The allowable bending stress is $f_b = 20$ ksi. Due to the 0.98-kips/ft weight of the 46-ft-long chord, the dead-load bending moment with a continuity factor of $^1/_{10}$ is

$$M_{DL} = 0.98(46)^2 \times \frac{12}{10} = 2488 \text{ kip} \cdot \text{in}$$

For the section modulus $S_{gx} = 97{,}770/27.875 = 3507$ in^3, the dead-load bending stress is

$$f_b = \frac{2488}{3507} = 0.709 \text{ ksi}$$

For combined bending and compression, the AASHTO Specifications require that the following interaction formula be satisfied:

$$\frac{f_c}{F_a} + \frac{f_b}{F_b} \cdot \frac{C_m}{1 - f_c/F_e'} \tag{13.10}$$

The coefficient C_m is taken as 0.85 for the condition of transverse loading on a compression member with joint translation prevented. For bending about the x axis, with a slenderness ratio of $KL/r_x = 29.60$, as determined in Art. 13.10.1, the Euler buckling stress with a 2.12 safety factor is

$$F_e' = \frac{\pi^2 E}{2.12(KL/r_x)^2} = \frac{\pi^2 \times 29{,}000}{2.12(29.60)^2} = 154 \text{ ksi}$$

Substitution of the preceding stresses in Eq. (13.10) yields

$$\frac{15.74}{16.33} + \frac{0.709}{20} \cdot \frac{0.85}{1 - 15.74/154} = 0.964 + 0.034 = 0.998 < 1 \qquad \text{OK}$$

The section is satisfactory for compression.

Tension in Chord from Service Loads. The section shown in Fig. 13.8 has to withstand a tension load of 391 kips on the net area of 263 in^2 computed in Art. 13.10.1. It was determined in Art. 13.10.1 that the capacity was controlled by the design gross section, and while SLD allowable stresses are 0.50 F_u on the net section and 0.55 F_y on the design gross section, the same conclusion is reached here. The allowable tensile stress F_t is 20 ksi. The uniform tension stress on the design gross section is

$$f_t = \frac{391}{263} = 1.49 \text{ ksi}$$

As computed in Art. 13.10.1, the moment of inertia of the design gross section is 95,550 in^4 and the corresponding section modulus in S_n = 3428 in^3. Also, as computed previously for compression in the chord, the dead-load bending moment M_{DL} = 2488 kip·in. Hence, the maximum bending stress is

$$f_b = \frac{2488}{3428} = 0.726 \text{ ksi}$$

The allowable bending stress F_b is 20 ksi.

For combined axial tension and bending, the sum of the ratios of actual stress to allowable stress is

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} = \frac{1.49}{20} + \frac{0.726}{20} = 0.075 + 0.036 = 0.111 < 1 \qquad \text{OK}$$

The section is satisfactory for tension.

Fatigue Design. Set Art. 13.10.1.

13.11 MEMBER DESIGN EXAMPLE—LRFD

The design of a truss hanger by the AASHTO LRFD Specifications is presented subsequently. This is preceded by the following introduction to the LRFD member design provisions.

13.11.1 LRFD Member Design Provisions

Tension Members. The net area, A_n, of a member is the sum of the products of thickness and the smallest net width of each element. The width of each standard bolt hole is taken as the nominal diameter of the bolt plus 0.125 in. The width deducted for oversize and slotted holes, where permitted in AASHTO LRFD Art. 6.13.2.4.1, is taken as 0.0625 in greater than the hole size specified in AASHTO LRFD Art. 6.13.2.4.2. The net width is determined for each chain of holes extending across the member along any transverse, diagonal, or zigzag line, as discussed in Art. 13.9.

In designing a tension member, it is conservative and convenient to use the least net width for any chain together with the full tensile force in the member. It is sometimes possible to achieve an acceptable, but slightly less conservative design, by checking each possible chain with a tensile force obtained by subtracting the force removed by each bolt ahead of that chain (bolt closer to midlength of the member), from the full tensile force in the member. This approach assumes that the full force is transferred equally by all bolts at one end.

Members and splices subjected to axial tension must be investigated for two conditions: yielding on the gross section [Eq. (13.11)], and fracture on the net section [Eq. (13.12)]. Determination of the net section requires consideration of the following:

- The gross area from which deductions will be made, or reduction factors applied, as appropriate. The determination of the gross section requires consideration of all holes larger than those typically used for connectors such as bolts, e.g., pin holes, access holes, and perforations.
- Deductions for all holes in the design cross section.
- Correction of the bolt-hole deductions for the stagger rule.
- Application of a reduction factor U, to account for shear lag.
- Application of an 85% maximum area efficiency factor for splice plates and other splicing elements.

The factored tensile resistance P_r is the lesser of the values given by Eqs. (13.11) and (13.12):

$$P_r = \varphi_y P_{\text{ny}} = \varphi_y F_y A_g \tag{13.11}$$

$$P_r = \varphi_u P_{nu} = \varphi_y F_u A_n U \tag{13.12}$$

where P_{ny} = nominal tensile resistance for yielding in gross section (kips)
F_y = yield strength (ksi)
A_g = gross cross-sectional area of the member (in^2)
P_{nu} = nominal tensile resistance for fracture in net section (kips)
F_u = tensile strength (ksi)
A_n = net area of the member as described above (in^2)
U = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements; as described below for other cases
φ_y = resistance factor for yielding of tension members, 0.95
φ_u = resistance factor for fracture of tension members, 0.80

The reduction factor U does not apply when checking yielding on the gross section because yielding tends to equalize the nonuniform tensile stresses over the cross section caused by shear lag.

Unless a more refined analysis or physical tests are utilized to determine shear lag effects, the reduction factors specified in the AASHTO LRFD Specifications may be used to account for shear lag in connections as explained in the following.

The reduction factor U for sections subjected to a tension load transmitted directly to each of the cross-sectional elements by bolts or welds may be taken as

$$U = 1.0 \tag{13.13}$$

For bolted connections, the following three values of U may be used depending on the details of the connection:

- For rolled I-shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress,

$$U = 0.90 \tag{13.14}$$

- For all other members having no fewer than three fasteners per line in the direction of stress,

$$U = 0.85 \tag{13.15}$$

- For all members having only two fasteners per line in the direction of stress,

$$U = 0.75 \tag{13.16}$$

Due to strain hardening, a ductile steel loaded in axial tension can resist a force greater than the product of its gross area and its yield strength prior to fracture. However, excessive elongation due to uncontrolled yielding of gross area not only marks the limit of usefulness, it can precipitate failure of the structural system of which it is a part. Depending on the ratio of net area to gross area and the mechanical properties of the steel, the component can fracture by failure of the net area at a load smaller than that required to yield the gross area. General yielding of the gross area and fracture of the net area both constitute measures of component strength. The relative values of the resistance factors for yielding and fracture reflect the different reliability indices deemed proper for the two modes.

The part of the component occupied by the net area at fastener holes generally has a negligible length relative to the total length of the member. As a result, the strain hardening is quickly reached and, therefore, yielding of the net area at fastener holes does not constitute a strength limit of practical significance, except, perhaps, for some built-up members of unusual proportions.

For welded connections, A_n is the gross section less any access holes in the connection region.

Compression Members. Bridge members in axial compression are generally proportioned with width–thickness ratios such that the yield point can be reached before the onset of local buckling. For such members, the nominal compressive resistance P_n is taken as

$$\text{If } \lambda \leq 2.25, \text{ then } P_n = 0.66^{\lambda} F_y A_s \tag{13.17}$$

$$\text{If } \lambda > 2.25, \text{ then } P_n = \frac{0.88 F_y A_s}{\lambda} \tag{13.18}$$

for which

$$\lambda = \left(\frac{Kl}{r_s \pi}\right)^2 \frac{F_y}{E} \tag{13.19}$$

where A_s = gross cross-sectional area (in^2)
F_y = yield strength (ksi)
E = modulus of elasticity (ksi)
K = effective length factor
l = unbraced length (in)
r_s = radius of gyration about the plane of buckling (in)

To avoid premature local buckling, the width-to-thickness ratios of plate elements for compression members, other than flanges of built-up I-sections, must satisfy the following relationship:

$$\frac{b}{t} \leq k\sqrt{\frac{E}{F_y}} \tag{13.20}$$

where k = plate-buckling coefficient, b = plate width (in), and t = thickness (in). See Table 13.6 for values for k and descriptions of b.

TABLE 13.6 Values of k for Calculating Limiting Width–Thickness Ratios

Element	Coefficient k	Width b
a. Plates supported along one edge		
Flanges and projecting legs or plates	0.56	Half-flange width of hot-rolled I-sections. Full-flange width of channels. Distance between free edge and first line of bolts or weld in plates. Full width of an outstanding leg for pairs of angles in continuous contact.
Stems of rolled tees	0.75	Full depth of tee.
Other projecting elements	0.45	Full width of outstanding leg for single-angle strut or double-angle strut with separator. Full projecting width for others
b. Plates supported along two edges		
Box flanges and cover plates	1.40	Clear distance between webs minus inside corner radius on each side for box flanges. Distance between lines of welds or bolts for flange cover plates.
Webs and other plate elements	1.49	Clear distance between flanges minus fillet radii for webs of rolled beams. Clear distance between edge supports for all others.
Perforated cover plates	1.86	Clear distance between edge supports.

Source: Adapted from *AASHTO LRFD Bridge Design Specification*, American Association of State Highway and Transportation Officials, 444 North Capital St., N.W., Ste. 249, Washington, DC, 2001.

The half-width of flanges of built-up I sections must satisfy

$$\frac{b}{t} \le 0.64\sqrt{\frac{k_c E}{F_y}} \tag{13.21}$$

and

$$35 \le K_c \le 0.76 \tag{13.22}$$

in which

$$k_c = \frac{4}{\sqrt{D_0/t_w}} \tag{13.23}$$

where b_c = half-width of flange, in
D_0 = web depth, in

Members under Tension and Flexure. A component subjected to tension and flexure must satisfy the following interaction equations:

$$\text{If } \frac{P_u}{P_r} < 0.2, \text{ then}$$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \tag{13.24}$$

$$\text{If } \frac{P_u}{P_r} \ge 0.2, \text{ then}$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \tag{13.25}$$

where P_r = factored tensile resistance, kips
M_{rx}, M_{ry} = factored flexural resistances about the x and y axes, respectively, kip·in
M_{ux}, M_{uy} = moments about x and y axes, respectively, resulting from factored loads, kip·in
P_u = axial force effect resulting from factored loads, kips

Interaction equations in tension and compression members are a design simplification. Such equations involving exponents of 1.0 on the moment ratios are usually conservative. More exact, nonlinear interaction curves are also available and are discussed in the literature. If these interaction equations are used, additional investigation of service limit state stresses is necessary to avoid premature yielding.

A flange or other component subject to a net compressive stress due to tension and flexure should also be investigated for local buckling.

Members under Compression and Flexure. For a component subjected to compression and flexure, the axial compressive load P_u and the moments M_{ux} and M_{uy} are determined for concurrent factored loadings by elastic analytical procedures. The following relationships must be satisfied:

$$\text{If } \frac{P_u}{P_r} < 0.2, \text{ then}$$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \tag{13.26}$$

$$\text{If } \frac{P_u}{P_r} \geq 0.2, \text{ then}$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \tag{13.27}$$

where P_r = factored compressive resistance, φP_n, kips
M_{rx} = factored flexural resistance about the x axis, kip·in
M_{ry} = factored flexural resistance about the y axis, kip·in
M_{ux} = factored flexural moment about the x axis calculated as specified below, kip·in
M_{uy} = factored flexural moment about the y axis calculated as specified below, kip·in
φ = resistance factor for compression members

The moments about the axes of symmetry, M_{ux} and M_{uy}, may be determined by either (1) a second-order elastic analysis that accounts for the magnification of moment caused by the factored axial load, or (2) the approximate single-step adjustment specified in AASHTO LRFD Art. 4.5.3.2.2b.

13.11.2 LRFD Design to Truss Hanger

The following example, prepared in the SI system of units, illustrates the design of a tensile member that also supports a primary live-load bending moment. The existence of the bending moment is not common in truss members, but can result from unusual framing. In this example, the bending moment serves to illustrate the application of various provisions of the LRFD Specifications.

A fabricated H-shaped hanger member is subjected to the unfactored design loads listed in Table 13.7. The applicable AASHTO load factors for the strength I limit state and the fatigue limit state are listed in Table 13.8. The impact factor I is 1.15 for the fatigue limit state and 1.33 for all other limit states.

For the overall bridge cross section, the governing live-load condition places three lanes of live load on the structure with a distribution factor DF of 2.04 and a multiple presence factor, MPF, of 0.85. For the fatigue limit state, the placement of the single fatigue truck produces a distribution factor of 0.743. The multiple presence factor is not applied to the fatigue limit state.

The factored force effect Q in the member is calculated for the axial force and the moment in Table 13.7 from the following equation to obtain the factored member load and moment:

$$Q = \eta[\lambda_{DC}DC + \lambda_{DW}DW + \lambda_{LL+I}(DF)(MPF)(LL_{TR} \times I + LL_{LA})] \tag{13.28}$$

where DF is the distribution factor, MPF is the multiple presence factor, I is the impact factor, and the other terms are defined in Tables 13.7 and 13.8. For example, for the axial load, Q is calculated as follows:

$$\begin{aligned} Q &= 1.10[1.25 \times 1344 + 1.50 \times 149 + 1.75(2.04)(0.85)(32.9 \times 1.33 + 82.4)] \\ &= 2515 \text{ kN} \end{aligned}$$

TABLE 13.7 Unfactored Design Loads

Load component	Axial tension load P, kN	Bending moment M_x, kN·m	Bending moment M_y, kN·m
Dead load of structural components, DC	1344	0	−9.01
Dead load of wearing surfaces and utilities, DW	149	0	−1.07
Truck live load per lane, LL_{TR}	32.9	0	35.8
Lane live load per lane, LL_{LA}	82.4	0	90.0
Fatigue live load, LL_{FA}	44.0, −1.10	0	15.0, −4.40

TABLE 13.8 AASHTO Load Factors

Type of factor	Strength I limit state*	Fatigue limit state
Ductility, η_D	1.00	1.0
Redundancy, η_R	1.05	1.0
Importance, η_I	1.05	1.0
$\eta = \eta_D \eta_P \eta_I$†	1.10	1.0
Dead load, γ_{DC}	1.25/0.90	—
Dead load, γ_{DW}	1.50/0.65	—
Live load + impact, *LL* + *I*	1.75	0.75

*Basic load combination relating to normal vehicular use of bridge without wind.

†$\eta \geq 0.95$ for loads for which a maximum load factor is appropriate; $1/\eta \leq 1.10$ for loads for which a minimum load factor is appropriate.

Table 13.9 summarizes the nominal force effects for the member.

The preliminary section selected is shown in Fig. 13.9. The member length is 20 m, the yield stress 345 MPa, the tensile strength 450 MPa, and the diameter of A325 bolts is 24 mm. Section properties are listed in Table 13.10.

Tensile Resistance. The tensile resistance is calculated as the lesser of Eqs. (13.11) and (13.12). From Eq. (13.11), gross section yielding, $P_r = 0.95 \times 345 \times 26{,}456/1000 = 8671$ kN. From Eq. (13.12), net section fracture, assuming the force effects are transmitted to all components so that $U = 1.00$, $P_r = 0.80 \times 450 \times 20{,}072/1000 = 7226$ kN. Thus, net section fracture controls and $P_r = 7226$ kN.

Flexural Resistance. Because net section fracture controls, use net section properties for calculating flexural resistance. Also, because $M_x = 0$, only investigate weak-axis bending. The nominal moment strength M_n is defined by AASHTO in this case as the plastic moment. Thus, for an H-section about the weak axis, in terms of the yield stress F_y and section modulus S,

$$M_{\mathrm{ny}} = 1.5 F_y S \tag{13.29}$$

Substituting y-axis values, $M_{ny} = 1.5 \times 345 \times 1.49 \times 10^6/1000^2 = 771$ kN·m. The factored flexural resistance M_r is defined as

$$M_r = \varphi_f M_n \tag{13.30}$$

where φ_f is the resistance factor for flexure (1.00). Therefore, in this case, $M_{ry} = 1.00 M_{ny} = 771$ kN·m.

Combined Tension and Flexure. This will be checked for the strength I limit state using the nominal force effects listed in Table 13.9. First calculate $P_u/P_r = 2515/7226 = 0.348$. Because this exceeds 0.2, Eq. (13.25) applies. Substitute appropriate values as follows:

$$\frac{2515}{7226} + \frac{8}{9}\left(0 + \frac{450}{771}\right) = 0.87 \leq 1.00 \qquad \text{OK}$$

TABLE 13.9 Factored Design Loads (Nominal Force Effects)

Limit state	Axial tension load P_u, kN	Bending moment M_{ux}, kN·m	Bending moment M_{uy}, kN·m
Strength I	2515	0	450
Fatigue	28.2, −0.70	0	9.61, −2.82

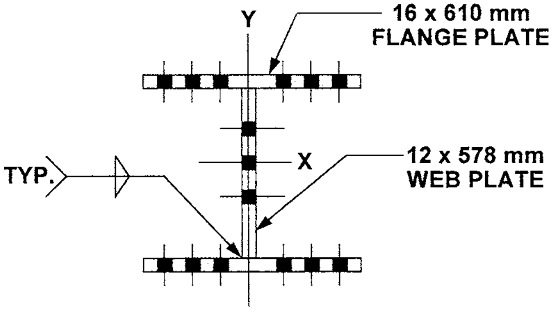


FIGURE 13.9 Cross section of H-shaped hanger.

Slenderness Ratio. AASHTO requires that tension members other than rods, eyebars, cables, and plates satisfy certain slenderness ratio (l/r) requirements. For main members subject to stress reversal, $l/r \leq 140$. In the present case the least radius of gyration is $r = \sqrt{I_{yg}/A_g} = \sqrt{6.05 \times 10^8/26{,}456} = 151$ mm and $l/r = 20{,}000/151 = 132$. This is within the limit of 140.

Fatigue Limit State. The member is fabricated from plates with continuous fillet welds parallel to the applied stress. Slip-critical bolts are used for the end connections. Both of these are Category B fatigue details. The average daily truck traffic, ADTT, is 2250, and three lanes are available to trucks. The number of trucks per day in a single lane, averaged over the design life, is calculated from the AASHTO expression

$$\text{ADTT}_{SL} = p \times \text{ADTT} \tag{13.31}$$

where p is the fraction of truck traffic in a single lane as follows: 1.00 for one truck lane, 0.85 for two truck lanes, and 0.80 for three or more truck lanes. Therefore, $\text{ADTT}_{SL} = 0.80 \times 2250 = 1800$. The nominal fatigue resistance is calculated as a maximum permissible stress range as follows:

$$\Delta F = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{TH} \tag{13.32}$$

where

$$N = (365)(75)(n)(\text{ADTT}_{SL}) \tag{13.33}$$

In the above, A is a fatigue constant that varies with the fatigue detail category, n is the number of stress range cycles per truck, and $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold. These constants

TABLE 13.10 Section Properties for Example Problem

Area	A_g	26,456 mm^2
	A_n	20,072 mm^2
Moment of inertia	I_{xg}	1.92×10^9 mm^4
	I_{xn}	1.44×10^9 mm^4
	I_{yg}	6.05×10^8 mm^4
	I_{yn}	4.56×10^8 mm^4
Section modulus	S_{xg}	6.30×10^6 mm^3
	S_{xn}	4.71×10^6 mm^3
	S_{yg}	1.98×10^6 mm^3
	S_{yn}	1.49×10^6 mm^3

are found in the AASHTO LRFD Specifications for the present case as follows: $A = 39.3 \times 10^{11}$ MPa^3, $n = 1.0$, and $(\Delta F)_{TH} = 110$ MPa. Substituting in Eq. (13.32),

$$\Delta F = \left(\frac{39.3 \times 10^{11}}{365 \times 75 \times 1.0 \times 1800} \right)^{1/3} = 43.0 \text{ MPa and } \frac{1}{2}(\Delta F)_{TH} = 55 \text{ MPa}$$

Therefore, $\Delta F = 55$ MPa. Next, calculate the stress range for the force effects in Table 13.9. For the web-to-flange welds, which lie near the neutral axis, only the axial load is considered, and net section properties are used as the worst case:

$$\frac{28.2 - (-0.70)}{20,072} \times 1000 = 1.44 \text{ MPa} < 55 \text{ MPa} \qquad \text{OK}$$

For the extreme fiber at the slip-critical connections, both axial load and flexure is considered, and gross section properties are used:

$$\frac{28.2 - (-0.70)}{26,456} \times 10^3 + \frac{9.61 - (-2.82)}{1.98 \times 10^6} \times 10^6 = 7.37 \text{ MPa} < 55 \text{ MPa} \qquad \text{OK}$$

Thus, fatigue does not control and the member selection is satisfactory. A separate check shows that the bolts are also adequate.

13.12 TRUSS JOINT DESIGN PROCEDURE

At every joint in a truss, working lines of the intersecting members preferably should meet at a point to avoid eccentric loading (Art. 13.2). While the members may be welded directly to each other, most frequently they are connected to each other by bolting to gusset plates. Angle members may be bolted to a single gusset plate, whereas box and H shapes may be bolted to a pair of gusset plates.

A gusset plate usually is a one-piece element. When necessary, it may be spliced with groove welds. When the free edges of the plate will be subjected to compression, they usually are stiffened with plates or angles. Consideration should be given in design to the possibility of the stresses in gusset plates during erection being opposite in sense to the stresses that will be imposed by service loads.

Gusset plates are sometimes designed by the **method of sections,** based on conventional strength-of-materials theory. The method of sections involves investigation of stresses on various planes through a plate and truss members. Analysis of gusset plates by finite-element methods, however, may be advisable where unusual geometry exists.

Transfer of member tensile forces into and out of a gusset plate invokes the potential for block shear around the connector groups. A 30° angle of distribution with respect to the gage line, as illustrated in Fig. 13.10 (lines 1–5 and 4–6) has often been assumed as a supplementary check on tension transferred into gusset plates.

The SLD and LFD provisions offer relatively little guidance regarding the investigation of block shear in tension members. The LRFD provisions are more complete regarding the factored resistance to be compared to a factored load. These provisions could reasonably be applied to LFD. Application to SLD would require use of allowable stresses and reconsideration of the applicability of the resistance factor.

The LRFD provisions require that the factored resistance of the combination of parallel and perpendicular planes be determined using either Eq. (13.34) or Eq. (13.35) as appropriate. All possible failure planes in the member and connection plates must be considered. Such planes include those parallel and perpendicular to the applied forces. The planes parallel to the applied force are considered to resist only shear stresses. The planes perpendicular to the applied force are considered to resist only tension stresses.

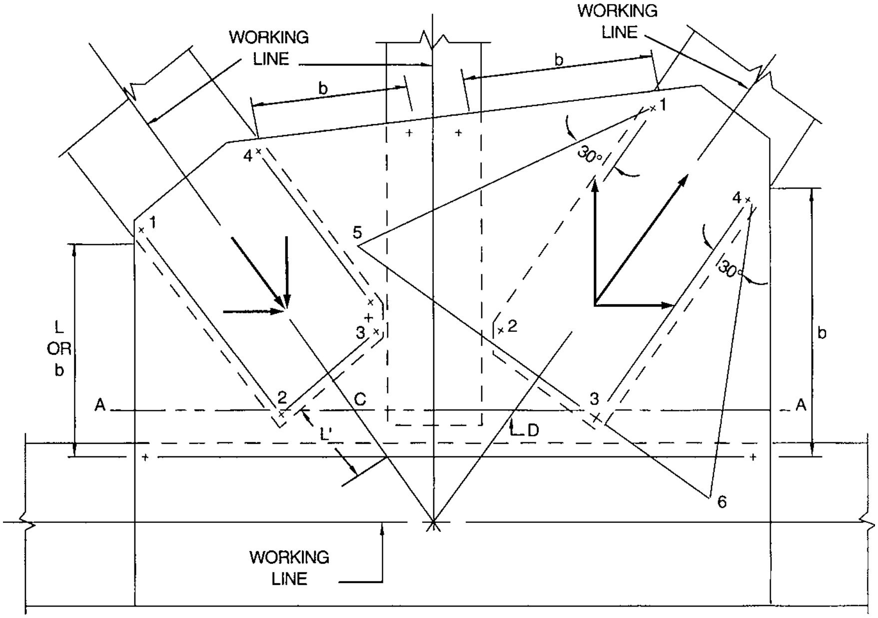


FIGURE 13.10 Typical design sections for a gusset plate.

If $A_{tn} \geq 0.58A_{vn}$, then:

$$R_r = \phi_{bs}(0.58F_yA_{vg} + F_uA_{tn}) \tag{13.34}$$

Otherwise:

$$R_r = \phi_{bs}(0.58F_uA_{vn} + F_yA_{tg}) \tag{13.35}$$

where A_{vg} = gross area along the plane resisting shear stress, in^2
A_{vn} = net area along the plane resisting shear stress, in^2
A_{tg} = gross area along the plane resisting tension stress, in^2
A_{tn} = net area along the plane resisting tension stress, in^2
F_y = specified minimum yield strength of the connected material, ksi
F_u = specified minimum tensile strength of the connected material, ksi
ϕ_{bs} = resistance factor for block shear specified as 0.80

The following summarizes a procedure for load factor design of a truss joint. Splices are assumed to occur within the truss joints. (See examples in Arts. 13.13 and 13.14.) The concept employed in the procedure can also be applied to working-stress design.

1. Lay out the centerlines of truss members to an appropriate scale with gage lines.
2. Detail the fixed parts, such as floorbeam, strut, and lateral connections.
3. Determine the grade and size of bolts to be used.
4. Detail the end connections of truss diagonals. The connections should be designed for the average of the design strength of the diagonals and the factored load they carry but not less than 75% of the design strength. The design strength should be taken as the smallest of the following: (*a*) member strength, (*b*) column capacity, and (*c*) strength based on the width–thickness ratio *b*/*t*. A diagonal should have at least the major portion of its ends normal to the working line (square), so that milling across the ends will permit placing of templates for bolt-hole alignment

accurately. The corners of the diagonal should be as close as possible to the cover plates of the chord and verticals. Bolts for connection to a gusset plate should be centered about the neutral axis of the member.

5. Design fillet welds connecting a flange plate of a welded box member to the web plates, or the web plate of an H member to the flange plates, to transfer the connection load from the flange plate into the web plates over the length of the gusset connection. Weld lengths should be designed to satisfy fatigue requirements. The weld size should be shown on the plans if the size required for loads or fatigue is larger than the minimum size allowed.

6. Avoid the need for fills between gusset plates and welded-box truss members by keeping the out-to-out dimension of web plates and the in-to-in dimension of gusset plates constant.

7. Determine gusset-plate outlines. This step is influenced principally by the diagonal connections.

8. Select a gusset-plate thickness t to satisfy the following criteria, as illustrated in Fig. 13.10:

a. The loads for which a diagonal is connected may be resolved into components parallel to and normal to line A–A in Fig. 13.10 (horizontal and vertical). A shearing stress is induced along the gross section of line A–A through the last line of bolts. Equal to the sum of the horizontal components of the diagonals (if they act in the same direction), this stress should not exceed $F_y/1.35\sqrt{3}$, where F_y is the yield stress of the steel, ksi.

b. A compression stress is induced in the edge of the gusset plate along Section A–A (Fig. 13.10) by the vertical components of the diagonals (applied at C and D) and the connection load of the vertical or floorbeam, when compressive. The compression stress should not exceed the permissible column stress for the unsupported length of the gusset plate (L or b in Fig. 13.10). A stiffening angle should be provided if the slenderness ratio $L/r = L\sqrt{12}/t$ of the compression edge exceeds 120, or if the permissible column stress is exceeded. The L/r of the section formed by the angle plus a 12-in width of the gusset plate should be used to recheck that $L/r \leq 120$ and the permissible column stress is not exceeded. In addition to checking the L/r of the gusset in compression, the width–thickness ratio b/t of every free edge should be checked to ensure that it does not exceed $348/\sqrt{F_y}$.

c. At a diagonal (Fig. 13.10),

$$V_1 + V_2 \geq P_d \tag{13.36}$$

where P_d = load from the diagonal, kips
V_1 = shear strength, kips, along lines 1–2 and 3–4
$= A_g F_y/\sqrt{3}$
A_g = gross area, in^2, along those lines
V_2 = strength, kips, along line 2–3 based on $A_n F_y$ for tension diagonals or $A_g F_a$ for compression diagonals
A_n = net area, in^2, of the section
F_a = allowable compressive stress, ksi

The distance L' in Fig. 13.10 is used to compute F_a for sections 2–3 and 5–6.

d. Assume that the connection stress transmitted to the gusset plate by a diagonal spreads over the plate within lines that diverge outward at 30° to the axis of the member from the first bolt in each exterior row of bolts, as indicated by path 1–5–6–4 (on the right in Fig. 13.10). Then, the stress on the section normal to the axis of the diagonal at the last row of bolts (along line 5–6) and included between these diverging lines should not exceed F_y on the net-section for tension diagonals and F_a for compression diagonals.

9. Design the chord splice (at the joint) for the full capacity of the chords. Arrange the gusset plates and additional splice material to balance, as much as practical, the segment being spliced.

10. When the chord splice is to be made with a web splice plate on the inside of a box member (Fig. 13.11), provide extra bolts between the chords and the gusset on each side of the inner splice plate when the joint lies along the centerline of the floorbeam. This should be done because in the diaphragm, bolts at floorbeam connections deliver some floorbeam reaction across

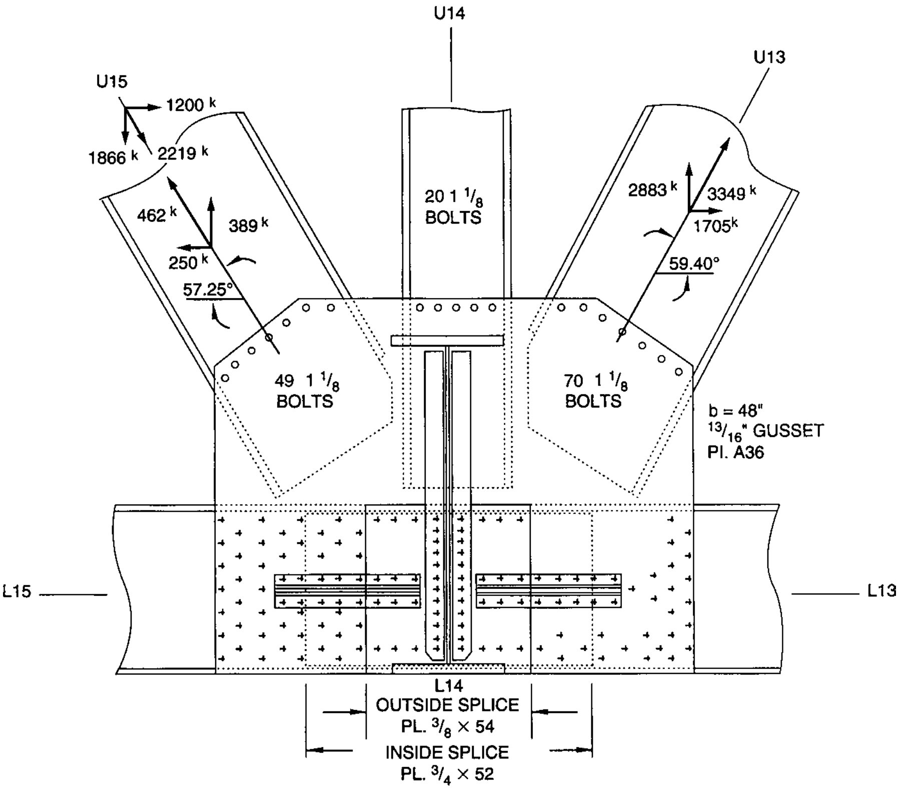


FIGURE 13.11 Truss joint for example of load factor design.

the chords. When a splice plate is installed on the outer side of the gusset, back of the floorbeam connection angles (Fig. 13.11), the entire group of floorbeam bolts will be stressed, both vertically and horizontally, and should not be counted as splice bolts.

11. Confirm that bolt spacing conforms to requirements for edge distance, pitch, and sealing.
12. For members subjected to tension, verify adequacy of gusset and splice material with respect to block shear.
13. Determine the size of standard perforations and the distances from the ends of the member.

13.13 EXAMPLE—LOAD FACTOR DESIGN OF TRUSS JOINT

The LFD Specifications for trusses have provisions for determining the required number of bolts by using a ratio of live load to total load to proportion strength-design loads to approximate the overload provisions. In this example, the overload provisions for slip-critical connections are used directly.

The shear capacity on line *A–A* has been reduced compared to other lines to be investigated because line *A–A* will be investigated for bending shear on a rectangular beam for which the peak stress is higher, compared to the average stress, than for conventional I-girders.

The joint shown in Fig. 13.11 is to be designed to satisfy the criteria listed in Table 13.11. Fasteners to be used are $1^1/_8$-in-diameter A325 high-strength bolts in a slip-critical connection with Class A surfaces, which have a single plane resistance of 18.27 kips per bolt.

TABLE 13.11 Allowable Stresses for Truss Joint, ksi*

Design section	Yield stress of steel, ksi 36	50
Shear on line *A–A*	15.5	21.8
Shear on lines 1–2 and 3–4	20.9	29.0
Tension on lines 2–3 and 5–5	36.0	50.0

*Figures 13.10 and 13.11.

Diagonal U15–L14. The diagonal is subjected to factored loads of 2219 kips compression and 462 kips tension. It has a design strength of 2379 kips. The AASHTO SLD Specifications require that the connection to the gusset plate transmit 75% of the design strength or the average of the factored load and the design strength, whichever is larger. Thus, the design load for the connection is

$$P = \frac{2219 + 2379}{2} = 2299 \text{ kips} > 0.75 \times 2379$$

The design load of 2299 kips is 3.6% greater than the factored compression load, 2219 kips. The overload criterion for the member results in a design force of 1707 kips. For consistency, this force will be increased by 7.2%. Therefore, since the member is connected to two gusset plates, the number of bolts required for diagonal U15-L14 is

$$N = \frac{1707 \times 1.036}{2 \times 18.27} = 49 \text{ per side}$$

Diagonal L14–U13. The diagonal is subjected to factored loads of a maximum of 3272 kips tension and a minimum of 650 kips tension. It has a design strength of 3425 kips. The design load for the connection is

$$P = \frac{3272 + 3425}{2} = 3349 \text{ kips} > 0.75 \times 3425$$

$$N = \frac{2517 \times 1.024}{2 \times 18.27} = 70 \text{ per side}$$

Vertical U14–L14. The vertical carries a factored compression load of 362 kips. It has a design strength of 1439 kips, limited by *b/t* at a perforation. The design load for the connection is

$$P = 0.75 \times 1439 = 1079 \text{ kips} > \frac{362 + 1439}{2}$$

Since the vertical does not carry any live load, the load factor is 1.5. Hence, the number of $1\frac{1}{8}$-in bolts required for the vertical is

$$N = \frac{1079}{2 \times 18.27} = 20 \text{ per side}$$

Given that the connection design load of 1079 kips is so much larger than the calculated factored load of 362 kips, 20 bolts per side is clearly adequate.

Splice of Chord Cover Plates. Each cover plate of the box chord is to be spliced with a plate on the inner and outer face (Fig. 13.12). A36 steel will be used for the splice material, as for the chord. Fasteners are $\frac{7}{8}$-in-diameter A325 bolts, with a capacity of 12.63 kips per shear plane.

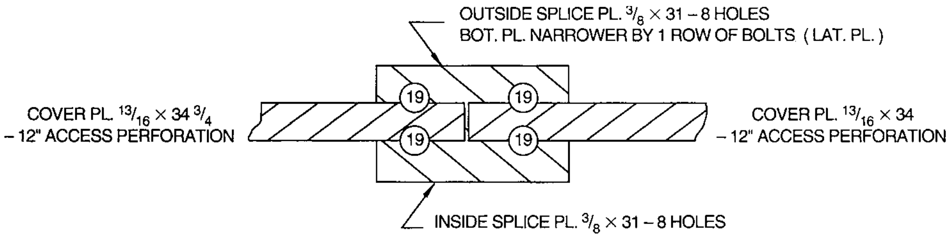


FIGURE 13.12 Cross section of chord cover-plate splice for example of load factor design.

The cover plate on chord L14–L15 (Fig. 13.11) is $^{13}/_{16} \times 34^{3}/_{4}$ in but has 12-in-wide access perforations. Usable area of the plate is 18.48 in^2. The cover plate for chord L13–L14 is $^{13}/_{16} \times 34$ in, also with 12-in-wide access perforations. Usable area of this plate is 17.88 in^2. Design of the chord splice is based on the 17.88-in^2 area. The difference of 0.60 in^2 between this area and that of the larger cover plate will be made up on the L14–L15 side of the web-plate splice as "cover excess."

Where the design section of the joint elements is controlled by allowances for bolts, only the excess exceeding 15% of the gross section area is deducted from the gross area to obtain the design area. (This is the designer's interpretation of the applicable requirements for splices in the AASHTO SLD Specifications. The interpretation is based on the observation that, for the typical dimensions of members, holes, bolt patterns, and grades of steel used on the bridge in question, the capacity of tension members was often controlled by the design gross area as illustrated in Arts. 13.10.1 and 13.10.2. The current edition of the specifications should be consulted on this and other interpretations, inasmuch as the specifications are under constant reevaluation.)

Dividing the strength-based capacity of the plate by 1.30 to estimate its share of the member overload force, the number of bolts needed for a cover-plate splice is

$$N = \frac{17.88 \times 36}{2 \times 1.30 \times 12.63} = 20 \text{ per side}$$

Try two splice plates, each $^3/_8 \times 31$ in, with a gross area of 23.26 in^2. Assume eight l-in-diameter bolt holes in the cross section. The area to be deducted for the holes then is

$$2 \times 0.375(8 \times 1 - 0.15 \times 31) = 2.51 \text{ in}^2$$

Consequently, the area of the design net section is

$$A_n = 23.26 - 2.51 = 20.75 \text{ in}^2 > 17.88 \text{ in}^2 \qquad \text{OK}$$

Tension Splice of Chord Web Plate. A splice is to be provided between the $1^1/_4 \times 54$-in web of chord L14–L15 and the $1^5/_8 \times 54$-in web of the L13–L14 chord. Because of the difference in web thickness, a $^3/_8$-in fill will be placed on the inner face of the $1^1/_4$-in web (Fig. 13.13). The gusset plate

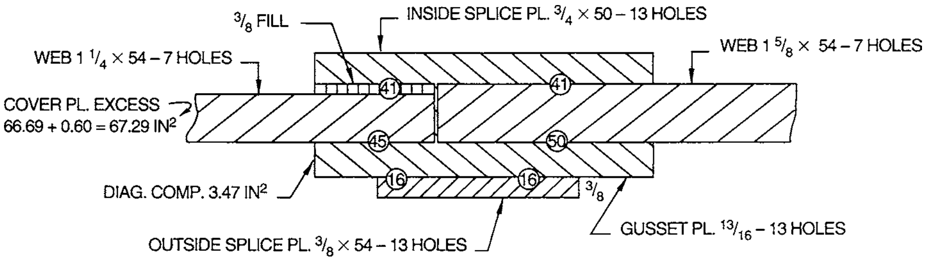


FIGURE 13.13 Cross section of chord web-plate splice for example of load factor design.

can serve as part of the needed splice material. The remainder is supplied by a plate on the inner face of the web and a plate on the outer face of the gusset. Fasteners are 1¹⁄₈-in-diameter A325 bolts.

The web of the L13–L14 chord has a gross area of 87.75 in^2. After deduction of the 15% excess of seven 1¹⁄₄-in-diameter bolt holes, the design area of this web is 86.69 in^2.

The web on the L14–L15 chord has a gross area of 67.50 in^2. After deduction of the 15% excess of seven bolt holes from the chord splice and addition of the "cover excess" of 0.60 in^2, the design area of this web is 67.29 in^2.

The gusset plate is ¹³⁄₁₆ in thick and 118 in high. Assume that only the portion that overlaps the chord web; that is, 54 in, is effective in the splice. To account for the eccentric application of the chord load to the gusset, an effectiveness factor may be applied to the overlap, with the assumption that only the overlapping portion of the gusset plate is stressed by the chord load.

The **effectiveness factor** E_f is defined as the ratio of the axial stress in the overlap due to the chord load to the sum of the axial stress on the full cross section of the gusset and the moment due to the eccentricity of the chord relative to the gusset centroid.

$$E_f = \frac{P/A_o}{P/A_g + Pey/I} \tag{13.37}$$

where P = chord load
A_o = overlap area = $54t$
A_g = full area of gusset plate = $118t$
e = eccentricity of $P = 118/2 - 54/2 = 32$ in
$y = 118/2 = 59$ in
$I = 118^3 t/12 = 136{,}900t$ in^4

Substitution in Eq. (13.37) yields

$$E_f = \frac{P/54t}{P/118t + 32 \times 59P/136{,}900t} = 0.832$$

The gross area of the gusset overlap is ¹³⁄₁₆ × 54 = 43.88 in^2. After deduction of the 15% excess of thirteen 1¹⁄₄-in-diameter bolt holes, the design area is 37.25 in^2. Then, the effective area of the gusset as a splice plate is 0.832 × 37.25 = 30.99 in^2.

In addition to the 67.29 in^2 of web area, the gusset has to supply an area for transmission of the 250-kip horizontal component from diagonal U15–L14 (Fig. 13.11). With F_y = 36 ksi, this area equals 250/(36 × 2) = 3.47 in^2. Hence, the equivalent web area from the L14–L15 side of the joint is 67.29 + 3.47 = 70.76 in^2. The number of bolts required to transfer the load to the inside and outside of the web should be determined based on the effective areas of gusset that add up to 70.76 in^2 but that provide a net moment in the joint close to zero.

The sum of the moments of the web components about the centerline of the combination of outside splice plate and gusset plate is 3.47 × 0.19 + 67.29 × 1.22 = 0.66 + 82.09 = 82.75 in^3. Dividing this by 2.59 in, the distance to the center of the inside splice plate, yields an effective area for the inside splice plate of 31.95 in^2. Hence, the effective area of the combination of the gusset and outside splice plates is 70.76 – 31.95 = 38.81 in^2. This is then distributed to the plates in proportion to thickness: gusset, 24.96 in^2, and splice plate, 13.85 in^2.

The number of 1¹⁄₈-in A325 bolts required to develop a plate with area A is given by

$$N = \frac{AF_y}{1.30 \times 18.27} = \frac{36A}{23.75} = 1.516A$$

Table 13.12 list the number of bolts for the various plates.

Check of Gusset Plates. The adequacy of the gusset plate will be investigated by traditional methods, followed by one investigation of block shear. At section A–A (Fig. 13.11), each plate is 128 in wide and 118 in high, ¹³⁄₁₆ in thick. The design shear stress is 15.5 ksi (Table 13.11). The sum of

TABLE 13.12 Number of Bolts for Plate Development

Plate	Area, in^2	Bolts
Inside splice plate	31.95	49
Outside splice plate	13.85	21
Gusset plate on L14–L15 side	(13.85 + 24.96 – 3.47) = 35.34	54
Gusset plate on L13–L14 side	(13.85 + 24.96) = 38.81	59

the horizontal components of the loads on the truss diagonals is 1244 + 1705 = 2949 kips. This produces a shear stress on section *A–A* of

$$f_v = \frac{2949}{2 \times 128 \times {}^{13}\!/_{16}} = 14.18 \text{ ksi} < 15.5 \text{ ksi} \qquad \text{OK}$$

The vertical component of diagonal U15–L14 produces a moment about the centroid of the gusset of 1,934 × 21 = 40,600 kip·in and the vertical component of U13–L14 produces a moment 2,883 × 20.5 = 59,100 kip·in. The sum of these moments is $M = 99{,}700$ kip·in. The stress at the edge of one gusset plate due to this moment is

$$f_b = \frac{6M}{td^2} = \frac{6 \times 99{,}700}{2({}^{13}\!/_{16})128^2} = 22.47 \text{ ksi}$$

The vertical, carrying a 362-kip load, imposes a stress

$$f_c = \frac{P}{A} = \frac{362}{2 \times 128 \times {}^{13}\!/_{16}} = 1.74 \text{ ksi}$$

The total stress then is $f = 22.47 + 1.74 = 24.21$ ksi.

The width b of the gusset at the edge is 48 in. Hence, the width–thickness ratio is $b/t = 48/({}^{13}\!/_{16}) = 59$. From step *b* in Art. 13.12, the maximum permissible b/t is $348/\sqrt{F_y} = 348/\sqrt{36} = 58 < 59$. The edge has to be stiffened. Use a stiffener angle 3 × 3 × ½ in.

For computation of the design compressive stress, assume the angle acts with a 12-in width of gusset plates. The slenderness ratio of the edge is 48/0.73 = 65.75. The maximum permissible slenderness ratio is

$$\sqrt{2\pi^2 \frac{E}{F_y}} = \sqrt{2\pi^2 \times \frac{29{,}000}{36}} = 126 > 65.75$$

Hence, the design compressive stress is

$$\begin{aligned} f_a &= 0.85F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{L}{r}\right)^2\right] \\ &= 0.85 \times 36\left[1 - \frac{36}{4\pi^2 \times 29{,}000}\left(\frac{48}{0.73}\right)^2\right] \\ &= 26.44 \text{ ksi} > 24.21 \text{ ksi} \qquad \text{OK} \end{aligned} \tag{13.38}$$

Next, the gusset plate is checked for shear and compression at the connection with diagonal U15–L14. The diagonal carries a factored compression load of 2299 kips. Shear paths 1–2 and 3–4 (Fig. 13.10) have a gross length of 93 in. From Table 13.11, the design shear stress is 20.9 ksi. Hence, design shear on these paths is

$$V_d = 2 \times 20.8 \times 93 \times {}^{13}\!/_{16} = 3143 \text{ kips} > 2299 \text{ kips} \qquad \text{OK}$$

Path 2–3 need not be investigated for compression. For compression on path 5–6, a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5–6 between the 30° lines is 82 in. The design stress, computed from Eq. (13.38) with a slenderness ratio of 52.9, is 27.9 ksi. The design strength of the gusset plate then is

$$P = 2 \times 27.9 \times 82 \times {}^{13}/_{16} = 3718 \text{ kips} > 2299 \text{ kips} \qquad \text{OK}$$

Also, the gusset plate is checked for shear and tension at the connection with diagonal L14–U13. The diagonal carries a tension load of 3272 kips. Shear paths 1–2 and 3–4 (Fig. 13.10) have a gross length of 98 in. From Table 13.11, the allowable shear stress is 20.9 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 20.9 \times 98 \times {}^{13}/_{16} = 3328 \text{ kips} > 3272 \text{ kips} \qquad \text{OK}$$

For path 2–3, capacity in tension with $F_y = 36$ ksi is

$$P_{23} = 2 \times 36 \times 27 \times {}^{13}/_{16} = 1580 \text{ kips}$$

For tension on path 5–6 (Fig. 13.10), a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5–6 between the 30° lines is a net of 83 in. The allowable tension then is

$$P_{56} = 2 \times 36 \times 83 \times {}^{13}/_{16} = 4856 \text{ kips} > 3272 \text{ kips} \qquad \text{OK}$$

As an illustration of investigating block shear in a tensile connection, consider path 1–2, 2–3, 3–4 for member L14–U13. Assume that paths 1–2 and 3–4 are 88 in long and contain 10 bolt holes in each leg, and that path 3–4 is 37 in long and contains seven bolt holes.

$$A_{\text{tn}} = (37 - 7 \times 1.25) \times {}^{13}/_{16} = 22.95 \text{ in}^2$$

$$A_{vn} = (88 - 20 \times 1.25) \times {}^{13}/_{16} = 51.19 \text{ in}^2$$

Note the usual assumption that the hole diameter is $^1/_{16}$ in larger than the bolt diameter for standard-size holes, and that an additional $^1/_{16}$ in is considered damaged and is not included for capacity calculations.

$$22.95 < 0.58 \times 51.19 = 29.69$$

Block shear is investigated using Eq. (13.35):

$$R_r = 2 \times 0.80\,(0.58 \times 58 \times 51.19 + 36 \times 37 \times {}^{13}/_{16}) = 4487 \text{ kips} > 3272 \text{ kips} \qquad \text{OK}$$

Welds to Develop Cover Plates. The fillet weld sizes selected are listed in Table 13.13 with their capacities, for an allowable stress of 26.10 ksi, based on a 58-ksi minimum specified tensile strength for A36 steel. The minimum specified strength of the welding rod would have to be at least equal to that of the base metal. A $^5/_{16}$-in weld is selected for the diagonals. It has a capacity of 5.76 kips/in.

TABLE 13.13 Weld Capacities—Load Factor Design

Weld size, in	Capacity of weld, kips/in
$^5/_{16}$	5.76
$^3/_8$	6.92
$^7/_{16}$	8.07
$^1/_2$	9.23

The allowable compressive stress for diagonal U15–L14 is 22.03 ksi. Then, the length of fillet weld required is

$$\frac{22.03(^7/_8)23^1/_8}{2\times 5.76} = 38.7 \text{ in}$$

For $F_y = 36$ ksi, the length of fillet weld required for diagonal L14–U13 is

$$\frac{36(^1/_2)23^1/_8}{2\times 5.76} = 36.1 \text{ in}$$

13.14 EXAMPLE—SERVICE LOAD DESIGN OF TRUSS JOINT

The joint shown in Fig. 13.14 is to be designed for connections with $1^1/_8$-in-diameter A325 bolts with an allowable stress $F_v = 15$ ksi. AASHTO requires a reduction factor of 0.875 for A325 bolts over 1 inch in diameter. The shear capacity of the bolts then is 15 ksi × 0.994 in^2 × 0.875 = 13.04 kips per shear plane.

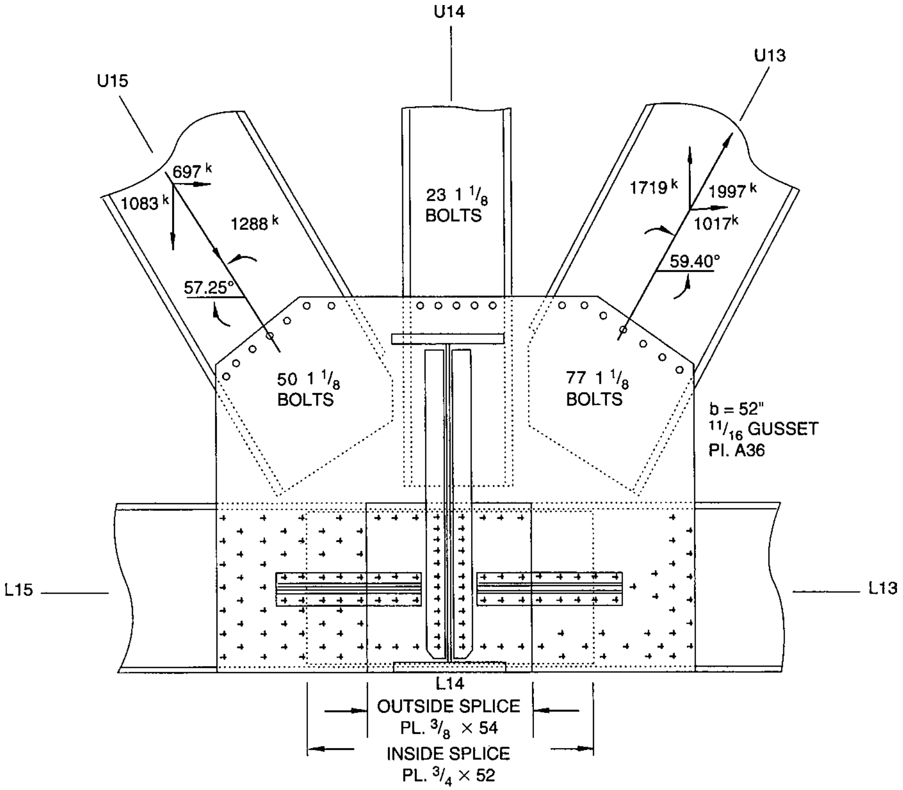


FIGURE 13.14 Truss joint for example of service load design.

Diagonal U15–L14. The diagonal is subjected to loads of 1250 kips compression and 90 kips tension. The connection is designed for 1288 kips, 3% over design load. The number of bolts required for the connection to the $^{11}/_{16}$-in-thick gusset plate is

$$N = \frac{1288}{2 \times 13.0} = 50 \text{ per side}$$

Diagonal L14–U13. The diagonal is subjected to a maximum tension of 1939 kips and a minimum tension of 628 kips. The connection is designed for 1997 kips, 3% over design load. The number of $1^{1}/_{8}$-in-diameter A325 bolts required is

$$N = \frac{1997}{2 \times 13.0} = 77 \text{ per side}$$

Vertical U14–L14. The vertical carries a compression load of 241 kips. The member is 74.53 ft long and has a cross-sectional area of 70.69 in^2. It has a radius of gyration $r = 10.52$ in and slenderness ratio of $KL/r = 74.53 \times 12/10.52 = 85.0$ with K taken as unity. The allowable compression stress then is

$$F_a = 16.98 - 0.00053\left(\frac{KL}{r}\right)^2$$

$$= 16.98 - 0.00053 \times 85.0^2 = 13.15 \text{ ksi} \tag{13.39}$$

The allowable unit stress for width–thickness ratio b/t, however, is $11.10 < 13.15$ and governs. Hence, the allowable load is

$$P = 70.69 \times 11.10 = 785 \text{ kips}$$

The number of bolts required is determined for 75% of the allowable load:

$$N = 0.75 \times \frac{785}{2 \times 13.0} = 23 \text{ bolts per side}$$

Splice of Chord Cover Plates. Each cover plate of the box chord is to be spliced with a plate on the inner and outer face (Fig. 13.15). A36 steel will be used for the splice material, as for the chord. Fasteners are $^{7}/_{8}$-in-diameter A325 bolts, with a capacity of 9.02 kips per shear plane.

The cover for L14–L15 (Fig. 13.14) is $^{13}/_{16}$ by $34^{3}/_{4}$ in but has 12-in-wide access perforations. Usable area of the plate is 18.48 in^2. The cover plate for L13–L14 is $^{13}/_{16} \times 34$ in, also with 12-in-wide access perforations. Usable area of this plate is 17.88 in^2. Design of the chord splice is based on the 17.88-in^2 area. The difference of 0.60 in^2 between this area and that of the larger cover plate will be made up on the L14–L15 side of the web plate splice as "cover excess."

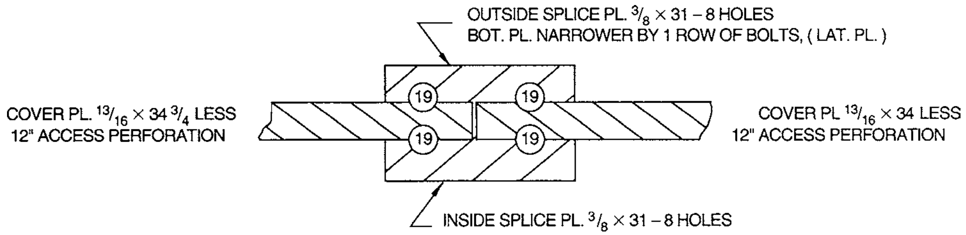


FIGURE 13.15 Cross section of chord cover-plate splice for example of service load design.

Where the net section of the joint elements is controlled by the allowance for bolts, only the excess exceeding 15% of the gross area is deducted from the gross area to obtain the design gross area, as in load-factor design (Art. 13.13).

For an allowable stress of 20 ksi in the cover plate, the number of bolts needed for the cover-plate splice is

$$N = \frac{17.88 \times 20}{2 \times 9.02} = 20 \text{ per side}$$

Try two splice plates, each $\frac{3}{8} \times 31$ in, with a gross area of 23.26 in^2. Assume eight 1-in-diameter bolt holes in the cross section. The area to be deducted for the holes then is

$$2 \times 0.375(8 \times 1 - 0.15 \times 31) = 2.51 \text{ in}^2$$

Consequently, the area of the design gross section is

$$A_n = 23.26 - 2.51 = 20.75 \text{ in}^2 > 17.88 \text{ in}^2 \qquad \text{OK}$$

Splice of Chord Web Plate. A splice is to be provided between the $1\frac{1}{4} \times 54$-in web of chord L14–L15 and the $1\frac{5}{8} \times 54$-in web of the L13–L14 chord. Because of the difference in web thickness, a $\frac{3}{8}$-in fill will be placed on the inner face of the $1\frac{1}{4}$-in web (Fig. 13.16). The gusset plate can serve as part of the needed splice material. The remainder is supplied by a plate on the inner face of the web and a place on the outer face of the gusset. Fasteners are $1\frac{1}{8}$-in-diameter A325 bolts, with a capacity of 13.0 kips.

The web of the L13–L14 chord has a design gross area of 87.75 in^2. After deduction of the 15% excess of seven $1\frac{1}{4}$-in-diameter bolt holes, the net area of this web is 86.69 in^2.

The web of the L14–L15 chord has a design gross area of 67.50 in^2. After deduction of the 15% excess of seven bolt holes from the chord splice and addition of the “cover excess” of 0.60 in^2, the net area of this web is 67.29 in^2.

The gusset plate is $\frac{11}{16}$ in thick and 123 in high. Assume that only the portion that overlaps the chord web, that is, 54 in, is effective in the splice. To account for the eccentric application of the chord load to the gusset, an effectiveness factor E_f [Eq. (13.37)] may be applied to the overlap (Art. 13.13). The moment of inertia of the gusset is $123^3t/12 = 155{,}100t$ in^4.

$$E_f = \frac{P/54t}{P/123t + P(123/2 - 54/2)(123/2)/155{,}100t} = 0.849$$

The gross area of the gusset overlap is $\frac{11}{16} \times 54 = 37.13$ in^2. After the deduction of the excess of thirteen $1\frac{1}{4}$-in-diameter bolt holes, the net area is 31.52 in^2. Then, the effective area of the gusset as a splice plate is $0.849 \times 31.52 = 26.76$ in^2.

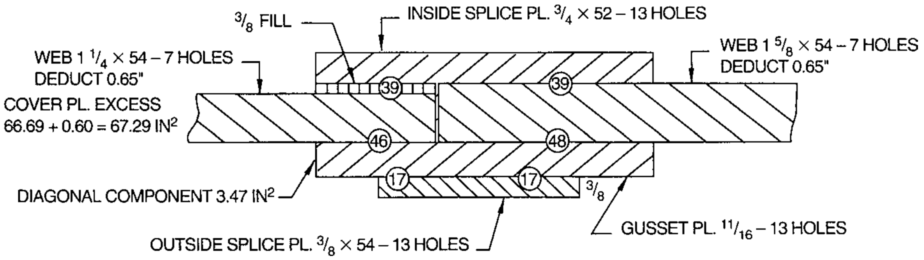


FIGURE 13.16 Cross section of chord web-plate splice for example of service load design.

TABLE 13.14 Number of Bolts for Plate Development

Plate	Area, in^2	Bolts
Inside splice plate	30.94	48
Outside splice plate	14.70	23
Gusset plate on L14–L15 side	(14.70 + 22.88 − 1.16) = 36.42	56
Gusset plate on L13–L14 side	(14.70 + 22.88) = 37.58	58

In addition to the 67.29 in^2 of web area, the gusset has to supply an area for transmission of the 49-kip horizontal component from diagonal U15–L14. With an allowable stress of 20 ksi, the area is $49/(20 \times 2) = 1.23\ \text{in}^2$. Hence, the equivalent web area from the L14–L15 side of the joint is $67.29 + 1.23 = 68.52\ \text{in}^2$. The number of bolts required to transfer the load to the inside and outside of the web should be based on the effective areas of gusset that add up to 68.52 in^2 but that provide a net moment in the joint close to zero.

The sum of the moments of the web components about the centerline of the combination of outside splice plate and gusset plate is $1.23 \times 0.19 + 67.29 \times 1.16 = 78.29$ kip · in. Dividing this by 2.53, the distance to the center of the inside splice plate, yields an effective area for the inside splice plate of 30.94 in^2. Hence, the effective area of the combination of the gusset and outside splice plates is $68.52 - 30.94 = 37.58\ \text{in}^2$. This is then distributed to the plates as follows: gusset, 22.88 in^2, and outside splice plate, 14.70 in^2.

The number of $1\frac{1}{8}$-in-diameter A325 bolts required to develop a plate with area A and allowable stress of 20 ksi is

$$N = \frac{20A}{13.04} = 1.534$$

Table 13.14 lists the number of bolts for the various plates.

Check of Gusset Plates. At section A–A (Fig. 13.11), each plate is 134 in wide and 123 in high, $^{11}/_{16}$ in thick. The allowable shear stress is 10 ksi. The sum of the horizontal components of the loads on the truss diagonals is 697 + 1017 = 1714 kips. This produces a shear stress on section A–A of

$$f_v = \frac{1714}{2 \times 134 \times {}^{11}/_{16}} = 9.30\ \text{ksi} < 10\ \text{ksi} \qquad \text{OK}$$

The vertical component of diagonal U15–L14 produces a moment about the centroid of the gusset of $1083 \times 21 = 22{,}740$ kip · in and the vertical component of U13–L14 produces a moment $1719 \times 20.5 = 35{,}240$ kip · in. The sum of these moments is 57,980 kip · in. The stress at the edge of one gusset plate due to this moment is

$$f_b = \frac{6M}{td^2} = \frac{6 \times 57{,}980}{2({}^{11}/_{16})134^2} = 14.09\ \text{ksi}$$

The vertical, carrying a 241-kip load, imposes a stress

$$f_c = \frac{P}{A} = \frac{241}{2 \times 134 \times {}^{11}/_{16}} = 1.31\ \text{ksi}$$

The total stress then is 14.09 + 1.31 = 15.40 ksi.

The width b of the gusset at the edge is 52 in. Hence, the width–thickness ratio is $b/t = 52/({}^{11}/_{16}) = 75.6$. From step 8$b$ in Art. 13.12, the maximum permissible b/t is $348\sqrt{F_y} = 348/\sqrt{36} = 58 < 75.6$. The edge has to stiffened. Use a stiffener angle $4 \times 3 \times \frac{1}{2}$ in.

For computation of the allowable compressive stress, assume the angle acts with a 12-in width of gusset plate. The slenderness ratio of the edge is 52/1.00 = 52.0. The maximum permissible slenderness ratio is

$$\sqrt{2\pi^2 \frac{E}{F_y}} = \sqrt{2\pi^2 \times \frac{29,000}{36}} = 126 > 552$$

Hence, the allowable stress from Eq. (13.39) is

$$F_a = 16.98 - 0.00053 \times 52^2 = 15.55 \text{ ksi} > 15.40 \text{ ksi} \qquad \text{OK}$$

Next, the gusset plate is checked for shear and compression at the connection with diagonal U15–L14. The diagonal carries a load of 1288 kips. Shear paths 1–2 and 3–4 (Fig. 13.10) have a gross length of 105 in. The allowable shear stress is 12 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 12 \times 105 \times {}^{11}\!/_{16} = 1733 \text{ kips} > 1288 \text{ kips} \qquad \text{OK}$$

Path 2–3 need not be investigated for compression. For compression on path 5–6, a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5–6 between the 30° lines is 88 in. The allowable stress, computed from Eq. (13.39) with a slenderness ratio $KL/r = 0.5 \times 25/0.198 = 63$, is 14.88 ksi. This permits the gusset to withstand a load

$$P = 2 \times 14.88 \times 88 \times {}^{11}\!/_{16} = 1800 \text{ kips} > 1288 \text{ kips}$$

Also, the gusset plate is checked for shear and tension at the connection with diagonal L14–U13. The diagonal carries a tension load of 1997 kips. Shear paths 1–2 and 3–4 (Fig. 13.10) have a gross length of 102 in. The allowable shear stress is 12 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 12 \times 102 \times {}^{11}\!/_{16} = 1683 \text{ kips}$$

For path 2–3, capacity in tension with an allowable stress of 20 ksi is

$$P_{23} = 2 \times 20 \times 21.6 \times {}^{11}\!/_{16} = 594 \text{ kips} > (1997 - 1683) \qquad \text{OK}$$

For tension on path 5–6 (Fig. 13.10), a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5–6 between the 30° lines is a net of 88 in. The allowable tension then is

$$P = 2 \times 20 \times 88 \times {}^{11}\!/_{16} = 2420 \text{ kips} > 1997 \text{ kips} \qquad \text{OK}$$

Welds to Develop Cover Plates. The fillet weld sizes selected are listed in Table 13.15 with their capacities, for an allowable stress of 15.66 ksi. The welding notes in Art. 13.13 apply. A $^5/_{16}$-in weld is selected for the diagonals. It has a capacity of 3.46 kips/in.

The allowable compressive stress for diagonal U15–L14 is 11.93 ksi. Then, length of fillet weld required is

$$\frac{11.93(^7/_8)23\,^1/_8}{2 \times 3.46} = 34.9 \text{ in}$$

TABLE 13.15 Weld Capacities—Service Load Design

Weld size, in	Capacity of weld, kips/in
$^5/_{16}$	3.46
$^3/_8$	4.15
$^7/_{16}$	4.84
$^1/_2$	5.54

Other required weld lengths are calculated in a similar manner.

$$\frac{20.99(^1/_2)23\,^1/_8}{2\times 3.46} = 35.1 \text{ in}$$

13.15 SKEWED BRIDGES

To reduce scour and to avoid impending stream flow, it is generally desirable to orient piers with centerlines parallel to direction of flow; therefore, skewed spans may be required. Truss construction does not lend itself to bridges where piers are not at right angles to the superstructure (skew crossings). Hence, these should be avoided and this can generally be done by using longer spans with normal piers. In economic comparisons, it is reasonable to assume some increased cost of steel fabrication if skewed trusses are to be used.

If a skewed crossing is a necessity, it is sometimes possible to establish a panel length equal to the skew distance W tan ϕ, where W is the distance between trusses and ϕ is the skew angle. This aligns panels and maintains perpendicular connections of floorbeams to the trusses (Fig. 13.17). If such a layout is possible, there is little difference in cost and skewed spans and normal spans. Design principles are similar. If the skewed distance is less than the panel length, it might be possible to take up the difference in the angle of inclination of the end post, as shown in Fig. 13.17. This keeps the cost down, but results in trusses that are not symmetrical within themselves and, depending on the proportions, could be very upleasing esthetically. If the skewed distance is greater than the panel length, it may be necessary to vary panel lengths along the bridge. One solution to such a skew is shown in Fig. 13.18, where a truss, similar to the truss in Fig. 13.17, is not symmetrical within itself and, again, might not be esthetically pleasing. The most desirable solution for skewed bridges is the alternative shown in Fig. 13.17.

Skewed bridges require considerably more analysis than normal ones, because the load distribution is nonuniform. Placement of loads for maximum effect, distribution through the floorbeams, and determination of panel point concentrations are all affected by the skew. Unequal deflections of the trusses require additional checking of sway frames and floor system connections to the trusses.

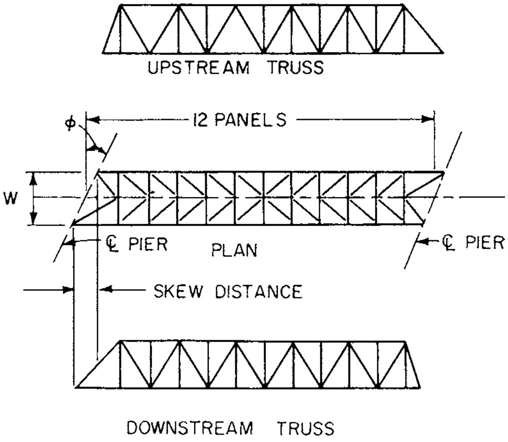


FIGURE 13.17 Skewed bridge with skew distance less than panel length.

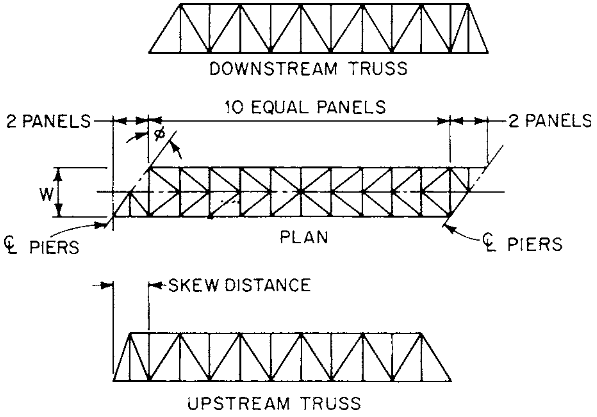


FIGURE 13.18 Skewed bridge with skew distance exceeding panel length.

13.16 TRUSS BRIDGES ON CURVES

When it is necessary to locate a truss bridge on a curve, designers should give special consideration to truss spacing, location of bridge centerline, and stresses.

For highway bridges, location of bridge centerline and stresses due to centrifugal force are of special concern. For through trusses, the permissible degree of curvature is limited because the roadway has to be built on a curve, while trusses are planar, constructed on chords. Thus, only a small degree of throw, or offset from a tangent, can be tolerated. Regardless of the type of bridge, horizontal centrifugal forces have to be transmitted through the floor system to the lateral system and then to supports.

For railroad truss bridges, truss spacing usually provides less clearance than the spacing for highway bridges. Thus, designers must take into account tilting of cars due to superelevation and the swing of cars overhanging the track. The centerline of a through-truss bridge on a curve often is located so that the overhang at mid-span equals the overhang at each span end. For bridges with more than one truss span, layout studies should be made to determine the best position for the trusses.

Train weight on a bridge on a curve is not centered on the centerline of track. Loads are greater on the outer truss than on the inner truss because the resultant of weight and centrifugal force is closer to the outer truss. Theoretically, the load on each panel point would be different and difficult to determine exactly. Because the difference in loading on inner and outer trusses is small compared with the total load, it is generally adequate to make a simple calculation for a percentage increase to be applied throughout a bridge.

Stress calculations for centrifugal forces are similar to those for any horizontal load. Floorbeams, as well as the lateral system, should be analyzed for these forces.

13.17 TRUSS SUPPORTS AND OTHER DETAILS

End bearings transmit the reactions from trusses to substructure elements, such as abutments or piers. Unless trusses are supported on tall slender piers that can deflect horizontally without exerting large forces on the trusses, it is customary to provide expansion bearings at one end of the span and fixed bearings at the other end.

Anchoring a truss to the support, a fixed bearing transmits the longitudinal loads from wind and live-load traction, as well as vertical loads and transverse wind. This bearing also must incorporate a hinge, curved bearing plate, pin arrangement, or elastomeric pads to permit end rotation of the truss in its plane.

An expansion bearing transmits only vertical and transverse loads to the support. It permits changes in length of trusses, as well as end rotation.

Many types of bearings are available. To ensure proper functioning of trusses in accordance with design principles, designers should make a thorough study of the bearings, including allowances for reactions, end rotations, and horizontal movements. For short trusses, a rocker may be used for the expansion end of a truss. For long trusses, it generally is necessary to utilize some sort of roller support. See also Arts. 10.22 and 11.9.

Inspection Walkways. An essential part of a truss design is provision of an inspection walkway. Such walkways permit thorough structural inspection and also are of use during erection and painting of bridges. The additional steel required to support a walkway is almost insignificant.

13.18 CONTINUOUS TRUSSES

Many river crossings do not require more than one truss span to meet navigational requirements. Nevertheless, continuous trusses have made possible economical bridge designs in many localities. Studies of alternative layouts are essential to ensure selection of the lowest-cost arrangement. The principles outlined in preceding articles of this section are just as applicable to continuous trusses as to simple spans. Analysis of the stresses in the members of continuous trusses, however, is more complex, unless computer-aided design is used. In this latter case, there is no practical difference in the calculation of member loads once the forces have been determined. However, if the truss is truly continuous, and, therefore, the truss in each span is statically indeterminant, the member forces are dependent on the stiffness of the truss members. This may make several iterations of member-force calculations necessary. But where sufficient points of articulation are provided to make each individual truss statically determinant, such as the case where a suspended span is inserted in a cantilever truss, the member forces are not a function of member stiffness. As a result, live-load forces need be computed only once, and dead-load member forces need to be updated only for the change in member weight as the design cycle proceeds. When the stresses have been computed, design proceeds much as for simple spans.

The preceding discussion implies that some simplification is possible by using cantilever design rather than continuous design. In fact, all other things being equal, the total weight of members will not be much different in the two designs if points of articulation are properly selected. More roadway joints will be required in the cantilever, but they, and the bearings, will be subjected to less movement. However, use of continuity should be considered because elimination of the joints and devices necessary to provide for articulation will generally reduce maintenance, stiffen the bridge, increase redundancy, and, therefore, improve the general robustness of the bridge.

CHAPTER 14

ARCH BRIDGES*

Kenneth J. Wright, P.E.
Vice President and Senior Professional Associate
HDR Engineering, Inc.
Pittsburgh, Pennsylvania

Matthew A. Brunner, P.E.
Professional Associate
HDR Engineering, Inc.
Pittsburgh, Pennsylvania

Basic principles of arch construction have been known and used successfully for centuries. Magnificent stone arches constructed under the direction of engineers of the ancient Roman Empire are still in service after 2000 years, as supports for aqueducts or highways. One of the finest examples is the Pont du Gard, built as part of the water-supply system for the city of Nîmes, France.

Stone was the principal material for arches until about two centuries ago. In 1779, the first metal arch bridge was built. Constructed of cast iron, it carried vehicles over the valley of the Severn River at Coalbrookedale, England. The bridge is still in service but now is restricted to pedestrian traffic. Subsequently, many notable iron or steel arches were built. Included was Eads' Bridge, with three tubular steel arch spans, 502, 520, and 502 ft, over the Mississippi River at St. Louis. This bridge was completed in 1874, closed due to safety concerns in 1991, then rehabilitated and reopened in 2003. It now carries large daily volumes of heavy highway traffic, as well as the MetroLink light-rail mass transit system.

Until 1900, stone continued as a strong competitor of iron and steel. After 1900, concrete became the principal competitor of steel for shorter-span arch bridges.

Development of structural steels made it feasible to construct long-span arches economically. The 1675-ft Bayonne Bridge, between Bayonne, New Jersey, and Staten Island, New York, was completed in 1931. The 1000-ft Lewiston–Queenston Bridge over the Niagara River on the United States–Canadian border was put into service in 1962. Availability of more high-strength steels and improved fabrication techniques expanded the feasibility of steel arches for long spans. Examples include the 1255-ft-span Fremont Bridge in Portland, Oregon, finished in 1973, the 1700-ft-span New River Gorge Bridge near Fayetteville, West Virginia, opened in 1977, and most recently, the 1800-ft-span Lupu Bridge in Shanghai, China, completed in 2003.

Historically, most steel arches that have been built lie in vertical planes. Accordingly, this section discusses design principles for such arches. A few arch bridges, however, have been constructed with

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*Revised; originally authored by George S. Richardson (deceased), Richardson, Gordon and Associates, Pittsburgh, in the First Edition, and updated by Arthur W. Hedgren, Jr., P.E. (retired), HDR Engineering, Inc., Pittsburgh, in the Second and Third Editions.

ribs inclined toward each other. This construction is effective in providing lateral stability and offers good appearance. Also, the decrease in average distance between the arch ribs of a bridge often makes possible the use of more economical Vierendeel-girder bracing instead of trussed bracing. Generally, though, inclined arches are not practicable for bridges with very wide roadways unless the span is very long, because of possible interference with traffic clearances. Further, inclined arch ribs result in more complex beveled connections between members.

14.1 TYPES OF ARCHES

In the most natural type of arch, the horizontal component of each reaction, or thrust, is carried into a buttress, which also carries the vertical reaction. This type will be referred to as the **true arch.** The application of arch construction, however, can be greatly expanded economically by carrying the thrust through a tie, a tension member between the ends of the span. This type will be referred to as a **tied arch.**

Either a truss or girder may be used for the arch member. Accordingly, arch bridges are classified as **trussed** or **solid-ribbed.**

Arch bridges are also classified according to the degree of articulation. A **fixed arch,** in which the construction prevents rotation at the ends of the span, is statically indeterminate, so far as external reactions are concerned, to the third degree. If the span is articulated at the ends, it becomes **two-hinged** and statically indeterminate to the first degree. In recent years, most arch bridges have been constructed as either fixed or two-hinged. Sometimes a hinge is included at the crown in addition to the end hinges. The bridge then becomes **three-hinged** and statically determinate.

In addition, arch bridges are classified as **deck** construction when the arches are entirely below the deck. This is the most usual type for the true arch. Tied arches, however, normally are constructed with the arch entirely above the deck and the tie at deck level. This type will be referred to as a **through arch.** Both true and tied arches, however, may be constructed with the deck at some intermediate elevation between springing (base of arch) and crown. These types are classified as **half-through.**

The arch also may be used as one element combined with another type of structure. For example, many structures have been built with a three-span continuous truss as the basic structure and with the central span arched and tied. This section is limited to structures in which the arch type is used independently.

14.2 ARCH FORMS

A great variety of forms have been used for trussed or solid-ribbed arch bridges. The following are some of the principal forms used.

Lindenthal's Hell Gate Bridge over the East River in New York has trusses deep at the ends and shallow at the crown. The bottom chord is a regular arch form. The top chord follows a reversed curve transitioning from the deep truss at the end to the shallow truss at the center. Accordingly, it is customary to refer to arch trusses of this form as Hell-Gate-type trusses. In another form commonly used, top and bottom chords are parallel. For a two-hinged arch, a crescent-shaped truss is another logical form.

For solid-ribbed arches, single-web or box girders may be used. Solid-ribbed arches usually are built with girders of constant depth. Variable-depth girders, tapering from deep sections at the springing to shallower sections at the crown, however, have been used occasionally for longer spans. As with trussed construction, a crescent-shaped girder is another possible form for a two-hinged arch.

Tied arches permit many variations in form to meet specific site conditions. In a true arch (without ties), the truss or solid rib must carry both thrust and moment under variable loading conditions. These stresses determine the most effective depth of truss or girder. In a tied arch, the thrust is carried by the arch truss or solid rib, but the moment for variable loading conditions is divided between arch and tie, somewhat in proportion to the respective stiffnesses of these two members. For this reason, for example, if a deep girder is used for the arch and a very shallow member for the tie, most of the moment for variable loading is carried by the arch rib. The tie acts primarily as a tension member. But if a relatively deep member is used for the tie, either in the form of a girder or a truss, it carries a high proportion of the moment, and a relatively shallow member may be used for the arch rib. In some cases, a truss has been used for the arch tie in combination with a shallow, solid rib for the arch. This combination is particularly applicable for double-deck construction.

Rigid-framed bridges, sometimes used for grade-separation structures, are basically another form of two-hinged or fixed arch. The generally accepted arch form is a continuous, smooth-curve member or a segmental arch (straight between panel points) with breaks located on a smooth-curve axis. For a rigid frame, however, the arch axis becomes rectangular in form. Nevertheless, the same principles of stress analysis may be used as for the smooth-curve arch form.

The many different types and forms of arch construction provide bridge engineers with numerous combinations to meet variable site conditions and desired esthetics.

14.3 SELECTION OF ARCH TYPE AND FORM

Some of the most important elements influencing selection of type and form of arch follow.

Foundation Conditions. If a bridge is required to carry a roadway or railroad across a deep valley with steep walls, an arch is probably a feasible and economical solution. (This assumes that the required span is within reasonable limits for arch construction.) The condition of steep walls indicates that foundation conditions should be suitable for the construction of small, economical abutments. Generally, it might be expected that under these conditions the solution would be a deck bridge. There may be other controls, however, that dictate otherwise. For example, the desirability of placing the arch bearings safely above high-water elevation, as related to the elevation of the deck, may indicate the advisability of a half-through structure to obtain a suitable ratio of rise to span. Also, variable foundation conditions on the walls of the valley may fix a particular elevation as much more preferable to others for the construction of the abutments. Balancing of such factors will determine the best layout to satisfy foundation conditions.

Tied-Arch Construction. At a bridge location where relatively deep foundations are required to carry heavy reactions, a true arch, transmitting reactions directly to buttresses, is not economical, except for short spans. There are two alternatives, however, that may make it feasible to use arch construction.

If a series of relatively short spans can be used, arch construction may be a good solution. In this case, the bridge would comprise a series of equal or nearly equal spans. Under these conditions, dead-load thrusts at interior supports would be balanced or nearly balanced by adjacent spans. With the short spans, unbalanced live-load thrusts would not be large. Accordingly, even with fairly deep foundations, intermediate pier construction may be almost as economical as for some other layout with simple or continuous spans. There are many examples of stone, concrete, and steel arches in which this arrangement has been used.

The other alternative to meet deep foundation requirements is tied-arch construction. The tie relieves the foundation of the thrust. This places the arch in direct competition with other types of structures for which only vertical reactions would result from the application of dead and live loading.

There has been some concern over the safety of tied-arch bridges because the ties can be classified as fracture-critical members. A fracture-critical member is one that would cause collapse of the bridge if it fractured. Since the horizontal thrust of a tied arch is resisted by its tie, most tied arches would collapse if the tie were lost. While some concern over fracture of welded tie girders is well-founded, methods are available for introducing redundancy in the construction of ties. These methods include using ties fabricated from bolted, built-up components and multiple posttensioning tendons. This type of structure should not be dismissed over these concerns, because it can be easily designed to address them. In particular, the high-performance steels (HPS) developed in recent years (Art. 1.1.5) provide superior notch toughness, minimizing concerns over fracture-critical members. Studies underway may eliminate the need for internal redundancy when HPS is used for tie girders.

Length of Span. Generally, determination of the best layout for a bridge starts with trial of the shortest feasible main span. Superstructure costs per foot increase rapidly with increase in span. Unless there are large offsetting factors that reduce substructure costs when spans are lengthened, the shortest feasible main span will be the most economical.

Arch bridges are applicable over a wide range of span lengths. The examples in Art. 14.8 cover a range from a minimum of 193 ft to a maximum of 1700 ft. With present high-strength steels and under favorable conditions, spans on the order of 2000 ft or more are feasible for economical arch construction.

In addition to foundation conditions, many other factors may influence the length of span selected at a particular site. Over navigable waters, span is normally set by clearance requirements of regulatory agencies. For example, the U.S. Coast Guard has final jurisdiction over clearance requirements over navigable streams. In urban or other highly built-up areas, the span may be fixed by existing site conditions that cannot be altered.

Truss or Solid Rib. Most highway arch bridges with spans up to 750 ft have been built with solid ribs for the arch member. There may, however, be particular conditions that would make it more economical to use trusses for considerably shorter spans. For example, for a remote site with difficult access, truss arches may be less expensive than solid-ribbed arches, because the trusses may be fabricated in small, lightweight sections, much more readily transported to the bridge site.

In the examples of Art. 14.8, solid ribs have been used in spans up to 1255 ft, as for the Fremont Bridge, Portland, Oregon. For spans over 750 ft, however, truss arches should be considered. Also, for spans under this length for very heavy live loading, as for railroad bridges, truss arches may be preferable to solid-rib construction.

For spans over about 600 ft, control of deflection under live loading may dictate the use of trusses rather than solid ribs. This may apply to bridges designed for heavy highway loading or heavy transit loading as well as for railroad bridges. For spans above 1000 ft, truss arches, except in some very unusual case, should be used.

Articulation. For true, solid-ribbed arches the choice between fixed and hinged ends will be a narrow one. In a true arch it is possible to carry a substantial moment at the springing line if the bearing details are arranged to provide for it. This probably will result in some economy, particularly for long spans. It is, however, common practice to use two-hinged construction.

An alternative is to let the arch act as two-hinged under partial or full dead load and then fix the end bearings against rotation under additional load.

Tied arches act substantially as two-hinged, regardless of the detail of the connection to the tie.

Some arches have been designed as three-hinged under full or partial dead load and then converted to the two-hinged condition. In this case, the crown hinge normally is located on the bottom chord of the truss. If the axis of the bottom chord follows the load thrust line for the three-hinged condition, there will be no stress in the top chord or web system of the truss. Top chord and web members will be stressed only under load applied after closure. These members will be relatively light and reasonably uniform in section. The bottom chord becomes the main load-bearing member.

If, however, the arch is designed as two-hinged, the thrust under all loading conditions will be nearly equally divided between top and bottom chords. For a given ratio of rise to span, the total horizontal thrust at the end will be less than that for the arrangement with part of the load carried as a three-hinged arch. Shifting from three to two hinges has the effect of increasing the rise of the arch over the rise measured from springing to centerline of bottom chord.

Esthetics. For arch or suspension-type bridges, a functional layout meeting structural requirements normally results in simple, clean-cut, and graceful lines. For long spans, no other bridge type offered serious competition so far as excellent appearance is concerned until about 1950. Since then, introduction of cable-stayed bridges and orthotropic-deck girder construction has made construction of good-looking girders feasible for spans of 2500 ft or more. Even with conventional deck construction but with the advantage of high-strength steels, very long girder spans are economically feasible and esthetically acceptable.

The arch then must compete with suspension, cable-stayed, and girder bridges so far as esthetic considerations are concerned. From about 1000 ft to the maximum practical span for arches, the only competitors are the cable-supported types.

Generally, architects and engineers prefer, when all other things are equal, that deck structures be used for arch bridges. If a through or half-through structure must be used, solid-ribbed arches are desirable when appearance is of major concern, because the overhead structure can be made very light and clean-cut (see Figs. 14.5 to 14.8 and 14.15 to 14.18).

Arch Form as Related to Esthetics. For solid-ribbed arches, designers are faced with the decision as to whether the rib should be curved or constructed on segmental chords (straight between panel points). A rib on a smooth curve presents the best appearance. Curved ribs, however, involve some increase in material and fabrication costs.

Another decision is whether to make the rib constant depth or tapered.

One factor that has considerable bearing on both these decisions is the ratio of panel length to span. As panel length is reduced, the angular break between chord segments is reduced, and a segmental arch approaches a curved arch in appearance. An upper limit for panel length should be about $^{1}/_{15}$ of the span.

In a study of alternative arch configurations for a 750-ft span, four solid-ribbed forms were considered. An architectural consultant rated these in the following order:

Tapered rib, curved

Tapered-rib on chords

Constant-depth rib, curved

Constant-depth rib on chords

He concluded that the tapered rib, 7 ft deep at the springing line and 4 ft deep at the crown, added considerably to the esthetic quality of the design as compared with a constant-depth rib. He also concluded that the tapered rib would minimize the angular breaks at panel points with the segmental chord axis. The tapered rib on chords was used in the final design of the structure. The effect of some of these variables on economy is discussed in Art. 14.6.

14.4 COMPARISON OF ARCH WITH OTHER BRIDGE TYPES

Because of the wide range of span length within which arch construction may be used (Art. 14.3), it is competitive with almost all other types of structures.

Comparison with Simple Spans. Simple-span girder or truss construction normally falls within the range of the shortest spans used up to a maximum of about 800 ft. Either true arches under favorable conditions or tied arches under all conditions are competitive within the range of 200 to 800 ft. (There will be small difference in cost between these two types within this span range.) With increasing emphasis on appearance of bridges, arches are generally selected rather than simple-span construction, except for short spans for which beams or girders may be used.

Comparison with Cantilever or Continuous Trusses. The normal range for cantilever or continuous-truss construction is on the order of 500 to 1800 ft for main spans. More likely, a top limit is about 1500 ft. Tied arches are competitive for spans within the range of 500 to 1000 ft. True arches are competitive, if foundation conditions are favorable, for spans from 500 ft to the maximum for the other types. The relative economy of arches, however, is enhanced where site conditions make possible use of relatively short-span construction over the areas covered by the end spans of the continuous or cantilever trusses.

The economic situation is approximately this: For three-span continuous or cantilever layouts arranged for the greatest economy, the cost per foot will be nearly equal for end and central spans. If a tied or true arch is substituted for the central span, the cost per foot may be more than the average for the cantilever or continuous types. If, however, relatively short spans are substituted for the end spans of these types, the cost per foot over the length of those spans is materially reduced. Hence, for a combination of short spans and a long arch span, the overall cost between end piers may be less than for the other types. In any case, the cost differential should not be large.

Comparison with Cable-Stayed and Suspension Bridges. Such structures normally are not used for spans of less than 500 ft. Above 3000 ft, suspension bridges are probably the most practical solution. In the shorter spans, self-anchored construction is likely to be more economical than independent anchorages. Arches are competitive in cost with the self-anchored suspension type or similar functional type with cable-stayed girders or trusses. There has been little use of suspension bridges for spans under 1000 ft, except for some self-anchored spans. For spans above 1000 ft, it is not possible to make any general statement of comparative costs. Each site requires a specific study of alternative designs.

14.5 ERECTION OF ARCH BRIDGES

Erection conditions vary so widely that it is not possible to cover many in a way that is generally applicable to a specific structure.

Cantilever Erection. For arch bridges, except short spans, cantilever erection usually is used. This may require use of two or more temporary piers. Under some conditions, such as an arch over a deep valley where temporary piers are very costly, it may be more economical to use temporary tiebacks.

Particularly for long spans, erection of trussed arches often is simpler than erection of solid-ribbed arches. The weights of individual members are much smaller, and trusses are better adapted to cantilever erection. The Hell-Gate-type truss (Art. 14.2) is particularly suitable because it requires little if any additional material in the truss on account of erection stresses.

For many double-deck bridges, use of trusses for the arch ties simplifies erection when trusses are deep enough and the sections large enough to make cantilever erection possible and at the same time to maintain a clear opening to satisfy temporary navigation or other clearance requirements.

Control of Stress Distribution. For trussed arches designed to act as three-hinged, under partial or full dead load, closure procedures are simple and positive. Normally, the two halves of the arch are erected to ensure that the crown hinge is high and open. A top-chord member at the crown is temporarily omitted. The trusses are then closed by releasing the tiebacks or lowering temporary intermediate supports. After all dead load for the three-hinged condition is on the span, the top chord is closed by inserting the final member. During this operation, consideration must be given to temperature effects to ensure that closure conditions conform to temperature-stress assumptions.

If a trussed arch has been designed to act as two-hinged under all conditions of loading, the procedure may be first to close the arch as three-hinged. Then, jacks are used at the crown to attain the calculated stress condition for top and bottom chords under the closing erection load and temperature condition. This procedure, however, is not as positive and not as certain of attaining agreement between actual and calculated stresses as the other procedure described. (There is a difference of opinion among bridge engineers on this point.)

Another means of controlling stress distribution may be used for tied arches. Suspender lengths are adjusted to alter stresses in both the arch ribs and the ties.

Fixed Bases. For solid-ribbed arches to be erected over deep valleys, there may be a considerable advantage in fixing the ends of the ribs. If this is not provided for in design, it may be necessary to provide temporary means for fixing bases for cantilever erection of the first sections of the ribs. If the structure is designed for fixed ends, it may be possible to erect several sections as cantilevers before it becomes necessary to install temporary tiebacks.

14.6 DESIGN OF ARCH RIBS AND TIES

Computers greatly facilitate preliminary and final design of all structures. They also make possible consideration of many alternative forms and layouts, with little additional effort, in preliminary design. Even without the aid of a computer, however, experienced designers can, with reasonable ease, investigate alternative layouts and arrive at sound decisions for final arrangements of structures.

Rise–Span Ratio. The generally used ratios of rise to span cover a range of about 1:5 to 1:6. For all but two of the arch examples in Art. 14.8, the range is from a maximum of 1:4.7 to minimum of 1:6.3. The flatter rise is more desirable for through arches, because appearance will be better. Cost will not vary appreciably within the rise limits of 1:5 to 1:6. These rise ratios apply both to solid ribs and to truss arches with rise measured to the bottom chord.

Panel Length. For solid-ribbed arches fabricated with segmental chords, panel length should not exceed $^1/_{15}$ of the span. This is recommended for esthetic reasons, to avoid large angular breaks at panel points. Also, for continuously curved axes, bending stresses in solid-ribbed arches become fairly severe if long panels are used. Other than this limitation, the best panel length for an arch bridge will be determined by the usual considerations, such as economy of deck construction.

Ratio of Depth to Span. In the examples in Art. 14.8, the true arches (without ties) with constant-depth solid ribs have depth–span ratios from 1:58 to 1:79. The larger ratio, however, is for a short span. A more normal range is 1:70 to 1:80. These ratios also are applicable to solid-ribbed tied arches with shallow ties. In such cases, since the ribs must carry substantial bending moments, depth requirements are little different from those for a true arch. For structures with variable-depth ribs, the depth–span ratio may be relatively small (Fig. 14.7).

For tied arches with solid ribs and deep ties, rib depth may be small, because the ties carry substantial moments, thus reducing the moments in ribs. For a number of such structures, the depth–span ratio ranges from 1:140 to 1:190, and for the Fremont Bridge, Portland, Oregon, is as low as 1:314. Note that such shallow ribs can be used only with girder or trussed ties of considerable depth.

For truss arches, whether true or tied, the ratio of crown depth to span may range from 1:25 to 1:50. Depth of tie has little effect on depth of truss required. Except for some unusual arrangement, the moment of inertia of the arch truss is much larger than the moment of inertia of its tie, which primarily serves as a tension member to carry the thrust. Hence, an arch truss carries substantial bending moments whether or not it is tied, and required depth is not greatly influenced by presence or absence of a tie.

Single-Web or Box Girders. For very short arch spans, single-web girders are more economical than box girders. For all the solid-ribbed arches in Art. 14.8, however, box girders were used for the arch ribs. These examples include a minimum span of 193 ft. Welded construction greatly facilitates use of box members in all types of structures.

For tied arches for which shallow ties are used, examples in Art. 14.8 show use of members made up of web plates with diaphragms and rolled shapes with posttensioned strands. More normally, however, the ties, like solids ribs, would be box girders.

Truss Arches. All the usual forms of bolted or welded members may be used in truss arches but usually sealed, welded box members are preferred. These present a clean-cut appearance. There also is an advantage in the case of maintenance.

Another variation of truss arches that can be considered is use of Vierendeel trusses (web system without diagonals). In the past, complexity of stress analysis for this type discouraged their use. With computers, this disadvantage is eliminated. Various forms of Vierendeel truss might well be used for both arch ribs and ties. There has been some use of Vierendeel trusses for arch bracing, as shown in the examples in Art. 14.8. This design provides an uncluttered, attractive bracing system.

Dead-Load Distribution. It is normal procedure for both true and tied solid-ribbed arches to use an arch axis conforming closely to the dead-load thrust line. In such cases, if the rib is cambered for dead load, there will be no bending in the rib under that load. The arch will be in pure compression. If a tied arch is used, the tie will be in pure tension. If trusses are used, the distribution of dead-load stress may be similarly controlled. Except for three-hinged arches, however, it will be necessary to use jacks at the crown or other stress-control procedures to attain the stress distribution that has been assumed. In an idealized uniformly loaded configuration, the thrust line would exhibit the shape of a funicular curve. Practically, even dead load is not perfectly uniform and is transmitted to the arch at panel points, resulting in a thrust line that deviates slightly from the idealized configuration.

Live-Load Distribution. One of the advantages of arch construction is that fairly uniform live loading, even with maximum-weight vehicles, creates relatively low bending stresses in either the rib or the tie. Maximum bending stresses occur only under partial unbalanced loading not likely to be realized under normal heavy traffic flow. Maximum live-load deflection occurs in the vicinity of the quarter point with live load over about half the span.

Wind Stresses. These may control design of long-span arches carrying two-lane roadways or of other structures for which there is relatively small spacing of ribs compared with span length. For a spacing–span ratio larger than 1:20, the effect of wind may not be severe. As this ratio becomes substantially smaller, wind may affect sections in many parts of the structure.

Thermal Stresses. Temperature causes stress variation in arches. One effect sometimes neglected but which should be considered is that of variable temperature throughout a structure. In a through, tied arch during certain times of the day or night, there may be a large difference in temperature between rib and tie due to different conditions of exposure. This difference in temperature easily reaches 30°F and may be much larger.

Deflection. For tied arches of reasonable rigidity, deflection under live load causes relatively minor changes in stress (secondary stresses). For a 750-ft span with solid-ribbed arches 7 ft deep at the springing line and 4 ft deep at the crown and designed for a maximum live-load deflection of $^1/_{800}$ of the span, the secondary effect of deflections was computed as less than 2% of maximum allowable unit stress. For a true arch, however, this effect may be considerably larger and must be considered, as required by design specifications.

Ratio of Dead Load to Total Load. For some 20 arch spans checked, the ratio of dead load to total load varied within the narrow range of 0.74 to 0.88. A common ratio is about 0.85. This does not mean that the ratio of dead-load stress to maximum total stress will be 0.85. This stress ratio may be fairly realistic for a fully loaded structure, at least for most of the members in the arch system. For partial live loading, however, which is the loading condition causing maximum live-load stress, the ratio of dead to total stress will be much lower, particularly as span decreases.

For most of the arches checked, the ratio of weight of arch ribs or, in the case of tied arches, weight of ribs and ties, to total load ranged from about 0.20 to 0.30. This is true despite the wide range of spans included and the great variety of steels used in their construction.

Use of high-strength steels helps to maintain a low ratio for the longer spans. For example, for the Fort Duquesne Bridge, Pittsburgh, a double-deck structure of 423-ft span with a deep truss as a tie, the ratio of weight of arch ribs plus truss ties to total load is about 0.22, or a normal factor within the range previously cited. For this bridge, arch ribs and trusses were designed with 77% of A440 steel and the remainder A36. These are suitable strength steels for this length of span.

For the Fort Pitt Bridge, Pittsburgh, with a 750-ft span and the same arrangement of structure with shallow girder ribs and a deep truss for the ties, the ratio of weight of steel in ribs plus trussed ties to total load is 0.33. The same types of steel in about the same percentages were used for this structure as for the Fort Duquesne Bridge. A higher-strength steel, such as A514, would have resulted in a much lower percentage for weight of arch ribs and trusses and undoubtedly in considerable economy. When the Fort Pitt arch was designed, however, the owner decided there had not been sufficient research and testing of the A514 steel to warrant its use in this structure.

For a corresponding span of 750 ft designed later for the Glenfield Bridge at Pittsburgh, a combination of A588 and A514 steels was used for the ribs and ties. The ratio of weight of ribs plus ties to total load is 0.19.

Incidentally, the factors for this structure, a single-deck bridge with six lanes of traffic plus full shoulders, are almost identical with the corresponding factors for the Sherman Minton Bridge at Louisville, Ky., an 800-ft double-deck structure with truss arches carrying three lanes of traffic on each deck. The factors for the Pittsburgh bridge are 0.88 for ratio of dead load to total load and 0.19 for ratio of weight of ribs plus ties to total load. The corresponding factors for the Sherman Minton arch are 0.85 and 0.19. Although these factors are almost identical, the total load for the Pittsburgh structure is considerably larger than that for the Louisville structure. The difference may be accounted for primarily by the double-deck structure for the latter, with correspondingly lighter deck construction.

For short spans, particularly those on the order of 250 ft or less, the ratio of weight of arch rib to total load may be much lower than the normal range of 0.20 to 0.30. For example, for a short span of 216 ft, this ratio is 0.07. On the other hand, for a span of only 279 ft, the ratio is 0.18, almost in the normal range.

A ratio of arch-rib weight to total load may be used by designers as one guide in selecting the most economical type of steel for a particular span. For a ratio exceeding 0.25, there is an indication that a higher-strength steel than has been considered might reduce costs and its use should be investigated, if available.

Effect of Form on Economy of Construction. For solid-ribbed arches, a smooth-curve axis is preferable to a segmental-chord axis (straight between panel points) so far as appearance is concerned. The curved axis, however, involves additional cost of fabrication. At the least, some additional material is required in fabrication of the arch because of the waste in cutting the webs to the curved shape. In addition to this waste, some material must be added to the ribs to provide for increased stresses due to bending. This occurs for the following reason: Since most of the load on the rib is applied at panel points, the thrust line is nearly straight between panel points. Curving the axis of the rib causes eccentricity of the thrust line with respect to the axis and thus induces increased bending moments, particularly for dead load. All these effects may cause an increase in the cost of the curved rib on the order of 5 to 10%.

For tied solid-ribbed arches for which it is necessary to use a very shallow tie, costs are larger than for shallow ribs and deep ties. (A shallow tie may be necessary to meet underclearance restrictions and vertical grades of the deck.) A check of a 750-ft span for two alternate designs, one with a 5-ft constant-depth rib and 12.5-ft-deep tie and the other with a 10-ft-deep rib and 4-ft-deep tie, showed that the latter arrangement, with shallow tie, required about 10% more material than the former, with deep tie. The actual increased construction cost might be more on the order of 5%, because of some constant costs for fabrication and erection that would not be affected by the variation in weight of material.

Comparison of a tapered rib with a constant-depth rib indicates a small percentage saving in material in favor of the tapered rib. Thus, costs for these two alternatives would be nearly equal.

14.7 DESIGN OF OTHER ELEMENTS

A few special conditions relating to elements of arch bridges other than the ribs and ties should be considered in design of arch bridges.

Floor System. Tied arches, particularly those with high-strength steels, undergo relatively large changes in length of deck due to variation in length of tie under various load conditions. Historically, it was considered necessary to provide deck joints at intermediate points to provide for erection conditions and to avoid high participation stresses. However, maintenance concerns regarding leaking of deck joints have resulted in a shift toward continuous decks. Proper detailing and erection sequencing can minimize these stresses.

Bracing. During design of the Bayonne Bridge arch (Art. 14.8), a study in depth explored the possibility of eliminating most of the sway bracing (bracing in a vertical plane between ribs). In addition to detailed analysis, studies were made on a scaled model to check the effect of various arrangements of this bracing. The investigators concluded that, except for a few end panels, the sway bracing could be eliminated. Though many engineers still adhere to an arbitrary specification requirement calling for sway bracing at every panel point of any truss, more consideration should be given to the real necessity for this. Furthermore, elimination of sway frames not only reduces costs but it also greatly improves the appearance of the structure. For several structures from which sway bracing has been omitted, there has been no adverse effect.

Various arrangements may be used for lateral bracing systems in arch bridges. For example, a diamond pattern, omitting cross struts at panel points, is often effective. Also, favorable results have been obtained with a Vierendeel truss.

In the design of arch bracing, consideration must be given to the necessity for the lateral system to prevent lateral buckling of the two ribs functioning as a single compression member. The lateral bracing thus is the lacing for the two chords of this member. The use of inclined ribs, referred to as a basket-handle configuration, can greatly influence the type and amount of lateral bracing required.

Hangers. These must be designed with sufficient rigidity to prevent adverse vibration under aerodynamic forces or as very slender members (wire rope or bridge strand). A number of long-span structures incorporate the latter device. Vibration problems have developed with some bridges for which rigid members with high slenderness ratios have been used. Corrosion resistance and provision for future replacement are other concerns which must be addressed in design of wire hangers. While not previously discussed in this section, the use of inclined hangers has been employed for some tied arch bridges. This hanger arrangement can add considerable stiffness to the arch-tie structure and cause it to function similar to a truss system with crossing diagonals. For such an arrangement, stress reversal, fatigue, and more complex details must be investigated and addressed.

14.8 EXAMPLES OF ARCH BRIDGES

Thanks to the cooperation of several engineers in private and public practice, detailed information on about 25 arch bridges has been made available. Sixteen have been selected from this group to illustrate the variety of arch types and forms in the wide range and span length for which steel arches have been used. Many of these bridges have been awarded prizes in the annual competition of the American Institute of Steel Construction.

The examples include only bridges constructed within the United States, though there are many notable arch bridges in other countries. A noteworthy omission is the imaginative and attractive Port Mann Bridge over the Fraser River in Canada. C.B.A. Engineering Ltd., consulting engineers, Vancouver, British Columbia, were the design engineers. By use of an orthotropic deck and stiffened, tied, solid-ribbed arch, an economical layout was developed with a central span of 1200 ft, flanked by side spans of 360 ft each. A variety of steels were used, including A373, A242, and A7.

Following are data on arch bridges that may be useful in preliminary design. (*Text continues on p. 14.43.*)

FIGURE 14.1

NEW RIVER GORGE BRIDGE

LOCATION: Fayetteville, West Virginia
TYPE: Trussed-deck arch, 40 panels, 36 at 40 ± to 43 ± ft
SPAN: 1,700 ft RISE: 353 ft RISE/SPAN = 1:4.8
NO. OF LANES OF TRAFFIC: 4
HINGES: 2 CROWN DEPTH: 34 ft DEPTH/SPAN = 1:50

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	8,600
Railings and parapets	1,480
Floor steel for roadway	3,560
Arch trusses	11,180
Arch bracing	1,010
Arch bents and bracing	2,870
TOTAL	28,700

SPECIFICATION FOR LIVE LOADING: H520-44
EQUIVALENT LIVE + IMPACT LOADING PER ARCH FOR FULLY LOADED STRUCTURE: 1,126 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch	A588
Floor system	A588

OWNER: State of West Virginia
ENGINEER: Michael Baker Jr., Inc.
FABRICATOR/ERECTOR: American Bridge Division, U.S. Steel Corporation
DATE OF COMPLETION: October 1977

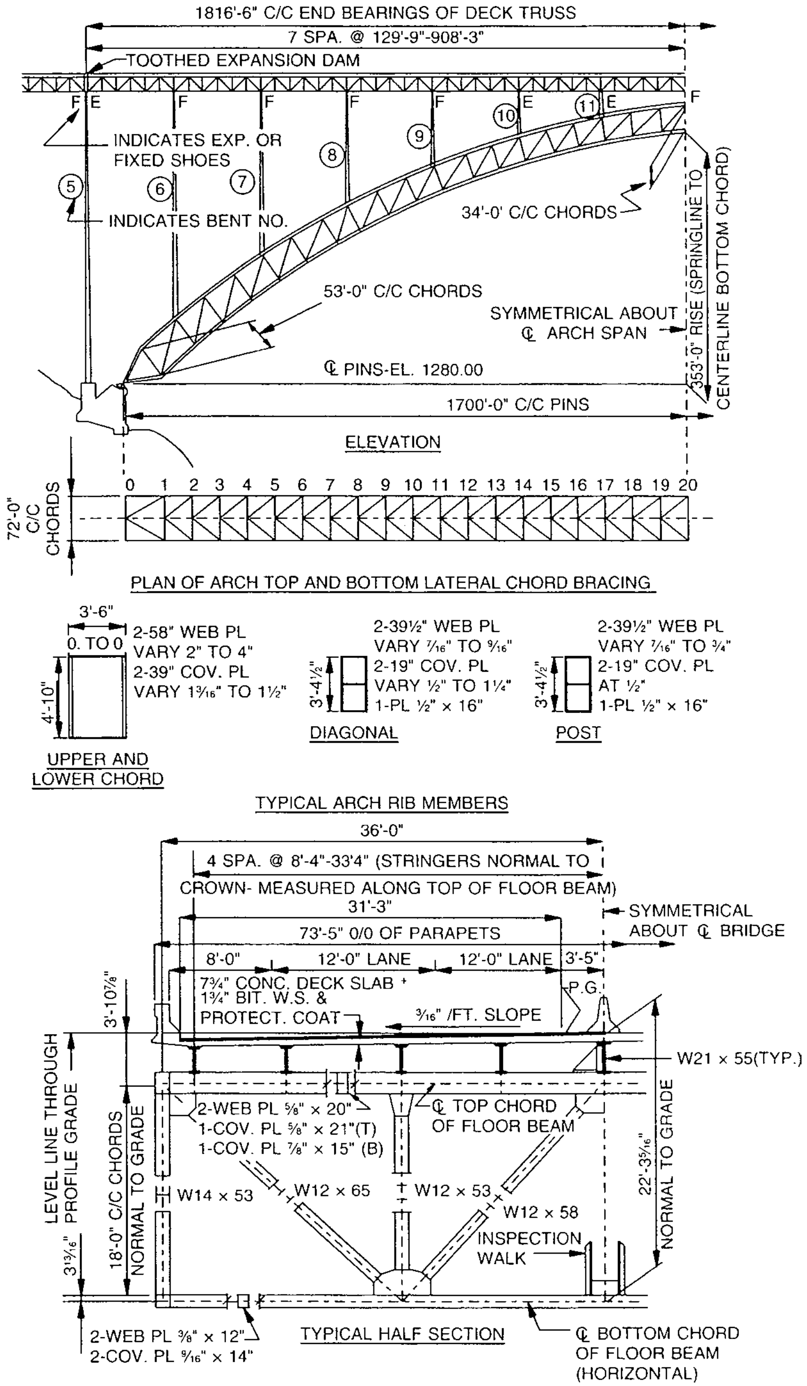


FIGURE 14.2 Details of New River Gorge Bridge.

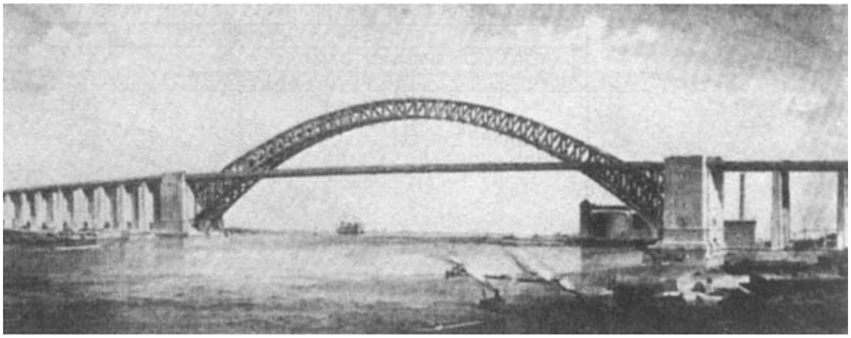


FIGURE 14.3

BAYONNE BRIDGE

LOCATION: Between Bayonne, New Jersey, and Port Richmond, Staten Island, New York
TYPE: Half-through truss arch, 40 panels at 41.3 ft
SPAN: 1,675 ft RISE: 266 ft RISE/SPAN = 1:6.3
NO. OF LANES OF TRAFFIC: 4 plus 2 future rapid transit
HINGES: 2 CROWN DEPTH: 37.5 FT DEPTH/SPAN = 1:45

AVERAGE DEAD LOAD:	LB PER FT
Track, paving	6,340
Floor steel and floor bracing	6,160
Arch truss and bracing	14,760
Arch hangers	540
Miscellaneous	200
TOTAL	28,000

SPECIFICATION LIVE LOADING:	LB PER FT
2 rapid-transit lines at 6,000 lb per ft	12,000
4 roadway lanes at 2,500 lb per ft	10,000
2 sidewalks at 600 lb	1,200
TOTAL (unreduced)	23,200

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE WITH REDUCTION FOR MULTIPLE LANES AND LENGTH OF LOADING: 2,800 lb per lin ft
TYPES OF STEEL IN STRUCTURE: About 50% carbon steel, 30% silicon steel, and 20% high-alloy steel (carbon-manganese)
OWNER: The Port Authority of New York and New Jersey
ENGINEER: O. H. Ammann, Chief Engineer
FABRICATOR: American Bridge Co., U.S. Steel Corp. (also erector)
DATE OF COMPLETION: 1931

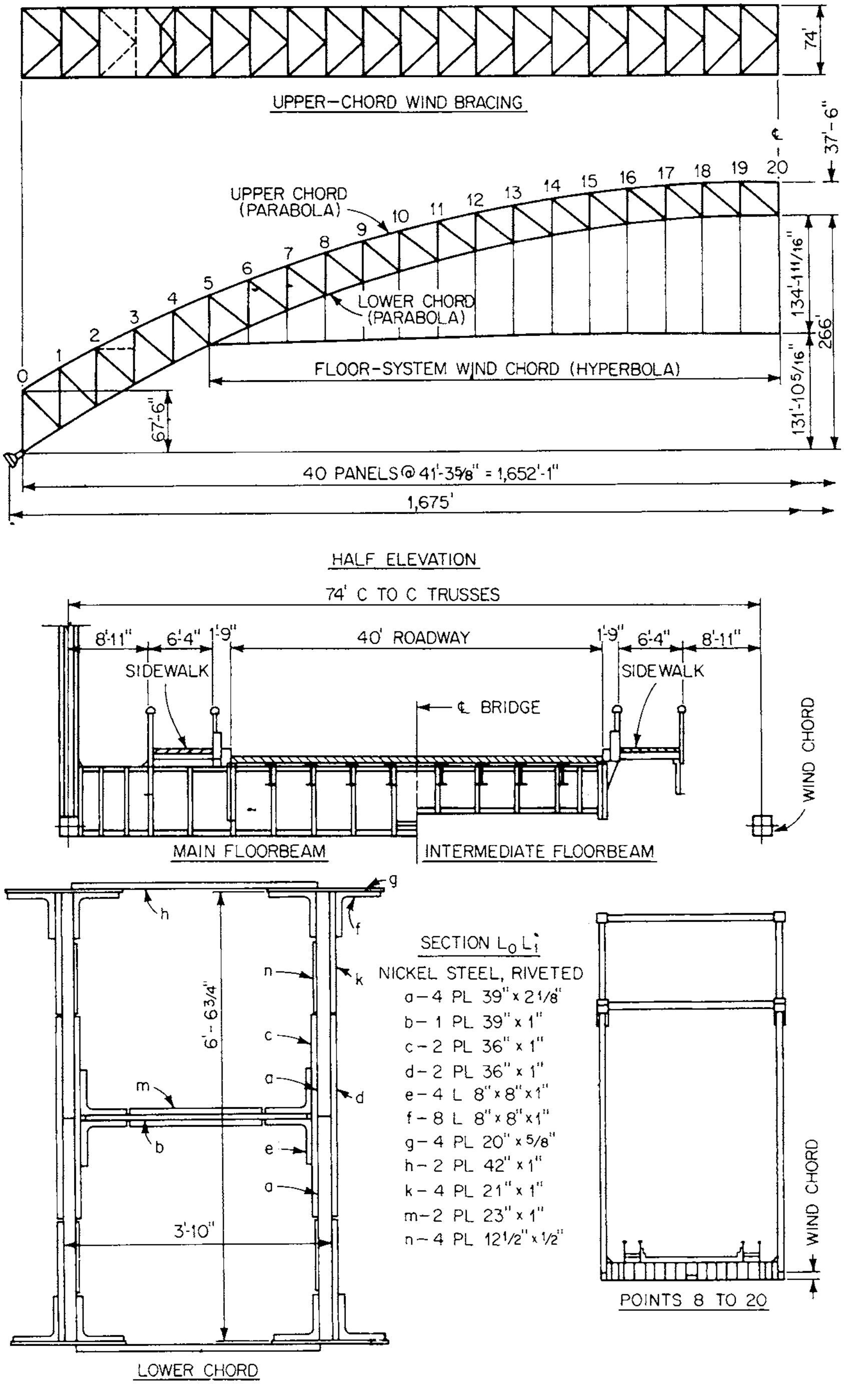


FIGURE 14.4 Details of Bayonne Bridge.

FIGURE 14.5

FREMONT BRIDGE

LOCATION: Portland, Oregon
TYPE: Half-through, tied, solid-ribbed arch, 28 panels at 44.83 ft
SPAN: 1,255 ft RISE: 341 ft RISE/SPAN = 1:3.7
NO. OF LANES OF TRAFFIC: 4 each upper and lower roadways
HINGES: 2 DEPTH: 4 ft DEPTH/SPAN = 1:314

AVERAGE DEAD LOAD:	LB PER FT
Decks and surfacing	10,970
Railings and parapets	1,280
Floor steel for roadway	4,000
Floor bracing	765
Arch ribs	2,960
Arch bracing	1,410
Arch hangers or columns and bracing	1,250
Arch tie girders	4,200
TOTAL	26,835

SPECIFICATION FOR LIVE LOADING: AASHTO HS20-44
EQUIVALENT LIVE + IMPACT LOADING PER ARCH FOR FULLY LOADED STRUCTURE: 2,510 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch ribs and tie girders	A514, A588, A441, A36
Floor system	A588, A441, A36

OWNER: State of Oregon, Department of Transportation
ENGINEER: Parsons, Brinckerhoff, Quade & Douglas
FABRICATOR: American Bridge Division, U.S. Steel Corp.
ERECTOR: Murphy Pacific Corporation
DATE OF COMPLETION: 1973

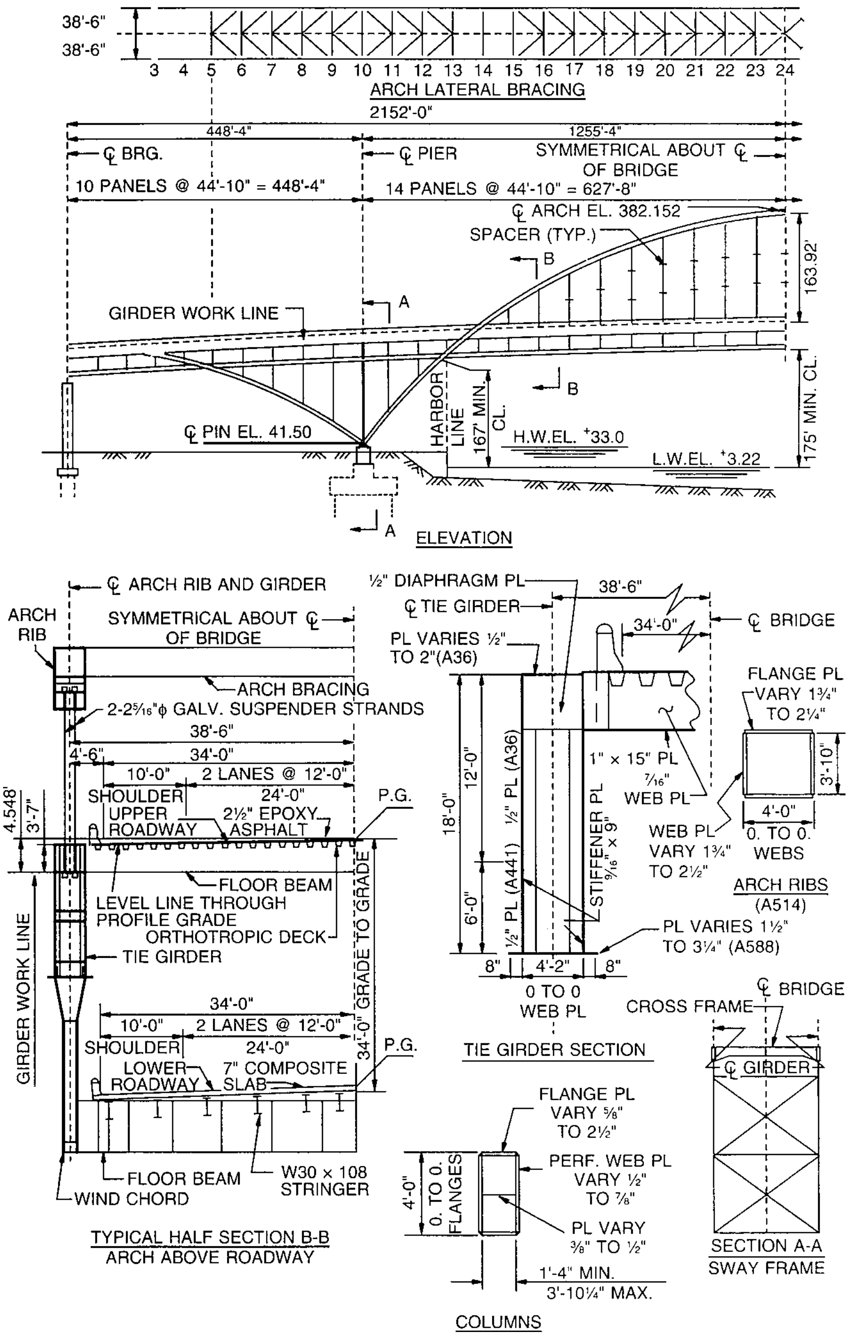


FIGURE 14.6 Details of Fremont Bridge.

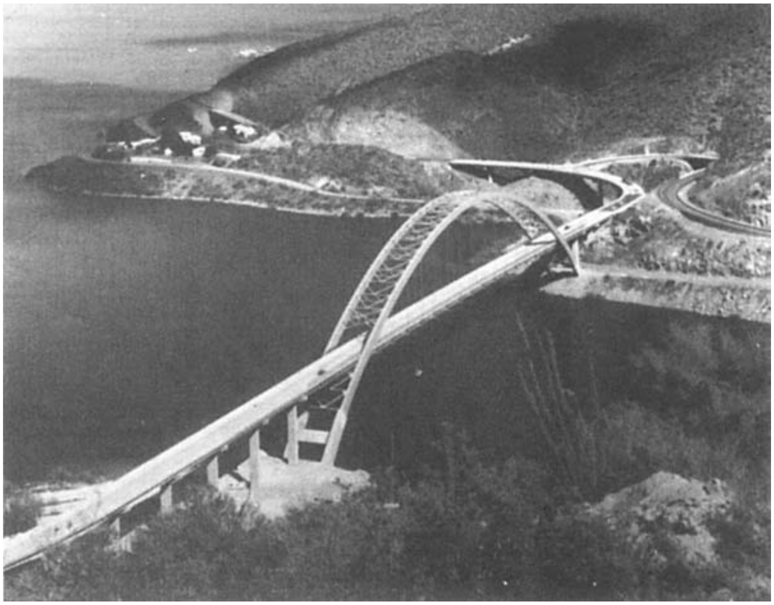


FIGURE 14.7

ROOSEVELT LAKE BRIDGE

LOCATION: Roosevelt, Arizona, SR 188
TYPE: Half-through, solid-rib arch, 16 panels at 50 ft
SPAN: 1,080 ft RISE: 230 ft RISE/SPAN = 1:4.7
NO. OF LANES OF TRAFFIC: 2
HINGES: 0 CROWN DEPTH: 8 ft DEPTH/SPAN = 1:135

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	4,020
Railings and parapets	800
Floor steel for roadway	1,140
Floor bracing	190
Arch ribs	4,220
Arch bracing	790
Arch hangers	80
TOTAL	11,240

SPECIFICATION FOR LIVE LOADING: HS20-44
EQUIVALENT LIVE + IMPACT LOADING PER ARCH FOR FULLY LOADED STRUCTURE: 971 lb per ft

TYPES OF STEEL IN STRUCTURE:	
Arch ribs and ties	A572
Hanger floorbeams and stringers	A572
All others	A36

OWNER: Arizona Department of Transportation
ENGINEER: Howard, Needles, Tammen & Bergendoff
CONTRACTOR: Edward Kraemer & Sons, Inc.
FABRICATOR: Pittsburgh DesMoines Steel Co./Schuff Steel
ERECTOR: John F. Beasley Construction Co.
DATE OF COMPLETION: October 23, 1991, Public Opening

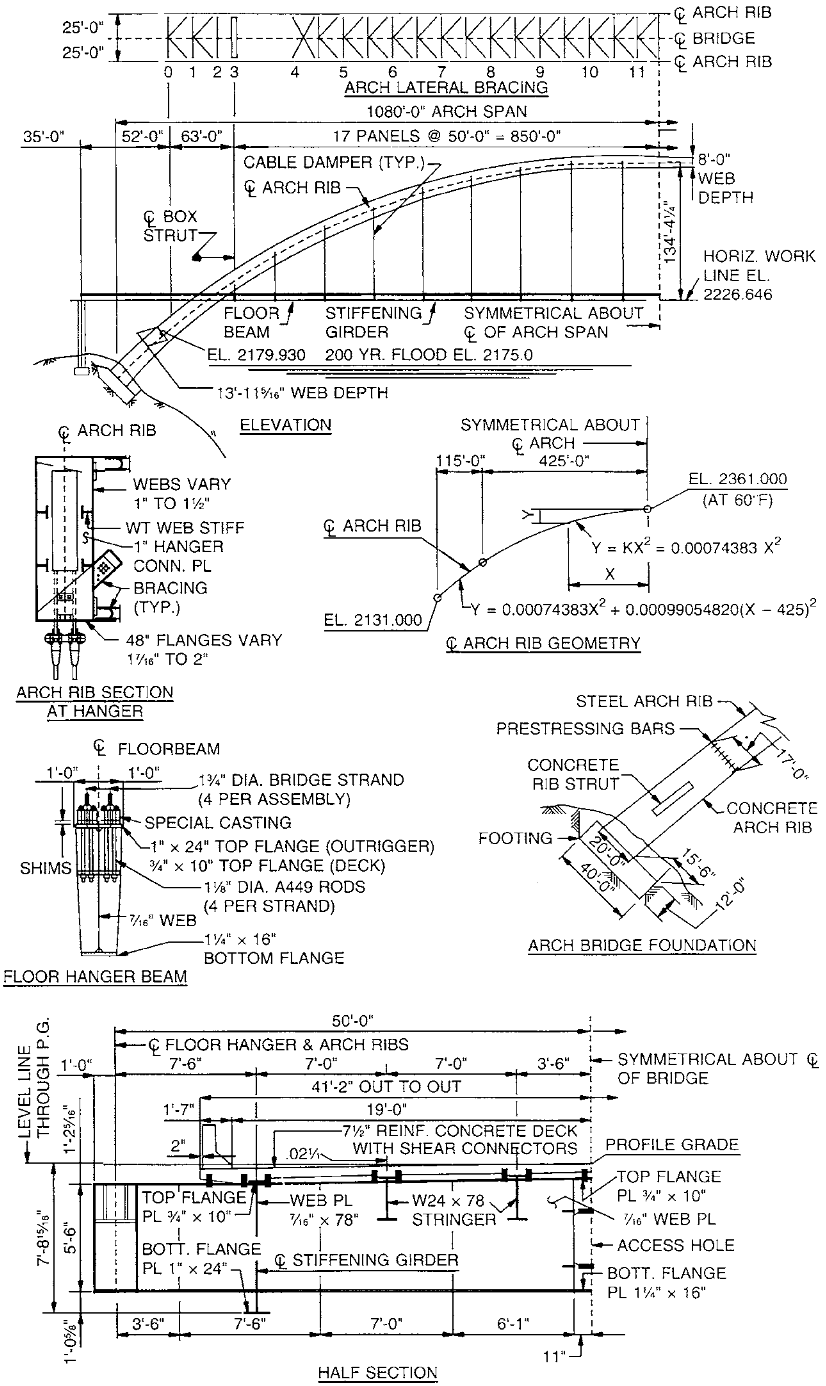


FIGURE 14.8 Details of Roosevelt Lake Bridge.

FIGURE 14.9

LEWISTON–QUEENSTON BRIDGE

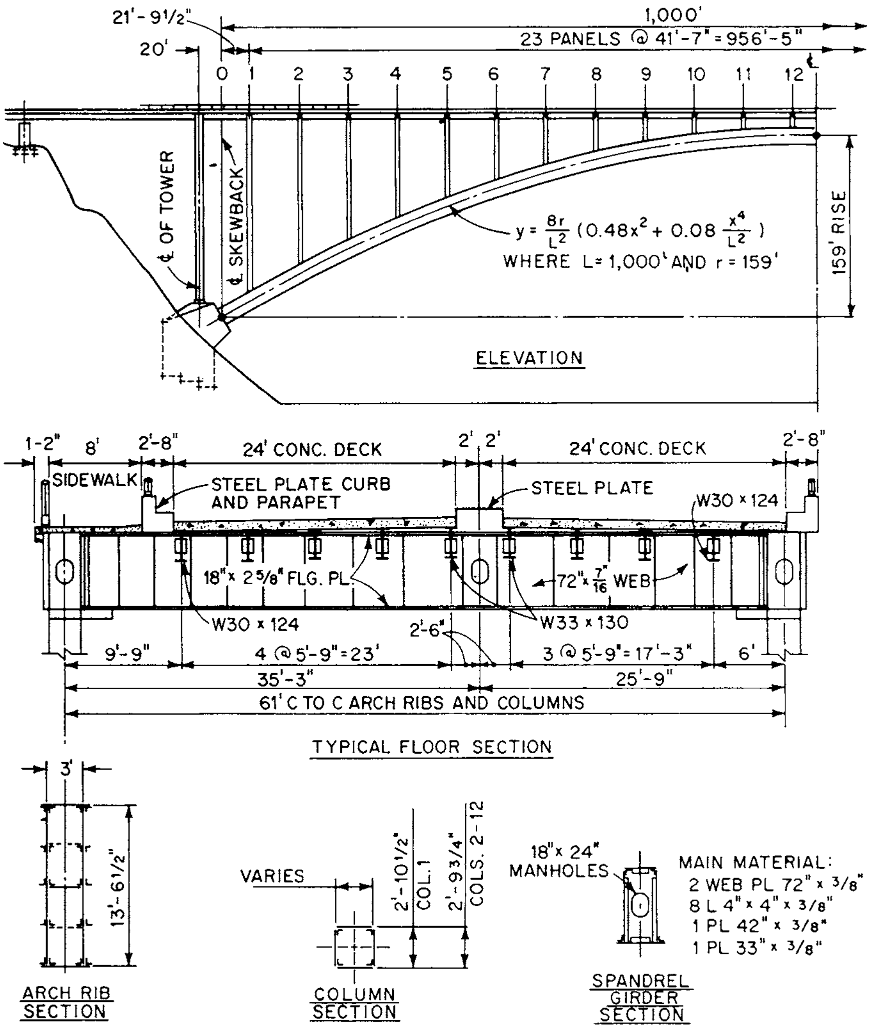
LOCATION: Over the Niagara River between Lewiston, New York, and Queenston, Ontario
TYPE: Solid-ribbed deck arch, 23 panels at 41.6 ft
SPAN: 1,000 ft RISE: 159 ft RISE/SPAN = 1:6.3
NO. OF LANES OF TRAFFIC: 4
HINGES: 0 DEPTH: 13.54 ft DEPTH/SPAN = 1:74

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	5,700
Slabs for sidewalks	495
Railings and parapets	780
Floor steel for roadway and sidewalks	2,450
Floor bracing	110
Arch ribs	7,085
Arch bracing	1,060
Miscellaneous—utilities, excess, etc	300
TOTAL	19,370

SPECIFICATION LIVE LOADING: HS20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,357 lb per ft

TYPES OF STEEL IN STRUCTURE:		%
Arch ribs	A440	100
Spandrel columns	A7	94
	A440	6
Rib bracing and end towers	A7	100
Floor system		A373 and A7

OWNER: Niagara Falls Bridge Commission
ENGINEER: Hardesty & Hanover
FABRICATOR: Bethlehem Steel Co. and Dominion Steel and Coal Corp., Ltd., Subcontractor
DATE OF COMPLETION: November 1, 1962



COMPOSITION OF ARCH RIB AND COLUMN SECTIONS			
	PANEL POINT 1	PANEL POINT 3	PANEL POINT 11
ARCH RIBS	6 COVER PL 54" x 7/8" 8 L 8" x 8" x 1 1/8" 2 WEB PL 162" x 15/16" 2 WEB PL 146" x 5/8" 6 L 7" x 4" x 5/8" 6 L 4" x 4" x 1/2" 1 PL 32" x 13/16"	4 COVER PL 54" x 3/4" 8 L 8" x 8" x 1" 2 WEB PL 162" x 15/16" 6 L 7" x 4" x 5/8" 6 L 4" x 4" x 1/2" 1 PL 33" x 13/16"	4 COVER PL 54" x 9/16" 8 L 8" x 8" x 1" 2 WEB PL 162" x 15/16" 6 L 7" x 4" x 5/8" 6 L 4" x 4" x 1/2" 1 PL 33" x 13/16"
COLS	2 COVER PL 37" x 1" 4 L 6" x 6" x 9/16" 2 WEB PL 32" x 5/8"	2 COVER PL 35" x 5/8" 4 L 6" x 6" x 9/16" 2 WEB PL 32" x 5/8"	2 COVER PL 27" x 1/2" 4 L 6" x 6" x 9/16" 2 WEB PL 32" x 5/8"

MATERIAL FOR ARCH RIBS, A440 STEEL

FIGURE 14.10 Details of Lewiston-Queenston Bridge.

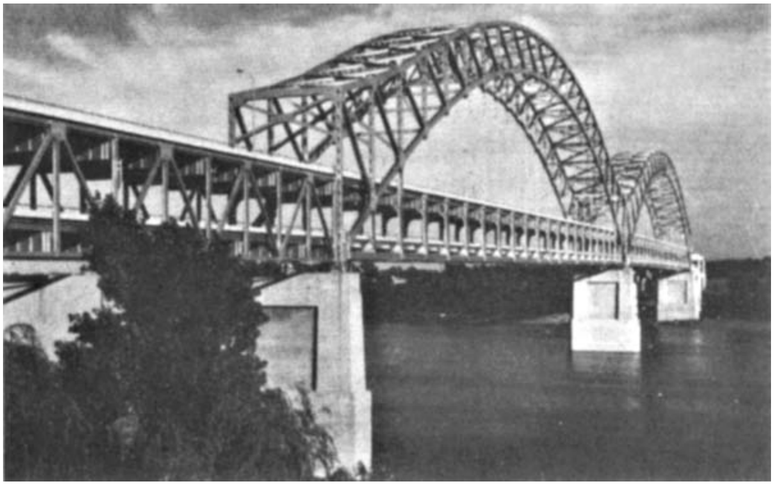


FIGURE 14.11

SHERMAN MINTON BRIDGE

LOCATION: On Interstate 64 over the Ohio River between Louisville, Kentucky, and New Albany, Indiana
TYPE: Tied through-truss arch, 22 panels at 36.25 ft
SPAN: 800 ft RISE: 140 ft RISE/SPAN = 1:5.7
NO. OF LANES OF TRAFFIC: 6, double deck
HINGES: 2 CROWN DEPTH: 30 ft DEPTH/SPAN = 1:27

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	7,600
Slabs for sidewalks	1,656
Railings and parapets	804
Floor steel for roadway and sidewalks	2,380
Floor bracing	420
Arch trusses	3,400
Arch bracing	880
Arch hangers and bracing	160
Arch ties	1,040
Miscellaneous—utilities, excess, etc. (including future searing surface)	1,680
TOTAL	20,020

SPECIFICATION LIVE LOADING: H20-S16
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,755 LB PER FT

TYPES OF STEEL IN STRUCTURE:		%
Arch trusses	A514	69
	A242	18
	A373	13
Floor system	A242	36
	A7	62
	A373	2

OWNER: Indiana Department of Transportation and Kentucky Transportation Cabinet
ENGINEER: Hazelet & Erdal, Louisville, Kentucky
FABRICATOR: R. C. Mahon Co.
DATE OF COMPLETION: December 22, 1961, opened to traffic

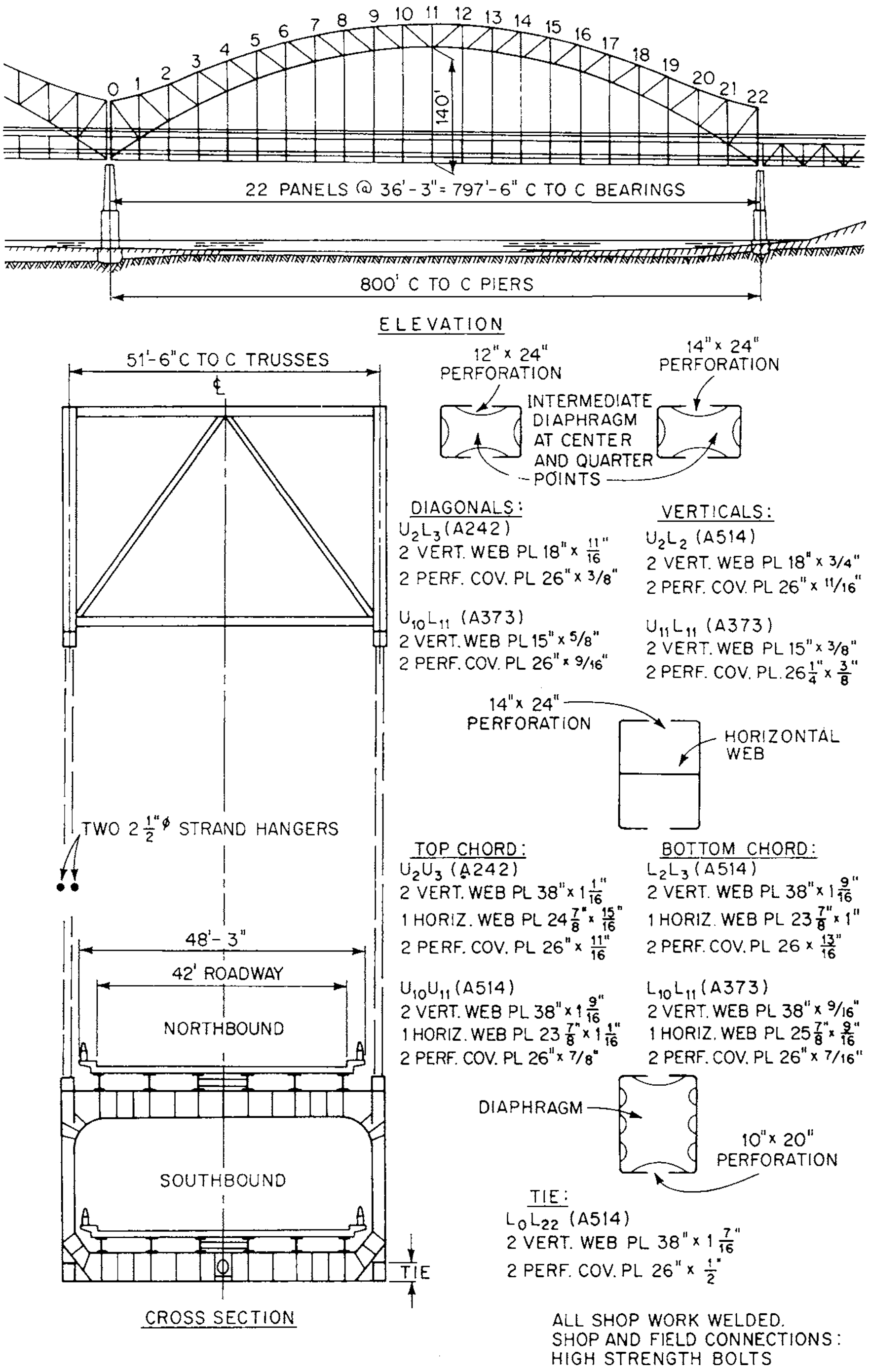


FIGURE 14.12 Details of Sherman Minton Bridge.

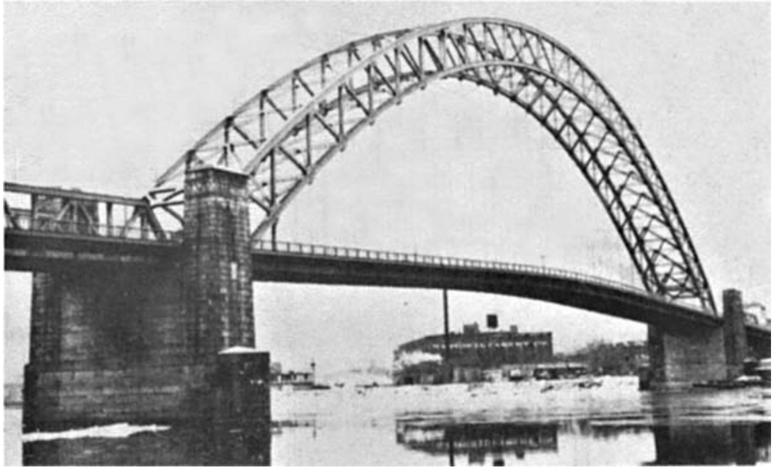


FIGURE 14.13

WEST END–NORTH SIDE BRIDGE

LOCATION: Pittsburgh, Pennsylvania, over Ohio River
TYPE: Tied through-truss arch, 28 panels at 27.8 ft
SPAN: 778 ft RISE: 151 ft RISE/SPAN = 1:5.2
NO. OF LANES OF TRAFFIC: 4, including 2 street-railway tracks
HINGES: Two CROWN DEPTH: 25 DEPTH/SPAN = 1:31

AVERAGE DEAD LOAD:	LB PER FT
Roadway, sidewalks, and railings	4,870
Floor steel and floor bracing	2,360
Arch trusses	4,300
Arch ties	2,100
Arch bracing	550
Hangers	360
Utilities and excess	600
TOTAL	15,140

SPECIFICATION LIVE LOADING: Allegheny County Truck & Street Car
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,790 lb per ft
TYPES OF STEEL IN STRUCTURE:
All main material in arch trusses and ties including splice material—silicon steel.

Floor system and bracing	A7
Hangers	Wire rope

OWNER: Pennsylvania Department of Transportation
ENGINEER: Department of Public Works, Allegheny County
FABRICATOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: 1932

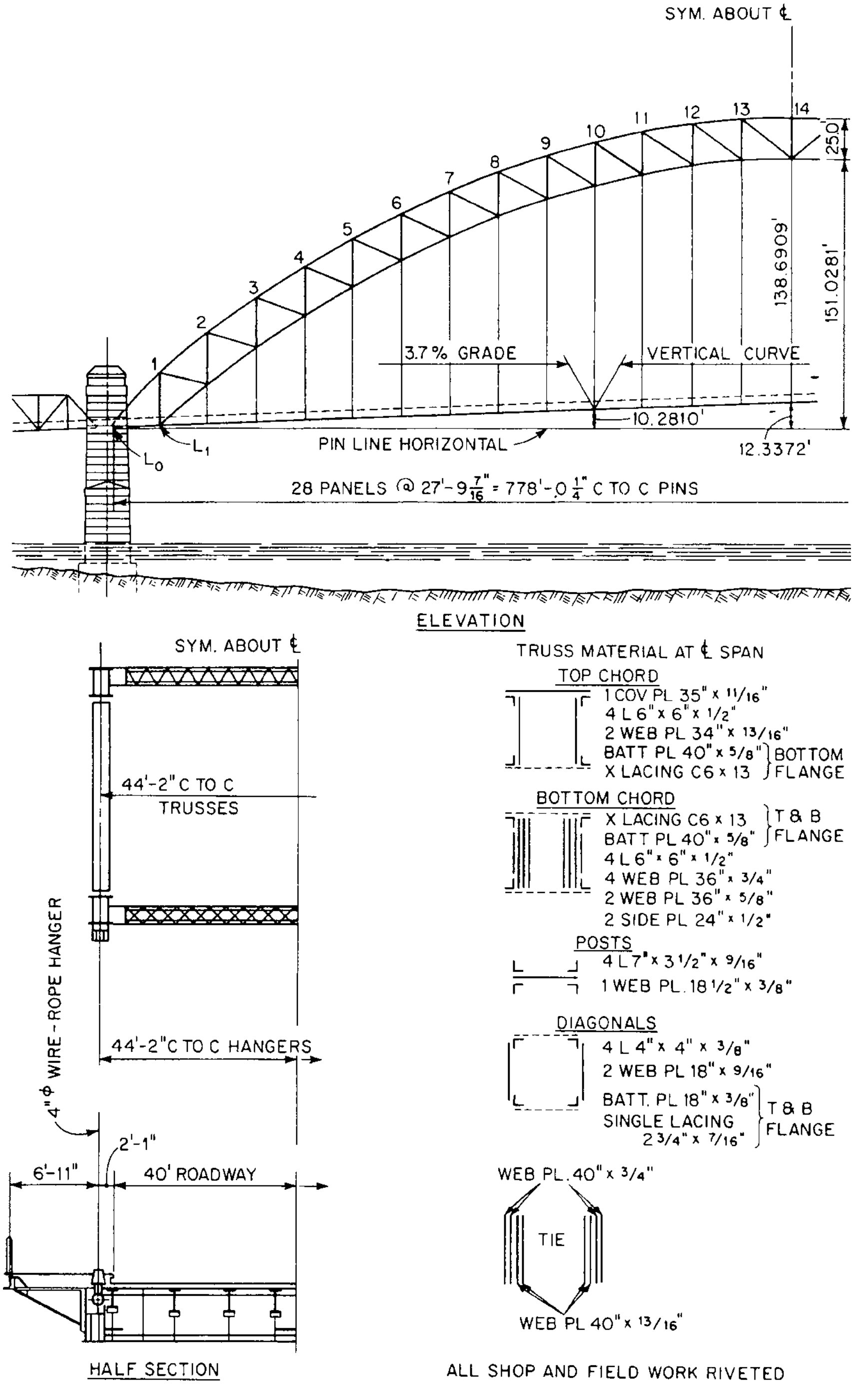


FIGURE 14.14 Details of West End–North Side Bridge.

FIGURE 14.15

FORT PITT BRIDGE

LOCATION: Pittsburgh, Pennsylvania, over the Monongahela River
TYPE: Solid-ribbed, tied, through-arch, 30 panels at 25 ft
SPAN: 750 ft RISE: 122.2 ft RISE/SPAN = 1:6.2
NO. OF LANES OF TRAFFIC: 4, each level of double deck
HINGES: 2 DEPTH: 5.4 ft DEPTH/SPAN = 1:139

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadways, slabs for sidewalks, railings and parapets, on both decks	16,100
Floor steel for roadway and sidewalks, on both decks	4,860
Floor bracing (truss bracing)	480
Arch ribs	5,480
Arch bracing	1,116
Arch hangers (included with rib and tie)	
Arch ties (trusses)	8,424
Miscellaneous—utilities, excess, etc	400
TOTAL	36,860

SPECIFICATION LIVE LOADING: HS20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 2,500 lb per ft
TYPES OF STEEL IN STRUCTURE:

Arch ribs and trussed ties	A242	64
	A7	36
Floor system	A242	90
	A7	10

OWNER: Pennsylvania Department of Highways
ENGINEER: Richardson, Gordon and Associates
FABRICATOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: 1957

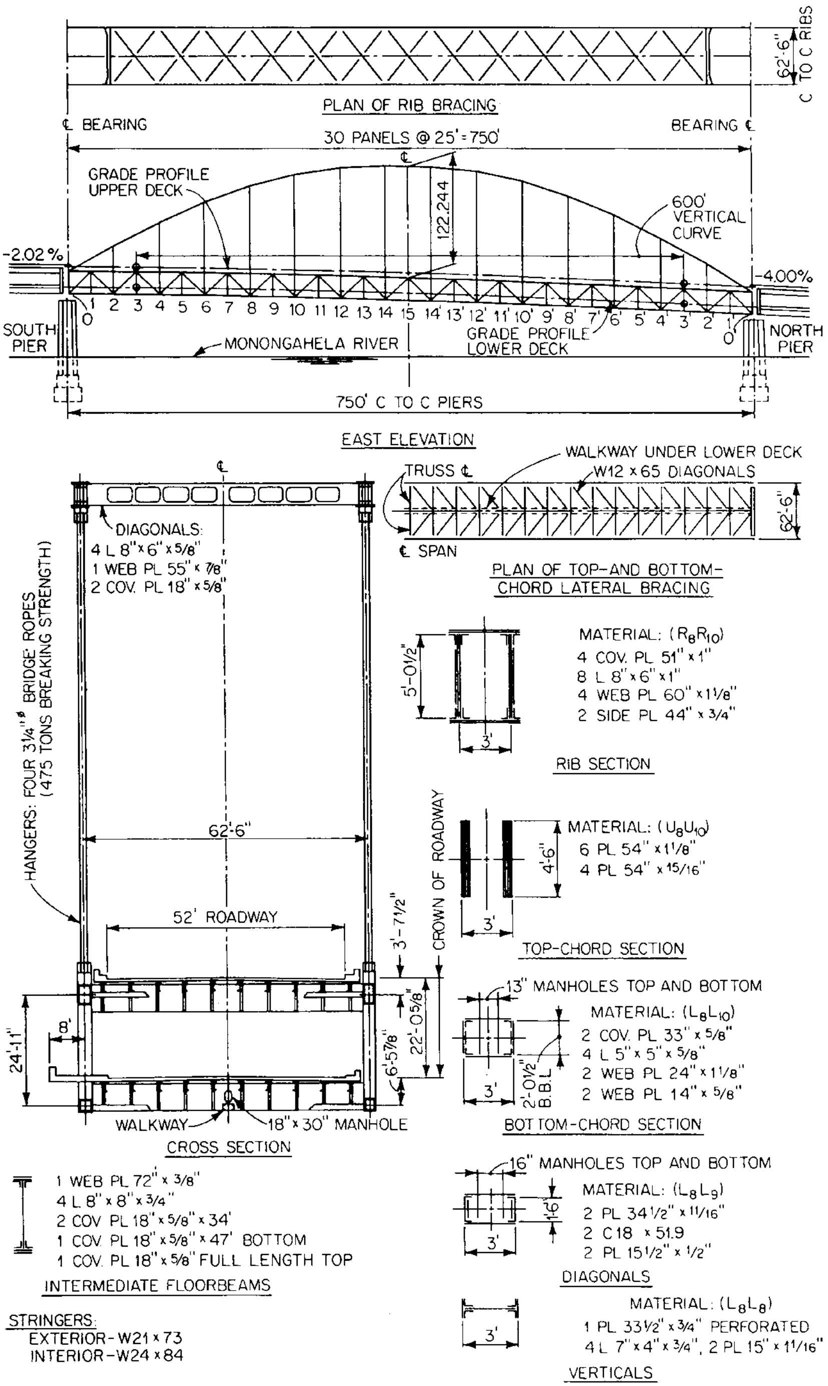


FIGURE 14.16 Details of Fort Pitt Bridge.

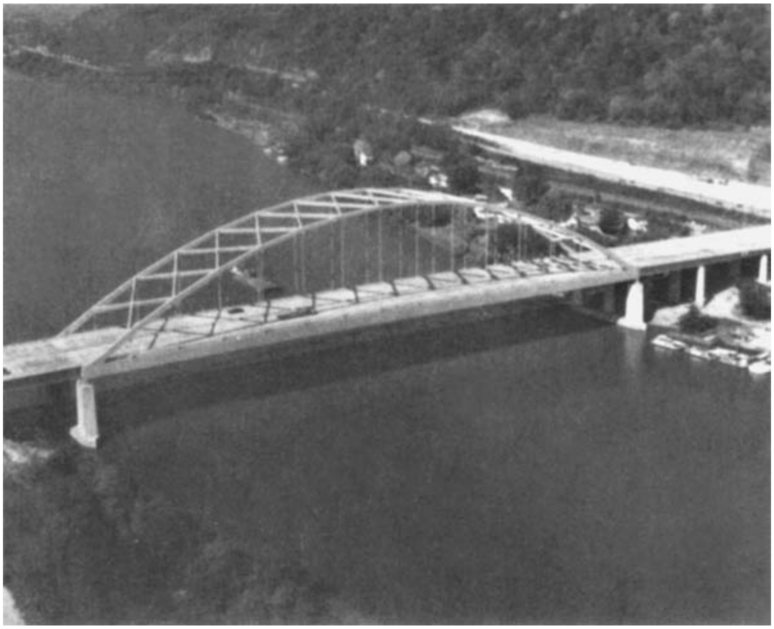


FIGURE 14.17

GLENFIELD BRIDGE

LOCATION: I-79 crossing of Ohio River at Neville Island, Pennsylvania
TYPE: Tied, through, solid-ribbed arch, 15 panels at 50 ft
SPAN: 750 ft RISE: 124.4 ft RISE/SPAN = 1:6
NO. OF LANES OF TRAFFIC: 6 plus 10-ft berms
HINGES: 0 CROWN DEPTH: 4 ft DEPTH/SPAN = 1:187

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	13,980
Railings and parapets	1,090
Floor steel for roadway	3,397
Floor bracing	392
Arch ribs	2,563
Arch bracing	1,639
Arch hangers	94
Arch ties	3,400
Miscellaneous—utilities, excess, etc	589
TOTAL	27,144

SPECIFICATION LIVE LOADING: H20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,920 lb per ft

TYPES OF STEEL IN STRUCTURE:		%
Arch ribs and ties	A514	64
	A588	36
Ribs and bottom-lateral bracing	A36	100
Hangers		Wire rope

OWNER: Pennsylvania Department of Transportation
ENGINEER: Richardson, Gordon and Associates
FABRICATOR: Bristol Steel and Iron Works, Inc., and Pittsburgh DesMoines Steel Co.
ERECTOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: 1976

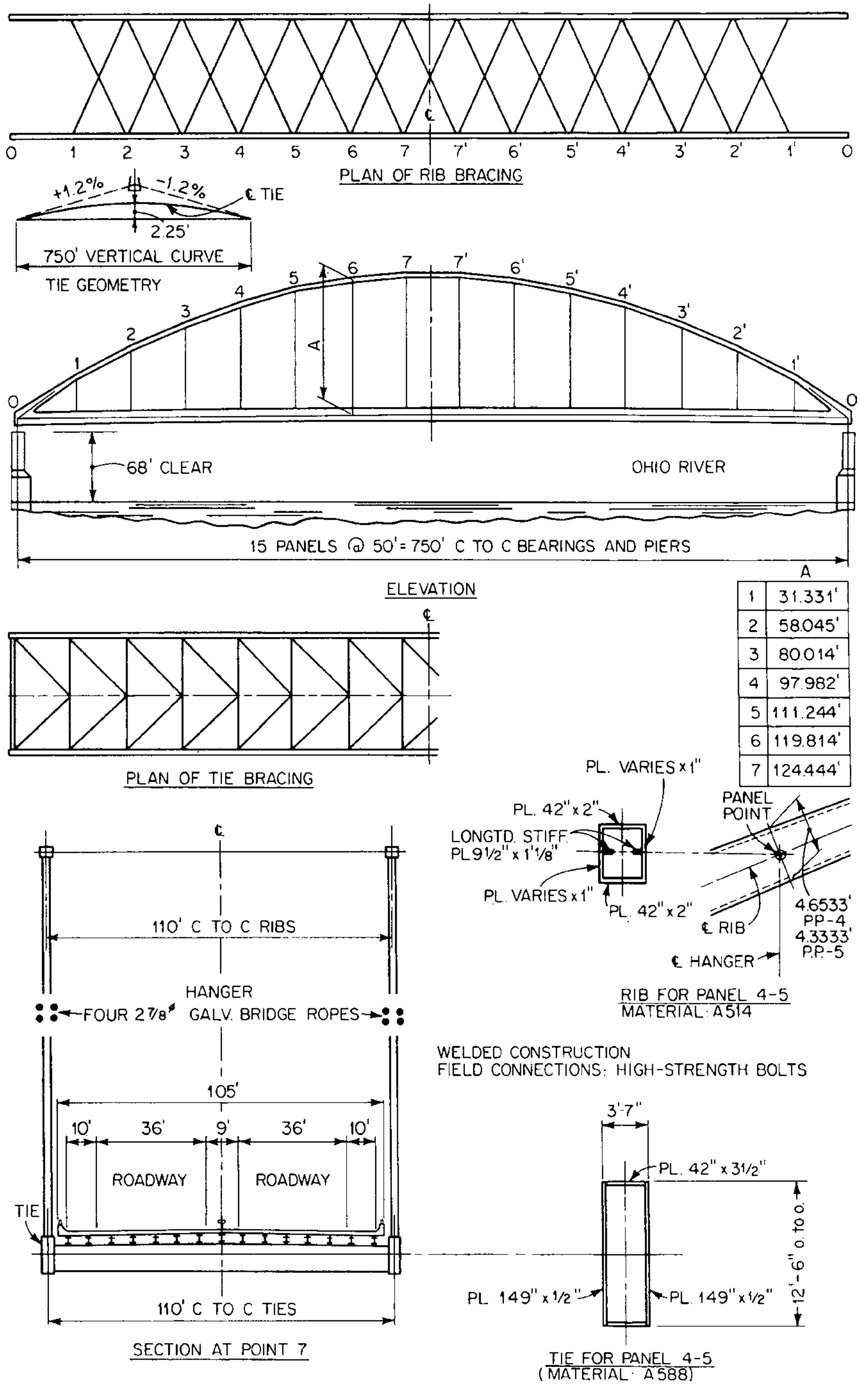


FIGURE 14.18 Details of Glenfield Bridge.

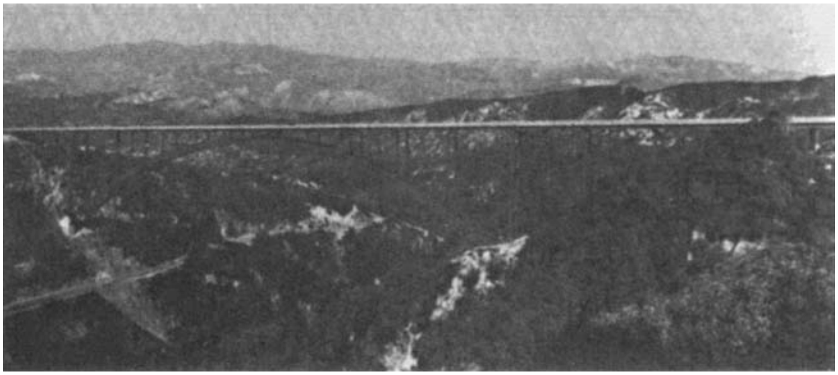


FIGURE 14.19

COLD SPRING CANYON BRIDGE

LOCATION: About 13.5 miles north of city limit of Santa Barbara, California
TYPE: Solid-ribbed deck arch, 11 panels, 9 at 63.6 ft
SPAN: 700 ft RISE: 119.2 ft RISE/SPAN = 1:5.9
NO. OF LANES OF TRAFFIC: 2
HINGES: 2 DEPTH: 9 ft DEPTH/SPAN = 1:78

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	3,520
Railings and parapets	1,120
Floor steel for roadway	620
Floor bracing	75
Arch ribs	3,400
Arch bracing	530
Arch posts and bracing	210
TOTAL	9,475

SPECIFICATION LIVE LOADING: H20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 904 lb per ft

TYPES OF STEEL IN STRUCTURE:	
Arch ribs	A373
Floor system	A373

OWNER: State of California
ENGINEER: California Department of Transportation
FABRICATOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: December 1963

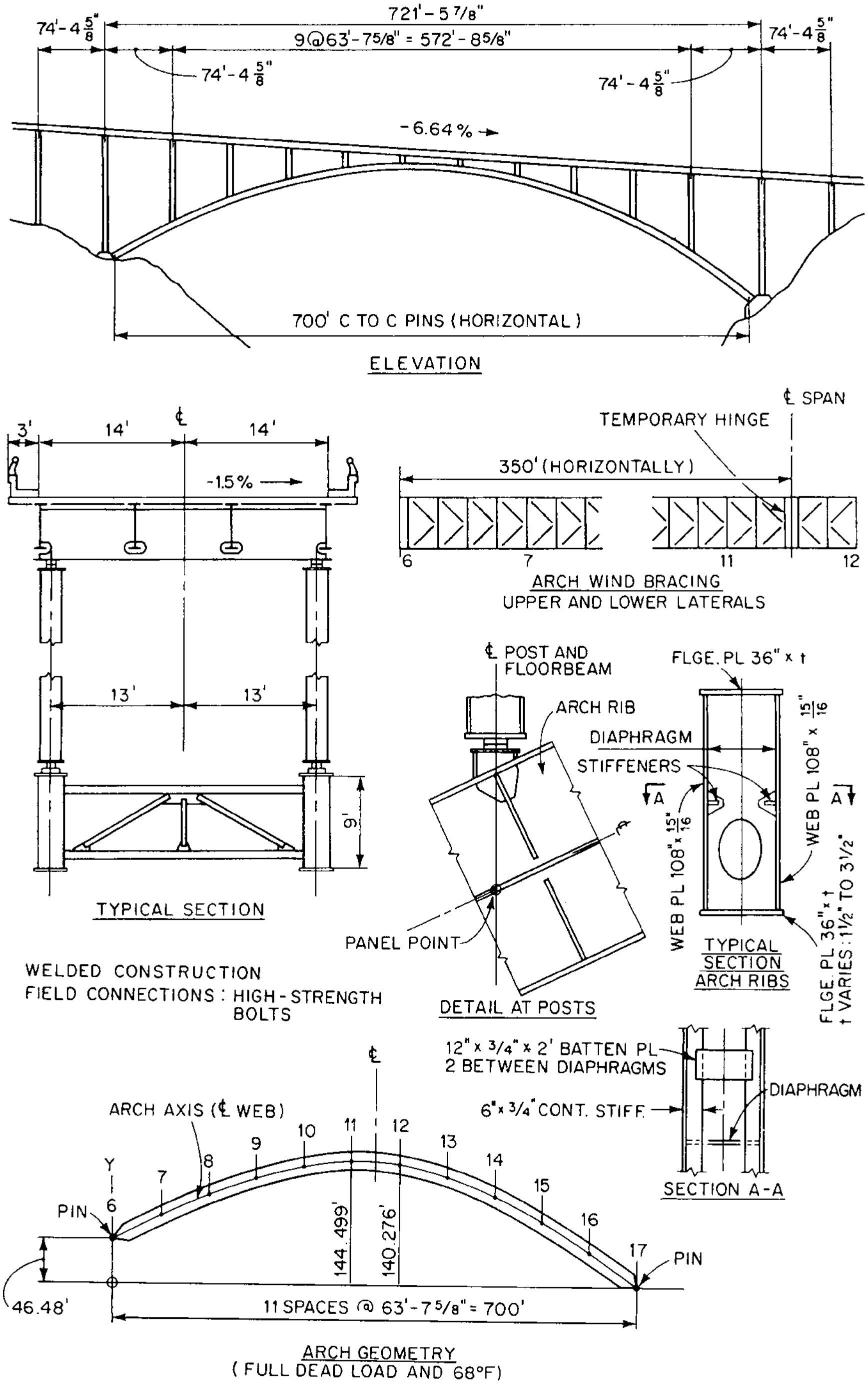


FIGURE 14.20 Details of Cold Spring Canyon Bridge.

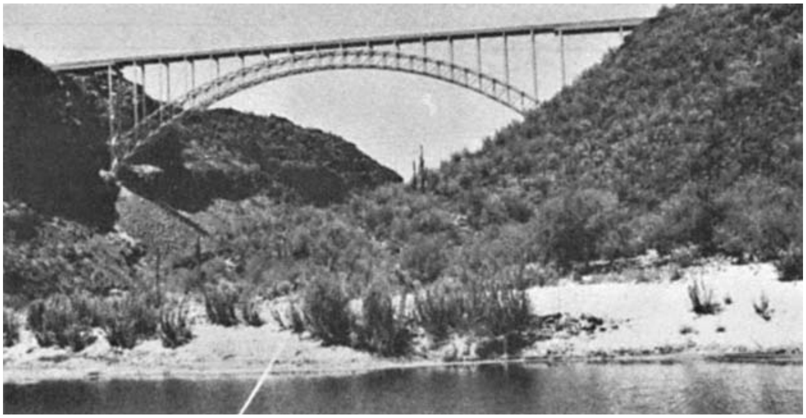


FIGURE 14.21

BURRO CREEK BRIDGE

LOCATION: Arizona State Highway 93, about 75 miles southeast of Kingman, Arizona
TYPE: Trussed deck arch, 34 panels at 20 ft
SPAN: 680 ft RISE: 135 ft RISE/SPAN = 1:5.0
NO. OF LANES OF TRAFFIC: 2
HINGES: 2 CROWN DEPTH: 20 FT DEPTH/SPAN = 1:34

Deck slab and surfacing for roadway	3,140
Slab for sidewalks	704
Railings and parapets	470
Floor steel for roadway	800
Floor bracing	203
Arch trusses	2,082
Arch bracing	580
Arch posts and bracing	608
TOTAL	8,587

SPECIFICATION LIVE LOADING: H20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,420 lb per ft
TYPES OF STEEL IN STRUCTURE:

		%
Arch trusses	A441	61
	A36	39
Other components	A36	

OWNER: Arizona Department of Transportation
ENGINEER: Bridge Division
FABRICATOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: March 23, 1966

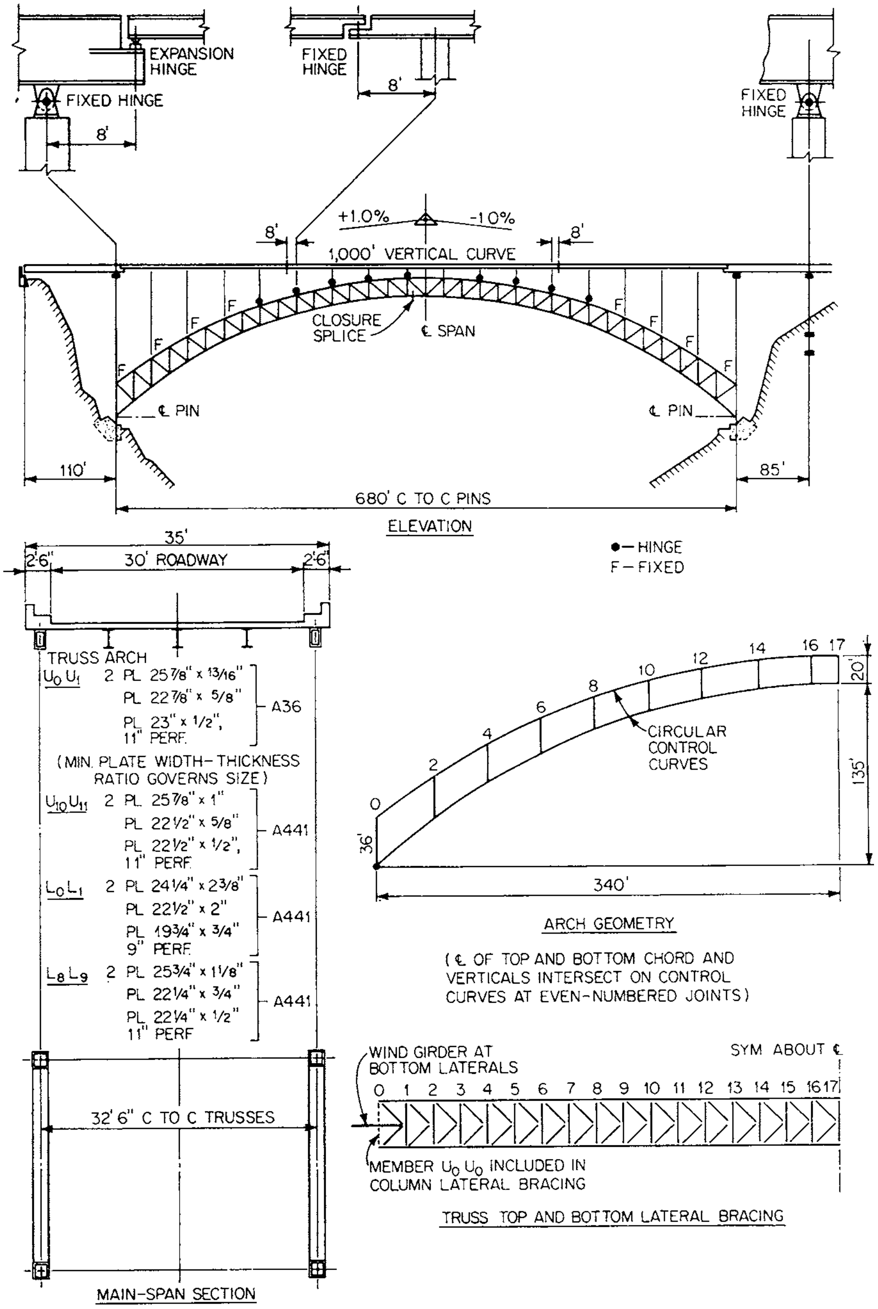


FIGURE 14.22 Details of Burro Creek Bridge.

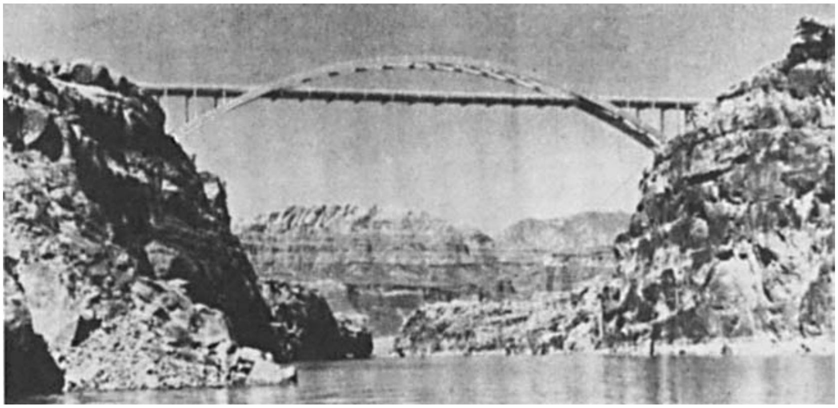


FIGURE 14.23

COLORADO RIVER ARCH BRIDGE

LOCATION: Utah State Route 95 over Colorado River, near Garfield–San Juan county line
TYPE: Half-through, solid-ribbed arch, 21 panels, 19 at 27.5 ft
SPAN: 550 ft RISE: 90 ft RISE/SPAN = 1:6.1
NO. OF LANES OF TRAFFIC: 2
HINGES: 0 DEPTH: 7 ft DEPTH/SPAN = 1:79

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	2,804
Railings and parapets	605
Floor steel for roadway	615
Floor bracing	60
Arch ribs	2,200
Arch bracing	370
Arch hangers and bracing	61
TOTAL	6,715

SPECIFICATION LIVE LOADING: HS20-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 952 lb per ft
STEEL IN THIS STRUCTURE: A36, except arch hangers, which are bridge strand.
OWNER: State of Utah
ENGINEER: Structures Division, Utah Department of Transportation
FABRICATOR: Western Steel Co., Salt Lake City, Utah
DATE OF COMPLETION: November 18, 1966

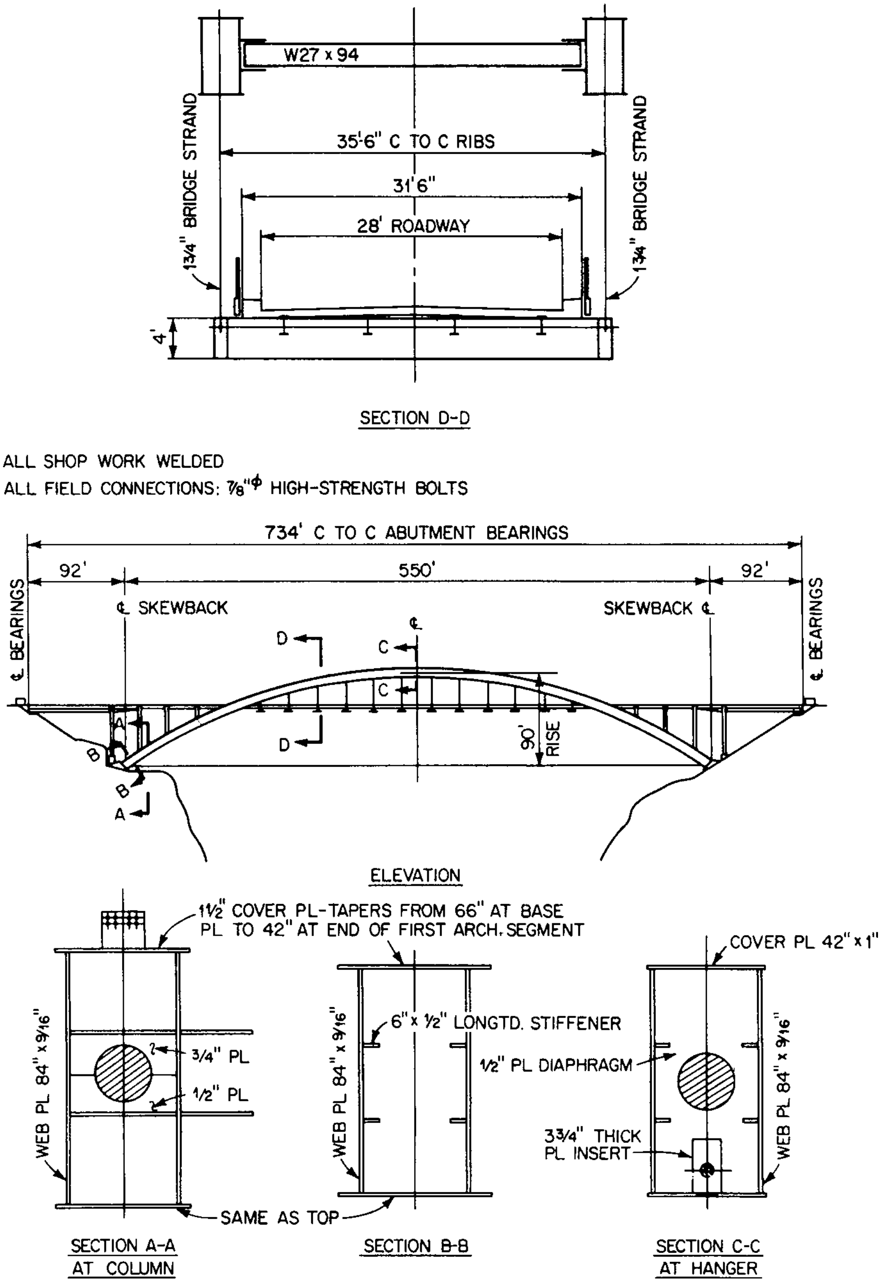


FIGURE 14.24 Details of Colorado River Arch Bridge.

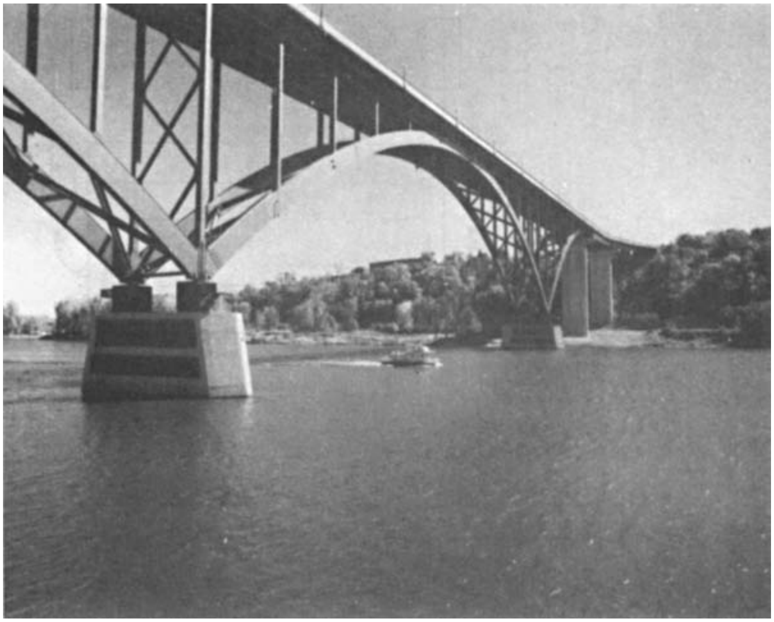


FIGURE 14.25

SMITH AVENUE HIGH BRIDGE

LOCATION: Smith Avenue over Mississippi River in St. Paul, Minnesota
TYPE: Solid-ribbed, tied, deck arch, 26 panels at 40 ft
SPAN: 520 ft RISE: 109.35 ft RISE/SPAN = 1:4.8
NO. OF LANES OF TRAFFIC: 2
HINGES: 0 DEPTH: 8 ft DEPTH/SPAN = 1:65

AVERAGE DEAD LOAD:	LB PER FT
Deck slab, sidewalks, railings and surfacing for roadway	9,370
Floor steel for roadway	920
Arch ribs	2,810
Arch bracing	360
Arch ties	200
Arch columns and bracing	300
TOTAL	13,960

SPECIFICATION FOR LIVE LOADING: HS20-44
EQUIVALENT LIVE + IMPACT LOADING FOR ARCH FOR FULLY LOADED STRUCTURE: 2,250 lb per ft

TYPES OF STEEL IN STRUCTURE:	
Arch ribs and ties	A588
Floor system	A588

OWNER: Minnesota Department of Transportation
ENGINEER: Strgar Roscoe Fausch/T. Y. Lin International
FABRICATOR: Lunda Construction
DATE OF COMPLETION: July 25, 1987

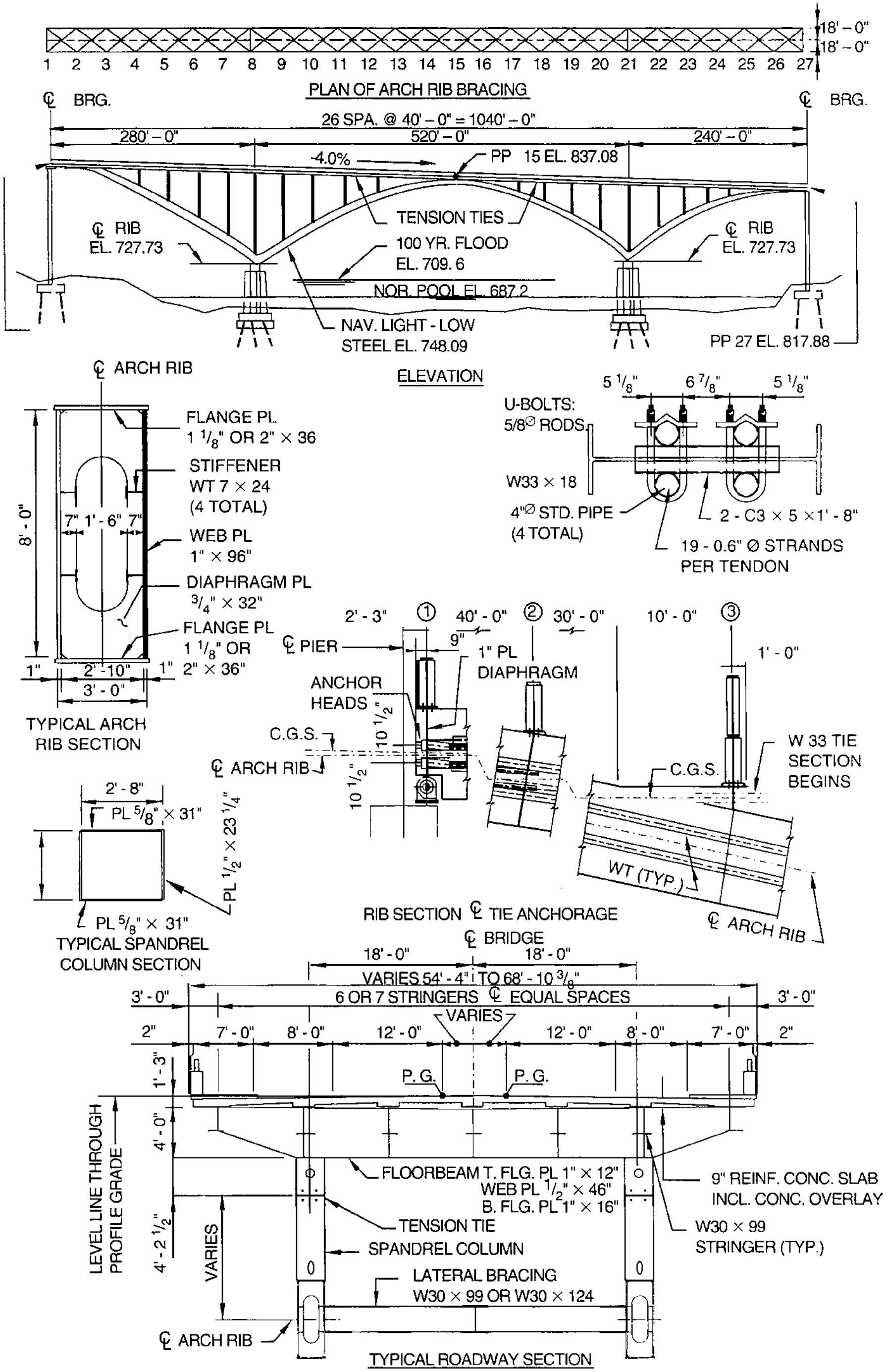


FIGURE 14.26 Details of Smith Avenue High Bridge.

FIGURE 14.27

LEAVENWORTH CENTENNIAL BRIDGE

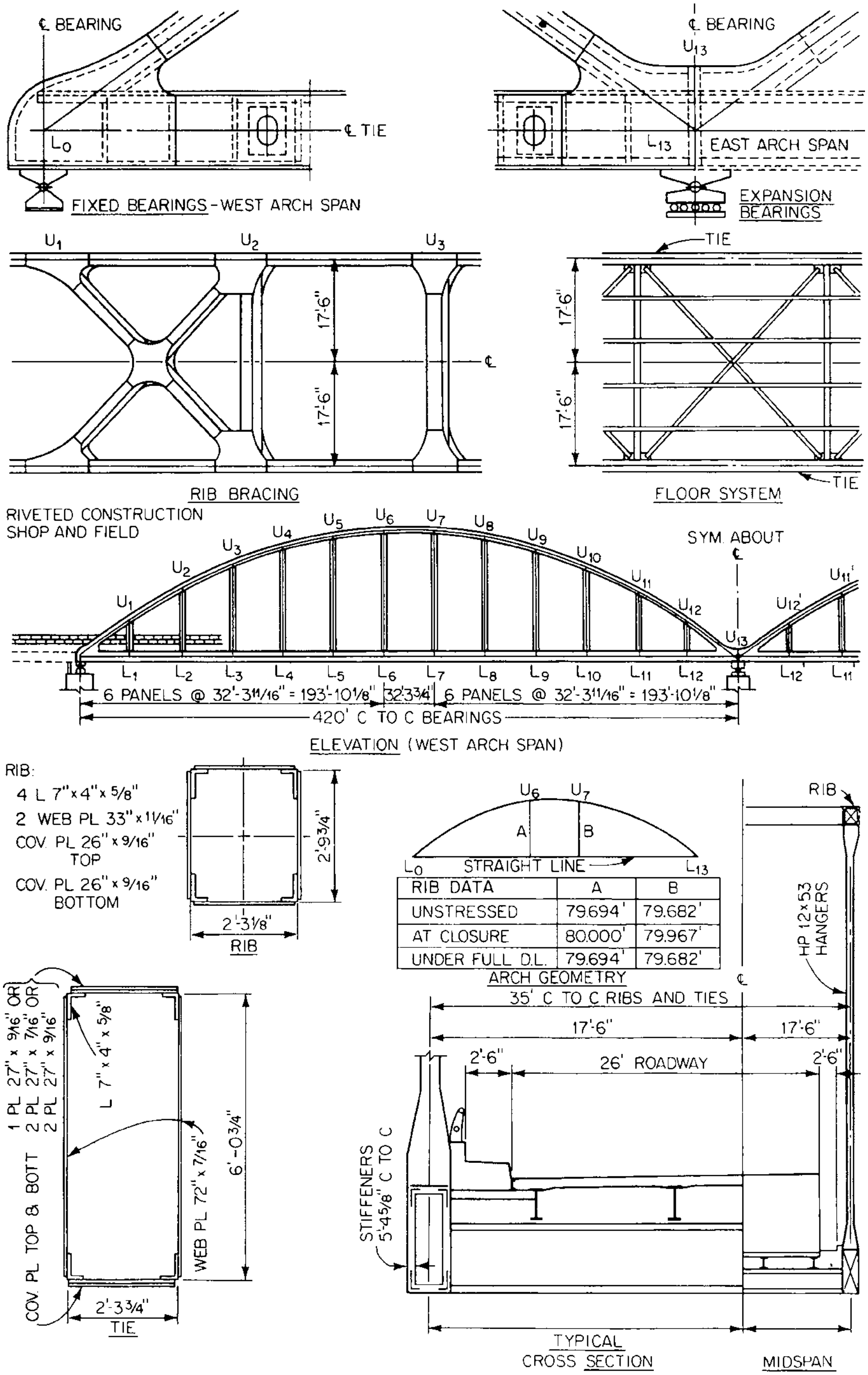
LOCATION: Leavenworth, Kansas, over Missouri River
TYPE: Tied, through, solid-ribbed arch, 13 panels at 32.3 ± ft
SPAN: 420 ft RISE: 80 ft RISE/SPAN = 1:5.2
NO. OF LANES OF TRAFFIC: 2
HINGES: 0 DEPTH: 2.8 ft DEPTH/SPAN = 1:150

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	2,710
Railings and parapets (aluminum)	32
Floor steel for roadway	820
Floor steel for sidewalks	202
Floor bracing	116
Arch ribs	986
Arch bracing	420
Arch hangers and bracing	200
Arch ties	1,104
Miscellaneous—utilities, excess, etc	110
TOTAL	6,700

SPECIFICATION LOADING: H20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 885 lb per ft
TYPES OF STEEL IN STRUCTURE:

Arch	A7	25
	A242	75
Ties	A242	
Floor system and bracing	A7	
Hangers	A7	

OWNER: Kansas Department of Transportation and Missouri Highway and Transportation Department
ENGINEER: Howard, Needles, Tammen & Bergendoff
FABRICATOR: American Bridge Division, U.S. Steel Corp.
DATE OF COMPLETION: April 1955



RIB DATA	A	B
UNSTRESSED	79.694'	79.682'
AT CLOSURE	80.000'	79.967'
UNDER FULL D.L.	79.694'	79.682'

FIGURE 14.28 Details of Leavenworth Centennial Bridge.

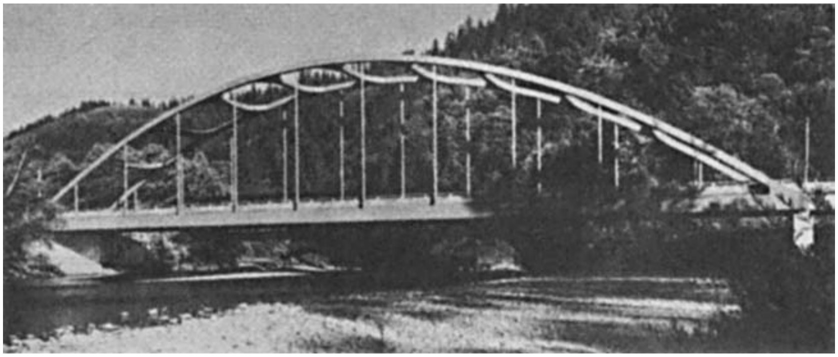


FIGURE 14.29

NORTH FORK STILLAGUAMISH RIVER BRIDGE

LOCATION: Cicero, Snobomish County, Washington
TYPE: Tied, through, solid-ribbed arch, 11 panels at 25.3 ft
SPAN: 278.6 ft RISE: 51 ft RISE/SPAN = 1:5.5
NO. OF LANES OF TRAFFIC: 2
HINGES: 0 DEPTH: 2 ft DEPTH/SPAN = 1:139

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	2,500
Railings and parapets	1,000
Floor steel for roadway	475
Floor bracing	59
Arch ribs	684
Arch bracing	400
Arch hangers or posts and bracing	83
Arch ties	799
TOTAL	6,000

SPECIFICATION LIVE LOADING: HS20
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,055 lb per ft
TYPES OF STEEL IN STRUCTURE:

Arch ribs and ties	A36	28
	A440 and A441	72
Floor system	A36	63
	A440 and A441	37
Hangers	A36	

OWNER: Washington Department of Transportation
ENGINEER: Bridges and Structures Division, Washington DOT
FABRICATOR: Northwest Steel Fabricators, Vancouver, Wash.
GENERAL CONTRACTOR: Dale M. Madden, Inc., Seattle, Wash.
DATE OF COMPLETION: 1966

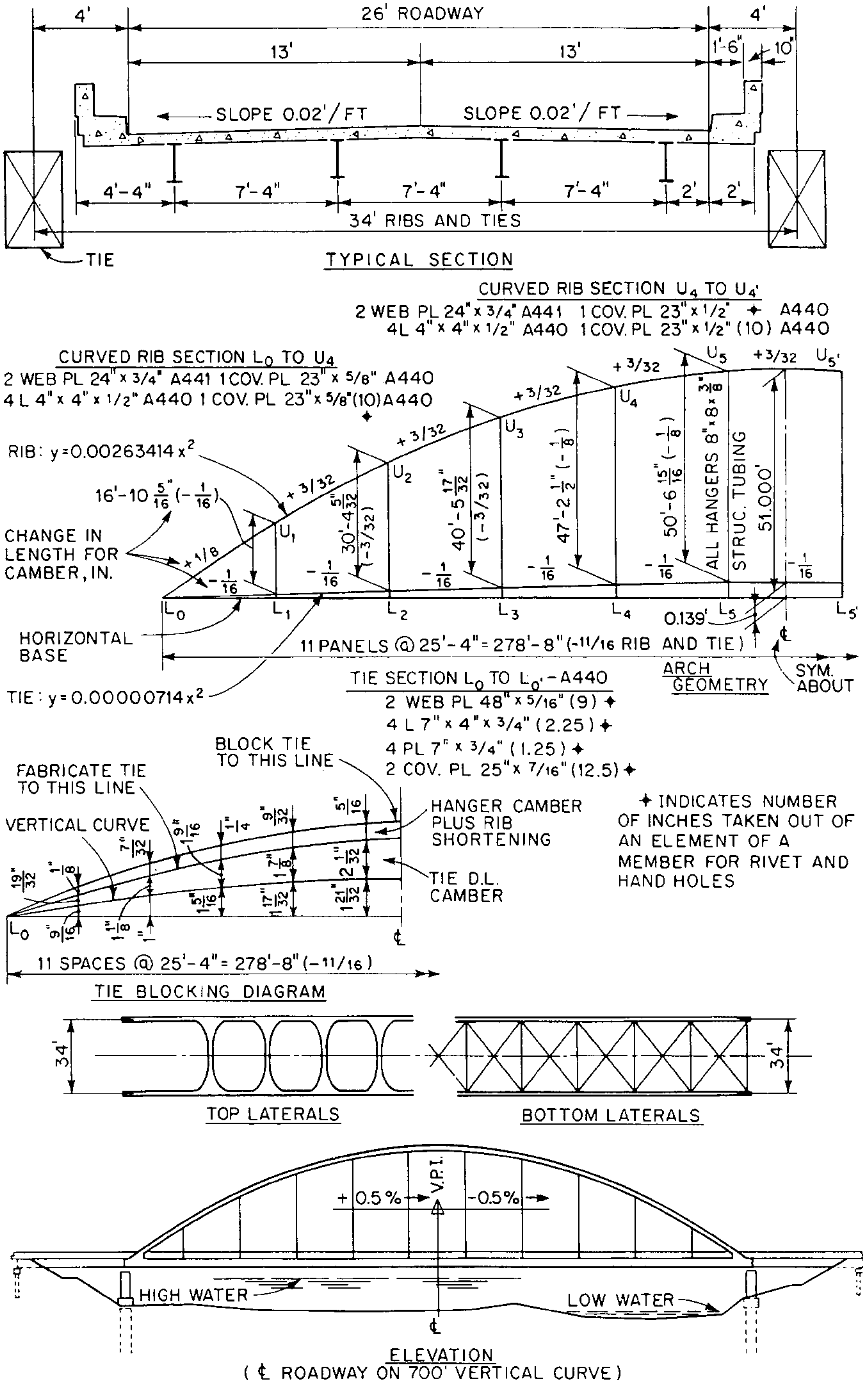


FIGURE 14.30 Details of North Fork Stillaguamish River Bridge.

FIGURE 14.31

SOUTH STREET BRIDGE OVER I-84

LOCATION: South Street over Route I-84, Middlebury, Connecticut
TYPE: Solid-ribbed deck arch, 7 panels, 5 at 29 ft
SPAN: 193 ft RISE: 29 ft RISE/SPAN = 1:6.7
NO. OF LANES OF TRAFFIC: 2
HINGES: 2 DEPTH: 3.3 ft DEPTH/SPAN = 1:58

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway	4,000
Railings and parapets	500
Floor steel for roadway	560
Arch ribs	1,070
Arch bracing	230
Arch posts and bracing	20
Miscellaneous—utilities, excess, etc	50
TOTAL	6,430

SPECIFICATION LIVE LOADING: H20-S16-44
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,498 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch ribs	A373	98
	A242	2
Floor system	A373	

OWNER: Connecticut Department of Transportation
ENGINEER: Connecticut DOT
FABRICATOR: The Ingalls Iron Works Co.
DATE OF COMPLETION: 1964

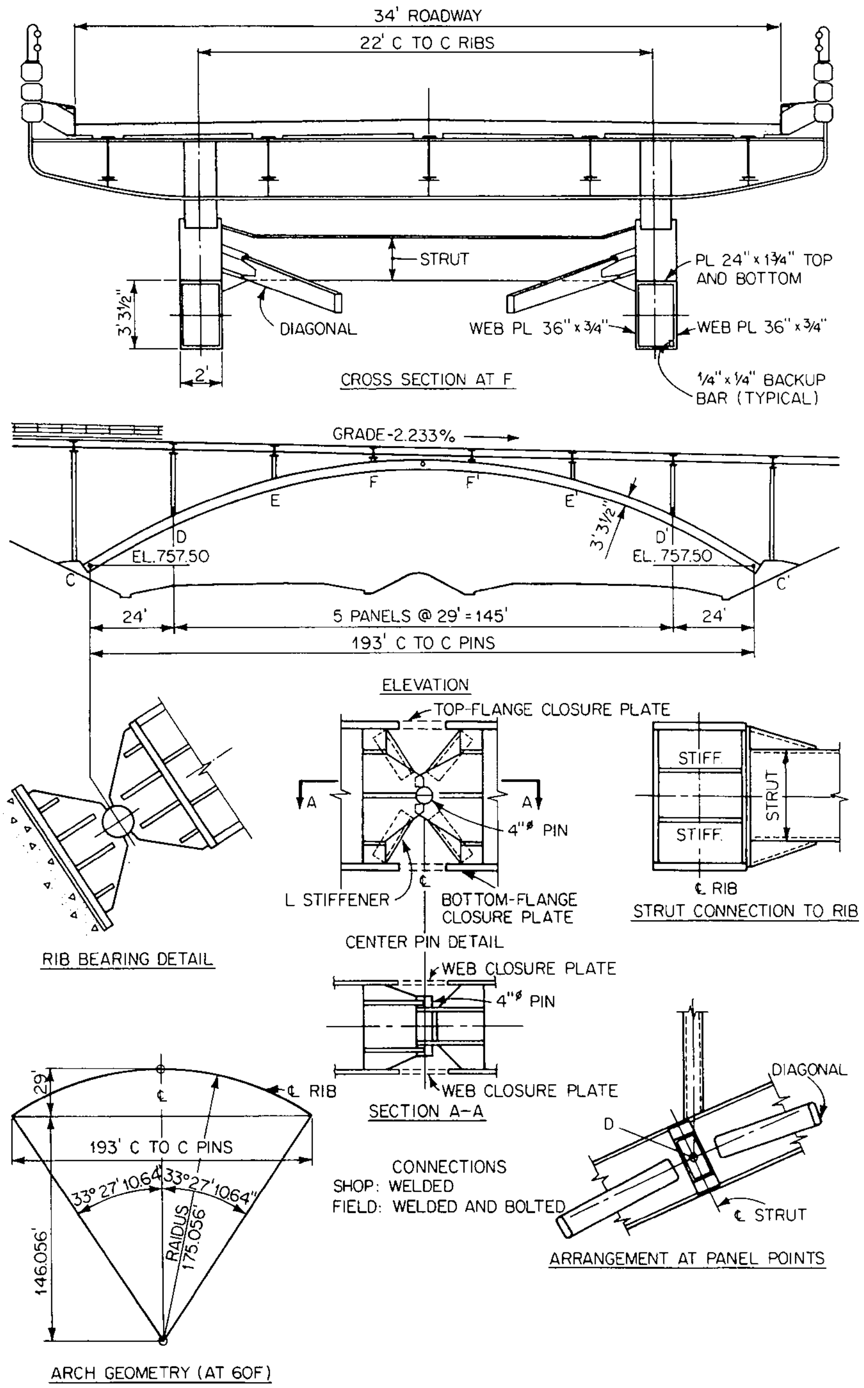


FIGURE 14.32 Details of South Street Bridge over I-84.

14.9 GUIDELINES FOR PRELIMINARY DESIGNS AND ESTIMATES

The usual procedure followed by most designers in preliminary designs of bridges involves the following steps:

1. Preliminary layout of structure
2. Preliminary design of floor system and calculation of weights and dead load
3. Preliminary layout of bracing systems and estimates of weights and loads
4. Preliminary estimate of weight of main load-bearing structure
5. Preliminary stress analysis
6. Check of initial assumptions for dead load

Preliminary Weight of Arch. The ratios given in Art. 14.6 can guide designers in making a preliminary layout with nearly correct proportions. The 16 examples of arches in Art. 14.8 also can be helpful for that purpose and, in addition, valuable in making initial estimates of weights and dead loads.

Equations (14.1) and (14.2), shown graphically in Fig. 14.33, were developed to facilitate estimating weights of main arch members.

For a true arch of low-alloy, high-strength steel (without ties), the ratio R of weight of rib to total load on the arch may be estimated from

$$R = 0.032 + 0.000288S \tag{14.1}$$

where S = span, ft.

For a tied arch of low-alloy, high-strength steel, the ratio R of weight of rib and tie to total load on the arch may be estimated from

$$R = 0.088 + 0.000321S \tag{14.2}$$

This equation was derived from a study of seven of the structures in Art. 14.8 that are tied arches made of low-alloy, high-strength steels predominantly for ribs, trusses, and ties. Equation (14.1), however, is not supported by as many examples of actual designs and may give values on the high side for truss arches. Despite the small number of samples, both equations should give reasonably accurate estimates of weight for preliminary designs and cost estimates of solid-ribbed and truss arches and for comparative studies of different types of structures.

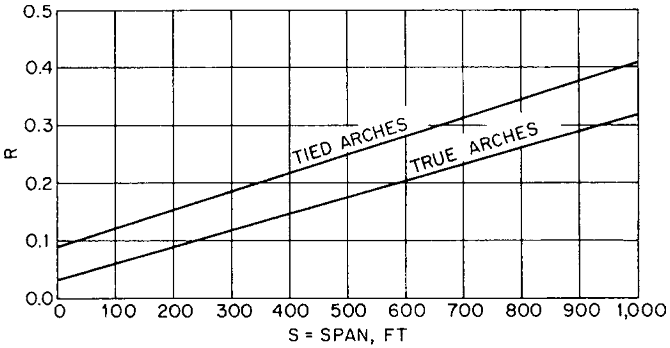


FIGURE 14.33 Chart gives ratio R of weight of rib, or rib and tie, to total load for arches fabricated predominantly with low-alloy steel.

With R known, the weight W, lb/ft, of arch, or arch plus tie, is given by

$$W = \frac{R(D+L)}{1-R} \tag{14.3}$$

where D = dead load on arch, lb/ft, excluding weight of arch, or arch plus tie and L = equivalent live load plus impact, lb/ft, on arch when the structure is fully loaded. D is determined from preliminary design of bridge components other than arches and ties.

Effect of Type of Steel on Arch Weights. The following approximate analysis may be used to determine the weight of arch rib or arch rib and tie based on the weight of arch for some initial design with one grade of steel and an alternative for some other grade with different physical properties.

Let F_b = basic unit stress for basic design, ksi
F_a = basic unit stress for alternative design, ksi
D = dead load, lb/ft, excluding weight of rib, or rib and tie
L = equivalent live load plus impact, lb/ft, for fully loaded structure
W_b = weight of rib, or rib and tie, lb/ft for basic design
W_a = weight of rib, or rib and tie, lb/ft, for alternate design
P_b = total load, lb, carried by 1 lb of rib, or 1 lb of rib and tie, for basic design
P_a = total load, lb, carried by 1 lb of rib, or 1 lb of rib and tie, for alternate design

The load supported per pound of member may be assumed proportional to the basic unit stress. Hence,

$$\frac{P_b}{P_a} = \frac{F_b}{F_a} \tag{14.4}$$

Also, the load per pound of member equals the ratio of the total load, lb/ft, on the arch to weight of member, lb/ft. Thus,

$$P_b = \frac{D+L+W_b}{W_b} = 1 + \frac{D+L}{W_b} \tag{14.5}$$

Similarly, and with use of Eq. (14.4),

$$P_a = 1 + \frac{D+L}{W_a} = \frac{P_b F_a}{F_b} \tag{14.6}$$

Solving for the weight of rib, or rib plus tie, gives

$$W_a = \frac{(D+L)F_b/F_a}{P_b - F_b/F_a} \tag{14.7}$$

Use of the preceding equations will be illustrated by application to the Sherman Minton Bridge (Figs. 14.11 and 14.12). Its arches were fabricated mostly of A514 steel. Assume that a preliminary design has been made for the floor system and bracing. A preliminary estimate of weight of truss arch and tie is required.

From the data given for this structure in Art. 14.8, the total load per arch, excluding truss arch and tie, is

$$D + L = \frac{15{,}580}{2} + 1755 = 9545 \text{ lb/ft}$$

From Eq. (14.2), or from Fig. 14.33, with span S = 797.5 ft, if the arch had been constructed of low-alloy steel, the ratio of weight of rib and tie to total load would be about

$$R = 0.088 + 0.000321 \times 797.5 = 0.34$$

By Eq. (14.3), the weight of rib and tie, if made of low-alloy steel, would have been

$$W_b = 9545 \times \frac{0.34}{1 - 0.34} = 4900 \text{ lb/ft}$$

For the A514 steel actually used for the arch, an estimate of weight of rib and tie may be obtained from Eqs. (14.5) and (14.7). When these are applied, the following basic unit stresses may be used:

Normal grades of low-carbon steel—$F = 18$ ksi
Low-alloy, high-strength steels—$F = 24$ ksi
A514 high-strength steel—$F = 45$ ksi

These stresses make some allowance for reductions due to thickness, reductions due to compression, and other similar factors. A check against a number of actual designs indicates that these values give about the correct ratios for the above grades of steel. Accordingly, the calculations for estimating weight of rib plus tie of A514 steel are as follows:

$$\frac{F_b}{F_a} = \frac{24}{45} = 0.53$$

By Eq. (14.5),

$$P_b = 1 + \frac{9545}{4900} = 2.95$$

Then, by Eq. (14.7), the weight of rib and tie is estimated at

$$W_a = 9545 \times \frac{0.53}{2.95 - 0.53} = 2090 \text{ lb/ft}$$

Use 2100 in preliminary design calculations.

Weight of truss arch and tie as constructed is $^1/_2$(3400 + 1040) = 2200 lb/ft, checking the estimate within about 5%.

14.10 BUCKLING CONSIDERATIONS FOR ARCHES

Since all arches are subjected to large compressive stresses and also usually carry significant bending moments, stability considerations must be addressed. The American Association of State Highway and Transportation Officials (AASHTO) “Standard Specifications for Highway Bridges” and “LRFD Bridge Design Specifications” contain provisions intended to ensure stability.

For true arches, the design should provide stability in the vertical plane of the arch, with the associated effective buckling length, and also provide for moment amplification effects. For tied arches with the tie and roadway suspended from the arch, moment amplification in the arch rib need not be considered. For such arches, the effective length can be considered the distance along the arch between hangers. However, with the relatively small cross-sectional area of the cable hangers, the effective length may be slightly longer than the distance between hangers.

For prevention of buckling in the lateral direction, a lateral bracing system of adequate stiffness should be provided. Effective lengths equal to the distance between rib bracing points are usually assumed. Special consideration should be given to arch-end portal areas. Lateral torsional buckling for open I-section ribs is much more critical than for box ribs and must be prevented.

Local buckling of web and flange plates is avoided by designs conforming to the limiting plate width-to-thickness ratios in the AASHTO specifications. If longitudinal web stiffeners are provided, additional criteria for their design are available (Art. 10.11).

14.11 EXAMPLE—DESIGN OF TIED-ARCH BRIDGE

The typical calculations that follow are based on the design of the 750-ft, solid-ribbed arch for the Glenfield Bridge (Figs. 14.17 and 14.18). The original design was in accordance with AASHTO "Standard Specifications for Highway Bridges," 1965. In previous editions of this publication, the example had been revised in general accordance with later editions of the "Standard Specifications." For this edition, the design has been revised in general accordance with the AASHTO "LRFD Bridge Design Specifications," Third Edition, 2004 (LRFD Specifications) for the load and resistance factor design method. Conditions that do not meet applicable code provisions are noted.

The structure is a tied, through arch with 50-ft-long panels. The tie has a constant depth of 12 ft 6 in. The arch rib is segmental (straight between panel points) and tapers in depth from 7 ft at the springing line to 4 ft at the crown.

The tied arches are assumed to be fabricated so that the dead loads, except member dead loads between panel points, are carried by axial stresses. The floor system is assumed to act independently. Thus, it does not participate in the longitudinal behavior of the arches. The following illustrates the design of selected components and some typical structural details.

14.11.1 Design of Floor System

The floor system was originally designed for HS20-44 loading. The minimum design load provided for by the LRFD Specifications is the HL-93 loading, a combination of the previously specified HS20 truck and lane loading. (See Art. 10.5.2.)

Slab Design. Assumed cross sections of the roadway slab are shown in Fig. 14.34. While the design of deck slabs using the traditional method (based on flexure) is still permitted by the LRFD Specifications, the use of the empirical deck-slab design methodology is also supported. This empirical method, which is based on extensive research on the load-carrying action (internal arching) and failure mode (punching shear) within deck systems, recognizes the overly conservative factor of safety of the traditional method. Use of the empirical method, which is gaining acceptance throughout the design community, reduces the amount of deck-slab reinforcement steel required

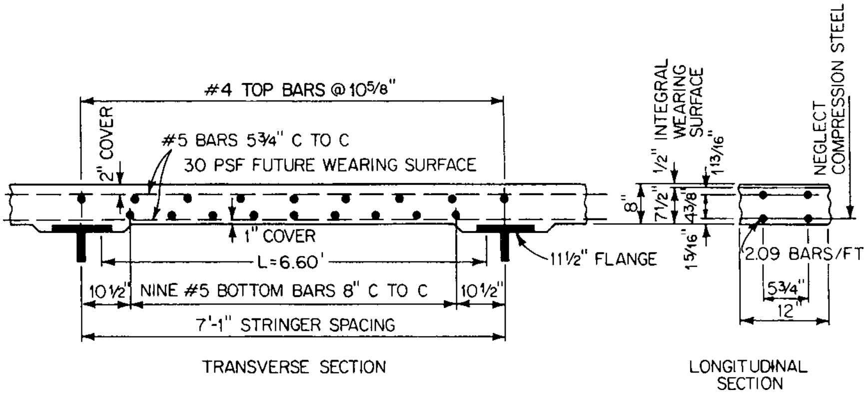


FIGURE 14.34 Concrete deck of tied-arch Glenfield Bridge (Fig. 14.18) is supported on steel stringers. (Current AASHTO Specifications require that concrete cover over top reinforcing steel be at least $2^1/_2$ in instead of the 2 in used for the Glenfield Bridge.)

(from typical values of 300 lb/yd^3 for a traditional design to 225 lb/yd^3 for an empirical design), while still maintaining a conservative factor of safety.

The empirical design method is governed by Sec. 9.7.2 of the LRFD Specifications, and is applicable (other than for the deck overhang) if certain conditions are satisfied (as listed in Sec. 9.7.2.4 of the LRFD Specifications). If designed today, it is likely that the Glenfield Bridge would have met all of these conditions, and could have been detailed with lighter reinforcement (a minimum of 0.27 in^2/ft in each of the bottom layers, and 0.18 in^2/ft in each of the top layers) than that shown in Fig. 14.34. However, the original design, which provided for slab concrete with a 28-day strength $f_c' = 3$ ksi, Grade 40 reinforcing steel ($F_y = 40$ ksi), and supporting structural elements (stringers) that are noncomposite with the slab, does not meet the criteria of Sec. 9.7.2.4, and therefore the empirical design method cannot be employed.

Traditional deck-slab design included in the LRFD Specifications permits the use of either an approximate method (equivalent-strip method, which is similar to the approach used in the AASHTO Standard Specifications) or a refined method. This example demonstrates the approximate method, governed by LRFD Specifications Sec. 4.6.2.1. In this approximate method, the deck is divided into strips perpendicular to the supporting components (stringers). In this case, the strips span in the transverse direction. The calculations of bending moments in the slab take into account continuity.

For dead load, the maximum moment per foot is independent of the strip width, and is taken as $+M = wL^2/12.5$ and $-M = wL^2/10$, where w is the dead load, kips/ft^2, and L is the effective slab span measured between stringer flange quarter points = 7.083 – 0.96/2 = 6.60 ft. Slab dead loads and dead-load moments are as follows. (See Art. 10.5.1 for load classifications such as *DC* and *DW*.)

Slab dead load, kips/ft^2

Concrete (DC) = $0.150 \times 0.667 = 0.100$

Future wearing surface (DW) = 0.030

Slab dead-load moments (kip · ft/ft):

$$+M\,(DC) = 0.100 \times \frac{(6.60)^2}{12.5} = 0.348$$

$$+M\,(DW) = 0.030 \times \frac{(6.60)^2}{12.5} = 0.105$$

$$-M\,(DC) = 0.100 \times \frac{(6.60)^2}{10} = 0.436$$

$$-M\,(DW) = 0.030 \times \frac{(6.60)^2}{10} = 0.131$$

For live load, based on LRFD Specifications Sec. 3.6.1.3.3, the transverse strips are to be designed for the wheels of the 32.0-kip axle and the lane load. According to Sec. C4.6.2.1.3, the load per unit width of the equivalent strip is obtained by dividing the total load on one design traffic lane by the calculated strip width. For this cast-in-place concrete deck, the width of the primary strip (in inches) for positive and negative moment is dependent on the stringer spacing, $S = 7.083$ ft. This differs from the "effective slab span" used previously. The strip width is calculated as

$+M$: 26.0 + 6.6(7.083) = 72.7 in = 6.06 ft

$-M$: 48.0 + 3.0(7.083) = 69.2 in = 5.77 ft

Given this information, two options are available to compute the maximum live-load moments. The first entails an analysis (i.e., by computer modeling) of the strip as a continuous beam for various positions of the live load, which is then divided by the appropriate strip widths computed above. In lieu of this more rigorous analysis, the LRFD Specification (in App. A4) provides a tabulation of upper-bound maximum live-load moments for various deck geometries. For this example, values for the 7.083-ft stringer spacing will be interpolated from App. A. These tabular values include multiple presence factors and dynamic load allowance:

Slab live-load moments (kip · ft/ft):

$+M = 5.25$

$-M = 5.22$ (using 3 in from stringer centerline to design section)

Total factored moments (kip · ft/ft) are computed as

$$+M = 1.25(0.348) + 1.50(0.105) + 1.75(5.25) = 9.78$$

$$-M = 1.25(0.436) + 1.50(0.131) + 1.75(5.22) = 9.88$$

Using the provisions of Sec. 5 of the LRFD Specifications and these moment demands, the amount of reinforcing steel required per foot of deck (using $f'_c = 3$ ksi and $F_y = 40$ ksi) has been determined to be approximately 0.51 in^2/ft for positive bending (bottom of slab using $d = 6.6875$ in) and 0.62 in^2/ft for negative bending (top of slab using $d = 5.6875$ in). The area of transverse reinforcing provided both top and bottom, $A_s = 0.65$ in^2/ft, is adequate.

The percentage of the main reinforcement that must be supplied for distribution reinforcing in the longitudinal direction is computed from $220/(L)^{0.5}$, but need not exceed 67%, in accordance with Sec. 9.7.3.2 of the LRFD Specifications.

$$\frac{220}{(6.60)^{0.5}} = 85.6 \qquad \text{therefore use } 67\%$$

The required area of distribution steel = 0.67 × 0.65 = 0.44 in^2/ft. Number 5 bars on 8-in centers supply an area of 0.47 in^2/ft > 0.44.

Stringer Design. The stringers are designed as three-span continuous noncomposite beams on rigid supports 50 ft apart (Fig. 14.35). (While floorbeams do not provide perfectly rigid supports, the effects of their flexibility have been studied and found to be small.) The 101-ft-wide roadway is assumed to be carried by 15 stringers, each a W33 × 130, made of A36 steel ($F_y = 36$ ksi). Had this bridge been designed more recently, it is likely that the stringers would have been made composite with the deck slab.

The dead load is considered to consist of four parts: the initial weight of stringer and concrete deck (Table 14.1*a*); the superimposed weight of median, parapets, and railings (Table 14.1*b*); the superimposed weight of future wearing surface; and a concentrated load at mid-span from a diaphragm and connections (Fig. 14.35).

Initial dead load per stringer (*DC*) = 0.868 kip/ft

Superimposed dead load per stringer (*DC*) = 1.094/15 = 0.073 kip/ft

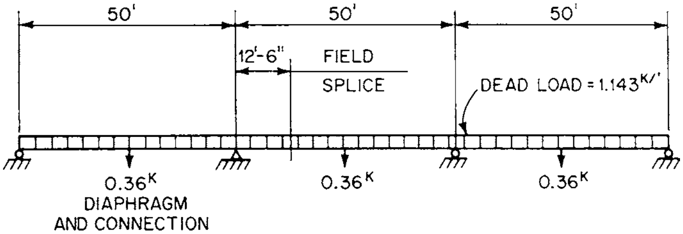


FIGURE 14.35 Stringers of Glenfield Bridge are continuous over three spans, with a splice in the center span. Dead load is classified as 0.941 kip/ft *DC* and 0.202 kip/ft *DW*.

TABLE 14.1 Dead Load per Stringer, kips/ft

(*a*) Initial dead load	
W33 × 130 =	0.130
8-in concrete slab: 0.150(8/12)7.083 =	0.708
Concrete haunch =	0.030
Total =	0.868 kip/ft
(*b*) Superimposed dead load on 15 stringers	
Bridge median =	0.030
Two railings: 2 × 0.030 =	0.060
Two parapets =	1.004
Total =	1.094 kips/ft

Superimposed dead load per stringer (*DW*) = (0.030 × 101)/15 = 0.202 kip/ft

Concentrated dead load at mid-span (*DC*) = 0.36 kip

If performed by hand, the calculation of live-load distribution factors for the stringers in accordance with the LRFD Specifications is significantly more complex than under previous specifications. However, with the aid of one of numerous software packages available, these computations are readily performed, along with the analysis of the stringers for both dead and live loads. Critical design moments and reactions are summarized in Table 14.2.

The first check to be performed is the flexural resistance of the noncomposite stringer in the positive-moment region—compression flange. The compression flange is assumed to be continuously braced by the deck slab, and therefore, lateral bending stresses are taken equal to zero. This compression flange is checked in accordance with LRFD Sec. 6.10.8.1.3:

$$f_{bu} \leq \Phi_f R_h F_{yf}$$

$$f_{bu} = \frac{M}{S} = \frac{(1205.8)(12)}{406} = 35.64 \text{ ksi}$$

$$\Phi_f = 1.00$$

$$R_h = 1.0 \qquad \text{for rolled sections}$$

$$F_{yf} = 36 \text{ ksi}$$

$$f_{bu} = 35.64 \text{ ksi} < (1.00)(1.0)(36 \text{ ksi}) = 36 \text{ ksi} \qquad \text{OK}$$

Next, check the flexural resistance of the noncomposite stringer in the positive-moment region—tension flange, in accordance with Sec. 6.10.8.1.2. Because the tension flange is braced at discrete locations, the lateral bending stresses (such as those due to wind as covered in the LRFD Specifications

TABLE 14.2 Design Moments and Reactions for Stringer End Span

	Maximum positive bending moment at 0.4 point of end span, ft·kips	Maximum negative bending moment at first interior support, ft·kips	Maximum reaction at first interior support, kips
Dead load (*DC*)	191.0	237.3	52.2
Dead load (*DW*)	40.5	50.4	11.1
LL + I	517.9	376.5	71.7
Factored (Strength I)	1205.8	1031.1	207.4

The maximum factored shear (Strength I) occurs at the first interior support. V_u = 182.2 kips

Sec. 4.6.2.7) should be computed. However, the wind load is included in the Strength III group, which does not include live load. (See Art. 10.4.1 for group designations.) The wind load on these shallow rolled stringers is relatively small and does not control. For the Strength I group, $f_\ell = 0$.

$$f_{bu} + {}^1\!/_3 f_\ell \le \Phi_f F_{nt} \qquad \text{with} \qquad f_\ell = 0, f_{bu} \le \Phi_f F_{nt}$$

$$f_{bu} = \frac{M}{S} = \frac{(1205.8)(12)}{406} = 35.64 \text{ ksi}$$

$$F_{nt} = R_h F_{yt} = (1.0)(36 \text{ ksi}) = 36 \text{ ksi}$$

$$f_{bu} = 35.64 \text{ ksi} < (\Phi_f = 1.00)F_{nt} = 36 \text{ ksi} \qquad \text{OK}$$

The next check to be performed is the flexural resistance of the noncomposite stringer in the negative-moment region—compression flange. As in the positive-moment region, the Strength III group does not control. For the Strength I group, $f_\ell = 0$. This compression flange is checked in accordance with Sec. 6.10.8.1.1:

$$f_{bu} + {}^1\!/_3 f_\ell \le \Phi_f F_{nc} \qquad \text{with} \qquad f_\ell = 0, f_{bu} \le \Phi_f F_{nc}$$

$$f_{bu} = \frac{M}{S} = \frac{(1031.1)(12)}{406} = 30.48 \text{ ksi}$$

Local buckling resistance is found from

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{11.5}{2(0.855)} = 6.725$$

$$\lambda_{pf} = 0.38\left(\frac{E}{F_{yc}}\right)^{1/2} = 10.785$$

$$\lambda_f = 6.725 < \lambda_{pf} = 10.785$$

Therefore

$$F_{nc} = R_b R_h F_{yc}$$

$$R_h = 1.0$$

$$R_b = 1.0 \qquad \text{for } \frac{2D_c}{t_w} < \lambda_{rw}$$

$$F_{nc} = R_b R_h F_{yc} = (1.0)(1.0)(36 \text{ ksi}) = 36 \text{ ksi}$$

$$f_{bu} = 30.48 \text{ ksi} < (\Phi_f = 1.00)F_{nc} = 36 \text{ ksi} \qquad \text{OK}$$

Lateral torsional buckling resistance is found from

$$L_b = 25 \text{ ft} = 300 \text{ in}$$

$$L_p = 1.0 r_t \left(\frac{E}{F_{yc}}\right)^{1/2} = (1.0)(2.88)(28.38) = 82$$

$$L_r = \pi r_t \left(\frac{E}{F_{yr}}\right)^{1/2} = (3.14)(2.88)(33.92) = 307$$

$$L_p < L_b < L_r$$

Therefore

F_{nc} is computed by Eq. 6.10.8.2.3-2 as 25.54 ksi

$$f_{bu} = 30.48 \text{ ksi} > (\Phi_f = 1.00)F_{nc} = 25.54 \text{ ksi} \qquad \text{No Good}$$

Other flexural checks that would be completed (which do not control for this example) include, tension flange in the negative-moment region by Sec. 6.10.8.1.3, net section fracture at the splice location by Sec. 6.10.1.8, web bend-buckling resistance by Sec. 6.10.1.9, permanent deformations at the Service II limit state by Sec. 6.10.4.2, and fatigue and fracture considerations by Sec. 6.10.5.

To evaluate shear resistance at the strength limit state (Sec. 6.10.9), the unstiffened web of the stringers must meet:

$$V_u \leq V_r = \Phi_v V_n$$

$$V_u = 182.2 \text{ kips}$$

$$V_n = V_{cr} = CV_p$$

By calculation, $C = 1.0$ for this stocky web.

$$V_p = 0.58F_{yw}Dt_w = 0.58(36)(31.38)(0.58) = 380.0 \text{ kips}$$

$$V_n = (1.0)(380.0 \text{ kips}) = 380.0 \text{ kips}$$

$$V_u = 182 \text{ kips} < (\Phi_v = 1.00)V_n = 380.0 \text{ kips} \qquad \text{OK}$$

Under previous specifications, the fact that $V_u < 0.75\Phi_v V_n$ would preclude the need for bearing stiffeners for these stringers. However, under the LRFD Specifications (Sec. 6.10.11.2), web local yielding and web crippling without bearing stiffeners should be checked in accordance with Art. D6.5.

The LRFD Specifications, in Sec. 2.5.2.6.2, make deflection criteria optional (leaving prescribed limits, if any, up to the governing agency). Under previous specifications, deflection under live load plus impact (dynamic-load allowance) was limited to $L/800$, where L is the span. Previous computations have given the deflection at the 0.4 point of the end span, $\delta = 0.472$ in.

$$\frac{\delta}{L} = \frac{0.472}{(50)(12)} = \frac{1}{1271} < \frac{1}{800}$$

This deflection would meet the criteria from previous specifications.

Stringer Splice Design. Because of the 150-ft length of the three-span beam, it will be erected in two pieces. A field splice is located in the center span, 12 ft 6 in from a support (Fig. 14.35). Figures 14.36, 14.37, and 14.38 show details of the stringer splice originally designed in accordance with the provisions of the AASHTO "Standard Specifications for Highway Bridges," 1965. This splice will be checked below for adequacy in accordance with the LRFD Specifications.

The current splice provisions require calculations that are fairly tedious and time-consuming to perform by hand. Splice-design calculations in accordance with the LRFD Specifications have been automated in the computer program AISIsplice, available from the American Iron and Steel Institute, 1140 Connecticut Ave., N.W., Suite 705, Washington, DC 20036. This program provides detailed output with all pertinent code checks performed for either a splice design or evaluation (analysis).

For this example, the connections will be made with $^7/_8$-in-diameter A325 bolts. Class A surface conditions are assumed, and bolts will act in double-shear with threads excluded from the shear planes.

Shear resistance of bolts from Sec. 6.13.2.7 (to be compared to Strength I loads):

$$R_r = \Phi_s R_n = \Phi_s(0.48A_b F_{ub} N_s) = (0.8)(0.48)(0.6013)(120)(2) = 55.4 \text{ kips}$$

Slip resistance of bolts from Sec. 6.13.2.8 (to be compared to Service II loads):

$$R_r = R_n = K_h K_s N_s P_t = (1.00)(0.33)(2)(39) = 25.7 \text{ kips}$$

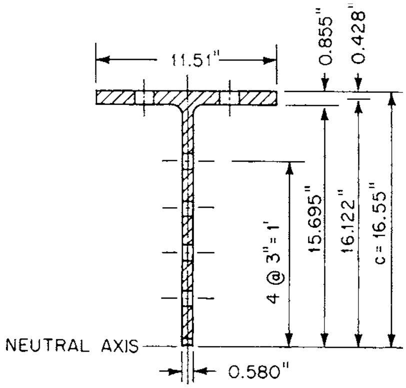


FIGURE 14.36 Stringer cross section at splice.

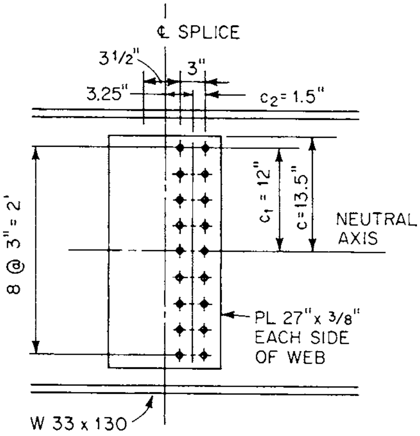


FIGURE 14.37 Web splice for stringer.

The maximum moments and shears at the splice are given in Table 14.3. The maximum factored shear at the splice is calculated from this as

(Strength I) $V_u = 1.25(11.9) + 1.50(2.5) + 1.75(50.8) = 107.5$ kips

(Service II) $V_u = 1.00(11.9) + 1.00(2.5) + 1.30(50.8) = 80.4$ kips

As computed previously, the shear resistance of the web, $V_r = 380.0$ kips. For $V_u \le 0.5V_r$, the design shear is

(Strength I) $V_{uw} = 1.5V_u = 1.5(107.5) = 161.3$ kips

(Service II) $V_{ser} = V_u = 80.4$ kips

Thus, the design shear requirements are satisfied.

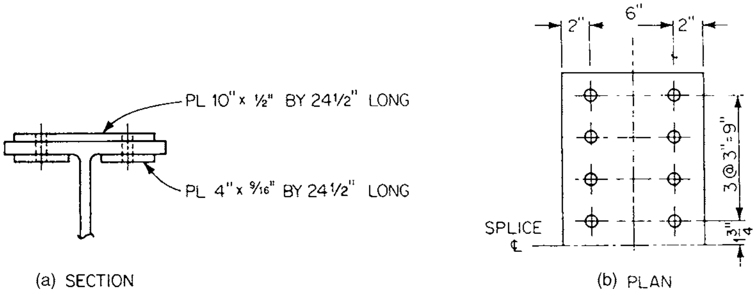
Calculated gross section properties are as follows:

I steel = 6710.0 in^4

S top and bottom of steel = 405.6 in^3

S top and bottom of web = 427.7 in^3

The design stress at each surface of the flanges is listed in Table 14.4 for different conditions. From this, the factored stress at the mid-thickness of the flange is calculated as



(a) SECTION (b) PLAN

FIGURE 14.38 Flange splice for stringer.

TABLE 14.3 Actual Moments and Shears for Stringer Splice

	Moment, ft·kips	Shear, kips
Dead load (*DC*)	–17.4	11.9
Dead load (*DW*)	–3.6	2.5
+ (*LL* + I)	267.2	50.8
– (*LL* + I)	–228.2	–12.4
Fatigue + (*LL* + I)	96.5	20.4
Fatigue – (*LL* + I)	–65.0	–4.7

$$\text{(Strength I)} \qquad f_{cf} = \frac{13.33 + 12.62}{2} = 12.98 \text{ ksi}$$

$$\text{(Service II)} \qquad f_s = \frac{9.68 + 9.16}{2} = 9.42 \text{ ksi}$$

$$[\text{Fatigue} + (LL + I)] \qquad f_{fat} = \frac{2.14 + 2.03}{2} = 2.09 \text{ ksi}$$

$$[\text{Fatigue} - (LL + I)] \qquad f_{fat} = \frac{-1.44 + -1.37}{2} = -1.41 \text{ ksi}$$

Fatigue stress range = 2.09 + 1.41 = 3.50 ksi

Determine effective area of the tension flange (Sec. 6.13.6.1.4c):

$$A_e = \left(\frac{\Phi_u F_u}{\Phi_y F_{yt}}\right) A_n \le A_g = (0.855)(11.5) = 9.83 \text{ in}^2$$

$$A_n = (0.855)[11.5 - 2(1)] = 8.12 \text{ in}^2$$

$$A_e = \left[\frac{(0.80)(58)}{(0.95)(36)}\right] \times (8.12) = 11.01 \text{ in}^2 > A_g \qquad \text{therefore } A_e = A_g = 9.83 \text{ in}^2$$

No reduction to the flange area is necessary, and the section properties for the gross section will be used.

In past AASHTO specifications, the design of the splices of flexural members were to be made for the larger of either the average of the capacity of the member and the actual loads at the splice, or 75% of the capacity of the member. Though a similar approach is included in the LRFD Specifications, the formulation is somewhat different. The approach is to calculate a design stress, F_{cf}, where

TABLE 14.4 Design Stress in Flange, ksi

	Top/bottom of steel	Top/bottom of web
Dead load (*DC*)	–0.51	–0.49
Dead load (*DW*)	–0.11	–0.10
+ (*LL* + I)	7.92	7.50
Factored (Strength I)	13.33	12.62
Factored (Service II)	9.68	9.16
Fatigue + (*LL* + I)	2.14	2.03
Fatigue – (*LL* + I)	–1.44	–1.37

$$F_{cf} = \frac{|f_{cf} / R_h| + \alpha\Phi_f F_{yf}}{2} \geq 0.75\alpha\Phi_f F_{yf}$$

For this design, $R_h = 1.0$, $\alpha = 1.0$, $\Phi_f = 1.0$, $F_{yf} = 36$ ksi, and this equation reduces to the larger of

$$F_{cf} = \frac{12.98 + 36}{2} = 24.49 \text{ ksi}$$

$$F_{cf} = (0.75)(36) = 27.00 \text{ ksi} \qquad \text{(Controls)}$$

The stringer flange splice plates and connections will be designed for the axial load that would produce a stress equivalent to the design stress, F_{cf}, for the Strength I group and the actual stress, f_s, for the Service II group:

(Strength I) $P_u = (27.00 \text{ ksi})(0.855 \text{ in})(11.5 \text{ in}) = 265.5$ kips

(Service II) $P_{ser} = (9.42 \text{ ksi})(0.855 \text{ in})(11.5 \text{ in}) = 92.6$ kips

(Fatigue) $P_{fat} = (3.50 \text{ ksi})(0.855 \text{ in})(11.5 \text{ in}) = 34.4$ kips

The area of the outside splice plate as shown in Fig. 14.38 is 5.00 in^2. The total area of the inside splice plates is 4.50 in^2. The percentage difference = 0.50/4.50 = 11.1%. Since this difference is greater than 10%, the flange splice design force will be divided on a pro-rated basis between the inside and outside plates:

(Strength I) $$P_u \text{ (outside)} = \frac{(265.5)(5.00)}{(9.50)} = 139.7 \text{ kips}$$

(Strength I) $$P_u \text{ (inside)} = \frac{(265.5)(4.50)}{(9.50)} = 125.8 \text{ kips}$$

(Service II) $$P_{ser} \text{ (outside)} = \frac{(92.6)(5.00)}{(9.50)} = 48.7 \text{ kips}$$

(Service II) $$P_{ser} \text{ (inside)} = \frac{(92.6)(4.50)}{(9.50)} = 43.9 \text{ kips}$$

(Fatigue) $$P_{fat} \text{ (outside)} = \frac{(34.4)(5.00)}{(9.50)} = 18.1 \text{ kips}$$

(Fatigue) $$P_{fat} \text{ (inside)} = \frac{(34.4)(4.50)}{(9.50)} = 16.3 \text{ kips}$$

The following equations are recommended in the LRFD Specifications to determine a design moment, M_{uw}, and a design horizontal force resultant, H_{uw}, to be applied at the mid-depth of the web for designing the web splice plates and their connections at the strength limit state:

$$M_{uw} = \left(\frac{t_w D^2}{12}\right)(|R_h F_{cf} - R_{cf} f_{ncf}|)$$

$$H_{uw} = \left(\frac{t_w D}{2}\right)(R_h F_{cf} + R_{cf} f_{ncf})$$

where t_w is web thickness and D is web depth.

$$R_{cf} = \left|\frac{F_{cf}}{f_{cf}}\right| = \left|\frac{27.00}{12.98}\right| = 2.08$$

f_{ncf} (for this noncomposite section is equal and opposite to f_{cf}) = −12.98 ksi

$$M_{uw} = \left[\frac{(0.58)(31.39^2)}{12}\right](|(1.0)(27.00) - (2.08)(-12.98)|) = 2571.6\ \text{kip}\cdot\text{in} = 214.3\ \text{kip}\cdot\text{ft}$$

$$H_{uw} = \left[\frac{(0.58)(31.39)}{2}\right][(1.0)(27.00) + (2.08)(-12.98)] = 0.0\ \text{kip}$$

For the Service II check for slip, M and H must be calculated as well. The modified equations for the service limit state are

$$M_{ser} = \left(\frac{t_w D^2}{12}\right)(|f_s - f_{os}|)$$

$$H_{ser} = \left(\frac{t_w D}{2}\right)(f_s + f_{os})$$

f_{os} (for this noncomposite section is equal and opposite to f_s) = −12.98 ksi

$$M_{ser} = \left[\frac{(0.58)(31.39^2)}{12}\right](|9.42 - -9.42|) = 897.2\ \text{kip}\cdot\text{in} = 74.8\ \text{kip}\cdot\text{ft}$$

$$H_{ser} = \left[\frac{(0.58)(31.39)}{2}\right](9.42 + -9.42) = 0.0\ \text{kip}$$

Flange Splice Plates. The following checks are performed for the Strength I limit state to determine the splice plate area required, A_{greq} and A_{nreq}.

1. Yielding of the gross section:
- Compression (inside plates):

$$A_{greq} = \frac{P_u}{\Phi_c F_y} = \frac{125.8}{(0.90)(36)} = 3.88\ \text{in}^2 < 4.50\ \text{in}^2$$

- Compression (outside plate):

$$A_{greq} = \frac{P_u}{\Phi_c F_y} = \frac{139.7}{(0.90)(36)} = 4.31\ \text{in}^2 < 5.00\ \text{in}^2$$

- Tension (inside plates):

$$A_{greq} = \frac{P_u}{\Phi_y F_y} = \frac{125.8}{(0.95)(36)} = 3.68\ \text{in}^2 < 4.50\ \text{in}^2$$

- Tension (outside plate):

$$A_{greq} = \frac{P_u}{\Phi_y F_y} = \frac{139.7}{(0.95)(36)} = 4.08\ \text{in}^2 < 5.00\ \text{in}^2$$

2. Fracture of the net section:
- Tension (inside plates):

$$A_{nreq} = \frac{P_u}{\Phi_u F_u} = \frac{125.8}{(0.80)(58)} = 2.71\ \text{in}^2 < 3.38\ \text{in}^2$$

- Tension (outside plate):

$$A_{nreq} = \frac{P_u}{\Phi_u F_u} = \frac{139.7}{(0.80)(58)} = 3.01 \text{ in}^2 < 4.00 \text{ in}^2$$

3. Block shear rupture: Calculations according to Sec. 6.13.5.2 show that this limit state does not control.

In addition, at the fatigue limit state, the gross area of the splice plates must be checked. The base metal at the bolted splices is a Category B detail, with a constant-amplitude fatigue threshold equal to 16.0 ksi. The nominal fatigue resistance, $(\Delta F)_n$, is taken as $\geq {}^1\!/_2\,(16.0) = 8.0$ ksi.

Tension (inside plates):

$$A_{greq} = \frac{P_{fat}}{(\Delta F)_n} = \frac{16.3}{8.0} = 2.04 \text{ in}^2 < 4.50 \text{ in}^2$$

Tension (outside plate):

$$A_{greq} = \frac{P_{fat}}{(\Delta F)_n} = \frac{18.1}{8.0} = 2.26 \text{ in}^2 < 5.00 \text{ in}^2$$

Flange Splice Bolts. The following checks are performed to determine the number of bolts required on each side of the centerline of the splice. In addition to these checks, pitch, gage, edge, and end-distance requirements of Sec. 6.13.2.6 must be checked. For this splice, these requirements are met. Note that, due to the use of both inside and outside splice plates, the bolts are in double-shear.

1. Bolt shear (Strength I)—Check worst-case slip plane (adjacent outside plate):

$$\text{No. of bolts required} = \frac{139.7 \text{ kips}}{(55.4 \text{ kips per bolt for 2 shear planes}/2)} = 5.04 < 8$$

2. Slip (Service II)—Check the entire load on both slip planes:

$$\text{No. of bolts required} = \frac{92.6 \text{ kips}}{(25.7 \text{ kips per bolt for 2 shear planes})} = 3.60 < 8$$

3. Plate bearing at bolt holes (clear spacing between holes = 2 in; clear edge distance $L_c = 1.0$ in; d = bolt diameter; t = plate thickness; F_u = specified minimum plate tensile strength):

- For clear spacing > 2.0d, $R_n = 2.4dtF_u = (2.4)(0.875)(t)(58) = 121.8t$ for the interior holes
- For clear end distance < 2.0d, $R_n = 1.2L_c tF_u = (1.2)(1.0)(t)(58) = 69.6t$ for the end holes
- The weighted average is $[(6)(121.8t) + (2)(69.6t)]/8 = 108.8t$
- $R_r = \Phi_{bb}R_n = (0.80)(108.8t) = 87.0t$
- Outside plate bolts required = 139.7/(87.0)(0.5) = 3.21 < 8
- Inside plate bolts required = 125.8/(87.0)(0.5625) = 2.57 < 8
- Stringer flange bolts required = 265.5/(87.0)(0.855) = 3.57 < 8

Web Splice Bolts. The following checks are performed to determine the number of bolts required on each side of the centerline of the web splice. In addition to these checks, pitch, gage, edge, and end-distance requirements of Sec. 6.13.2.6 must be checked. For this splice, these requirements are met. Note that, due to the use of splice plates on both sides of the web, the bolts are in double-shear.

1. Bolt group properties:

- Number of bolts each side of splice = 18
- Polar moment of inertia = $I_p = (9)(2)[3^2(9^2 - 1) + 3^2(2^2 - 1)]/12 = 1120.5 \text{ in}^2$

- Radius to outermost bolt from c.g. of bolt group = $r = (12^2 + 1.5^2)^{1/2} = 12.09$ in
- Polar section modulus = $I_p/r = 1120.5/12.09 = 92.7$ in

2. Slip (Service II):
   - $M_{ser} = 897.2$ kip·in
   - $V_{ser} = 80.4$ kips located 3.25 in from the c.g. of the bolt group
   - $H_{ser} = 0.0$ kips
   - $M_v = (80.4)(3.25) = 261.3$ kip·in
   - $M_{total} = 897.2 + 261.3 = 1158.5$ kip·in
   - $P_{shear\text{-}v} = 80.4/18 = 4.47$ kips
   - $P_{shear\text{-}h} = 0.0/18 = 0$ kips
   - $P_{moment} = 1158.5/92.7 = 12.50$ kips
   - $P_{moment\text{-}v} = (1.5/12.09)(12.50) = 1.55$ kips
   - $P_{moment\text{-}h} = (12/12.09)(12.50) = 12.41$ kips
   - $P_v = 4.47 + 1.55 = 6.02$ kips
   - $P_h = 0 + 12.41 = 12.41$ kips
   - $P_{total} = [(6.02)^2 + (12.41)^2]^{1/2} = 13.79$ kips < 25.7 kips—OK
3. Bolt shear (Strength I):
   - $M_{uw} = 2571.6$ kip·in
   - $V_{uw} = 161.3$ kips located 3.25 in from the c.g. of the bolt group
   - $H_{uw} = 0.0$ kips
   - $M_v = (161.3)(3.25) = 524.2$ kip·in
   - $M_{total} = 2571.6 + 524.2 = 3095.8$ kip·in
   - $P_{shear\text{-}v} = 161.3/18 = 8.96$ kips
   - $P_{shear\text{-}h} = 0.0/18 = 0$ kips
   - $P_{moment} = 3095.8/92.7 = 33.40$ kips
   - $P_{moment\text{-}v} = (1.5/12.09)(33.40) = 4.14$ kips
   - $P_{moment\text{-}h} = (12/12.09)(33.40) = 33.15$ kips
   - $P_v = 8.96 + 4.14 = 13.10$ kips
   - $P_h = 0 + 33.15 = 33.15$ kips
   - $P_{total} = [(13.10)^2 + (33.15)^2]^{1/2} = 35.64$ kips < 55.4 kips—OK

Web Splice Plates. The size of the splice plates required for the web (Fig. 14.37) is determined by the requirements for both shear and moment.

For connection elements in shear, in accordance with Sec. 6.13.5.3, the factored resistance is

$$R_r = \Phi_v R_n = \Phi_v(0.58A_g F_y) = (1.0)(0.58)(2 \times 0.375 \times 27)(36) = 422.8 \text{ kips}$$

The actual factored shear = 161.3 kips, and is therefore OK.

Additionally, fracture on the net section is checked:

$$R_r = \Phi_u R_n = \Phi_u(0.58A_n F_u) = (0.8)(0.58)(2 \times 0.375 \times (27 - 9))(58) = 363.3 \text{ kips}$$

The actual factored shear = 161.3 kips, and therefore is OK.

Bending in the web plates is checked for the combination of the moment previously calculated, $M_{uw} = 214.3$ kip·ft = 2571.6 kip·in, and the moment due to eccentricity of the shear = (161.3)(3.25) = 524.2 kip·in. Therefore, the total factored (Strength I) moment = 3095.8 kip·in. The moment of inertia of the web plates = 1230 in^4; and the section modulus = 91.1 in^3. This results in a maximum bending stress = 3095.8/91.1 = 33.98 ksi < 36 ksi, and therefore is OK.

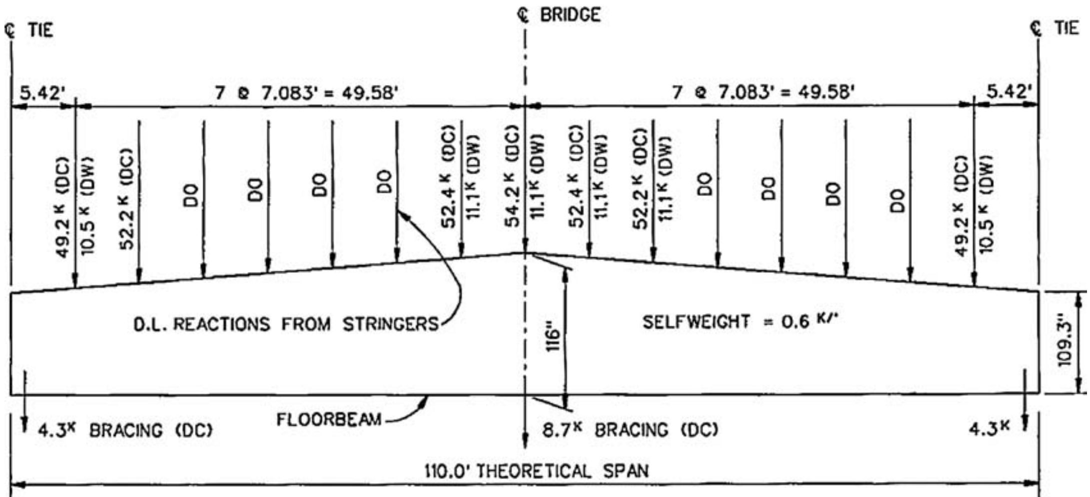
A similar check performed for the service limit state shows $M_{ser} = 74.8$ kip·ft = 897.2 kip·in, and the moment due to eccentricity of the shear = (80.4)(3.25) = 261.3 kip·in. Therefore, the total factored (Service II) moment = 1158.5 kip·in. This results in a maximum bending stress = 1158.5/91.1 = 12.72 ksi < (36 ksi)(0.8) = 28.8 ksi, and therefore is OK.

Design of Interior Floorbeam. Floorbeams, spaced 50 ft center to center (c to c), are designed as hybrid girders over the center 60 ft of their 110-ft spans. The web is fabricated of A588 steel

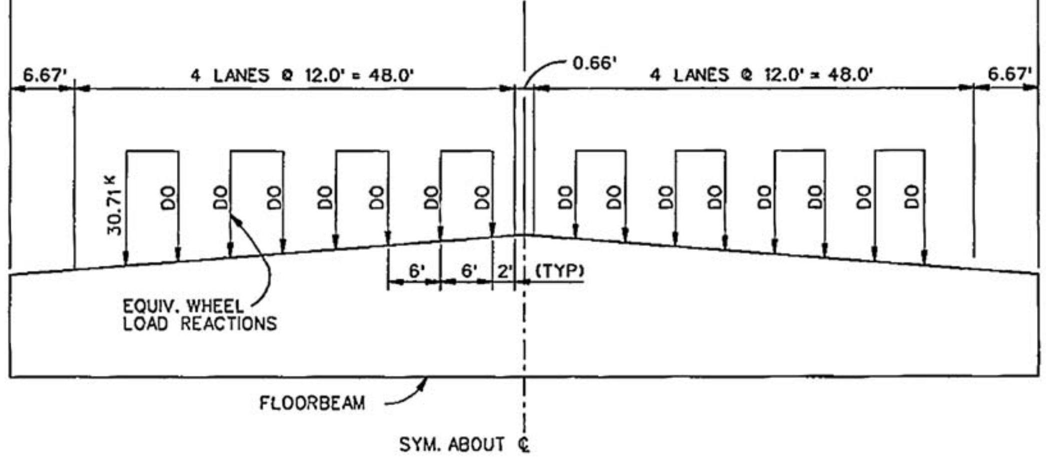
($F_y = 50$ ksi). Its depth varies from 109.3 in at centerline of ties to 116 in at centerline of bridge, to accommodate the cross slope of the roadway. For the flange, A514 steel ($F_y = 100$ ksi) as well as A588 is used, to keep flange thickness constant over the full width of the bridge. The floorbeams are noncomposite. The LRFD Specifications, in Sec. 6.10.1.3, recommend that the yield stress of the web should not be less than 70% of the yield stress of the flange. Although this hybrid floorbeam violates this guideline, the provisions of the LRFD Specifications are still valid.

The dead and live loads on a typical interior floorbeam are indicated in Fig. 14.39. The equivalent wheel-load reactions shown include the dynamic load allowance and a multiple presence factor of 0.65. Table 14.5 indicates the maximum moments and shears in the interior floorbeam. The hybrid section is used in the region of maximum moment. The floorbeam properties required for bending analysis are given in Table 14.6.

Floorbeam Flexure Check. Since the floorbeams are assumed to be simply supported at the ends, they are in positive bending for their entire length (top flange in compression). The first check to be



(a) DEAD LOADS



(b) LIVE LOADS FOR MAXIMUM MOMENT

FIGURE 14.39 Loads on an interior floorbeam of Glenfield Bridge.

TABLE 14.5 Maximum Moments and Shears in Interior Floorbeam

	Moment at centerline of bridge, ft·kips	Shear at tie, kips
Dead load (*DC*)	12,379	431.4
Dead load (*DW*)	2,377	82.7
(*LL* + I)	7,782	245.7
Factored (Strength I)	$M_{ux} = 32{,}658$	$V_u = 1{,}093.3$

performed is the flexural resistance of the noncomposite floorbeam—compression flange at the point of maximum moment (mid-span). The compression flange is assumed to be braced at discrete locations, by the supported stringers, therefore the unbraced length is $L_b = 7.083$ ft = 85 in. The lateral bending stresses, such as those due to wind as covered in the LRFD Specifications, Sec. 4.6.2.7, should be computed. However, the wind load would be included in the Strength III group, which does not include live load, and has been determined not to control. For the Strength I group, it will be assumed that $f_\ell = 0$. This compression flange is checked in accordance with Sec. 6.10.8.1.1:

$$f_{bu} + 1/3\, f_\ell \le \Phi_f F_{nc} \qquad \text{with} \qquad f_\ell = 0, f_{bu} \le \Phi_f F_{nc}$$

$$f_{bu} = \frac{M}{S} = \frac{(32{,}658)(12)}{6789} = 57.73 \text{ ksi}$$

Local buckling resistance according to Sec. 6.10.8.2.2 is

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{24}{2(2.0)} = 6.00$$

$$\lambda_{pf} = 0.38\left(\frac{E}{F_{yc}}\right)^{1/2} = 0.38\left(\frac{29{,}000}{100}\right)^{1/2} = 6.47$$

$$\lambda_f = 6.00 < \lambda_{pf} = 6.47$$

Therefore,

$$F_{nc} = R_b R_h F_{yc}$$

R_h according to Sec. 6.10.1.10.1 is found as

$$\beta = \frac{2D_n t_w}{A_{fn}} = \frac{(2)(58)(0.5625)}{(24)(2)} = 1.36$$

$$\rho = \frac{F_{yw}}{f_n} = \frac{50}{100} = 0.50$$

$$R_h = \frac{12 + \beta(3\rho - \rho^3)}{12 + 2\beta} = \frac{13.87}{14.72} = 0.942$$

TABLE 14.6 Properties of Hybrid Section of Floorbeam

Section	Steel	Yield stress F_y, ksi	Area, in^2	Distance to centroid *d*, in	Moment of inertia *I*, in^4
Web: 116 × $^9/_{16}$	A588	50	65.25	—	73,167
Flanges: 2 – 24 × 2	A514	100	96.00	59.0	334,176
			161.25		407,343

Section modulus $S = 407{,}343/60 = 6{,}789$ in^3

R_b according to Sec. 6.10.1.10.2 is as follows. Since one longitudinal stiffener is provided, determine bend-buckling coefficient, k. $d_s = 24$ in and $d_s/D_c = (24/58) = 0.41$, which is near the 0.40 optimum location.

$$k = \frac{5.17}{(d_s/D)^2} = \frac{5.17}{(24/116)^2} = 120.8$$

$$0.95\left(\frac{Ek}{F_{yc}}\right)^{0.5} = 0.95\left[\frac{(29{,}000)(120.8)}{100}\right]^{0.5} = 177.8$$

$$\frac{D}{t_w} = \frac{116}{0.5625} = 206.2 > 177.8$$

Therefore,

$$R_b = 1\left(\frac{a_{wc}}{1200 + 300a_{wc}}\right)\left(\frac{2D_c}{t_w - \lambda_{rw}}\right) \leq 1.0$$

$$\lambda_{rw} = 5.7\left(\frac{E}{F_{yc}}\right)^{0.5} = 5.7\left(\frac{29{,}000}{100}\right)^{0.5} = 97.1$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} = \frac{(2)(58)(0.5625)}{(24)(2)} = 1.36$$

$$R_b = 1 - \left\{\frac{1.36}{[1200 + 300(1.36)]}\right\}\{[(2)(58)/(0.5625)] - 97.1\} = 0.908 < 1.0$$

$$F_{nc} = R_b R_h F_{yc} = (0.908)(0.942)(100 \text{ ksi}) = 85.53 \text{ ksi}$$

$$f_{bu} = 57.73 \text{ ksi} < (\Phi_f = 1.00)F_{nc} = 85.53 \text{ ksi} \qquad \text{OK}$$

Lateral torsional buckling resistance by Sec. 6.10.8.2.3 is

$$L_b = 7.083 \text{ ft} = 85 \text{ in}$$

r_t is calculated in accordance with Eq. 6.10.8.2.3-9 as 6.26 in.

$$L_p = 1.0r_t\left(\frac{E}{F_{yc}}\right)^{1/2} = (1.0)(6.26)(17.03) = 106.6$$

$L_b < L_p$, therefore F_{nc} is computed by Eq. 6.10.8.2.3-1 as $F_{nc} = R_b R_h F_{yc}$, which is similar to the local buckling resistance calculations provided above, which are satisfactory.

Next, check the flexural resistance of the tension (bottom) flange, in accordance with Sec. 6.10.8.1.2. Similar to the top flange, for Strength I group, $f_\ell = 0$.

$$f_{bu} + {}^1\!/_3 f_\ell \leq \Phi_f F_{nt} \qquad \text{with} \qquad f_\ell = 0, f_{bu} \leq \Phi_f F_{nt}$$

$$f_{bu} = 57.73 \text{ ksi} \qquad \text{as above}$$

$$F_{nt} = R_h F_{yt} = (0.942)(100 \text{ ksi}) = 94.20 \text{ ksi}$$

$$f_{bu} = 57.73 \text{ ksi} < (\Phi_f = 1.00)F_{nt} = 94.20 \text{ ksi} \qquad \text{OK}$$

A final check shown here will be the web bend-buckling check according to Sec. 6.10.1.9. As calculated previously, the bend-buckling coefficient $k = 120.8$ for this web with one longitudinal stiffener.

$$R_h F_{yc} = (0.942)(100 \text{ ksi}) = 94.20 \text{ ksi}$$

$$\frac{F_{yw}}{0.7} = \frac{50}{0.7} = 71.43 \text{ ksi} \qquad \text{(Controls)}$$

$$F_{crw} = \frac{0.9Ek}{(D/t_w)^2} = \frac{0.9(29{,}000)(120.8)}{(206.2)^2} = 74.15 \text{ ksi} > 71.43 \text{ ksi} \qquad \text{use } 71.43 \text{ ksi}$$

$$F_{crw} = 71.43 \text{ ksi} > f_{bu} = 57.73 \text{ ksi} \qquad \text{OK}$$

Additional checks not shown here would include checks of permanent deformations at the Service II limit state and appropriate fatigue and fracture checks.

Longitudinal Stiffener for Floorbeam. As stated previously, one longitudinal stiffener, 6 × $^9/_{16}$ -in A588 steel, is provided on the floorbeam webs. A check of the web slenderness, Sec. 6.10.2.1.2 shows:

$$\frac{D}{t_w} = \frac{116}{0.5625} = 206.2 < 300 \text{ at mid-span} \qquad \text{OK}$$

$$\frac{D}{t_w} = \frac{109.3}{0.4375} = 249.8 < 300 \text{ at the ends} \qquad \text{OK}$$

The stiffener must satisfy the *b*/*t* ratio specified in Sec. 6.10.11.3.2:

$$\frac{b_\ell}{t_s} \le 0.48\left(\frac{E}{F_{ys}}\right)^{0.5} = 0.48\left(\frac{29{,}000}{50}\right)^{0.5} = 11.56$$

$$\frac{b_\ell}{t_s} = \frac{6}{0.5625} = 10.67 < 11.56 \qquad \text{OK}$$

Though not shown here, the longitudinal stiffener must meet the moment-of-inertia and radius-of-gyration requirements of Sec. 6.10.11.3.3. The spacing of transverse stiffeners at mid-span, d_0, is taken as 85 in.

From Sec. 6.10.11.3, the flexural stress in the longitudinal stiffener (computed by using the ratio of distances from the neutral axis), f_s, due to the factored loads at the strength limit state must satisfy:

$$f_s \le \Phi_f R_h F_{ys} = (1.0)(0.942)(50) = 47.10 \text{ ksi}$$

$$f_s = \left(\frac{35}{60}\right)(57.73) = 33.68 \text{ ksi} < 47.10 \text{ ksi} \qquad \text{OK}$$

Floorbeam Shear Check. At the ends of the floorbeam (at the point of maximum shear), the web is $^7/_{16}$ -in A588 steel, approximately 109.3 in deep. An unstiffened web is not adequate for the floorbeams. According to Sec. 6.10.9, for a web with a longitudinal stiffener to be considered stiffened, the transverse stiffener spacing must be ≤ 1.5*D*. The capacity of the stiffened web should be computed for both the end panel and interior panels. Capacity calculations for the end panel, where post-buckling shear resistance due to tension-field action is not present, are shown in the following. Based on the original design, it is assumed that the first transverse stiffener is spaced 22 in from the end of the floorbeam. In general:

$$V_u \le V_r = \Phi_v V_n$$

$$V_u = 1093.3 \text{ kips}$$

End panel calculations, Sec. 6.10.9.3.3, are

$$V_n = V_{cr} = CV_p$$

$$V_p = 0.58F_{yw}Dt_w = 0.58(50)(109.3)(0.4375) = 1386.7 \text{ kips}$$

$$k = 5 + \frac{5}{(d_0/D)^2} = 5 + \frac{5}{(22/109.3)^2} = 128.4$$

$$1.12\left(\frac{Ek}{F_{yw}}\right)^{0.5} = 1.12\left[\frac{(29{,}000)(128.4)}{(50)}\right]^{0.5} = 305.6$$

$$\frac{D}{t_w} = \frac{109.3}{0.4375} = 249.8 < 305.6 \qquad \text{so} \qquad C = 1.0$$

$$V_n = (1.0)(1386.7) = 1386.7 \text{ kips}$$

$$V_u = 1093.3 \text{ kips} < (\Phi_v = 1.00)V_n = 1386.7 \text{ kips} \qquad \text{OK}$$

Beyond the first two panels, which are 22 in wide, the width of transverse stiffener bays increases to 42.5 in. This case will also be checked as an interior panel. Within the controlling interior panel, the shear has decreased to approximately $V_u = 1006.5$ kips.

Interior panel calculations, Sec. 6.10.9.3.2, are

$$\frac{2Dt_w}{b_{fc}t_{fc} + b_{ft}t_{ft}} = \frac{(2)(109.3)(0.4375)}{(24)(2) + (24)(2)} = 0.996 \le 2.5$$

Therefore,

$$V_n = V_{cr} = V_p\left\{\frac{[C + [0.87(1 - C)]}{[1 + (d_0/D)^2]^{0.5}}\right\}$$

$$V_p = 0.58F_{yw}Dt_w = 0.58(50)(109.3)(0.4375) = 1386.7 \text{ kips}$$

$$k = 5 + \frac{5}{(d_0/D)^2} = 5 + \frac{5}{(42.5/109.3)^2} = 38.1$$

$$1.40\left(\frac{Ek}{F_{yw}}\right)^{0.5} = 1.40\left[\frac{(29{,}000)(38.1)}{50}\right]^{0.5} = 208.0$$

$$\frac{D}{t_w} = \frac{109.3}{0.4375} = 249.8 > 208.0 \qquad \text{so} \qquad C = 1.0$$

$$C = \left[\frac{1.57}{(D/t_w)^2}\right]\left(\frac{Ek}{F_{yw}}\right) = \left[\frac{1.57}{(109.3/0.4375)^2}\right]\left[\frac{(29{,}000)(38.1)}{50}\right] = 0.56$$

$$V_n = (1386.7)\left(\frac{0.56 + [0.87(1 - 0.56)]}{\{[1 + (42.5/109.3)^2]^{0.5}\}}\right) = 1271.3 \text{ kips}$$

$$V_u = 1006.5 \text{ kips} < (\Phi_v = 1.00)V_n = 1271.3 \text{ kips} \qquad \text{OK}$$

Floorbeam Transverse Stiffener Design. The floorbeam transverse stiffeners must be designed in accordance with Sec. 6.10.11.1. The moment of inertia of a transverse stiffener is dependent on a factor, *J*, computed below. In addition, where longitudinal stiffeners are used, the moment of inertia of the transverse stiffener must also satisfy additional criteria.

$$J = 2.5\left(\frac{D}{d_0}\right)^2 - 2.0 \geq 0.5 \qquad I_t \geq d_0 t_w^3 J \qquad I_t \geq \left(\frac{b_t}{b_\ell}\right)\left(\frac{D}{3.0d_0}\right)I_\ell$$

For the end panels, with an end-panel stiffener spacing $d_0 = 22$ in,

$$J = 2.5\left(\frac{109.3}{22}\right)^2 - 2.0 = 59.7$$

$$I_t \geq (22)(0.4375)^3(59.7) = 110.0 \text{ in}^4$$

For the first transverse stiffeners, a pair of stiffeners 7 × $^1/_2$ in is specified. These have a moment of inertia of

$$I = \frac{1}{12(0.5)(14)^3} = 114.3 \text{ in}^4 > 110.0 \text{ in}^4$$

Also,

$$I_t \geq \left(\frac{7}{6}\right)\left[\frac{109.3}{(3.0)(22)}\right](29.91) = 57.78 \text{ in}^4$$

(I_ℓ has been computed elsewhere.)

Since 114.3 > 57.78, the moment of inertia of this stiffener pair is adequate.

For other panels, with a stiffener spacing elsewhere of $d_0 = 42.5$ in,

$$J = 2.5\left(\frac{109.3}{42.5}\right)^2 - 2.0 = 14.5$$

$$I_t \geq (42.5)(0.4375)^3(14.5) = 51.6 \text{ in}^4$$

For the remaining transverse stiffeners, a pair of stiffeners 6 × $^3/_8$ in is specified. These have a moment of inertia of

$$I = \frac{1}{12(0.375)(12)^3} = 54.0 \text{ in}^4 > 51.6 \text{ in}^4 \qquad \text{OK}$$

Also,

$$I_t \geq \left(\frac{6}{6}\right)\left[\frac{109.3}{(3.0)(42.5)}\right](29.91) = 25.64 \text{ in}^4$$

Since 54.0 > 25.64, the moment of inertia of this stiffener pair is adequate.

The proportions of these stiffeners must be checked against the requirements of Sec. 6.10.11.1.2. The width of the stiffeners must satisfy $b_t \geq 2.0 + d/30$ and $16t_p \geq b_t \geq b_f/4$.

For the end panels, with 7 × $^1/_2$-in stiffeners:

$$7 \geq 2.0 + \frac{d}{30} = 2.0 + \frac{120}{30} = 6.0 \qquad \text{OK}$$

$$16t_p = 16(0.5) = 8.0 \geq 7 \qquad \text{OK}$$

$$7 \geq \frac{b_f}{4} = \frac{24}{4} = 6 \qquad \text{OK}$$

For other panels, with 6 × $^3/_8$-in stiffeners:

$$6 \geq 2.0 + \frac{d}{30} = 2.0 + \frac{120}{30} = 6.0 \qquad \text{OK}$$

$$16t_p = 16(0.375) = 6.0 \geq 6 \qquad \text{OK}$$

$$6 \geq \frac{b_f}{4} = \frac{24}{4} = 6 \qquad \text{OK}$$

Finally, the area of transverse stiffeners required to carry the forces imposed by tension-field action of the web must satisfy an area requirement. These calculations will be performed for the interior panel checked in the previous section.

$$A_s \geq \left[0.15\left(\frac{BD}{t_w}\right)(1-C)\left(\frac{V_u}{\Phi_v V_n}\right) - 18\right]\left(\frac{F_{yw}}{F_{crs}}\right)(t_w^2)$$

where

$$F_{crs} = \frac{(0.31E)}{(b_t/t_p)^2} \leq F_{ys}$$

$$F_{crs} = \frac{(0.31)(29{,}000)}{(6/0.375)^2} = 35.12 \text{ ksi} \leq F_{ys} = 36 \text{ ksi}$$

$$A_s \geq \left[0.15(1.0)\left(\frac{109.3}{0.4375}\right)(1-0.56)\left(\frac{1006.5}{1271.3}\right) - 18\right]\left(\frac{50}{35.12}\right)(0.4375)^2 = -1.34$$

The fact that this solution is negative indicates that the web alone is sufficient to resist the tension-field load, therefore since the stiffeners meet the required stiffness and *b*/*t* requirements previously computed, they are adequate.

Bearing Stiffeners for Floorbeams. Bearing stiffeners must be provided under the stringers. A pair of A36 stiffener plates will be used. The bearing stiffeners are designed as columns to carry the reaction forces of the stringers, according to the provisions of Sec. 6.10.11.2. The factored (Strength I) reaction at a typical stringer location is

$$\text{Reaction} = 1.25(52.2) + 1.50(11.1) + 1.75(71.7) = 207.4 \text{ kips}$$

Assuming 6-in-wide stiffeners, the required minimum thickness required is calculated by Sec. 6.10.11.2.2:

$$b_t \leq 0.48t_p\left(\frac{E}{F_{ys}}\right)^{0.5} \qquad \text{yields} \qquad t_p \geq \frac{b_t}{0.48(E/F_{ys})^{0.5}} = \frac{6}{[(0.48)(29{,}000/36)^{0.5}]} = 0.44 \text{ in}$$

Based on the original design, 6 × $^9/_{16}$-in plates will be used, which meet the minimum thickness for *b*/*t* requirements. The factored bearing resistance is calculated in accordance with Sec. 6.10.11.2.3 as shown below. Note that a $^1/_2$-in clip of the bearing stiffener at the web-to-flange weld is used, and must be deducted from the bearing area:

$$(R_{sb})_r = \Phi_b(R_{sb})_n = \Phi_b(1.4A_{pn}F_{ys}) = (1.00)(1.4)[2(6-0.5)(0.5625)](36) = 311.9 \text{ kips} > 207.4 \text{ kips}$$

When computing the axial resistance of the bearing stiffeners, the bearing stiffeners are assumed to act as an equivalent column consisting of the stiffeners plus a centrally located strip of web extending $9t_w$ on each side of the stiffeners, as illustrated in Fig. 14.40. The area of the equivalent column is

$$A = 2(6)(0.5625) + 2(9)(0.4375)(0.4375) = 10.20 \text{ in}^2$$

and its moment of inertia is

$$I = 1/12(0.5625)(6 + 6 + 0.4375)^3 = 90.2 \text{ in}^4$$

Thus, the radius of gyration is

$$r_s = \left(\frac{I}{A}\right)^{0.5} = \left(\frac{90.2}{10.20}\right)^{0.5} = 2.97 \text{ in}$$

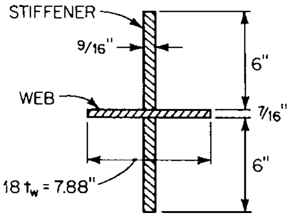


FIGURE 14.40 Bearing stiffeners for floorbeam.

and the slenderness ratio is

$$\frac{KL}{r} = \frac{(0.75)(116)}{2.97} = 29.3 < 120$$

$$\lambda = \left(\frac{KL}{r_s\pi}\right)^2\left(\frac{F_y}{E}\right) = \left(\frac{29.3}{\pi}\right)^2\left(\frac{36}{29,000}\right) = 0.108$$

Since $\lambda \leq 2.25$,

$$P_n = 0.66^{\lambda}F_yA_s = (0.66)^{0.108}(36)(10.20) = 351.1 \text{ kips}$$
$$P_r = \Phi_cP_n = (0.90)(351.1) = 316.0 \text{ kips} > 207.4 \text{ kips}$$

Therefore, two 6 × 9/16-in plates are adequate as bearing stiffeners. The fillet welds connecting the bearing stiffeners to the web should also be checked for adequacy.

Flange-to-Web Welds for Floorbeams. The flange-to-web welds must resist the horizontal shear at the flange-to-web interface. These are made the minimum size permitted for a 2-in flange, 5/16-in fillet welds. To check these welds, the properties of the floorbeam at the support given in Table 14.7 are needed. The horizontal shear flow, *s*, at the top of the web is computed as

$$s = \frac{V_uQ}{I} = \frac{(1093.3)(2675)}{(345,436)} = 8.47 \text{ kips/in}$$

This shear flow will be shared by the two welds connecting the top flange to the web, so, the horizontal shear flow per weld = 4.24 kips/in per weld. According to Sec. 6.13.3.2.4b, the factored resistance of a fillet-welded connection subject to shear on the effective area of the weld should be taken as the lesser of either the factored resistance of the weld metal or the factored resistance of the connected material in shear (Sec. 6.13.5.3).

The factored resistance of the weld metal is calculated as

$$R_r = 0.6\Phi_{e2}F_{exx} = (0.6)(0.80)(70) = 33.60 \text{ ksi}$$

for 70-ksi classification strength of weld metal.

TABLE 14.7 Properties of Floorbeam at Support

Section	Area, in^2	Distance to centroid d, in	Moment of inertia I, in^4
Web: 109 3/8 × 7/16	47.8	—	47,704
Flanges: 2 – 24 × 2	96.0	55.69	297,732
			345,436
At top of web, $Q = 24 \times 2 \times 55.69 = 2675$ in^3			

The design shear flow (kips/in) for the $^5/_{16}$-in fillet welds is then computed as

$$v = (33.60)(0.707)(0.3125)(2) = 14.85 \text{ kips/in} = 7.43 \text{ kips/in per weld}$$

The factored resistance of the connected material in shear is calculated as

$$R_r = 0.58\Phi_v F_y = (0.58)(1.00)(50) = 29.00 \text{ ksi}$$

The design shear flow (kips/in) on the connected material is then computed as

$$v = (29.00)(^9/_{16}) = 16.31 \text{ kips/in} > 14.85 \qquad \text{Does not control}$$

The controlling resistance, 7.43 kips/in per weld > 4.24 kips/in per weld, so the $^5/_{16}$-in fillet welds are adequate at the end supports.

14.11.2 Design of Arch Rib

Arch, tie, and hangers were analyzed by computer with the system assumed acting as an indeterminate plane frame. In the original design, the live load was taken as a moving load of 1.92 kips/ft, without a concentrated load or impact. At that time, a bridge of this type and span length (750 ft) was outside the range of the AASHTO Standard Specifications (which applied only up to a maximum span length of 500 ft). Therefore, the choice of live loading was subject to the judgment of the designer. Rather than reanalyze the system for the HL-93 loading, for purposes of this example, the live load from the original design will be used. Also, the original design made no distinction between *DC* and *DW* dead load. For the design of the arch rib, tie, and hangers, it is certain that the percentage of dead load that would be considered *DC* would have been in excess of 90%. For this example, conservatively, 90% of the dead load for the main arch members will be considered *DC* dead load.

The design procedure for the arch rib will be illustrated by the calculations for a rib section 54.78 ft (657 in) long at panel point U_3 (Fig. 14.18). The assumed cross section, of A514 steel, is shown in Fig. 14.41. The section properties given in Table 14.8 are needed. From the original computer analysis, the loads on the arch section are as given in Table 14.9.

The design of members for combined axial compression and flexure is governed by Sec. 6.9.2.2 of the LRFD Specifications. The first step in checking the adequacy of the rib section is to determine the nominal compressive resistance of the section according to Sec. 6.9.4.1.

$$\lambda = \left(\frac{Kl}{r_s\pi}\right)^2\left(\frac{F_y}{E}\right) = \left[\frac{(1.0)(39.4)}{\pi}\right]^2\left(\frac{100}{29{,}000}\right) = 0.867$$

Since $\lambda \le 2.25$,

$$P_n = 0.66^{\lambda} F_y A_s = (0.66)^{0.867}(100)(309.3) = 21{,}574 \text{ kips}$$

$$P_r = \Phi_c P_n = (0.90)(21{,}574) = 19{,}417 \text{ kips}$$

$$\frac{P_u}{P_r} = \frac{11{,}949}{19{,}417} = 0.62 > 0.2$$

Therefore, the axial-moment interaction equation applicable is

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$

Because $M_{uy} = 0$, the equation reduces to

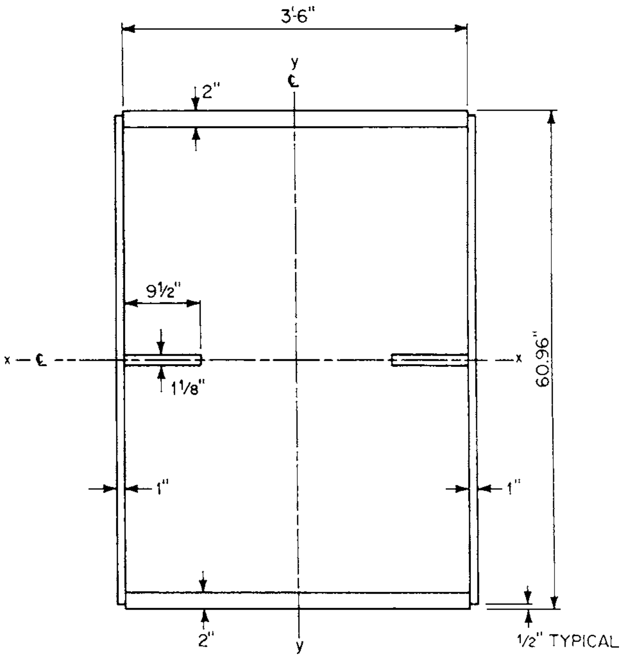


FIGURE 14.41 Arch-rib cross section.

TABLE 14.8 Properties of Arch Section

		Axis x–x			Axis y–y		
Section	Area	d_y	I_o	I_t	d_x	I_o	I_t
2—59.96 × 1	119.92	. . .	35,928	35,928	21.5	. . .	55,433
2—42 × 2	168.00	29.48	. . .	146,004	. . .	24,696	24,696
2—9½ × 1⅛	21.38				16.25	160	5,806
	309.30			181,932			85,935

$$\text{Radius of gyration } r_y = \sqrt{85{,}935/309.30} = 16.67 \text{ in}$$

$$r_x = \sqrt{181{,}932/309.30} = 24.24 \text{ in}$$

$$\text{Slenderness ratio } L/r_y = 54.78 \times 12/16.67 = 39.4$$

$$L/r_x = 54.78 \times 12/24.24 = 27.1$$

$$\text{Section modulus } S_x = 181{,}932/(60.96/2) = 5969 \text{ in}^3$$

TABLE 14.9 Loads on the Arch Rib Section

	Thrust, kips	Moment, ft·kips
Dead load (*DC*)	–7,587	330
Dead load (*DW*)	–843	37
Live load	–686	1,285
Factored (Strength I)	$P_u = -11{,}949$	$M_{ux} = 2{,}717$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \leq 1.0$$

$$M_{rx} = \Phi_f M_{nx} = (1.00)(M_{nx}) = M_{nx} \qquad \text{(to be determined by Sec. 6.12.2.2.2)}$$

$$M_{nx} = F_y S_x \left\{1 - \left[\frac{(0.064 F_y S_x \ell)}{(AE)}\right] \times \left(\frac{\Sigma(b/t)}{I_y}\right)^{0.5}\right\}$$

$$M_{nx} = (100)(5969)\left[1 - \left(\frac{0.064 \times 100 \times 5969 \times 657}{2449 \times 29{,}000}\right) \times \left(\frac{156}{85{,}935}\right)^{0.5}\right]$$

$$M_{nx} = 587{,}912 \text{ kip}\cdot\text{in} = 48{,}993 \text{ kip}\cdot\text{ft} \qquad \text{so} \qquad M_{rx} = 48{,}993 \text{ kip}\cdot\text{ft}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = 0.62 + \frac{8}{9}\left(\frac{2717}{48{,}993}\right) = 0.67 < 1.0 \qquad \text{OK}$$

It is not necessary to check the arch rib for fatigue, since it is not subject to tensile stresses.

Plate Buckling in Arch Rib. Compression plates are checked to ensure that width–thickness ratios meet LRFD Specifications requirements (Sec. 6.14.4.3 for flanges and Sec. 6.14.4.2 for webs).

For the flanges, $b/t \leq 1.06[E/(f_a + f_b)]^{0.5}$. The total stress due to axial load f_a and concurrent bending moment $f_b = P_u/A_s + M_{ux}/S_x$. Thus,

$$\text{Compressive stress} = \frac{11{,}949}{309.3} + \frac{(2717)(12)}{5969} = 38.63 + 5.46 = 44.09 \text{ ksi}$$

$$b/t \leq 1.06\left(\frac{29{,}000}{44.09}\right)^{0.5} = 27.19$$

$$\text{Flanges:} \qquad \frac{b}{t} = \frac{42}{2} = 21 < 27.19 \qquad \text{OK}$$

For the webs, $D/t_w \leq k(E/f_a)^{0.5}$. For one longitudinal stiffener, $k = 1.88$. Then

$$\text{Webs:} \qquad \frac{D}{t_w} \leq k\left(\frac{E}{f_a}\right)^{0.5} = 1.88\left(\frac{29{,}000}{38.63}\right)^{0.5} = 51.51$$

$$\text{Webs:} \qquad \frac{D}{t_w} = \frac{56.96}{1} = 56.96 > 51.51 \qquad \text{No good}$$

A new design would be adequate for this current provision if a $1^1/_8$-in-thick web was utilized, or if the total stress was decreased through the use of additional section area. Alternatively, a second longitudinal stiffener could be incorporated and the web thickness decreased.

Longitudinal Stiffener in Arch Rib. Requirements for the moment of inertia of the longitudinal stiffener (Fig. 14.42) about an axis at its base, parallel to the web, is governed by the equations of LRFD Table 6.14.4.2-1.

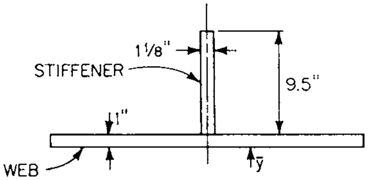


FIGURE 14.42 Stiffener on arch-rib web.

$$I_s \geq 0.75Dt_w{}^3$$

$$I_s \geq (0.75)(50.96)(1.0)^3 = 38.22 \text{ in}^4$$

The stiffener provides a moment of inertia about the face of the web of

$$I = \frac{bh^3}{3} = \frac{(1.125)(9.5)^3}{3} = 321.5 \text{ in}^4 >> 38.22 \text{ in}^4 \qquad \text{OK}$$

The width–thickness ratio of the stiffener as set forth in Sec. 6.14.4.2 is governed by

$$\frac{b}{t_s} \leq 0.408\left(\frac{E}{f_a + f_b/3}\right)^{0.5} = 0.408\left(\frac{29{,}000}{38.63 + 5.46/3}\right)^{0.5} = 10.92$$

$$\frac{b}{t_s} = \frac{9.5}{1.125} = 8.44 < 10.92 \qquad \text{OK}$$

14.11.3 Design of Tie

Design procedure for the tie will be illustrated for a section at panel point L_3 (Fig. 14.18). The assumed cross section, of A588 steel, is shown in Fig. 14.43. The tie is subject to combined axial tension and bending. In this case, the axial stress is so large that no compression occurs on the section due to bending. As for the arch rib, the live load from the original design will be used and 90% of the dead load for the tie will be considered *DC* dead load. The tension forces and bending moments in the tie section from the original computer analysis are given in Table 14.10. Properties of the tie section are given in Table 14.11.

The design of members for combined tension and flexure is governed by Sec. 6.8.2.3 of the LRFD Specifications. The first step in checking the adequacy of the tie section is to determine the factored tensile resistance of the section, P_r, according to Sec. 6.8.2.1. P_r = the lesser of the values given by either:

$$P_r = \Phi_y P_{ny} = \Phi_y F_y A_g = (0.95)(50)(443.0)$$
$$= 21{,}043 \text{ kips}$$

or

$$P_r = \Phi_u P_{nu} = \Phi_u F_u A_n U = (0.80)(65)(351.0)(1.0)$$
$$= 18{,}252 \text{ kips} \qquad \text{(Controls)}$$

Then

$$\frac{P_u}{P_r} = \frac{10{,}544}{18{,}252} = 0.58 > 0.2$$

Therefore, the applicable axial-moment interaction equation is

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

Because $M_{uy} = 0$, the equation reduces to

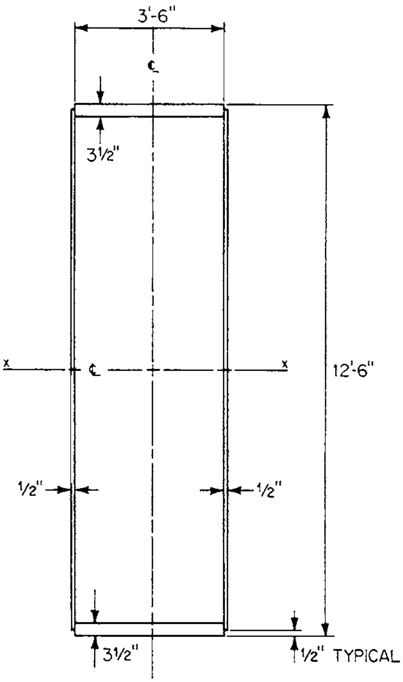


FIGURE 14.43 Tie cross section.

TABLE 14.10 Loads on the Tie Section

	Axial tension, kips	Moment, ft·kips
Dead load (*DC*)	6,926	424
Dead load (*DW*)	770	47
Live load	418	16,436
Factored (Strength I)	$P_u = 10{,}544$	$M_{ux} = 29{,}364$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \le 1.0$$

$$M_{rx} = \Phi_f M_{nx} = (1.00)(M_{nx}) = M_{nx} \qquad \text{(to be determined by Sec. 6.12.2.2.2)}$$

$$M_{nx} = F_y S_x \left\{ 1 - \left(\frac{0.064 F_y S_x \ell}{AE} \right) \times \left[\frac{\Sigma(b/t)}{I_y} \right]^{0.5} \right\}$$

$$M_{nx} = (50)(24{,}708)\left[1 - \left(\frac{0.064 \times 50 \times 24{,}708 \times 600}{6226 \times 29{,}000} \right) \times \left(\frac{620}{110{,}501} \right)^{0.5} \right]$$

$$M_{nx} = 1{,}211{,}086 \text{ kip}\cdot\text{in} = 100{,}924 \text{ kip}\cdot\text{ft} \qquad \text{so} \qquad M_{rx} = 100{,}924 \text{ kip}\cdot\text{ft}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = 0.58 + \frac{8}{9}\left(\frac{29{,}364}{100{,}924}\right) = 0.84 < 1.0 \qquad \text{OK}$$

Since the tie is a tension member, fatigue should be investigated. In the original design, under previous specifications the tie was checked as a nonredundant load-path structure and found to be acceptable. For future designs, it may be prudent to utilize high-performance steel in the tie girders to take advantage of the high fracture toughness of this material. It is anticipated that tie girders composed of such material may not be considered fracture-critical in the near future.

14.11.4 Design of Hangers

All hangers consist of four $2^7/_8$-in-diameter bridge ropes (breaking strength 758 kips per rope, 3032 kips for all four). From the original computer analysis of the arch-tie system, the most highly stressed

TABLE 14.11 Properties of Tie Section

		Axis *x–x*				Axis *y–y*			
Section	Area	d_y	Ad_y^2	I_0	I_t	d_x	Ad_x^2	I_0	I_t
PL2 – 149 × $^1/_2$	149.0	—	—	275,662	275,662	21.25	67,283	—	67,283
PL2 – 42 × $3^1/_2$	294.0	73.25	1,577,475	—	1,577,475	—	—	43,218	43,218
	443.0 in^2			$I_x = 1{,}853{,}137$ in^4					$I_y = 110{,}501$ in^4

Section modulus $S_x = 1{,}853{,}137/75 = 24{,}708$ in^3
Section modulus $S_y = 110{,}501/21.5 = 5{,}140$ in^3
Net section at tie splice = [149 – 2(29)(1)(0.5)] + [294 – 2(9)(1)(3.5)] = 351 in^2

hanger is L_4U_4 (Fig. 14.18). Together, these four ropes carry a 630-kip dead load (assumed 90% *DC* and 10% *DW*) and 99.5-kip live load, for a total of

Working load = 729.5 kips
Factored load (Service II) = 759.4 kips
Factored load (Strength I) = 977.4 kips

There is no explicit procedure for the design of such hangers in the LRFD Specifications. In the past, and on the original design of this structure, a factor of safety of 4 on the breaking strength would be compared against the working load. Because 758 kips > 729.5 kips, the design was considered acceptable. If a similar factor of safety were used with the Service II combination, the acceptability of the hangers would be borderline (758 kips compared to 759.4 kips).

The factored load is approximately 32% of the breaking strength of the ropes (a factor of safety of approximately 3), which is considered acceptable.

The live-load stress range for the hangers is small and considered acceptable. If a lower safety factor is used, or a larger live-load stress range is present, a more detailed fatigue investigation should be made. Also, provisions must be made to eliminate possible aerodynamic vibrations of the hangers, and details must be adequate for corrosion protection.

14.11.5 Bottom Lateral Bracing

The plan of the bracing used in the plane of the tie is shown in Fig. 14.18. Figure 14.44 shows the section used for the diagonal in the panel between L_0 and L_1. The steel is A36.

Because of lateral wind on the structure, the axial load on the 73-ft-long diagonal has been computed to be 295 kips [$P_u = (1.4)(295) = 413$ kips for the Strength III group]. The member also is subject to bending due to its own weight. The section properties given in Table 14.12 are needed.

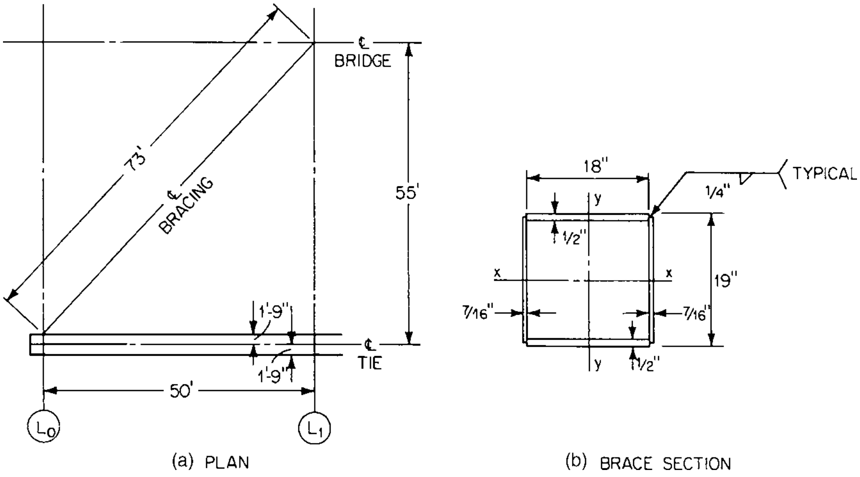
The weight of the member is

$$W = (34.18)(0.0034) = 0.12 \text{ kip/ft}$$

This produces a maximum dead-load bending moment (*DC*) at mid-span of

$$M = \frac{wL^2}{8} = \frac{(0.12)(73)^2}{8} = 79.9 \text{ ft}\cdot\text{kips}$$

$$M_{ux} = 1.25(79.9) = 99.9 \text{ ft}\cdot\text{kips}$$



(a) PLAN (b) BRACE SECTION

FIGURE 14.44 Diagonal brace in the plane of the ties.

TABLE 14.12 Properties of Diagonal in Bottom Lateral Bracing

		Axis x–x				Axis y–y			
Section	Area A	d_x	I_o	I_x	Ad_x^2	d_y	I_o	Ad_y^2	I_y
PL 2—18½ × 7/16	16.18	…	462	…	462	9.22	…	1375	1375
PL 2—18 × ½	18.00	9.25	…	1540	1540	…	486	…	486
	34.18				2002				1861

Distance c to c connections = 73 − 2 = 71 ft

Radius of gyration $r_x = \sqrt{2002/34.18} = 7.65$ in

$r_y = \sqrt{1861/34.18} = 7.38$ in

Effective length factor $K = 0.75$ (truss-type member connections)

Slenderness ratio $KL/r_y = 0.75(71 \times 12)/7.38 = 86.6 < 140$

Slenderness ratio $KL/r_x = 0.75(71 \times 12)/7.65 = 83.5 < 140$

Section modulus $S_x = 2002/9.5 = 211$ in^3

The design of members for combined axial compression and flexure is governed by Sec. 6.9.2.2 of the LRFD Specifications. The first step in checking the adequacy of the rib section is to determine the nominal compressive resistance of the section by Sec. 6.9.4.1.

$$\lambda = \left(\frac{Kl}{r_s\pi}\right)^2\left(\frac{F_y}{E}\right) = \left(\frac{86.6}{\pi}\right)^2\left(\frac{36}{29{,}000}\right) = 0.943$$

Since $\lambda \le 2.25$,

$$P_n = 0.66^{\lambda}F_yA_s = (0.66)^{0.943}(36)(34.18) = 831.6 \text{ kips}$$

$$P_r = \Phi_c P_n = (0.90)(831.6) = 748.4 \text{ kips}$$

$$\frac{P_u}{P_r} = \frac{413}{748.4} = 0.55 > 0.2$$

Therefore, the applicable axial-moment interaction equation is

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$

Because $M_{uy} = 0$, the equation reduces to

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \le 1.0$$

$$M_{rx} = \Phi_f M_{nx} = (1.00)(M_{nx}) = M_{nx} \qquad \text{(to be determined by Sec. 6.12.2.2.2)}$$

$$M_{nx} = F_yS_x\left[1 - \left(\frac{0.064F_yS_x\ell}{AE}\right) \times \left(\frac{\Sigma(b/t)}{I_y}\right)^{0.5}\right]$$

$$M_{nx} = (36)(211)\left[1 - \left(\frac{0.064 \times 36 \times 211 \times 876}{341 \times 29{,}000}\right) \times \left(\frac{159}{1861}\right)^{0.5}\right]$$

$$M_{nx} = 7500 \text{ kip}\cdot\text{in} = 625 \text{ kip}\cdot\text{ft} \qquad \text{so} \qquad M_{rx} = 625 \text{ kip}\cdot\text{ft}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = 0.55 + \frac{8}{9}\left(\frac{99.9}{625}\right) = 0.69 < 1.0 \qquad \text{OK}$$

Plate Buckling in Lateral Brace. Compression plates are checked to ensure that width–thickness ratios, *b*/*t*, meet LRFD Specifications requirements (Sec. 6.9.4.2). The general requirement is

$$\frac{b}{t} \le k\left(\frac{E}{F_y}\right)^{0.5}$$

where k is the plate-buckling coefficient. However, for members designed using the equations of Sec. 6.9.2.2, F_y may be replaced with the maximum calculated compressive stress due to the factored axial load and concurrent bending moment. For this section, assume this stress $= P_u/A_s + M_{ux}/S_x = 413/34.18 + (99.9)(12)/211 = 17.76$ ksi. Thus, with $k = 1.40$ (LRFD Table 6.9.4.2-1), the requirement becomes

$$\frac{b}{t} \le 1.40\left(\frac{29{,}000}{17.76}\right)^{0.5} = 56.57$$

$$\text{Flanges:} \quad \frac{b}{t} = \frac{18}{0.5} = 36.0 < 56.57 \qquad \text{OK}$$

$$\text{Webs:} \quad \frac{b}{t} = \frac{18.5}{0.4375} = 42.3 < 56.57 \qquad \text{OK}$$

The brace section is satisfactory.

14.11.6 Rib Bracing

The plan of the A36 steel bracing used for the arch rib is shown in Fig. 14.18. Figure 14.45 shows the section used for a brace in the first panel of bracing.

Rib bracing is designed to carry its own weight, wind on ribs and rib bracing, and an assumed buckling shear from compression of the ribs. Loads on the first-panel brace from the original analysis are given in Table 14.13, and section properties are computed in Table 14.14. The unsupported length of the rib brace is 58.7 ft (704 in).

The design of members for combined axial compression and flexure is governed by Sec. 6.9.2.2 of the LRFD Specifications. The first step in checking the adequacy of the rib section is to determine the nominal compressive resistance of the section according to Sec. 6.9.4.1.

$$\lambda = \left(\frac{Kl}{r_s\pi}\right)^2\left(\frac{F_y}{E}\right) = \left(\frac{56.6}{\pi}\right)^2\left(\frac{36}{29{,}000}\right) = 0.403$$

Since $\lambda \le 2.25$,

$$P_n = 0.66^{\lambda}F_yA_s = (0.66)^{0.403}(36)(92.5) = 2817 \text{ kips}$$

$$P_r = \Phi_c P_n = (0.90)(2817) = 2535 \text{ kips}$$

$$\frac{P_u}{P_r} = \frac{415}{2535} = 0.16 < 0.2$$

Therefore, the applicable axial-moment interaction equation is

$$\frac{P_u}{(2.0P_r)} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$

$$M_{rx} = \Phi_f M_{nx} = (1.00)(M_{nx}) = M_{nx}$$

$$M_{ry} = \Phi_f M_{ny} = (1.00)(M_{ny}) = M_{ny} \qquad \text{(Sec. 6.12.2.2.2)}$$

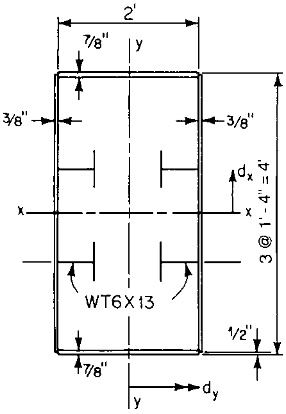


FIGURE 14.45 Section for brace between arch ribs.

TABLE 14.13 Loads on Brace between Arch Ribs

	Axial, kips	M_x, ft·kips	M_y, ft·kips
Dead load (*DC*)	—	1120	67.5
Wind	58.7	—	67.0
Buckling (use *LF* = 1.25)	266.0	—	—
Factored (Strength III)	$P_u = 415$	$M_{ux} = 1400$	$M_{uy} = 178$

$$M_{nx} = F_y S_x \left[1 - \left(\frac{0.064 F_y S_x \ell}{AE} \right) \times \left(\frac{\Sigma(b/t)}{I_y} \right)^{0.5} \right]$$

$$M_{nx} = (36)(1284)\left[1 - \left(\frac{0.064 \times 36 \times 1284 \times 704}{1149 \times 29{,}000} \right) \times \left(\frac{306}{8077} \right)^{0.5} \right]$$

$$M_{nx} = 45{,}662 \text{ kip}\cdot\text{in} = 3805 \text{ kip}\cdot\text{ft} \qquad \text{so} \qquad M_{rx} = 3805 \text{ kip}\cdot\text{ft}$$

$$M_{ny} = F_y S_y \left[1 - \left(\frac{0.064 F_y S_y \ell}{AE} \right) \times \left(\frac{\Sigma(b/t)}{I_x} \right)^{0.5} \right]$$

$$M_{ny} = (36)(652)\left[1 - \left(\frac{0.064 \times 36 \times 652 \times 657}{2449 \times 29{,}000} \right) \times \left(\frac{306}{30{,}804} \right)^{0.5} \right]$$

$$M_{nx} = 23{,}439 \text{ kip}\cdot\text{in} = 1953 \text{ kip}\cdot\text{ft} \qquad \text{so} \qquad M_{rx} = 1953 \text{ kip}\cdot\text{ft}$$

$$\frac{P_u}{2.0P_r} = \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} = \frac{0.16}{2.0} + \frac{1400}{3805} + \frac{178}{1953} = 0.54 < 1.0 \qquad \text{OK}$$

Plate Buckling in Brace. Compression plates are checked to ensure that width–thickness ratios meet LRFD Specifications requirements (Sec. 6.9.4.2). The general requirement is $b/t \le k(E/F_y)^{0.5}$.

TABLE 14.14 Properties of Rib Brace

		Axis *x–x*				Axis *y–y*			
Section	A	d_x	I_o	Ad_x^2	I_x	d_y	I_o	Ad_y^2	I_y
PL 2—47 × 3/8	35.2	…	6,490	…	6,490	12.19	…	5,230	5,230
PL 2—24 × 7/8	42.0	23.56	…	23,300	23,300	…	2,020	…	2,020
4—WT 6 × 13	15.3	8.00	35	979	1,014	7.14	47	780	827
	92.5				30,804				8,077

Radius of gyration $r_x = \sqrt{30{,}804/92.5} = 18.2$ in

$r_y = \sqrt{8{,}077/92.5} = 9.34$ in

Unsupported length = 58.7 ft

Effective length factor $K = 0.75$ (truss-type member connections)

Slenderness ratio $KL/r_x = 0.75 \times 58.7 \times 12/18.2 = 29.0$

Slenderness ratio $KL/r_y = 0.75 \times 58.7 \times 12/9.34 = 56.6$

Section modulus $S_x = 30{,}804/24 = 1{,}284$ in^3

$S_y = 8{,}077/12.38 = 652$ in^3

However, as explained previously, for members designed using the equations of Sec. 6.9.2.2, F_y may be replaced with the maximum calculated compressive stress due to the factored axial load and concurrent bending moments. For this section, assume this stress $= P_u/A_s + M_{ux}/S_x + M_{uy}/S_y = 415/92.5 + (1400)(12)/1284 + (178)(12)/652 = 20.85$ ksi. Thus, with $k = 1.40$ (LRFD Table 6.9.4.2-1), the requirement becomes

$$\frac{b}{t} \leq 1.40\left(\frac{29{,}000}{20.85}\right)^{0.5} = 52.21$$

$$\text{Flanges:} \qquad \frac{b}{t} = \frac{24}{0.875} = 27.4 < 52.21 \qquad \text{OK}$$

$$\text{Webs:} \qquad \frac{b}{t} = \frac{47}{0.375} = 125.3 > 52.21 \qquad \text{No good}$$

WT longitudinal stiffeners will be attached to the webs (Fig. 14.45), and it will be assumed that a node will occur at the location of these *WT*s. Thus, the *b*/*t* ratio will be rechecked based on a clear distance between the *WT* stiffener and flange.

$$\text{Web with stiffeners:} \qquad \frac{b}{t} = \frac{16}{0.375} = 42.67 < 52.21 \qquad \text{OK}$$

CHAPTER 15

CABLE-SUSPENDED BRIDGES

Walter Podolny, Jr., Ph.D., P.E.
Former Senior Structural Engineer, Office of Bridge Technology
Federal Highway Administration, U.S. Department of Transportation
Washington, D.C.

David Goodyear, P.E., S.E.
Senior Vice President
T. Y. Lin International
Olympia, Washington

Few structures are as universally appealing as cable-supported bridges. The origin of the concept of bridging large spans with cables, exerting their strength in tension, is lost in antiquity and undoubtedly dates back to a time before recorded history. Perhaps primitive humans, wanting to cross natural obstructions such as deep gorges and large streams, observed a spider spinning a web or monkeys traveling along hanging vines.

15.1 EVOLUTION OF CABLE-SUSPENDED BRIDGES

Early cable-suspended bridges were footbridges consisting of cables formed from twisted vines or hide drawn tightly to reduce sag. The cable ends were attached to trees or other permanent objects located on the banks of rivers or at the edges of gorges or other natural obstructions to travel. The deck, probably of rough-hewn plank, was laid directly on the cable. This type of construction was used in remote ages in China, Japan, India, and Tibet. It was used by the Aztecs of Mexico, the Incas of Peru, and by natives in other parts of South America. It can still be found in remote areas of the world.

From the sixteenth to nineteenth centuries, military engineers made effective use of rope suspension bridges. In 1734, the Saxon army built an iron-chain bridge over the Oder River at Glorywitz, reportedly the first use in Europe of a bridge with a metal suspension system. However, iron chains were used much earlier in China. The first metal suspension bridge in North America was the Jacob's Creek Bridge in Pennsylvania, designed and erected by James Finley in 1801. Supported by two suspended chains of wrought-iron links, its 70-ft span was stiffened by substantial trussed railing and timber planks.

Chains and flat wrought-iron bars dominated suspension-bridge construction for some time after that. Construction of this type was used by Thomas Telford in 1826 for the noted Menai Straits

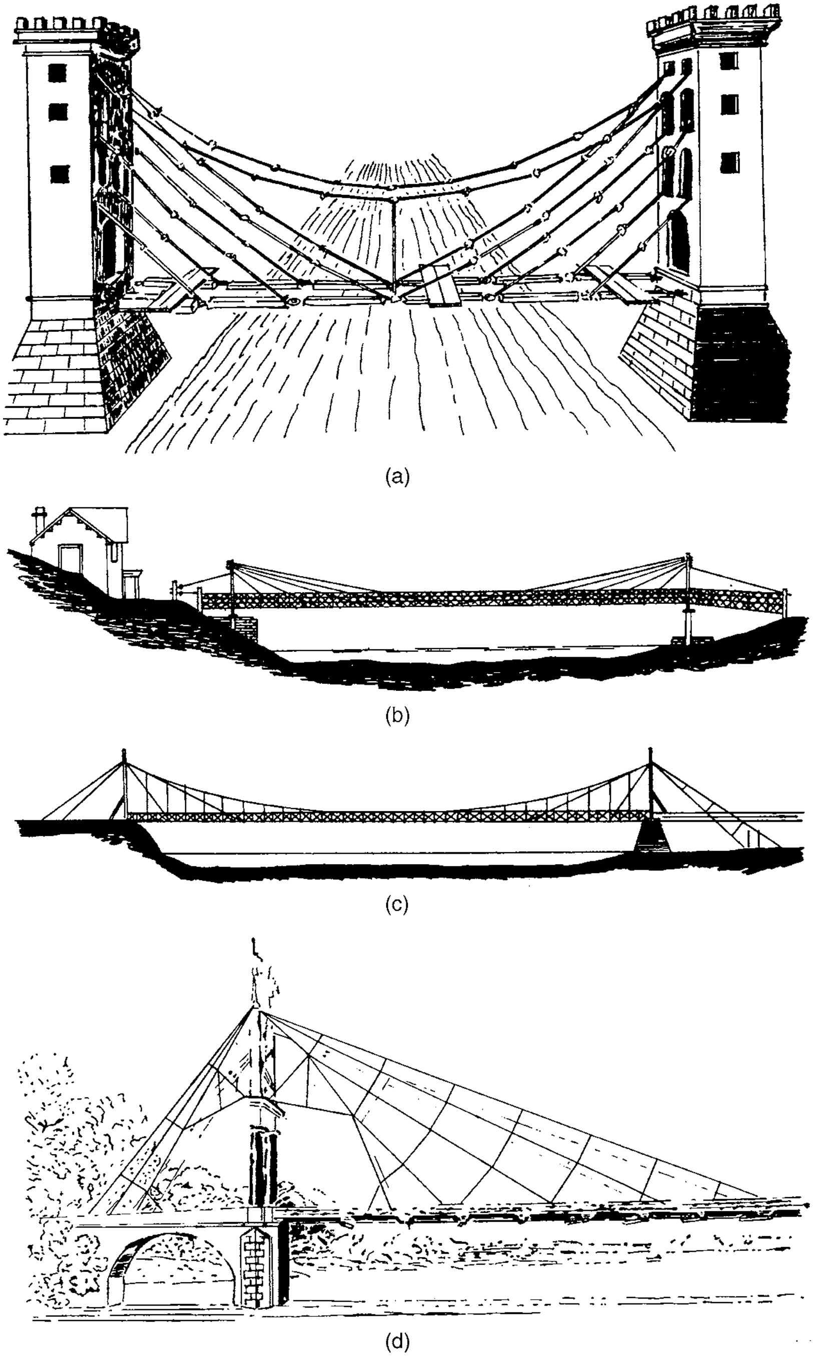


FIGURE 15.1 (*a*) Chain bridge by Faustus Verantius, 1607. (*b*) King's Meadow Footbridge. (*c*) Dryburgh Abbey Bridge. (*d*) Nienburg Bridge. (*Reprinted with permission from K. Roik et al., "Schrägseilbrüchen," Wilhelm Ernst & Sohn, Berlin.*)

Bridge, with a main span of 580 ft. But 10 years before, in 1816, the first wire suspension bridges were built, one at Galashiels, Scotland, and a second over the Schuylkill River in Philadelphia.

A major milestone in progress with wire cable was passed with erection of the 1010-ft suspended span of the Ohio River Bridge, at Wheeling, Virginia (later West Virginia), by Charles Ellet, Jr., in 1849. A second important milestone was the opening in 1883 of the 1595.5-ft wire-cable-supported span of the Brooklyn Bridge, built by the Roeblings.

In 1607, a Venetian engineer named Faustus Verantius published a description of a suspended bridge partly supported with several diagonal chain stays (Fig. 15.1*a*). The stays in that case were used in combination with a main supporting suspension (catenary) cable. The first use of a pure stayed bridge is credited to Löscher, who built a timber-stayed bridge in 1784 with a span of 105 ft (Fig. 15.2*a*). The pure stayed-bridge concept was apparently not used again until 1817, when two British engineers, Redpath and Brown, constructed the King's Meadow Footbridge (Fig. 15.1*b*), with a span of about 110 ft. This structure utilized sloping wire cable stays attached to cast-iron towers. In 1821, the French architect Poyet suggested a pure cable-stayed bridge (Fig. 15.2*b*) using bar stays suspended from high towers.

The pure cable-stayed bridge might have become a conventional form of bridge construction had it not been for an unfortunate series of circumstances. In 1818, a composite suspension and stayed pedestrian bridge crossing the Tweed River near Dryburgh-Abbey, England (Fig. 15.1*c*), collapsed as a result of wind action. In 1824, a cable-stayed bridge crossing the Saale River near Nienburg, Germany (Fig. 15.1*d*), collapsed, presumably from overloading. The famous French engineer C. L. M. H. Navier published in 1823 a prestigious work wherein his adverse comments on the failures of several cable-stayed bridges virtually condemned the use of cable stays to obscurity.

Despite Navier's adverse criticism of stayed bridges, a few more were built shortly after the fatal collapses of the bridges in England and Germany, for example, the Gischlard–Arnodin cable bridge (Fig. 15.2*c*), with multiple sloping cables hung from two masonry towers. In 1840, Hatley, an Englishman, used chain stays in a parallel configuration resembling harp strings (Fig. 15.2*d*). He maintained the parallel spacing of the main stays by using a closely spaced subsystem anchored to the deck and perpendicular to the principal load-carrying cables.

The principle of using stays to support a bridge superstructure did not die completely in the minds of engineers. John Roebling incorporated the concept in his suspension bridges, such as his Niagara Falls Bridge (Fig. 15.3); the Old St. Clair Bridge in Pittsburgh (Fig. 15.4); the Cincinnati Bridge across the Ohio River, and the Brooklyn Bridge in New York. The stays were used in addition to vertical suspenders to support the bridge superstructure. Observations of performance indicated that the stays and suspenders were not efficient partners. Consequently, although the stays were comforting safety measures in the early bridges, in the later development of conventional catenary suspension bridges the stays were omitted. The conventional suspension bridge was dominant until the latter half of the twentieth century.

The virtual banishment of stayed bridges during the nineteenth and early twentieth centuries can be attributed to the lack of sound theoretical analyses for determination of the internal forces of the total system. The failure to understand the behavior of the stayed system and the lack of methods for controlling the equilibrium and compatibility of the various highly indeterminate structural components appear to have been the major drawback to further development of the concept. Furthermore, the materials of the period were not suitable for stayed bridges.

Rebirth of stayed bridges appears to have begun in 1938 with the work of the German engineer Franz Dischinger. While designing a suspension bridge to cross the Elbe River near Hamburg (Fig. 15.5), Dischinger determined that the vertical deflection of the bridge under railroad loading could be reduced considerably by incorporating cable stays in the suspension system. From these studies and his later design of the Strömsund Bridge in Sweden (1955) evolved the modern cable-stayed bridge. However, the biggest impetus for cable-stayed bridges came in Germany after World War II with the design and construction of bridges to replace those that had been destroyed in the conflict.

(W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York; R. Walther et al., "Cable-Stayed Bridges," Thomas Telford, London; D. P. Billington and A. Nazmy, "History and Aesthetics of Cable-Stayed Bridges," *Journal of Structural Engineering*, vol. 117, no. 10, October 1990, American Society of Civil Engineers.)

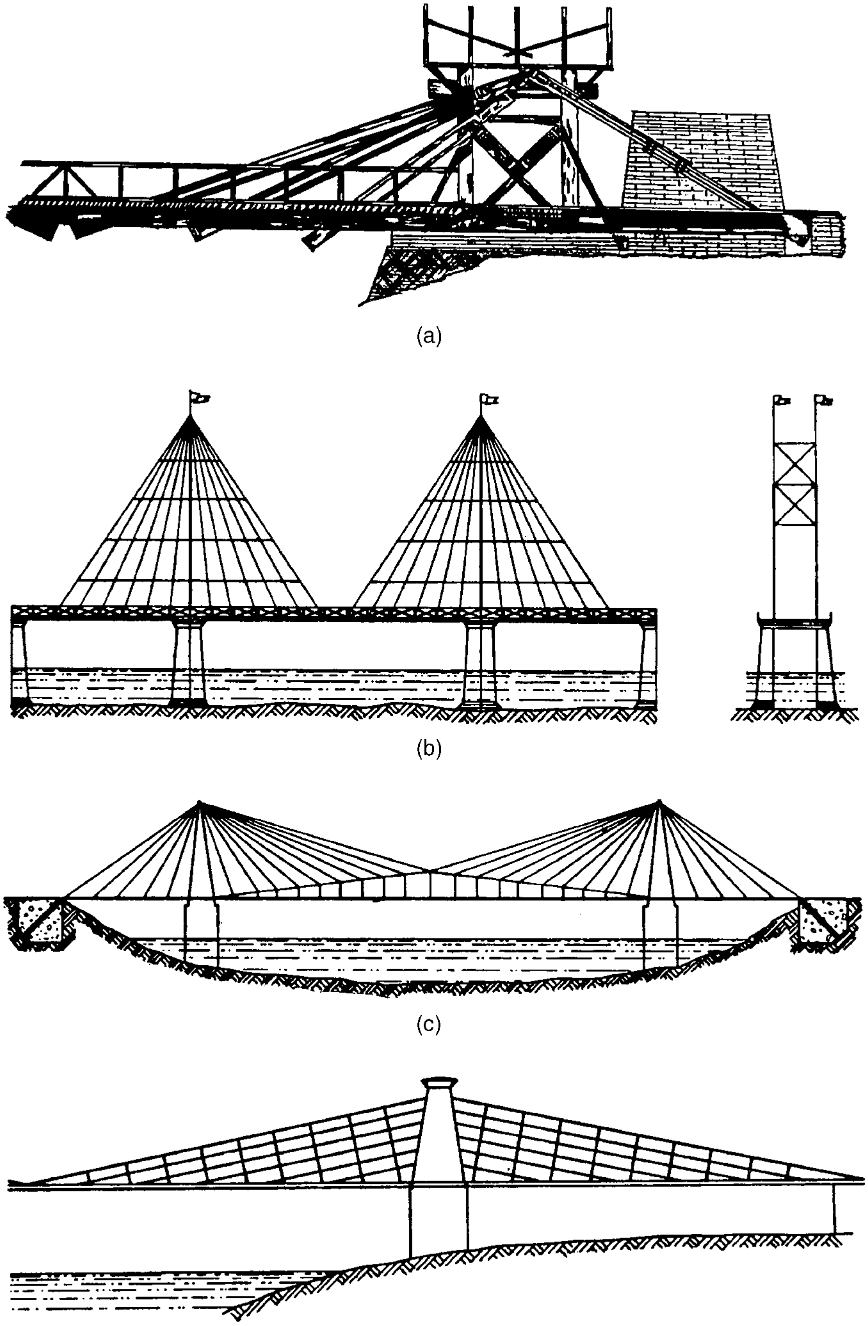


FIGURE 15.2 (*a*) Löscher-type timber bridge. (*b*) Poyet-type bridge. (*c*) Gischlard–Arnodin-type sloping-cable bridge. (*d*) Hatley chain bridge, (*Reprinted with permission from H. Thul, "Cable-Stayed Bridges in Germany," Proceedings of the Conference on Structural Steelwork, 1966, The British Constructional Steelwork Association, Ltd., London.*)

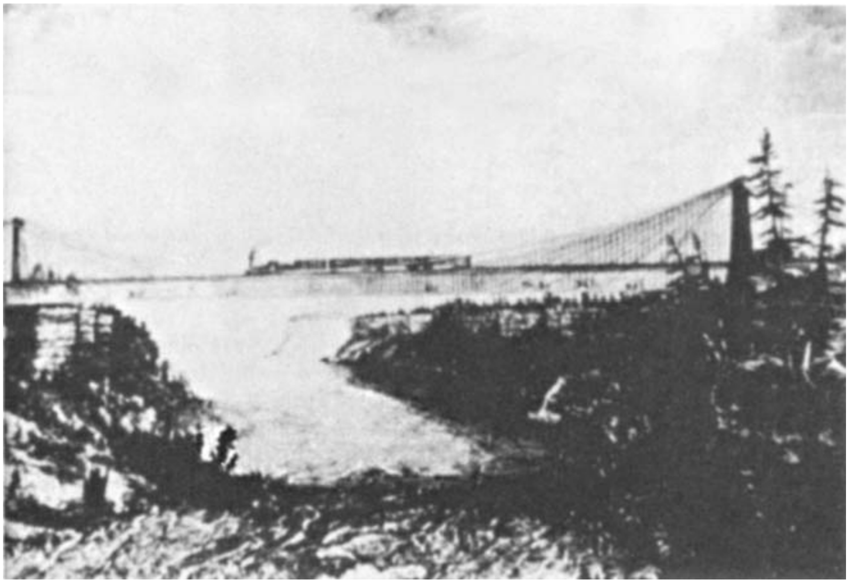


FIGURE 15.3 Niagara Falls Bridge.

FIGURE 15.4 Old St. Clair Bridge, Pittsburgh.

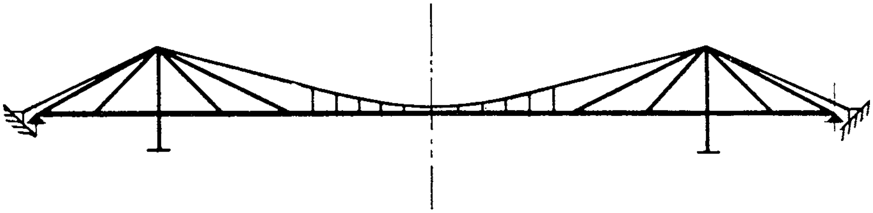


FIGURE 15.5 Bridge system proposed by Dischinger. (*Reprinted with permission from F. Dischinger, "Hangebrüchen for Schwerste Verkehrslasten," Der Bauingenieur, Heft 3 and 4, 1949.*)

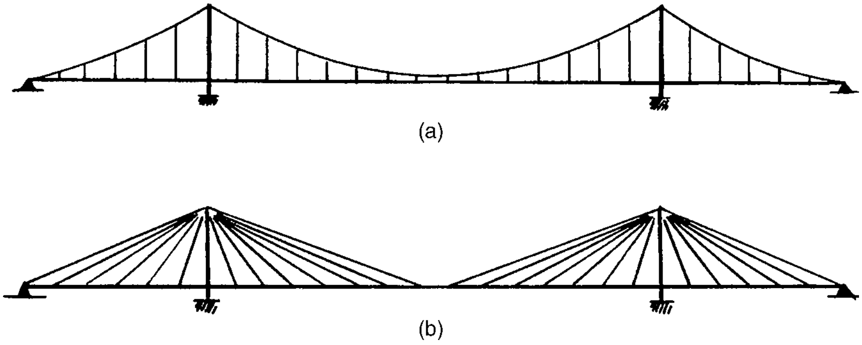


FIGURE 15.6 Cable-suspended bridge systems. (*a*) Suspension. (*b*) Cable-stayed. (*Reprinted with permission from W. Podolny, Jr. and J. B. Scalzi,* Construction and Design of Cable-Stayed Bridges, *2d ed., John Wiley & Sons, Inc., New York.*)

15.2 CLASSIFICATION OF CABLE-SUSPENDED BRIDGES

Cable-suspended bridges that rely on very-high-strength steel cables as major structural elements may be classified as suspension bridges or cable-stayed bridges. The fundamental difference between these two classes is the manner in which the bridge deck is supported by the cables. In suspension bridges, the deck is supported at relatively short intervals by vertical suspenders, which, in turn, are supported from a main cable (Fig. 15.6*a*). The main cables are relatively flexible and thus take a profile shape that is a function of the magnitude and position of loading. Inclined cables of the cable-stayed bridge (Fig. 15.6*b*) support the bridge deck directly with relatively taut cables, which, compared to the classical suspension bridge, provide relatively inflexible supports at several points along the span. The nearly linear geometry of the cables produces a bridge with greater stiffness than the corresponding suspension bridge.

Cable-suspended bridges are generally characterized by economy, lightness, and clarity of structural action. These types of structures illustrate the concept of form following function and present a graceful and esthetically pleasing appearance. Each of these types of cable-suspended bridges may be further subclassified; those subclassifications are presented in articles that follow.

Many early cable-suspended bridges were a combination of the suspension and cable-stayed systems (Art. 15.1). Such combinations can offer even greater resistance to dynamic loadings and may be more efficient for very long spans than either type alone. Note Steinman's design for the Salazar Bridge across the Tagus River in Portugal. The original structure, a conventional suspension

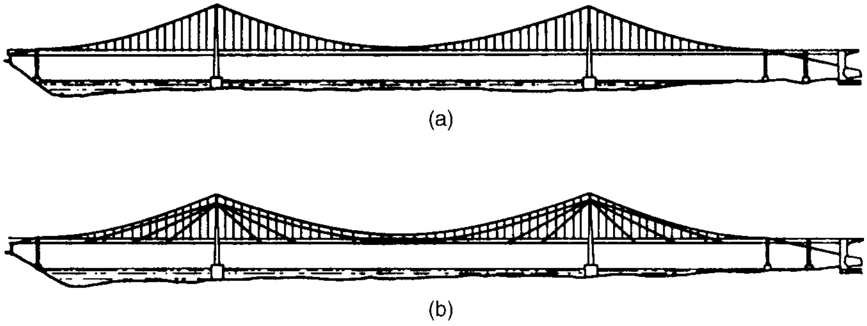


FIGURE 15.7 The Salazar Bridge. (*a*) Elevation of the bridge in 1993. (*b*) Elevation of future bridge. (*Reprinted with permission from W. Podolny, Jr., and J. B. Scalzi,* Construction and Design of Cable-Stayed Bridges, *John Wiley & Sons, Inc., New York.*)

bridge, is indicated in Fig. 15.7*a.* However, the bridge was retrofitted by adding a second set of suspension cables instead. Cable stays were to be installed to accommodate additional rail traffic (Fig. 15.7*b*).

(W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York.)

15.3 CLASSIFICATION AND CHARACTERISTICS OF SUSPENSION BRIDGES

Suspension bridges with cables made of high-strength, zinc-coated steel wires are suitable for the longest of spans. Such bridges usually become economical for spans in excess of 1000 ft, depending on specific site constraints. Nevertheless, many suspension bridges with spans as short as 300 or 400 ft have been built, to take advantage of their esthetic features.

The basic economic characteristic of suspension bridges, resulting from use of high-strength materials in tension, is lightness, due to relatively low dead load. But this, in turn, carries with it the structural penalty of flexibility, which can lead to large deflections under live load and susceptibility to vibrations. As a result, suspension bridges are more suitable for highway bridges than for the more heavily loaded railroad bridges. Nevertheless, for either highway or railroad bridges, care must be taken in design to provide resistance to wind- or seismic-induced oscillations, such as those that caused collapse of the first Tacoma Narrows Bridge in 1940.

15.3.1 Main Components of Suspension Bridges

A pure suspension bridge is one without supplementary stay cables and in which the main cables are anchored externally to anchorages on the ground. The main components of a suspension bridge are illustrated in Fig. 15.8. Most suspension bridges are stiffened; that is, as shown in Fig. 15.8, they utilize horizontal stiffening trusses or girders. Their function is to equalize deflections due to concentrated live loads and distribute these loads to one or more main cables. The stiffer these trusses or girders are, relative to the stiffness of the cables, the better this function is achieved. (Cables derive stiffness not only from their cross-sectional dimensions but also from their shape between supports, which depends on both cable tension and loading.)

For heavy, very long suspension spans, live-load deflections may be small enough that stiffening trusses would not be needed. When such members are omitted, the structure is an unstiffened suspension bridge. Thus, if the ratio of live load to dead load were, say, 1:4, the midspan deflection would be of the order of $^1/_{100}$ the sag, or 1/1000 of the span, and the use of stiffening trusses would ordinarily be unnecessary. (For the George Washington Bridge as initially constructed, the ratio of live load to dead load was approximately 1:6. Therefore, it did not include a stiffening truss.) However, stiffening trusses are often needed for torsional stiffness to separate the first vertical and first torsional frequencies, which is helpful for aerodynamic stability.

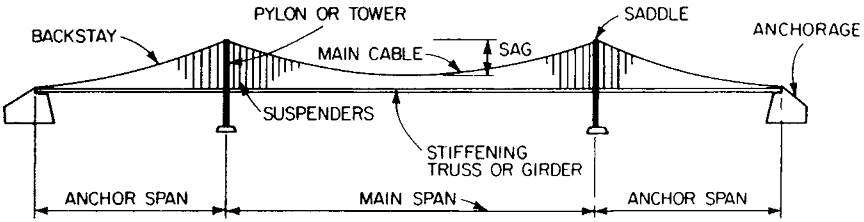


FIGURE 15.8 Main components of a suspension bridge.

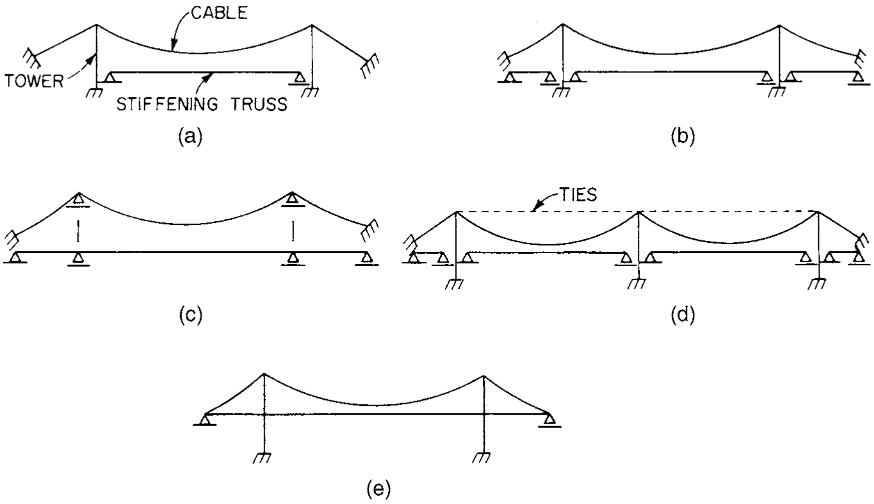


FIGURE 15.9 Suspension-bridge arrangements. (*a*) One suspended span, with pin-ended stiffening truss. (*b*) Three suspended spans, with pin-ended stiffening trusses. (*c*) Three suspended spans, with continuous stiffening truss. (*d*) Multispan bridge, with pin-ended stiffening trusses. (*e*) Self-anchored suspension bridge.

15.3.2 Types of Suspension Bridges

Several arrangements of suspension bridges are illustrated in Fig. 15.9. The main cable is continuous, over saddles at the pylons, or towers, from anchorage to anchorage. When the main cable in the side spans does not support the bridge deck (side spans independently supported by piers), that portion of the cable from the saddle to the anchorage is virtually straight and is referred to as a **straight backstay.** This is also true in the case shown in Fig. 15.9*a*, where there are no side spans.

Figure 15.9*d* represents a multispan bridge. This type is not considered efficient, because its flexibility distributes an undesirable portion of the load onto the stiffening trusses and may make horizontal ties necessary at the tops of the pylons. Ties were used on several French multispan suspension bridges of the nineteenth century. However, it is doubtful whether tied towers would be esthetically acceptable to the general public. Another approach to multispan suspension bridges is that used for the San Francisco–Oakland Bay Bridge (Fig. 15.10). It is essentially composed of two three-span suspension bridges placed end to end. This system has the disadvantage of requiring three piers in the central portion of the structure where water depths are likely to be a maximum.

Suspension bridges may also be classified by type of cable anchorage, external or internal. Most suspension bridges are **externally anchored** (earth-anchored) to a massive external anchorage (Fig. 15.9*a* to *d*). In some bridges, however, the ends of the main cables of a suspension bridge are attached to the stiffening trusses, as a result of which the structure becomes **self-anchored** (Fig. 15.9*e*). It does not require external anchorages.

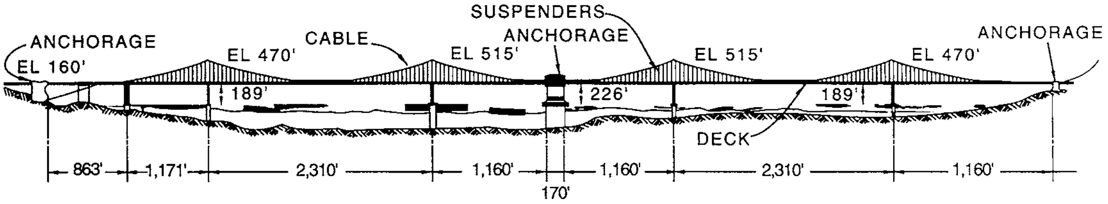


FIGURE 15.10 San Francisco–Oakland Bay Bridge.

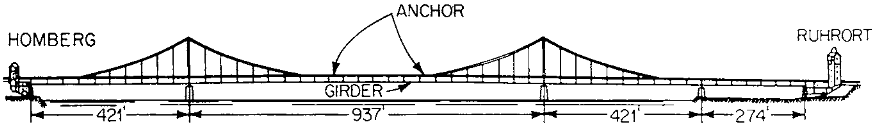


FIGURE 15.11 Bridge over the Rhine at Ruhrort–Homberg, Germany, a bridle-chord type.

The stiffening trusses of a self-anchored bridge must be designed to take the compression induced by the cables. The cables are attached to the stiffening trusses over a support that resists the vertical component of cable tension. The vertical upward component may relieve or even exceed the dead-load reaction at the end support. If a net uplift occurs, a pendulum-link tie-down should be provided at the end support.

Self-anchored suspension bridges are suitable for short to moderate spans (400 to 1000 ft) where foundation conditions do not permit external anchorages. Such conditions include poor foundation-bearing strata and loss of weight due to anchorage submergence. Typical examples of self-anchored suspension bridges are the Paseo Bridge at Kansas City, with a main span of 616 ft, and the former Cologne–Mülheim Bridge (1929) with a 1033-ft span.

Another type of suspension bridge is referred to as a **bridle-chord bridge.** Called by Germans *Zügelgurtbrücke*, these structures are typified by the bridge over the Rhine River at Ruhrort–Homberg (Fig. 15.11), erected in 1953, and the one at Krefeld–Urdingen, erected in 1950. It is a special class of bridge, intermediate between the suspension and cable-stayed types and having some of the characteristics of both. The main cables are curved but not continuous between towers. Each cable extends from the tower to a span, as in a cable-stayed bridge. The span, however, also is suspended from the cables at relatively short intervals over the length of the cables, as in suspension bridges.

A distinction to be made between some early suspension bridges and modern suspension bridges involves the position of the main cables in profile at mid-span with respect to the stiffening trusses. In early suspension bridges, the bottom of the main cables at maximum sag penetrated the top chord of the stiffening trusses and continued down to the bottom chord (Fig. 15.4, for example). Because of the design theory available at the time, the depth of the stiffening trusses was relatively large, as much as $^1/_{40}$ of the span. Inasmuch as the height of the pylons is determined by the sag of the cables and clearance required under the stiffening trusses, moving the mid-span location of the cables from the bottom chord to the top chord increases the pylon height by the depth of the stiffening trusses. In modern suspension bridges, stiffening trusses are much shallower than those used in earlier bridges and the increase in pylon height due to midspan location of the cables is not substantial (as compared with the effect in the Williamsburg Bridge in New York City, where the depth of the stiffening trusses is 25% of the main-cable sag).

Although most suspension bridges employ vertical suspender cables to support the stiffening trusses or the deck structural framing directly (Fig. 15.8), a few suspension bridges, for example, the Severn Bridge in England and the Bosphorus Bridge in Turkey, have inclined or diagonal suspenders (Fig. 15.12). In the vertical-suspender system, the main cables are incapable of resisting

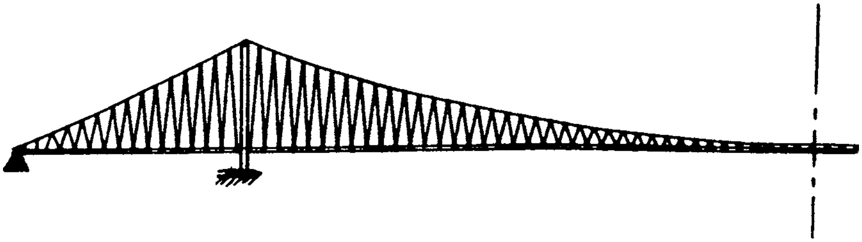


FIGURE 15.12 Suspension system with inclined suspenders.

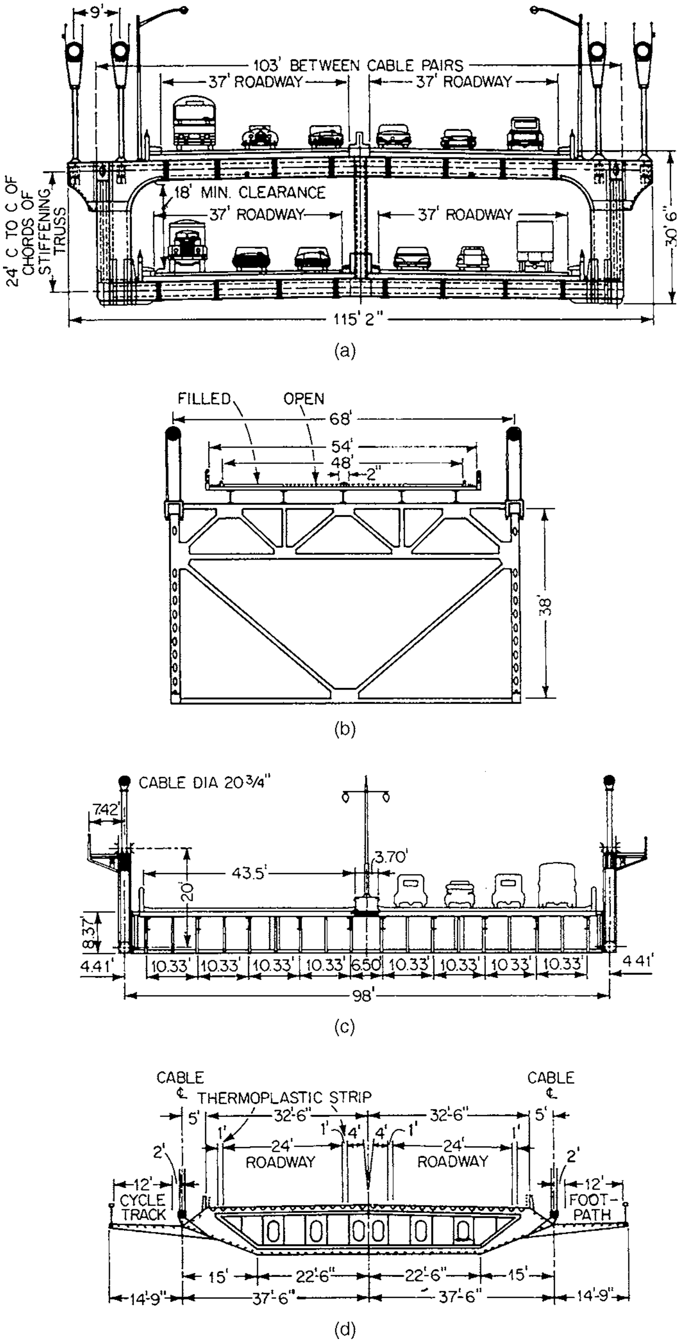


FIGURE 15.13 Typical cross sections of suspension bridges. (*a*) Verrazano Narrows. (*b*) Mackinac. (*c*) Triborough. (*d*) Severn.

shears resulting from external loading. Instead, the shears are resisted by the stiffening girders or by displacement of the main cables. In bridges with inclined suspenders, however, a truss action is developed, enabling the suspenders to resist shear. (Since the cables can support loads only in tension, design of such bridges should ensure that there always is a residual tension in the suspenders; that is, the magnitude of the compression generated by live-load shears should be less than the dead-load tension.) A further advantage of the inclined suspenders is the damping properties of the system with respect to aerodynamic oscillations. The drawback of inclined suspenders is the higher fatigue-stress range and parametric vibrations that are associated with inclined cables.

(N. J. Gimsing, *Cable-Supported Bridges: Concept and Design*, John Wiley & Sons, Inc., New York.)

15.3.3 Suspension Bridge Cross Sections

Figure 15.13 shows typical cross sections of suspension bridges. The bridges illustrated in Fig. 15.13*a*, *b*, and *c* have stiffening trusses, and the bridge in Fig. 15.13*d* has a steel box-girder deck. Use of plate-girder stiffening systems, forming an H-section deck with horizontal web, was largely superseded after the Tacoma Narrows Bridge failure by truss and box-girder stiffening systems for long-span bridges. The H deck, however, is suitable for short spans.

The Verrazano Narrows Bridge (Fig. 15.13*a*) employs 6-in-deep, concrete-filled, steel-grid flooring on steel stringers to achieve strength, stiffness, durability, and lightness. The double-deck structure has top and bottom lateral trusses. These, together with the transverse beams, stringers, cross frames, and stiffening trusses, are conceived to act as a tube resisting vertical, lateral, and torsional forces. The cross frames are rigid frames with a vertical member in the center.

The Mackinac Bridge (Fig. 15.13*b*) employs a $4^1/_4$-in steel-grid flooring. The outer two lanes were filled with lightweight concrete and topped with bituminous-concrete surfacing. The inner two lanes were left open for aerodynamic venting and to reduce weight. The single deck is supported by stiffening trusses with top and bottom lateral bracing as well as ample cross bracing.

The Triborough Bridge (Fig. 15.13*c*) has a reinforced-concrete deck carried by floorbeams supported at the lower panel points of through stiffening trusses.

The Severn Bridge (Fig. 15.13*d*) employs a 10-ft-deep torsion-resisting box girder to support an orthotropic-plate deck. The deck plate is stiffened by steel trough shapes, and the remaining plates by flat-bulb stiffeners. The box was faired to achieve the best aerodynamic characteristics.

15.3.4 Suspension Bridge Pylons

Typical pylon configurations, shown in Fig. 15.14, are portal frames. For economy, pylons should have the minimum width in the direction of the span consistent with stability but sufficient width at the top to take the cable saddle.

Most suspension bridges have cables fixed at the top of the pylons. With this arrangement, because of the comparative slenderness of pylons, top deflections do not produce large stresses. It is possible to use rocker pylons, pinned at the base and top, but they are restricted to use with short spans. Also, pylons fixed at the base and with roller saddles at the top are possible, but limited to use with medium spans. The pylon legs may, in any event, be tapered to allow for the decrease in area required toward the top.

The statical action of the pylon and the design of details depend on the end conditions.

Simply supported, main-span stiffening trusses are frequently suspended from the pylons on short pendulum hangers. Dependence is placed primarily on the short center-span suspenders to keep the trusses centered. In this way, temperature effects on the pylon can be reduced by half.

A list of major modern suspension bridges is provided in Table 15.1.

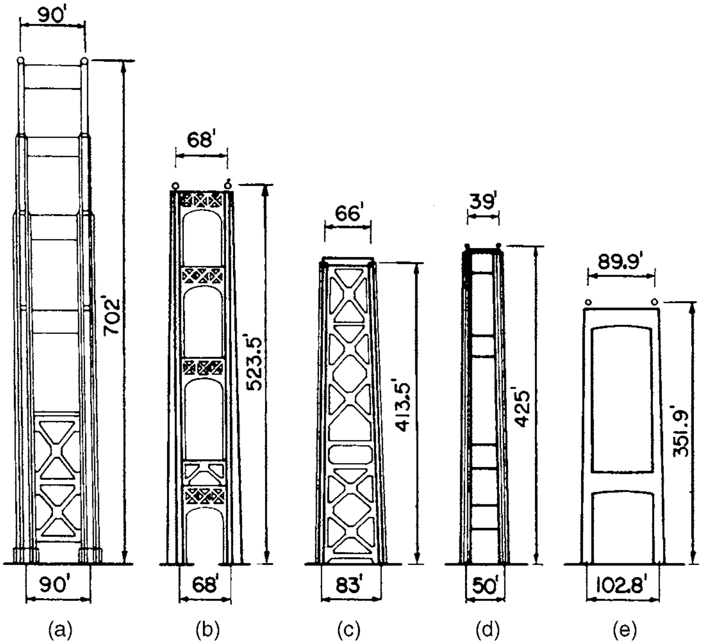


FIGURE 15.14 Suspension-bridge pylons: (*a*) Golden Gate, (*b*) Mackinac, (*c*) San Francisco-Oakland Bay, (*d*) First Tacoma Narrows, (*e*) Walt Whitman.

15.4 CLASSIFICATION AND CHARACTERISTICS OF CABLE-STAYED BRIDGES

The cable-stayed bridge has come into wide use since the 1950s for medium- and long-span bridges because of its economy, stiffness, esthetic qualities, and ease of erection without falsework. Cable-stayed bridges utilize taut cables connecting pylons to a span to provide intermediate support for the span. This principle has been understood by bridge engineers for at least two centuries, as indicated in Art. 15.1.

Cable-stayed bridges are economical for bridge spans intermediate between those suited for deck girders (usually up to 600 to 800 ft but requiring extreme depths, up to 33 ft) and the longer-span suspension bridges (over 2000 ft). The cable-stayed bridge, thus, finds application in the general range of 600- to 2000-ft spans, but spans as long as 3300 ft may be economically feasible.

A cable-stayed bridge has the advantage of greater stiffness over a suspension bridge. Cable-stayed single or multiple box girders with twin planes of stay cables possess large torsional and lateral rigidity. These factors make the structure stable against wind and aerodynamic effects.

15.4.1 Structural Characteristics of Cable-Stayed Bridges

The true action of a cable-stayed bridge is considerably different from that of a suspension bridge. As contrasted with the relatively flexible main cables of the latter, the inclined, taut cables of the cable-stayed structure furnish relatively stable point supports in the main span. Deflections are thus reduced. The structure, in effect, becomes a continuous girder over the piers, with additional intermediate, elastic (yet relatively stiff) supports in the span. As a result, the stayed girder may be shallow. Depths usually range from $^1/_{60}$ to $^1/_{80}$ of the main span, sometimes even as small as $^1/_{200}$ of the span.

Cable forces are usually balanced between the main and flanking spans, and the structure is internally anchored; that is, it requires no massive masonry anchorages. Second-order effects of the type

TABLE 15.1 Major Suspension Bridges

Name	Location	Length of main span		Year completed
		ft	m	
Akashi Kaiko	Japan	6529	1990	1998
Storebelt	Zealand-Sprago, Denmark	5328	1624	1997
Runyang[1]	Yangzhou, China	1490	1490	2005
Humber River	Hull, England	4626	1410	1981
Jiangyin Bridge	Yangtze R., Jiangsu Prov., China	4544	1385	
Tsing Ma Bridge	Hong Kong	4518	1377	1997
Hardanger Fjord	Norway	4347	1325	
Verrazano Narrows	New York, NY, USA	4260	1298	1964
Golden Gate	San Francisco, CA, USA	4200	1280	1937
Höga Kusten	400 km N. Stockholm, Sweden	3970	1210	1997
Mackinac Straits	Michigan, USA	3800	1158	1957
Minami Bisan-Seto	Japan	3609	1100	1988
2nd Bosphorus	Istanbul, Turkey	3576	1090	1988
1st Bosphorus	Istanbul, Turkey	3524	1074	1973
George Washington	New York, NY, USA	3500	1067	1931
3rd Kurushima Bridge	Japan	3379	1030	1999
2nd Kurushima Bridge	Japan	3346	1020	1999
Tagus River[2]	Lisbon, Portugal	3323	1013	1966
Forth Road	Queensferry, Scotland	3300	1006	1964
Kita Bisan-Seto	Japan	3248	990	1988
Severn	Beachley, England	3240	988	1966
Shimotsui Straits	Japan	3084	940	1988
Xiling Bridge	over Yangtze R., Xiling Gorge, China	2953	900	1996
Tigergate (Humen)	Pearl R., Guangdon Prov., China	2913	888	1997
Ohnaruto	Japan	2874	876	1985
Tacoma Narrows I[3]	Tacoma, WA, USA	2800	853	1940
Tacoma Narrows II	Tacoma, WA, USA	2800	853	1950
Tacoma Narrows III[1]	Tacoma, WA, USA	2800	853	(2007)
Askøy	Near Bergen, Norway	2787	850	1992
Innoshima	Japan	2526	770	1983
Akinada	Japan	2461	750	1999
Carquinez	Vallejo, CA, USA	2388	728	2003
Hakucho	Japan	2362	720	1997
Kanmon Straits	Kyushu-Honshu, Japan	2336	712	1973
Angostura	Ciudad Bolivar, Venezuela	2336	712	1967
San Francisco-Oakland Bay[4]	San Francisco, CA, USA	2310	704	1936
Bronx-Whitestone	New York, NY, USA	2300	701	1939
Pierre Laporte	Quebec, Canada	2190	668	1970
Delaware Memorial[5]	Wilmington, DE, USA	2150	655	1951
				1968
Seaway Skyway	Ogdensburg, NY, USA	2150	655	1960
Gjemnessund	Norway	2044	623	1992
Walt Whitman	Philadelphia, PA, USA	2000	610	1957
Tancarville	Tancarville, France	1995	608	1959
1st Kurushima Bridge	Japan	1969	600	1999
Lillebaelt	Lillebaelt Strait, Denmark	1969	600	1970
Kvisti[1]	Bergen, Hordland, Norway	1952	592	
Tokyo Port Connect. Br.	Tokyo, Japan	1870	570	1993
Ambassador	Detroit, MI, USA–Canada	1850	564	1929

(*Continued*)

TABLE 15.1 Major Suspension Bridges (*Continued*)

Name	Location	Length of main span		Year completed
		ft	m	
Skyway[3]	(Chicago World's Fair) USA	1850	564	1933
Hakata-Ohshima	Japan	1837	560	1988
Throgs Neck	New York, NY, USA	1800	549	1961
Benjamin Franklin[2]	Philadelphia, PA, USA	1750	533	1926
Skjomen	Narvik, Norway	1722	525	1972
Kvalsund	Hammerfest, Norway	1722	525	1977
Kwan Ann Great Bridge	Busan, Korea	1640	500	2002
Dazi Bridge	Lasa, Xizang Region, China	1640	500	1984
Kleve-Emmerich	Emmerich, Germany	1640	500	1965
Bear Mountain	Peekskill, NY, USA	1632	497	1924
Wm. Preston Lane, Jr.[5]	near Annapolis, MD, USA	1600	488	1952
Williamsburg[2]	New York, NY, USA	1600	488	1903
Newport	Newport, RI, USA	1600	488	1969
Chesapeake Bay	Sandy Point, MD, USA	1600	488	1952
Brooklyn[7]	New York, NY, USA	1595	486	1883
Lions Gate	Vancouver, B.C., Canada	1550	472	1939
Hirato Ohashi	Hirato, Japan	1536	468	1977
Sotra	Bergen, Norway	1535	468	1971
Hirato	Japan	1526	465	1977
Vincent Thomas	San Pedro–Terminal Is., CA, USA	1500	457	1963
Mid-Hudson	Poughkeepsie, NY, USA	1495	457	1930
Shantou Bay Bridge	Shantou, Guangdong Prov., China	1483	452	1995
Manhattan[2]	New York, NY, USA	1470	448	1909
MacDonald Bridge	Halifax, Nova Scotia, Canada	1447	441	1955
A. Murray Mackay	Halifax, Nova Scotia, Canada	1400	426	1970
Triborough	New York, NY, USA	1380	421	1936
Alvsborg	Goteburg, Sweden	1370	418	1966
Hadong-Namhae	Pusan, South Korea	1325	404	1973
Aquitaine	Bordeaux, France	1292	394	
Baclan	Garrone R., Bordeaux, France	1292	394	1967
Ame-Darja R.	Buhara-Ural, Russia	1280	390	1964
Clifton[3]	Niagara Falls, NY, USA	1268	386	1869
Cologne-Rodenkirchen I[3]	Cologne, Germany	1240	378	1941
Cologne-Rodenkirchen II[10]	Cologne, Germany	1240	378	1955
St. Johns	Portland, OR, USA	1207	368	1931
Wakato	Kita-Kyushu City, Japan	1205	367	1962
Mount Hope	Bristol, RI, USA	1200	366	1929
St. Lawrence River	Ogdensburg, NY–Prescot, Ont.	1150	351	1960
Ponte Hercilio[2,6]	Florianapolis, Brazil	1114	340	1926
Bidwell Bar Bridge	Oroville, CA, USA	1108	338	1965
Middle Fork Feather R.	California, USA	1105	337	1964
Varodd, Topdalsfjord	Kristiansand, Norway	1105	337	1956
Tamar Road	Plymouth, Great Britain	1100	335	1961
Deer Isle	Deer Isle, ME, USA	1080	329	1939
Rombaks	Narvik, Nordland, Norway	1066	325	1964
Maysville	Maysville, KY, USA	1060	323	1931
Ile d'Orleans	St. Lawrence R., Quebec, Canada	1059	323	1936
Ohio River	Cincinnati, OH, USA	1057	322	1867
Otto Beit	Zambezi R., Rhodesia	1050	320	1939
Dent	N. Fork, Clearwater R., ID, USA	1050	320	1971
Niagara[3]	Lewiston, NY, USA	1040	317	1850

TABLE 15.1 Major Suspension Bridges (*Continued*)

Name	Location	Length of main span ft	Length of main span m	Year completed
Cologne-Mulheim I[3]	Cologne, Germany	1033	315	1929
Cologne-Mulheim II	Cologne, Germany	1033	315	1951
Miampimi	Mexico	1030	314	1900
Wheeling	West Virginia, USA	1010	308	1848
(Wheeling Bridge reconstructed after collapse)				1856
Yong Jong	Seoul, Korea	984	300	2001
Konohana[8,9]	Osaka, Japan	984	300	1990
Elisabeth[6]	Budapest, Hungary	951	290	1903
Tjeldsund	Harstad, Norway	951	290	1967
Grand’ Mere	Quebec, Canada	948	289	1929
Cauca River	Colombia	940	287	1894
Jinhu Bridge	Taining, Fujian Prov., China	932	284	1989
Peach River	British Columbia, Canada	932	284	1950
Aramon	France	902	275	1901
Cornwall-Masena	St. Lawrence R., NY–Ontario	900	274	1958
Fribourg[3]	Switzerland	896	273	1834
Brevik	Telemark, Norway	892	272	1962
Royal George	Arkansas R., Canon City, CO, USA	880	268	1929
Kjerringstraumen	Nordland, Norway	853	260	1975
Vranov Lake Bridge	Czech Republic	827	252	1993
Railway Bridge[3]	Niagara River, NY, USA	821	250	1854
Dome, Grand Canyon	Dome, AZ, USA	800	244	1929
Point[3,6]	Pittsburgh, PA, USA	800	244	1877
Rochester	Rochester, PA, USA	800	244	1896
Niagara River	Lewiston, NY, USA	800	244	1899
Thousand Is. Int.	St. Lawrence R., USA–Canada	800	244	1938
Waldo Hancock	Penobscot R., Bucksport, ME, USA	800	244	1931
Anthony Wayne	Maumee R., Toledo, OH, USA	785	239	1931
Parkersburg	Parkersburg, WV, USA	755	236	1916
Footbridge[3]	Niagara R., NY, USA	770	235	1847
Vernaison	France	764	233	1902
Cannes Ecluse	France	760	232	1900
Ohio River	E. Liverpool, OH, USA	750	229	1905
Gotteron	Freiburg, Switzerland	746	227	1840
Iowa-Illinois Mem. I[3]	Moline, IL, USA	740	226	1934
Iowa-Illinois Mem. II	Moline, IL, USA	740	226	1959
Davenport	Davenport, IL, USA	740	226	1935
Monongahela R.	So. 10th St., Pittsburgh, PA, USA	725	221	1933
Rondout	Kingston, NY, USA	705	215	1922
Ohio River	E. Liverpool, OH, USA	705	215	1896
Clifton[3,6]	Bristol, England	702	214	1864
Ohio River[6]	St. Marys, OH, USA	700	213	1929
Ohio River[3,6]	Point Pleasant, OH, USA	700	213	1928
Sixth Street	Pittsburgh, PA, USA	700	213	1928
General U.S. Grant	Ohio R., Portsmouth, OH, USA	700	213	1927
Airline	St. Jo, TX, USA	700	213	1927
Red River	Nocona, TX, USA	700	213	1924
Ohio River	Steubenville, OH, USA	700	213	1904
Ohio River	Steubenville, OH, USA	689	210	1928
Isere	Veurey, France	688	210	1934

(*Continued*)

TABLE 15.1 Major Suspension Bridges (*Continued*)

Name	Location	Length of main span ft	Length of main span m	Year completed
Hungerford[3,6]	London, England	676	206	1845
Mississippi R.[3]	Minneapolis, MN, USA	675	206	1877
Meixihe Bridge	Fengjie, Sichuan Prov., China	673	205	1990
Lancz[6]	Budapest, Hungary	663	202	1845
White River	Des Arc, AR, USA	650	198	1928
Roche Bernard[3]	Vilaine, France	650	198	1836
Missouri River	Illinois, USA	643	196	1956
Caille[3]	Annecy, France	635	194	1839
Columbia R.	Beegee, WA, USA	632	193	1919

[1]Under construction. [2]Railroad and highway. [3]Not standing. [4]Twin spans. [5]Twin bridges. [6]Eyebar chain. [7]Includes cable stays. [8]Self-anchored. [9]Monocable. [10]Structure widened by addition of third cable (1994).

requiring analysis by a deflection theory are of relatively minor importance for the common, self-anchored type of cable-stayed bridge, characterized by compression in the main bridge girders.

15.4.2 Types of Cable-Stayed Bridges

Cable-stayed bridges may be classified by the type of material they are constructed of, by the number of spans stay-supported, by transverse arrangement of cable-stay planes, and by the longitudinal stay geometry.

A concrete cable-stayed bridge has both the superstructure girder and the pylons constructed of concrete. Generally, the pylons are cast-in-place, although in some cases the pylons may be precast-concrete segments above the deck level to facilitate the erection sequence. The girder may consist of either precast or cast-in-place concrete segments. Examples are the Talmadge Bridge in Georgia and the Sunshine Skyway Bridge in Florida.

All-steel cable-stayed bridges consist of structural steel pylons and one or more stayed steel box girders with an orthotropic deck (Fig. 15.15). Examples are the Luling Bridge in Louisiana and the Meridian Bridge in California (also constructed as a swing span).

Other so-called steel cable-stayed bridges are, in reality, composite structures with concrete pylons, structural-steel edge girders and floorbeams (and possibly stringers), and a composite cast-in-place or precast plank deck. The precast deck concept is illustrated in Fig. 15.16.

In general, span arrangements are single-span; two spans, symmetrical or asymmetrical; three spans; or multiple spans. Single-span cable-stayed bridges are a rarity, usually dictated by unusual site conditions. An example is the Ebro River Bridge at Navarra, Spain (Fig. 15.17). Generally, back stays are anchored to deadman anchorage blocks, analogous to the simple-span suspension bridge (Fig. 15.9*a*). Likewise, multiple-span cable-stayed bridges are also rare. Behavior of a more typical three-span cable-stayed bridge is dominated by the stiffness of back-stay cables anchored to the back pier. In the case of a multiple-span cable-stayed bridge, there are no back-stay cables for interior spans. Therefore, multispan cable-stayed bridges are typified by relatively stiff towers, which provide the necessary stiffness for interior spans. The most recent example of a multispan design is the Millau Viaduct in France.

15.4.3 Span Arrangements in Cable-Stayed Bridges

A few examples of two-span cable-stayed bridges are illustrated in Fig. 15.18. In two-span, asymmetrical cable-stayed bridges, the major spans are generally in the range of 60% to 70% of the total

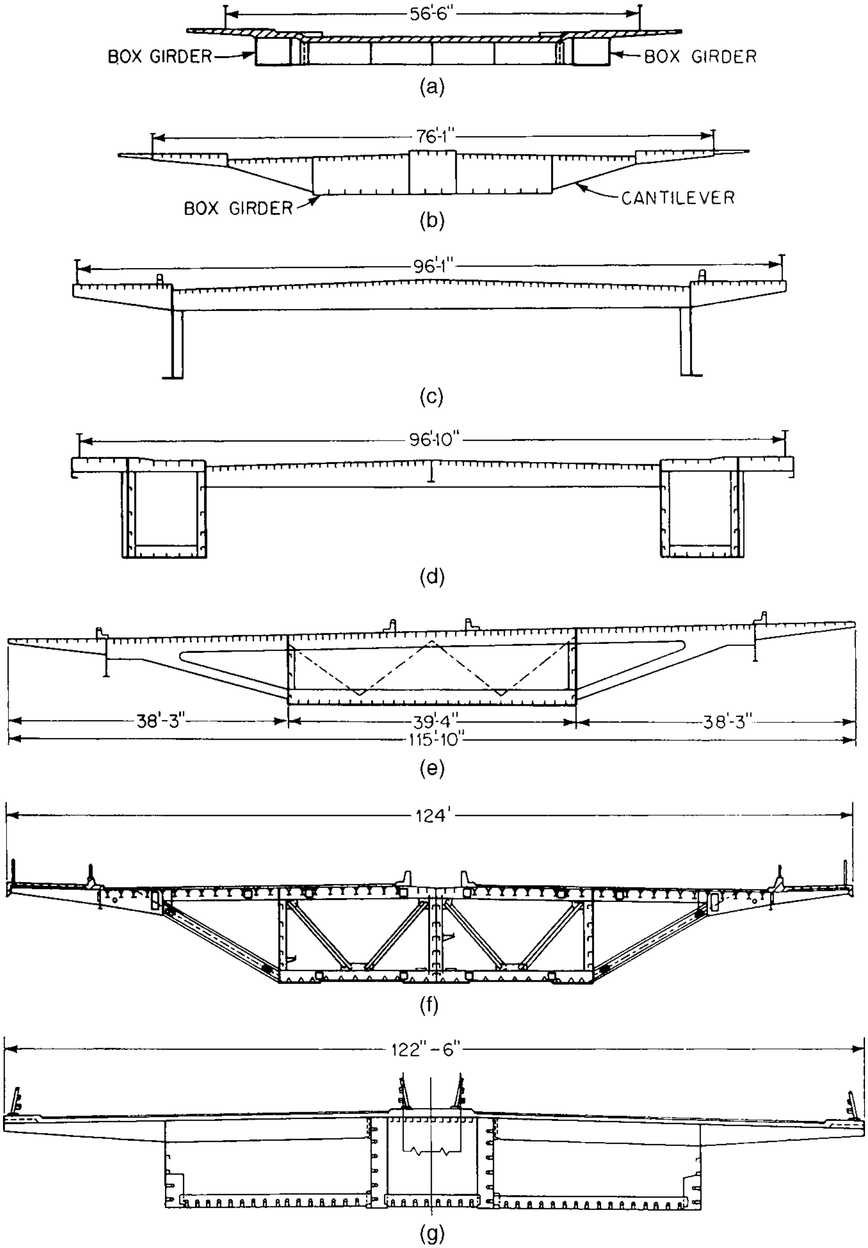


FIGURE 15.15 Typical cross sections of cable-stayed bridges. (*a*) Büchenauer Bridge with composite concrete deck and two steel box girders. (*b*) Julicherstrasse crossing with orthotropic-plate deck, box girder, and side cantilevers. (*c*) Kniebrucke with orthotropic-plate deck and two solid-web girders. (*d*) Severn Bridge with orthotropic-plate deck and two box girders. (*e*) Bridge near Maxau with orthotropic-plate deck, box girder, and side cantilevers. (*f*) Leverkusen Bridge with orthotropic-plate deck, box girder, and side cantilevers. (*g*) Lower Yarra Bridge with composite concrete deck, two box girders, and side cantilevers. (*Adapted from A. Feige, "The Evolution of German Cable-Stayed Bridges—An Overall Survey," Acier-Stahl-Steel (English version), no. 12, December 1966, reprinted in the AISC Journal, July 1967.*)

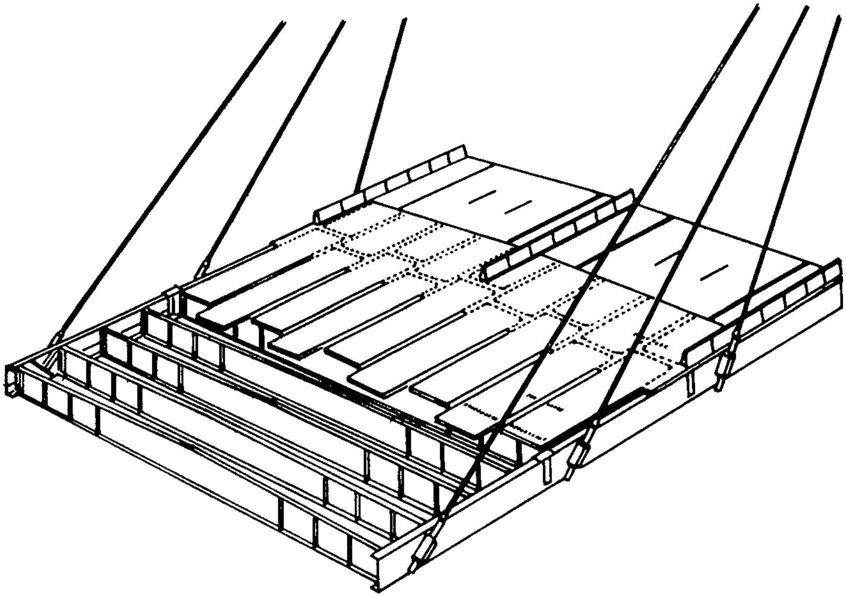


FIGURE 15.16 Composite steel-concrete superstructure girder of a cable-stayed bridge.

length of stayed spans. Exceptions are the Batman Bridge (Fig. 15.18*g*) and Bratislava Bridge (Fig. 15.18*h*), where the major spans are 80% of the total length of stayed spans. The reason for the longer major span is that these bridges have a single back stay anchored to the abutment rather than several back stays distributed along the side span.

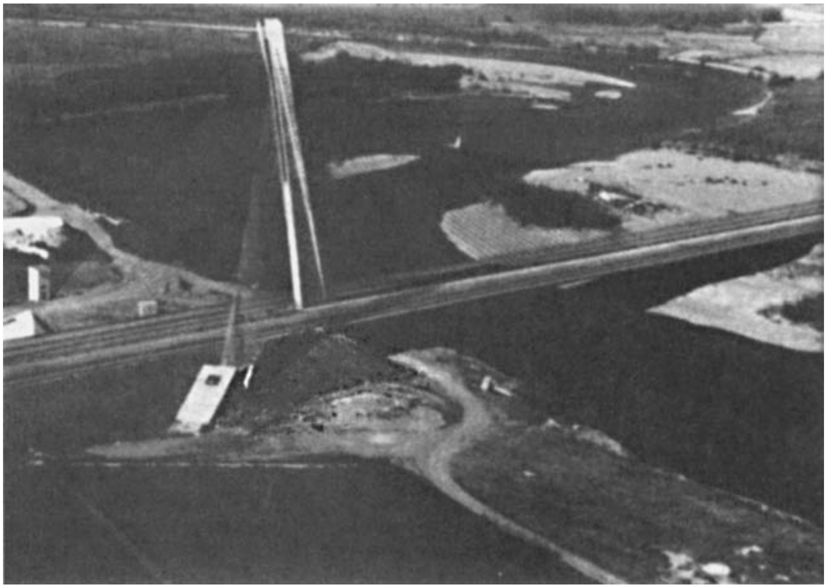


FIGURE 15.17 Ebro River Bridge, Navarra, Spain. (*Reprinted with permission from Stronghold International, Ltd.*)

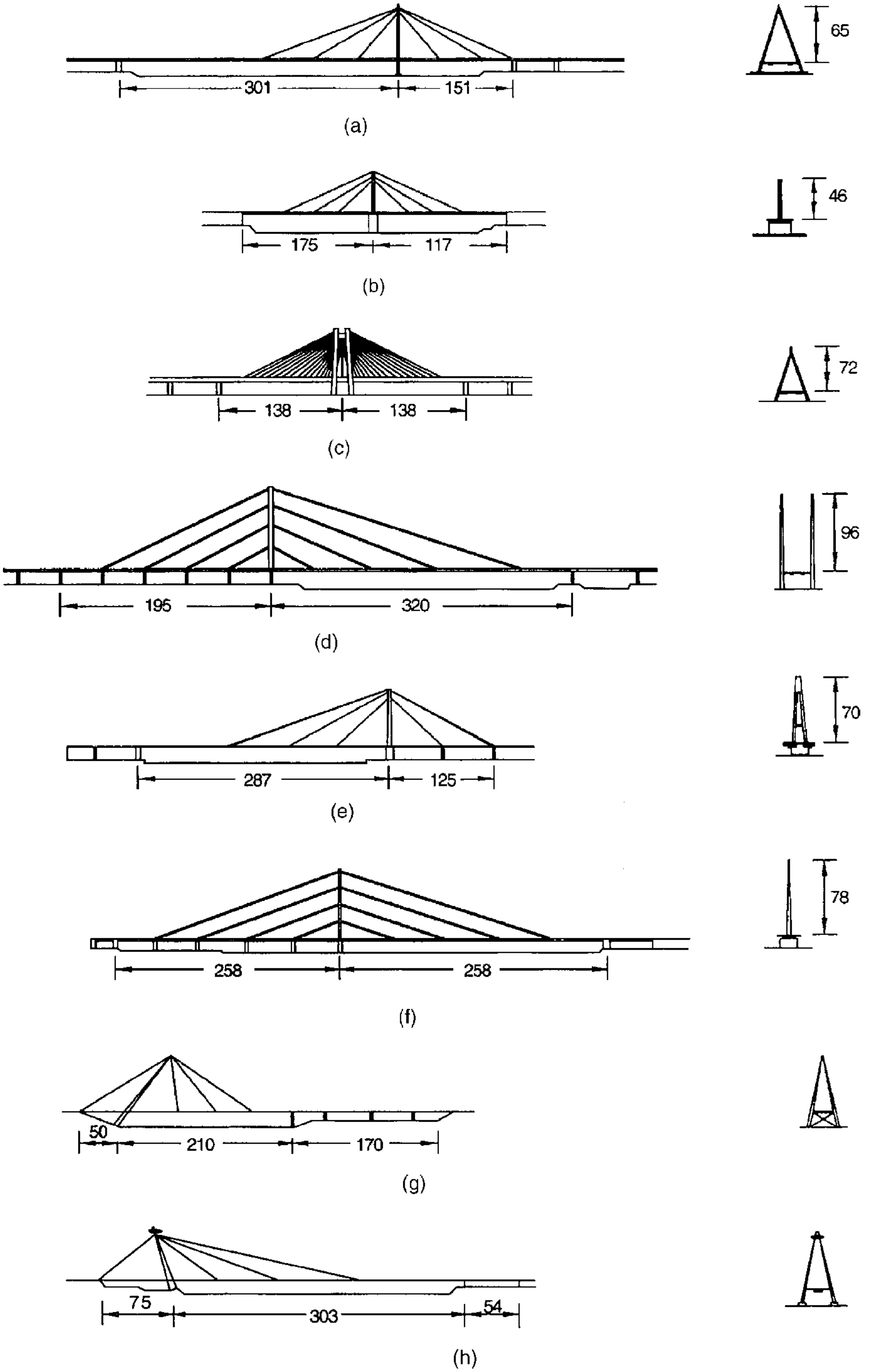


FIGURE 15.18 Examples of two-span cable-stayed bridges (dimensions in meters). (*a*) Cologne, Germany. (*b*) Karlsruhe, Germany. (*c*) Ludwigshafen, Germany. (*d*) Kniebrucke–Dusseldorf, Germany. (*e*) Manheim, Germany. (*f*) Dusseldorf–Oberkassel, Germany. (*g*) Batman, Australia. (*h*) Bratislava, Czechoslovakia.

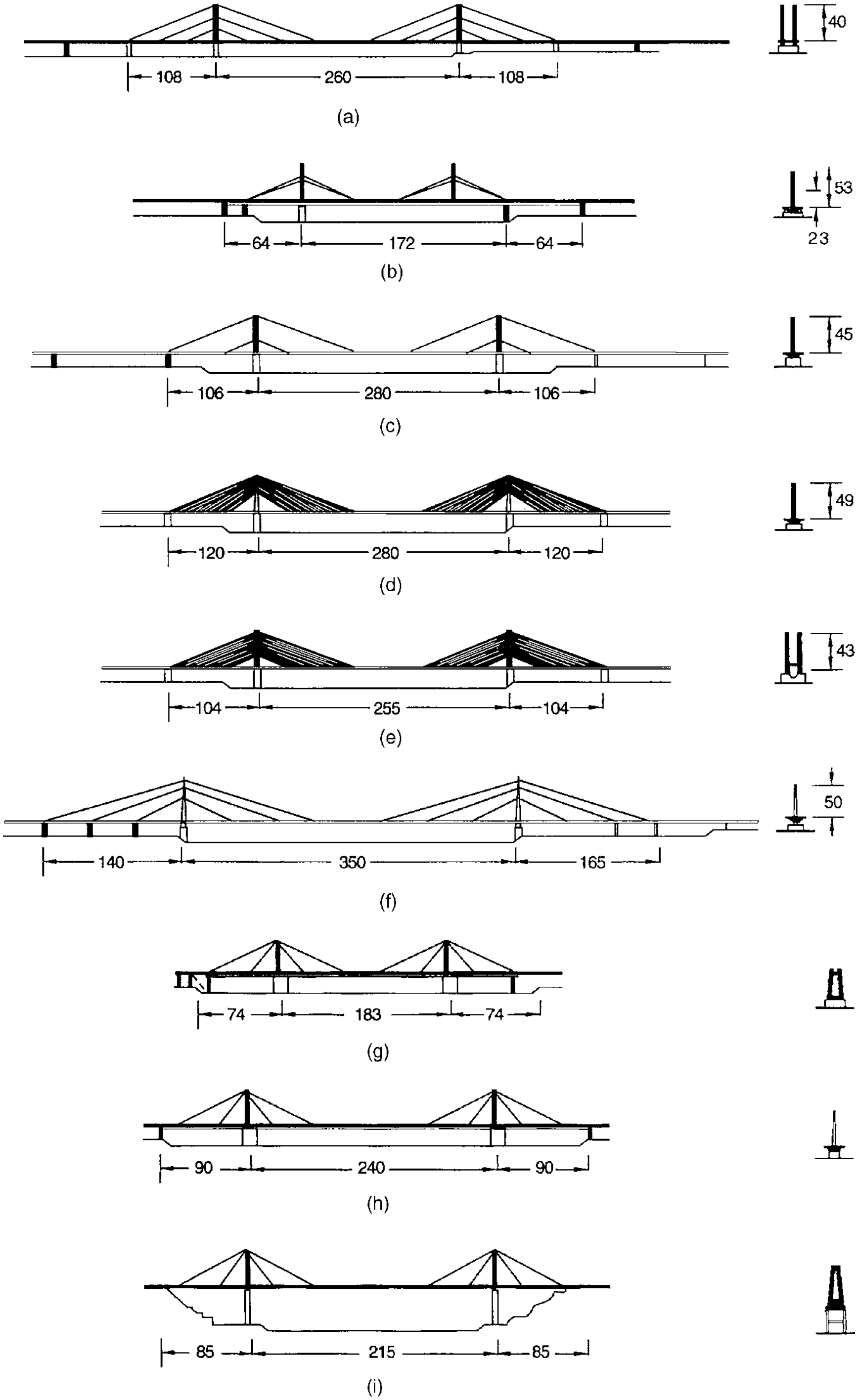


FIGURE 15.19 Examples of three-span cable-stayed bridges (dimensions in meters). (*a*) Dusseldorf-North, Germany. (*b*) Norderelbe, Germany. (*c*) Leverkusen, Germany. (*d*) Bonn, Germany. (*e*) Rees, Germany. (*f*) Duisburg, Germany. (*g*) Stromsund, Sweden. (*h*) Papineau, Canada. (*i*) Onomichi, Japan.

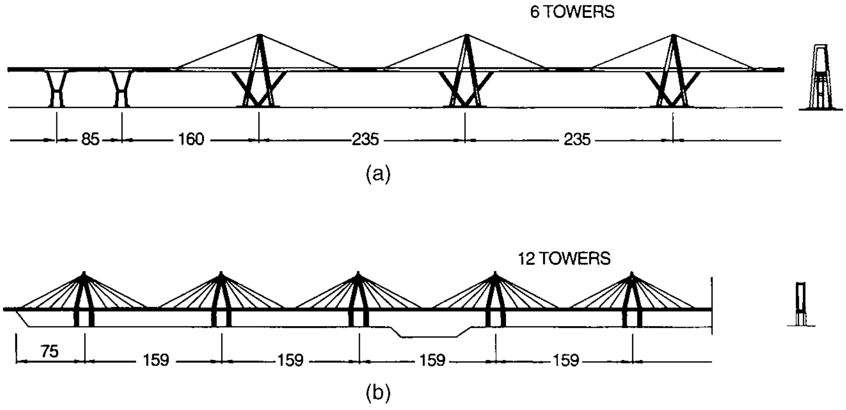


FIGURE 15.20 Examples of multispan cable-stayed bridges (dimensions in meters). (*a*) Maracaibo, Venezuela. (*b*) Ganga Bridge, India.

Three-span cable-stayed bridges (Fig. 15.19) generally have a center span with a length about 55% of the total length of stayed spans. The remainder is usually equally divided between the two anchor spans.

Multiple-span cable-stayed bridges (Fig. 15.20) normally have equal-length spans with the exception of the two end spans, which are adjusted to connect with approach spans or the abutment. The cable-stay arrangement is symmetrical on each side of the pylons. For convenience of fabrication and erection, the girder has "drop-in" sections at the center of the span between the two leading stays. The ratio of drop-in span length to length between pylons varies from 20%, when a single stay emanates from each side of the pylon, to 8% when multiple stays emanate from each side of the pylon.

15.4.4 Cable-Stay Configurations

Transverse to the longitudinal axis of the bridge, the cable stays may be arranged in a single or double plane with respect to the longitudinal centerline of the bridge and may be positioned in vertical or inclined planes (Fig. 15.21). Single-plane systems, located along the longitudinal centerline of the structure (Fig. 15.21*a*), generally require a torsionally stiff stayed box girder to resist the torsional forces developed by unbalanced loading. The laterally displaced vertical system (Fig. 15.21*b*) has

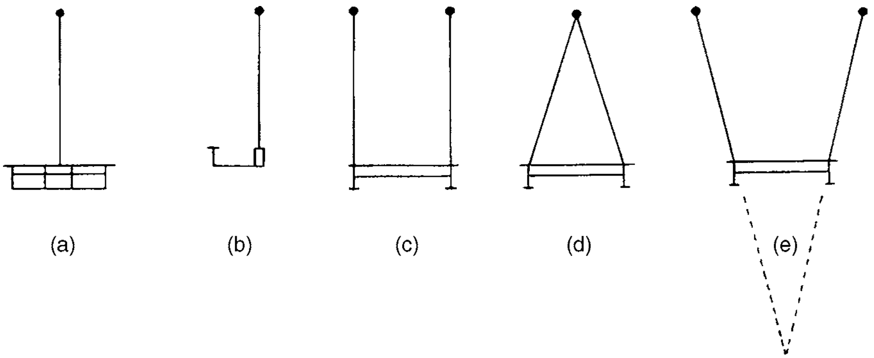


FIGURE 15.21 Cross sections of cable-stayed bridges showing variations in arrangements of cable stays. (*a*) Single-plane vertical. (*b*) Laterally displaced vertical. (*c*) Double-plane vertical. (*d*) Double-plane inclined. (*e*) Double-plane V-shaped. (*Reprinted with permission from W. Podolny, Jr., and J. B. Scalzi,* Construction and Design of Cable-Stayed Bridges, *2d ed., John Wiley & Sons, Inc., New York.*)

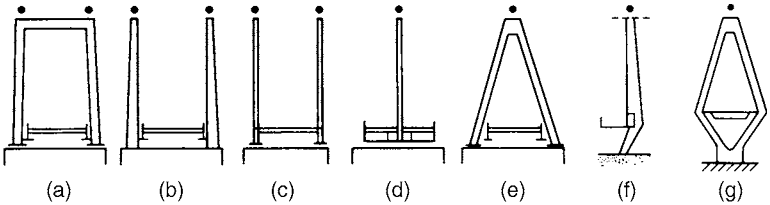


FIGURE 15.22 Shapes of pylons used for cable-stayed bridges. (*a*) Portal frame with top cross member. (*b*) Pylon fixed to pier and without top cross member. (c) Pylon fixed to girders and without top cross member. (*d*) Axial pylon fixed to superstructure. (*e*) A-shaped pylon. (*f*) Laterally displaced pylon fixed to pier. (*g*) Diamond-shaped pylon. (*Reprinted with permission from A. Feige, "The Evolution of German Cable-Stayed Bridges—An Overall Survey," Acier-Stahl-Steel (English version), no. 12, December 1966, reprinted in the AISC Journal, July 1967.*)

been used for a pedestrian bridge. The V-shaped arrangement (Fig. 15.21*e*) has been used for cable-stayed bridges supporting pipelines. This variety of transverse-stay geometry leads to numerous choices of pylon arrangements (Fig. 15.22).

There are four basic stay configurations in elevation (Fig. 15.23): radiating, harp, fan, and star. In the radiating system, all stays converge at the top of the pylon. In the harp system, the stays are parallel to each other and distributed over the height of the pylon. The fan configuration is a hybrid of the radiating and the harp system. The star system was used for the Norderelbe Bridge in Germany primarily for its esthetic appearance. The variety of configurations in elevation leads to a wide variation of geometric arrangements, as indicated by Fig. 15.23.

The number of stays used for support of the deck ranges from a single stay on each side of the pylon to a multistay arrangement, as illustrated in Figs. 15.18 to 15.20. Use of a few stays leads to large spacing between attachment points along the girder. This necessitates a relatively deep stayed girder and large concentrations of stay force to the girder, with attendant complicated connection details. A large number of stays has the advantage of reduction in girder depth, smaller diameter stays, simpler connection details, and relative ease of erection by the cantilever method. However, the number of terminal stay anchorages is increased and there are more stays to install.

A list of major modern cable-stayed bridges is provided in Table 15.2.

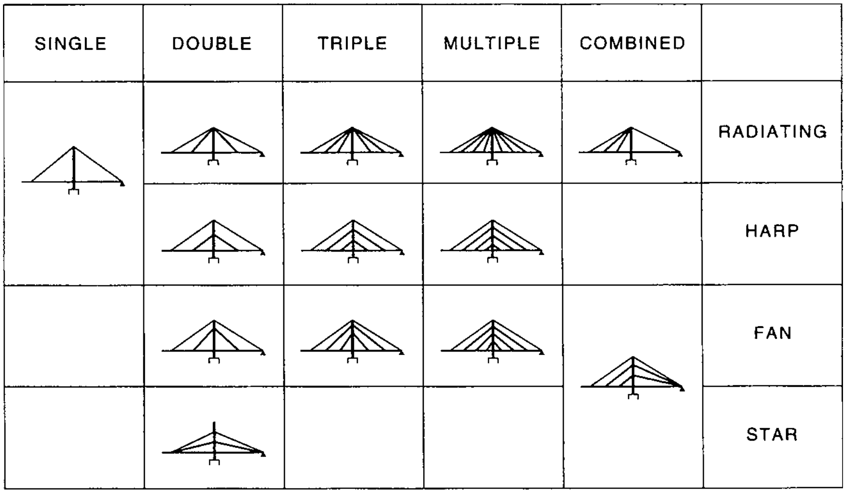


FIGURE 15.23 Stay configurations for cable-stayed bridges.

TABLE 15.2 Major Cable-Stayed Bridges

		Length of main or major span		
Name	Location	ft	m	Year completed
Stonecutters[1]	Hong Kong	3340	1018	(2008)
Tatara	Ehime, Japan	2920	890	1999
Normandy	Le Havre, France	2808	856	1995
Nanjing Yangtze R.	Nanjing, China	2060	618	1999
Wuhan Third Yangtze	Wuhan, Hubei, China	2028	628	1998
Qingzhou Minjiang	Fuzhou, China	1985	605	1996
Yang Pu	Shanghai, China	1975	602	1993
Xupu	Shanghai, China	1936	590	1997
Meiko Chuo	Aichi, Japan	1936	590	1997
Rion Anti Rion	Rion, Greece	1837	506	2004
Skarnsundet	near Trondheim, Norway	1739	530	1991
Tsurumi Tsubasa	Kanagawa, Japan	1673	510	1994
Oresund	Denmark/Sweden	1614	492	2000
Ikuchi	Hiroshima, Ehime, Japan	1608	490	1991
Higashi Kobe	Hyogo, Japan	1591	485	1993
Ting Kau[4]	Hong Kong	1558	475	1998
Seo Hae Grand	Pyung Taek City/Dang Jin County, South Korea	1542	470	1999
Annacis (Alex Fraser)	Vancouver, B.C., Canada	1526	465	1986
Yokohama Bay	Kanagawa, Japan	1509	465	1989
Second Hooghly R.	Calcutta-Howrah, India	1499	457	1992
2nd Severn Crossing	Severn R., England/Wales	1496	456	1996
Queen Elizabeth II	Thames R., Dartford, England	1476	450	1991
Dao Kanong, Chao Phraya R.	Bangkok, Thailand	1476	450	1987
Chongqing 2nd Br. over the Yangtze River	Chongqing, Sichuan Prov., China	1457	444	1991
Barrios De Luna	Cordillera, Spain	1444	440	1983
Tongling over Yangtze R.	Tongling, Anhui Prov., China	1417	432	1995
Kap Shui Mun[2,3]	Hong Kong	1411	430	1997
Helgeland	Sandnessjoen, Nordland, Norway	1394	425	1991
Quetzalapa	Quetzalapa, Mexico	1391	424	1993
Nan Pu	Shanghai, China	1388	423	1991
Vasco da Gama	Lisbon, Portugal	1378	420	1998
Hitsuishijima[2]	Kagawa, Japan	1378	420	1988
Iwagurojima[2]	Kagawa, Japan	1378	420	1988
Yunyang over Hanjiang R.	Yunjang, Hubei Prov., China	1358	414	1994
Meiko Higashi	Aichi, Japan	1345	410	1997
Erasmus	Rotterdam, Netherlands	1345	410	1996
Volga R.	Ulyanovsk, Russia	1335	407	
Wadi Leban	Riyadh, Saudi Arabia	1329	405	1996
Meiko-Nishi	Nagoya, Aichi, Japan	1329	405	1985
Bridge over the Waal River	Ewijck, Netherlands	1325	404	1976
Saint Nazaire	Saint Nazaire, France	1325	404	1975
Elorn River	Brest/Quimper, France	1312	400	1994
Rande	Vigo, Spain	1313	400	1977
Wuhan Bridge over Yangtze	Wuhan, Hubei Prov., China	1312	400	1995
Dame Point	Jacksonville, FL, USA	1300	396	1988
Sidney Lanier	Brunswick R., GA, USA	1250	381	2000

(*Continued*)

TABLE 15.2 Major Cable-Stayed Bridges (*Continued*)

Name	Location	Length of main or major span		Year completed
		ft	m	
Houston Ship Channel	Baytown, TX, USA	1250	381	1995
Hale Boggs Memorial	Luling, LA, USA	1222	372	1983
Dusseldorf—Flehe	Rhine River, Germany	1207	368	1979
Tjorn Bridge, Askerofjord	near Gothenberg, Sweden	1201	366	1982
William Natcher Bridge	Ohio R., Owensboro, KY, USA	1200	366	2001
Sunshine Skyway	Tampa, FL, USA	1200	366	1987
Tampico	Panuco R., Mexico	1181	360	1988
Yamatogawa	Osaka, Japan	1165	355	1982
Novi Sad	Yugoslavia	1152	351	1981
Bill Emerson Memorial	Rt. 74 over Miss. R., Cape Girardeau, MO, USA	1150	350.5	2003
My Thuan[1]	My Thuan, Vietnam	1148	350	2000
Batam—Tonton	Indonesia	1148	350	1998
Tempozan	Osaka, Japan	1148	350	1990
Ajigawa	Osaka, Japan	1148	350	1987
Duisburg-Neuenkamp	Rhine R., Germany	1148	350	1970
Glebe Island	Sydney, Australia	1132	345	1994
Jindo	Uldolmok Straits, Korea	1129	344	1984
Millau Viaduct	Millau, France	1122	342	2004
ALRT Fraser River Br.	Vancouver, B.C., Canada	1115	340	1988
Mesopotamia	Parana, Argentina	1115	340	1972
West Gate	Melbourne, Australia	1102	336	1978
Talmadge Memorial Bridge	Savannah, GA, USA	1100	335	1990
Hao Ping Hsi	Taiwan	1083	330	
Posadas Encarnacion	Argentina	1083	330	1986
Puente Brazo Largo[2]	Rio Parana, Argentina	1083	330	1976
Zarate[2]	Rio Parana, Argentina	1083	330	1975
Karnali River Bridge	Chisapani, Nepal	1066	325	1993
Kohlbrand	Hamburg, Germany	1066	325	1974
Int. Guadiana Bridge	Portugal/Spain	1063	324	1991
Maysville, over Ohio R.	Maysville, KY, USA	1050	320	2000
Qi Ao, mouth of Pearl R.	Zhuhai and Hong Kong, China	1050	320	1998
Pont de Brotonne, Seine R.	Rouen, France	1050	320	1977
Kniebrücke	Rhine R., Dusseldorf, Germany	1050	320	1969
Wuhu over Yangtze R.	Wuhu, China	1024	312	2000
Mezcala	Mexico City/Acapulco Highway	1024	312	1993
Daugava R.	Riga, Latvia	1024	312	1981
Emscher	Rhine R., Germany	1017	310	1990
Grenland Bridge	Frierfjord, Telemark, Norway	1001	305	1996
Dartford-Thurrock Bridge	Thames R., Great Britain	1001	305	1991
Erskine, River Clyde	Glasgow, Scotland	1000	305	1971
Ombla Bay	Dubrovnik, Yugoslavia	998	304	
Bratislava	Danube R., Czechoslovakia	994	303	1972
Severn	Cologne, Germany	990	302	1959
Rama VIII	Bangkok, Thailand	984	300	2002
Mezcala	Mexico	984	300	1993
Moscovksy, Dnieper R.	Kiev, Ukraine	984	300	1976
Pasco-Kennewick	Washington, USA	981	299	1978
Neuwied	Rhine R., Germany	958	292	1977
Rama VIII	Bangkok, Thailand	955	291	1999

TABLE 15.2 Major Cable-Stayed Bridges (*Continued*)

Name	Location	Length of main or major span ft	Length of main or major span m	Year completed
Faro Bridge	Faro, Denmark	951	290	1985
Donaubrücke	Deggenau, Germany	951	290	1975
Coatzacoalcos R.	Mexico	945	288	1984
Dongying Br. over Yellow R.	Kenli, Shandong, China	945	288	1987
Kurt-Schumacher	Mannheim-Ludwigshafen, Germany	941	287	1971
Erasmus Bridge	Rotterdam, Netherlands	932	284	1996
Wadi Kuf	Sipac, Libya	925	282	1971
Wadi Dib	Algeria	919	280	1998
Dolsan	Yeosu, Korea	919	280	1984
Leverkusen	Germany	919	280	1964
Friedrich-Ebert (Bonn-Nord)	Bonn, Germany	919	280	1967
Rheinbrucke	Speyer, Germany	902	275	1974
East Huntington	East Huntington, WV, USA	900	274	1985
Bayview Bridge	Quincy, IL, USA	900	274	1987
South Bridge, Dnieper R.	Kiev, Ukraine	889	271	1993
Willems	Rotterdam, Netherlands	886	270	1981
Ewijk, Waal R.	near Ewijk, Netherlands	886	270	1976
River Waal	Tiel, Netherlands	876	267	1975
Puente del Centenario	Spain	869	265	1992
Ikarajima	Japan	853	260	1996
Yonghe	Tianjin, China	853	260	1987
Theodor Heuss	Dusseldorf, Germany	853	260	1957
Burton	New Brunswick, Canada	850	259	1970
Oberkassel	Dusseldorf, Germany	846	258	1976
Waal R.	Zaltbommel, Netherlands	840	256	1994
Arade Bridge	Portimao, Portugal	840	256	1991
Rees	Rees-Kalkar, Germany	837	255	1967
Duisburg-Rheinhausen	Rhine R., Germany	837	255	1965
Save Rivert Railroad	Belgrade, Yugoslavia	833	254	1977
Tokachi	Japan	823	251	1995
Aswan	Egypt	820	250	1998
Raippaluto	Finland	820	250	1997
Weirton-Steubenville	West Virginia, USA	820	250	1990
Tokachi Chuo	Obihiro, Hokkaido, Japan	820	250	1989
Yobuko	Saga, Japan	820	250	1988
Suehiro	Tokushima, Japan	820	250	1975
Ishikari	Hokkaido, Japan	820	250	1972
General Belgrano	Argentina	804	245	1998
Chaco/Corrientes	Parana River, Argentina	804	245	1973
Papineau-Leblanc	Montreal, Canada	790	241	1969
Karkistensalmi	Finland	787	240	1997
Aomori	Aomon, Japan	787	240	1992
Jianwei	Sichuan Prov., China	787	240	1990
Kessock	Inverness, Scotland	787	240	1982
Yasaka Bridge	Ohta, Yamaguchi, Japan	787	240	1987
Kamone	Osaka, Japan	787	240	1975
Sun Bridge	Japan	784	239	1993
Kemi	Wakayama, Japan	784	239	2000

(*Continued*)

TABLE 15.2 Major Cable-Stayed Bridges (*Continued*)

Name	Location	Length of main or major span ft	m	Year completed
Sugawara-Shirokita	Osaka, Japan	780	238	1989
Cochrane	Mobile, AL, USA	780	238	1991
Lake Maracaibo	Venezuela	771	235	1962
Neuwied	Rhine R., Germany	770	235	1978
Wye River Bridge	England	770	235	1966
Albert Canal Bridge	Lanaye, Belgium	761	232	1985
Clark Bridge Replacement	Alton, IL, USA	756	230	1994
Shimen	Chongqing, Sichuan, China	755	230	1988
Chesapeake and Delaware Canal Bridge	Dover, DE, USA	750	229	1995
Donaubrucke	Hainburg, Austria	748	228	1972
Charles R. Bridge	Boston, MA, USA	745	227	2001
Penang	Malaysia	738	225	1985
Fengtai	Anhui, China	735	224	1990
Bengbu over Huaihe R.	Bengbu, Anhui Prov., China	735	224	1989
Luangawa	Zambia	730	223	1968
Jinan Br. over Yellow R.	Jinan, Shandong Prov., China	722	220	1982
Katsushika	Katsushika, Tokyo, Japan	722	220	1987
Rokko	Hyogo, Japan	722	220	1976
Hawkshaw	New Brunswick, Canada	713	217	1967
Longs Creek	New Brunswick, Canada	713	217	1966
Toyosato	Osaka, Japan	709	216	1976
Evripos Bridge	Greece	707	215	1988
Onomichi	Hiroshima, Japan	705	215	1968
Donaubrucke	Linz, Austria	705	215	1972
Quetzalapa	Mexico	699	213	1993
Pereira-Dasquetradas	Colombia	692	211	1997
Chuo	Japan	692	211	1993
Mei Shywe	Taiwan	689	210	
Ohshiba	Japan	689	210	1997
Xiangjiang North Br.	Changshu, Hunan Prov., China	689	210	1990
Chalkis	Greece	689	210	1989
Godsheide	Hasselet, Belgium	690	210	1978
Polcevera Viaduct	Genoa, Italy	682	208	1969
Arno	Florence, Italy	676	206	1977
Batman	Tasmania, Australia	675	206	1968
Burlington	Burlington, IA, USA	660	201	1993
Ayunose	Japan	656	200	1999
Alamillo	Guadalquivir R., Seville, Spain	656	200	1992
Shin Inagawa	Osaka, Japan	656	200	1997
Torikai-Niwaji (Yodogawa)	Settsu, Osaka, Japan	656	200	1987
Chung Yang	Taiwan	656	200	1984
Maogang	Shanghai, China	656	200	1982
Chichibu Park Bridge	Arakawa R., Saitama Pref., Japan	640	195	
Neches River	Texas, USA	640	195	1991
Dee Crossing	UK	635	194	1997
Ijssel	Kampen, Netherlands	635	194	1983
Tarascon Beaucaire	France	633	192.8	
James River	near Richmond, VA, USA	630	192	1989

TABLE 15.2 Major Cable-Stayed Bridges (*Continued*)

Name	Location	Length of main or major span ft	Length of main or major span m	Year completed
Sakitama	Sakitama, Japan	623	190	1991
Ashigara	Kanagawa, Japan	607	185	1991
Bybrua	Norway	607	185	1978
Aratsu	Fukuoka, Japan	604	184	1988
Wandre	Belgium	600	183	1989
Strömsund	Sweden	600	183	1955

[1]Under construction. [2]Railroad and highway. [3]Double deck. [4]Three pylons, span continuous.

15.5 CLASSIFICATION OF BRIDGES BY SPAN

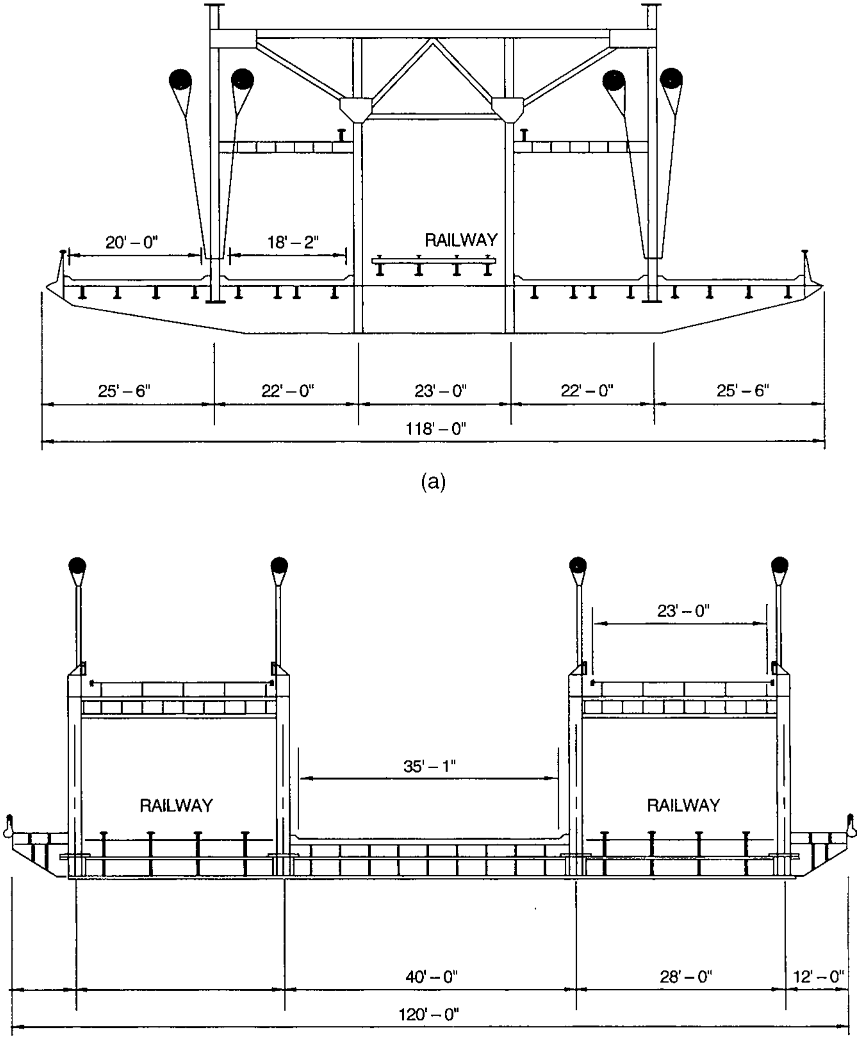
Bridges have been categorized in many ways. They have been categorized by their principal use, such as highway, railroad, pedestrian, pipeline, etc.; by the material used in their construction, such as stone, timber, wrought iron, steel, concrete, and prestressed concrete; by their structural form as girder, box-girder, movable, truss, arch, suspension, and cable-stayed; by structural behavior, such as simple span, continuous, and cantilever, and by their span dimension, such as short, intermediate, and long-span. The definition of long span continually changes with technology. As noted in Art. 15.4, the most economical range for cable-stayed bridges is from about 600 to 2000 ft. The lower end of this range was once considered long-span technology, but is now becoming more routine. Similarly, with suspension bridges, shorter spans for pedestrian bridges and even some for highway bridges are more routine in design.

The definition of long span is perhaps best characterized by the special design and construction conditions that apply. While the AASHTO *LRFD Specifications* go beyond the older AASHTO *Standard Specifications*, they both present prescriptive design procedures for bridges that generally do not require project-specific design criteria. In the case of long-span bridges, especially cable-suspended bridges, the requirements for special aerodynamic studies in lieu of code-based static wind forces are one discriminator in defining long spans. Another special aspect of long-span bridges is the need to carefully consider the effect of the erection process on design, where that process is fundamental to the performance of the completed bridge. And while not unique to long-span bridges, the need to consider spatial variation of ground motions for seismic design is another factor setting long-span bridges apart from more conventional designs.

15.6 CABLE-SUSPENDED BRIDGES FOR RAIL LOADING

Because of flexibility and susceptibility to vibration under dynamic loads, pure suspension bridges are rarely constructed for railway spans. They are sometimes used, however, where dead load constitutes a relatively large proportion of the total load. Where provisions for both railway and highway traffic is necessary, the addition of inclined cable stays from the pylon to the stiffening girder is advantageous, or a cable-stayed bridge may be used, for increased stiffness.

An important consideration in the design for rail loading (including rapid-transit trains) is the positioning of the tracks with respect to the transverse centerline of the deck structure. In the Williamsburg Bridge (Fig. 15.24*a*), the railway is positioned adjacent to the centerline, greatly minimizing torsional forces. In the Manhattan Bridge (Fig. 15.24*b*), the railway is positioned outboard



(b)

FIGURE 15.24 Position of rail loading on two suspension bridges. (*a*) Williamsburg Bridge. (*b*) Manhattan Bridge.

of the centerline, resulting in large torsional forces. As a result of this positioning, the Manhattan Bridge, over the years, has suffered damage and had to be retrofitted with a torsion tube to increase its resistance to torsional forces.

The Zarate-Brazo Largo Bridges in Argentina (two identical structures) are unique cable-stayed bridges not only from the standpoint of supporting highway and railroad traffic, but also in that the rail line is one side of the structures. This positioning necessitated an increased stiffness of the stays on the railroad side. (See W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York.)

15.7 SPECIFICATIONS AND LOADINGS FOR CABLE-SUSPENDED BRIDGES

Both the *Standard Specifications for Highway Bridges* and the newer *LRFD Bridge Design Specifications* by the American Association of State Highway and Transportation Officials (AASHTO) cover ordinary steel bridges, generally with spans less than 500 ft. Specifications of the American Railway Engineering and Maintenance Association (AREMA) for railway bridges are similarly written for spans less than 400 ft. There are no comprehensive standard specifications for long-span bridges. However, many elements of AASHTO and AREMA specifications are appropriate for design of components and local areas of long-span bridges. As a result of the special needs for high-volume traffic, flexibility, aerodynamic and seismic affects, and the sophistication of design and construction for long-span bridges, these projects may use the AASHTO or AREMA Specifications as a baseline criteria, but require supplemental design criteria to address the unique structural design requirements that are significant for major spans.

Structural analysis is usually applied to the following loading conditions: dead load, live load, impact, traction and braking, temperature changes, displacement of supports (including settlement), wind (both static and dynamic effects), seismic effects, and combinations of these. Guidelines for loadings on long-span bridges are given in P. G. Buckland, "North American and British Long-Span Bridge Loads," *Journal of Structural Engineering*, vol. 117, no. 10, October 1991, American Society of Civil Engineers (ASCE). Recommendations for stay cables are presented in "Recommendations for Stay-Cable Design, Testing and Installation," Committee on Cable-Stayed Bridges, Post-Tensioning Institute. See also "Guide for the Design of Cable-Stayed Bridges," ASCE Committee on Cable-Stayed Bridges.

15.8 CABLES

The concept of bridging long spans with flexible tension members, antecedes recorded history (Art. 15.1). Known ancient uses of metal cables include the following. A short length of copper cable discovered in the ruins of Ninevah, near Babylon, is estimated to have been made in about 685 B.C. in the Kingdom of Assyria. A piece of bronze rope was discovered in the ruins of Pompeii, which was destroyed by the eruption of Mt. Vesuvius in 79 A.D. The Romans made cables of wires and rope; on display in the Museo Barbonico at Naples, Italy, is a 1-in-diameter, 15-in-long specimen of their lang-lay bronze rope, in which the direction of lay of both wires and strands is the same.

These early specimens of rope consisted of hand-made wires. In succeeding centuries, the craftsmanship reached such a state of the art that only a very close inspection reveals that wires were hand-made. Viking craftsmanship produced such uniform wire that some authorities believe that mechanical drawing was used.

Machine-drawn wire first appeared in Europe during the fourteenth century, but there is controversy as to whether the first wire rope resembling the current uniform, high-quality product was produced by a German, A. Albert (1834), or an Englishman named Wilson (1832). The first U. S. machine-made wire rope was placed in service in 1846. Since then, with technological improvements, such as advances in manufacturing processes and introduction of high-strength steels, the quality of strand and rope has advanced to that currently available.

In structural applications, cable is generally used in a generic sense to indicate a flexible tension member. Several types of cables are available for use in cable-supported bridges. The form or configuration of a cable depends on its makeup; it can be composed of parallel bars, parallel wires, parallel strands or ropes, or locked-coil strands (Fig. 15.25). Parallel bars are not used for suspension bridges because of the curvature requirements at the pylon saddles. Nor are they used in cable-stayed bridges where a saddle is employed at the pylon, but they have been utilized in a stay where it terminates and is anchored at the pylon.

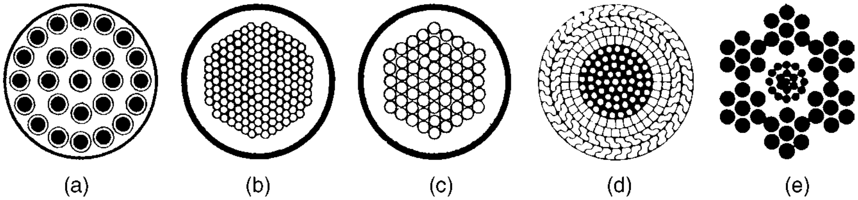


FIGURE 15.25 Various types of cables used for stays. (*a*) Parallel bars. (*b*) Parallel wires. (*c*) Parallel strands. (*d*) Helical lock-coil strands. (*e*) Ropes. (*Courtesy of VSL International, Ltd.*)

15.8.1 Definition of Terms

Cable. Any flexible tension member, consisting of one or more groups of wires, strands, ropes or bars.

Wires. A single, continuous length of metal drawn from a cold rod.

Prestressing wire. A type of wire usually used in posttensioned concrete applications. As normally used for cable stays, it consists of 0.25-in-diameter wire produced in the United States in accordance with ASTM A421 Type BA.

Structural strand (with the exception of parallel-wire strand). Wires helically coiled about a center wire to produce a symmetrical section (Fig. 15.26), produced in the United States in accordance with ASTM A586.

Lay. Pitch length of a wire helix.

Parallel-wire strand. Individual wires arranged in a parallel configuration without the helical twist (Fig. 15.26).

Locked-coil strand. An arrangement of wires resembling structural strands except that the wires in some layers are shaped to lock together when in place around the core (Fig. 15.26).

Structural rope. Several strands helically wound around a core that is composed of a strand or another rope (Fig. 15.27), produced in the United States in accordance with ASTM A603.

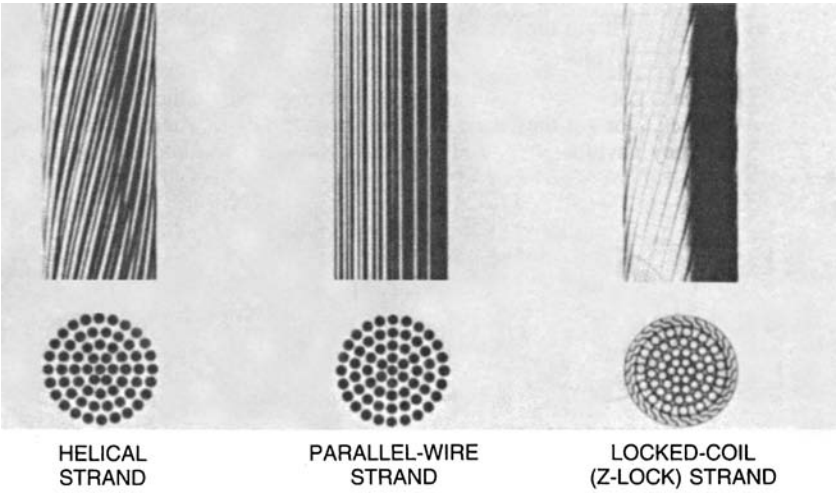


FIGURE 15.26 Types of strands. (*Courtesy of Bethlehem Steel Corporation.*)

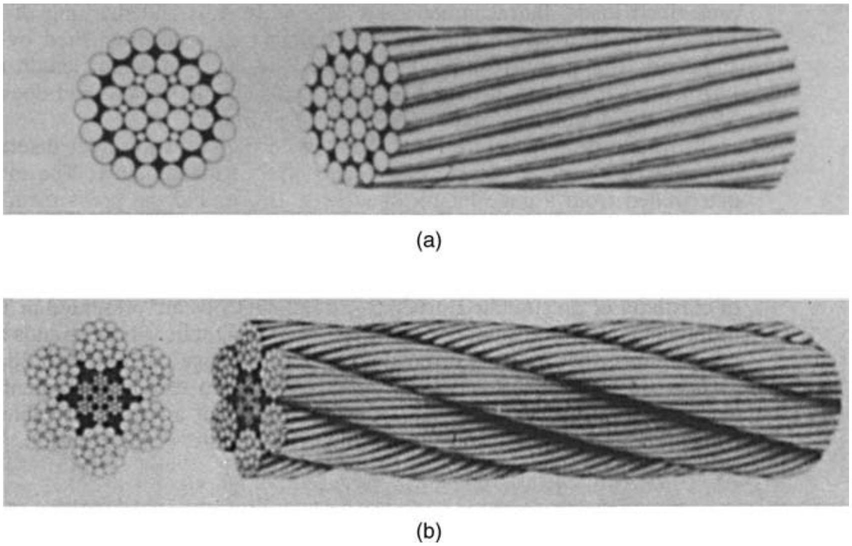


FIGURE 15.27 Configuration of (*a*) structural strand and (*b*) structural rope. (*Reprinted with permission from J. B. Scalzi et al., "Design Fundamentals of Cable Roof Structures," ADUSS 55-3580-01, U.S. Steel.*)

Prestressing strands. A 0.6-in-diameter seven-wire, low-relaxation strand generally used for prestressed concrete and produced in the United States in accordance with ASTM A416 (used for stay cables).

Bar. A solid, hot-rolled bar produced in the United States in accordance with ASTM A722 Type II (used for cable stays).

15.8.2 Structural Properties of Cables

A comparison of nominal ultimate and allowable tensile stress for various types of cables is presented in Table 15.3.

Structural strand has a higher modulus of elasticity, is less flexible, and is stronger than structural rope of equal size. The wires of structural strand are larger than those of structural rope of the same nominal diameter and, therefore, have a thicker zinc coating and better resistance to corrosion (Art. 15.10).

TABLE 15.3 Comparison of Nominal Ultimate and Allowable Tensile Stress for Various Types of Cables, ksi

Type	Nominal tensile strength F_{pu}	Allowable tensile strength F_t
Bars, ASTM A722 Type II	150	$0.45F_{pu} = 67.5$
Locked-coil strand	210	$0.33F_{pu} = 70$
Structural strand, ASTM A586*	220	$0.33F_{pu} = 73.3$
Structural rope, ASTM A603*	220	$0.33F_{pu} = 73.3$
Parallel wire	225	$0.40F_{pu} = 90$
Parallel wire, ASTM A421	240	$0.45F_{pu} = 108$
Parallel strand, ASTM A416	270	$0.45F_{pu} = 121.5$

*Class A zinc coating (see Art. 15.10).

TABLE 15.4 Minimum Modulus of Elasticity of Prestretched Structural Strand and Rope*

Type	Diameter, in	Modulus of elasticity, ksi
Strand	$\frac{1}{2}$ to $2\frac{9}{16}$	24,000
	$2\frac{5}{8}$ and larger	23,000
Rope	$\frac{3}{8}$ to 4	20,000

*For Class B or Class C weight of zinc-coated outer wires, reduce modulus 1000 ksi.

The total elongation or stretch of a structural strand is the result of several component deformations. One of these, termed **constructional stretch,** is caused by the lengthening of the strand lay due to subsequent adjustment of the strand wires into a dense cross section under load. Constructional stretch is permanent.

Structural strand and rope are usually prestretched by the manufacturer to approach a condition of true elasticity. Prestretching removes the constructional stretch inherent in the product as it comes from the stranding or closing machines. Prestretching also permits, under prescribed loads, the accurate measuring of lengths and marking of special points on the strand or rope to close tolerances. Prestretching is accomplished by the manufacturer by subjecting the strand to a predetermined load for a sufficient length of time to permit adjustment of the component parts to that load. The prestretch load does not normally exceed 55% of the nominal ultimate strength of the strand. As a result of generally lower stiffness and the effect of stretch on apparent modulus, structural rope is not used for major cable-stayed bridges. Both bridge strand and structural rope are used for suspension bridge hangers.

In bridge design, careful attention should be paid to correct determination of the cable modulus of elasticity, which varies with type of manufacture. The modulus of elasticity is determined from a gage length of at least 100 in and the gross metallic area of the strand or rope, including zinc coating, if present. The elongation readings used for computing the modulus of elasticity are taken when the strand or rope is stressed to at least 10% of the rated ultimate stress or more than 90% of the prestretching stress. The minimum modulus of elasticity of prestretched structural strand and rope is presented in Table 15.4. The values in the table are for normal prestretched, structural, helical-type strands and ropes; for parallel wire strands, the modulus of elasticity is in the range of 28,000 to 28,500 ksi.

For cable-stayed bridges, it is also necessary to use an equivalent reduced modulus of elasticity E_{eq} to account for the reduced stiffness of a long, taut cable due to sag under its own weight, especially during erection when there is less tension. The formula for this equivalent modulus was developed by J. H. Ernst:

$$E_{\text{eq}} = \frac{E}{1 + \dfrac{E(\gamma l)^2}{12\sigma_m^3}\left[\dfrac{(1+\mu)^4}{16\mu^2}\right]} \tag{15.1}$$

where E = modulus of elasticity of the steel from test
$\sigma_m = (\sigma_u + \sigma_o)/2 = \sigma_o(1 + \mu)/2$ = average stress
σ_u and σ_o = upper and lower stress limits, respectively
$\mu = \sigma_u/\sigma_o$
γ = weight of cable per unit of length per unit of cross-sectional area
l = horizontal projected length of cable

The bracketed term in the denominator becomes unity when $\sigma_o = \sigma_u$, that is, when the stress is constant. The reduction in modulus of elasticity of the cable due to sag is a major factor in limiting the maximum spans of cable-stayed bridges.

The effects of creep of cables of cable-supported bridges should be taken into account in design for certain types of cables. Lock-coil, rope, and epoxy-coated strands are known to exhibit creep behavior at service-level stresses. Creep is the elongation of cables under large, constant stress, for instance, from dead loads, over a period of time. The effects can be evaluated by modification of the cable equation in the deflection theory. As an indication of potential magnitude, an investigation of the Cologne–Mulheim Suspension Bridge indicated that, in a 100-year period, the effects of cable creep would be the equivalent of about one-fourth the temperature drop for which the bridge was designed.

15.8.3 Erection of Cables

Up until the 1960s, parallel-wire main cables for suspension bridges were formed with a spinning-wheel system, carrying one wire loop at a time in each spinning wheel over the pylons from anchorage to anchorage (Fig. 15.28). Depending on the size of the main cables, one, two, or four spinning wheels have been used, each carrying a loop of wire on the outgoing travel to a semicircular strand shoe, and placing the return wire into the strand bundle on the incoming travel (Art. 15.19). Wires were then adjusted for sag, compacted into strand bundles, which were in turn compacted into a circular cable sections (Fig 15.29).

Prefabricated parallel-wire strands are an economical alternative. Large main cables of suspension bridges may be made up of many such strands, laid parallel to each other in a selected geometric pattern. In the commonly used hexagonal, there may be 19, 37, 61, 91, or 127 large strands. In a rectangular pattern, there may be 6 or more strands in each horizontal row and 6 or more vertical rows, with suitable spacers. The strands may have up to 233 wires each, all shop-fabricated, socketed, tested, and packaged on reels. Their use can yield saving in erection time over the older process of aerial spinning of cables on the site.

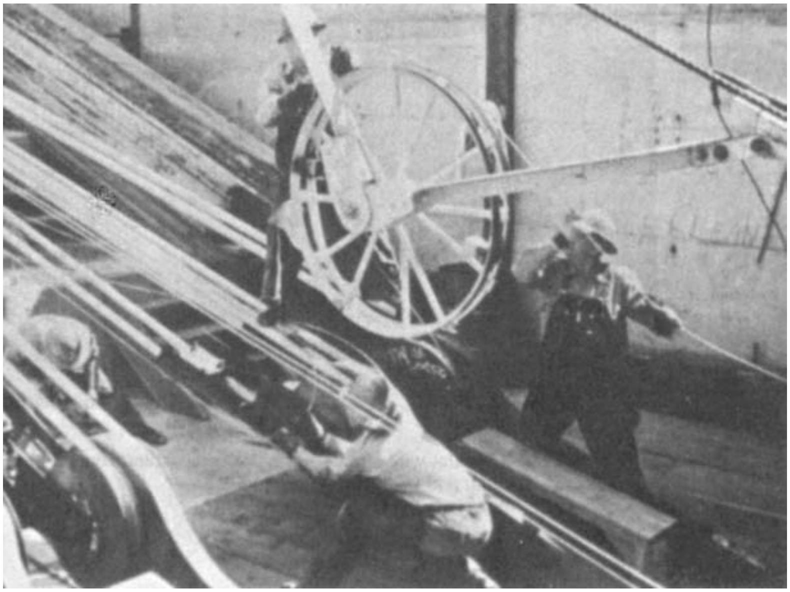
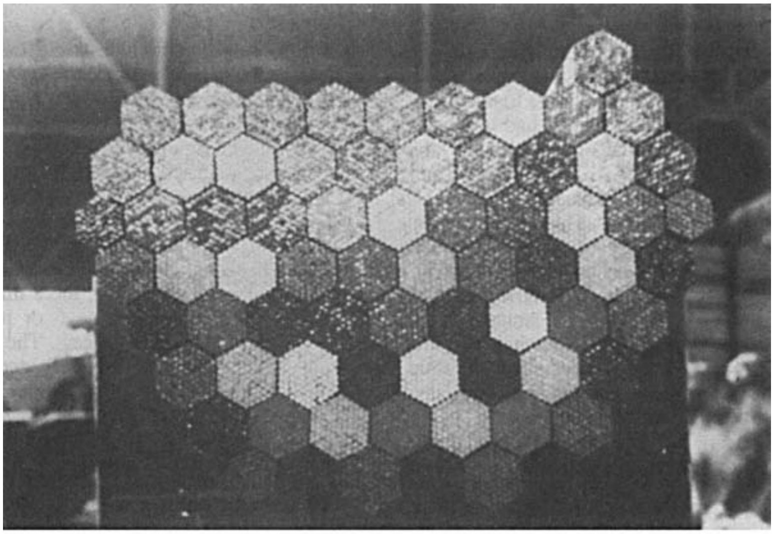
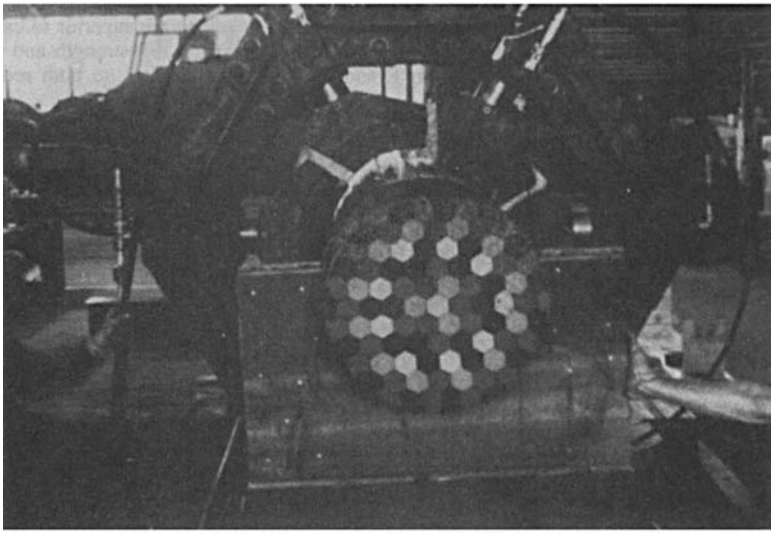


FIGURE 15.28 Transfer of wire from a spinning wheel to an eyebar-and-shoe arrangement at an anchorage.



(a)



(b)

FIGURE 15.29 Parallel-wire strand (*a*) before compaction from an hexagonal arrangement into a round cross section, and (*b*) after compaction.

For the Newport Bridge, which was completed in 1969, shop-fabricated, parallel-wire strands form the cables. Each cable is made up of 4636 wires, each 0.202 in diameter, shop-fabricated into 76 parallel-wire strands of 61 wires each. Thus, in place of thousands of spinning-wheel trips previously necessary, only 152 trips of a hauling rope were needed to form the two cables. Furthermore, thousands of sag adjustments of individual wires were eliminated from the field operation.

From a design point of view, parallel-wire cables are superior to cables made of helical-wire strands. Straight, parallel-laid wires deliver the full strength and modulus of elasticity of the steel, whereas strength and modulus of elasticity are both reduced (by about one-eighth) with helical placement. On the other hand, from the bridge-erection standpoint, standard helical-strand-type cables are superior to field-assembled parallel-wire type. Strands are readily erected and adjusted, with a minimum of equipment and labor. Therefore, they have been used on many small- to moderate-sized suspension bridges. Prefabricated parallel-wire strands, however, combine the erection advantages of strand-type cables with the superior in-place characteristics of parallel-wire cables.

For smaller cable bridges, cables with few strands may be arranged in an open form with strands separated. But for longer bridges, the strands are arranged in a closed form (Fig. 15.29*a*) in either a hexagonal or other geometric pattern. They then may be compacted by machine (Fig. 15.29*b*) and wrapped for protection. Note that a group of helical-type strands cannot be compacted into as dense a mass as a group of parallel-wire strands.

Cable-stayed bridges once used traditional structural strands or locked-coil strands for the stays. Since the 1980s, however, stays composed of prestressing steels have generally been used. Cable stays for cable-stayed bridges are similar to posttensioning tendons in that they consist of the following primary elements:

- Prestressing steel (parallel wires, strands, or bars).
- Sheathing (duct), which encapsulates the prestressing steel and may be a steel pipe or a high-density polyethylene pipe (HDPE).
- Some include a material that fills the void between the prestressing steel and the sheathing and may be a cementitious grout, petroleum wax, or other appropriate material.
- Anchorages.

There are two basic methods of manufacture and installation of stays: (1) assembly on site in final position and (2) prefabricated installation. Both methods have been successfully employed. Given various constraints for a specific project or site, it is generally a question of economics as to which method is employed. Prefabrication may be accomplished either at a factory remote from the construction site or, if feasible, at the project site (possibly on the bridge deck). Normally, factory-prefabricated stays are delivered to the site reeled on drums, complete with the bundle of wires or strands, the HDPE sheathing, and anchorages. (This method cannot be used with prestressing bars or steel pipe sheathing.) Usually, one or both anchorages are fitted to the stay.

At the site, prefabricated stays usually are erected into final inclined position either by crane or by a temporary guying system that is erected between anchorage points from which the stays are suspended. The stays are brought into final position by means of a winch or other suitable hydraulic equipment.

When a guying system is used, site assembly of stays in the final position begins with installation of the initial strand and stay pipe. This strand serves as the guy needed to support the stay pipe, which provides a guide for pulling in successive strands in the stay. Winches are used to pull guide shoes that run each strand up the stay until the cable is filled out. In the past, final stressing was often accomplished with a large jack that tensioned all strands in the stays as a unit. Contemporary practice is to tension each strand one at a time as it is installed, with special monostrand jacks and gages. With the knowledge of overall structural stiffness at the end nodes of each cable, the installer can compute the force or elongation of each strand installed in succession so that the final cable has a uniform stress across each strand. There are a number of proprietary systems that automate this installation process.

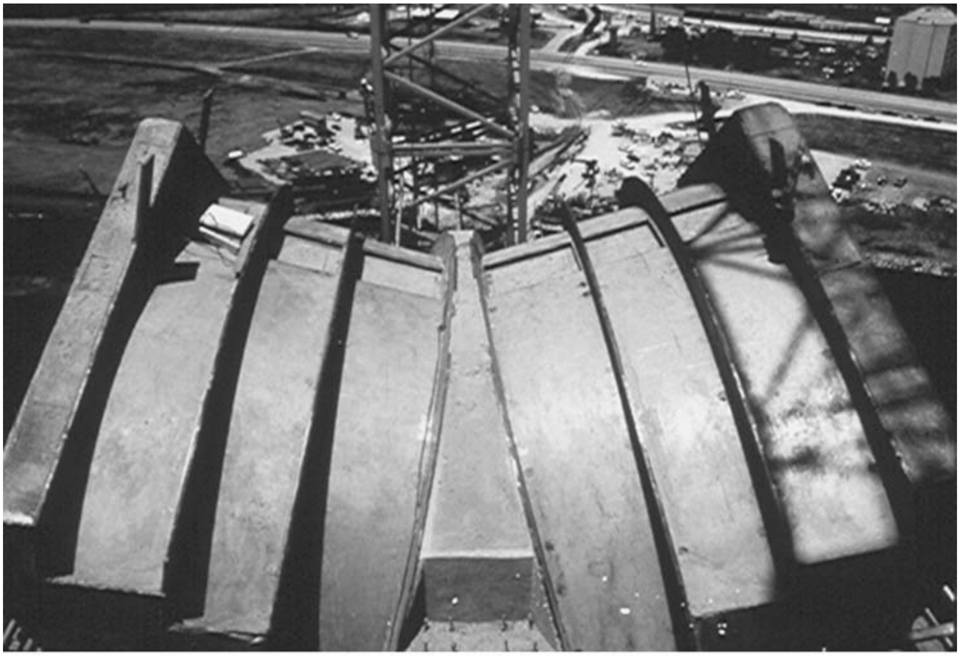
15.9 CABLE SADDLES, ANCHORAGES, AND CONNECTIONS

Saddles atop towers of suspension bridges may be large steel castings in one piece (Fig. 15.30) or, to reduce weight, partly of weldment. The size of the saddle may be determined by the permissible lateral pressures on the cables, which are a function of the radius of curvature of the saddle. Other saddles of special design may be required at side piers to deflect the anchor-span cables to the anchorages. Also, splay saddles are needed at the anchorages.

FIGURE 15.30 Pylon saddle.

In cable-stayed bridges, where the cable stays converge to the top of a pylon (radiating configuration) and are continuous over the pylon, massive saddles, similar to those for suspension-bridge towers, are used (Fig. 15.31). For the types of cable-stay configurations where the stays are distributed along a cellular- type pylon, similar (but smaller) saddles may be used at the pylon. If the pylon is solid concrete, the saddles are generally steel pipe, bent to the appropriate degree of curvature and embedded in the concrete.

Suspension bridge anchorages for the main cables are usually massive concrete blocks designed to resist, with mass and friction, the overturning and sliding effects of the main cable pull. (Where local conditions permit, as with the Forth Road Bridge, the cables may be anchored in tunnels in rock.) The anchorages contain embedded steel eyebar chains to which the main wire cables are connected. A typical arrangement, as used for the Verrazano Narrows Bridge, is shown in Fig. 15.32. A saddle is installed where the strands diverge to attach to the eyebars. Strand wires loop over a strand shoe and are attached to an eyebar (Fig. 15.28—see discussion of spinning in Art. 15.19).



(a)

(b)

FIGURE 15.31 Pylon saddle used for the Clark Cable-Stayed Bridge showing (*a*) details and (*b*) view of tower.

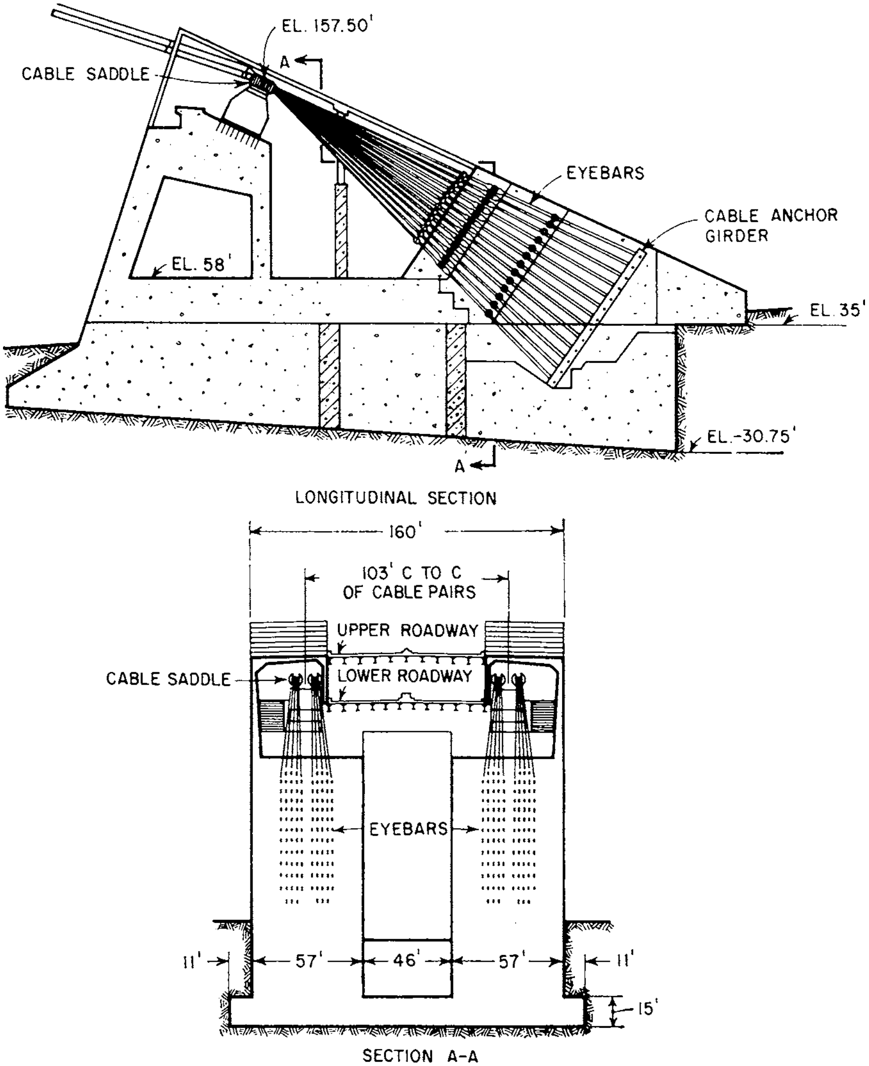


FIGURE 15.32 Anchorage for Verrazano Narrows Bridge.

A slightly different concept was used for the Newport Bridge (Fig. 15.33). In this case, the prefabricated strands of the main cable diverge and pass through 78 pipes held in position by a structural steel framework and transfer their loads to the anchorage through a bearing-type anchorage socket. The whole supporting framework is eventually encased in concrete. In this anchorage-block arrangement, the strand sockets bear on the back of the anchorage block instead of connecting with a tension linkage at the front of the block.

In suspension bridges, the suspender cables are attached to the main cables by cable bands. These are usually made of paired, semicylindrical steel castings with clamping bolts. There are basically two arrangements for attaching the suspenders. The first is typified by the detail used for the Forth Road Bridge (Fig. 15.34). In this arrangement, the cable band has grooves to accommodate looping

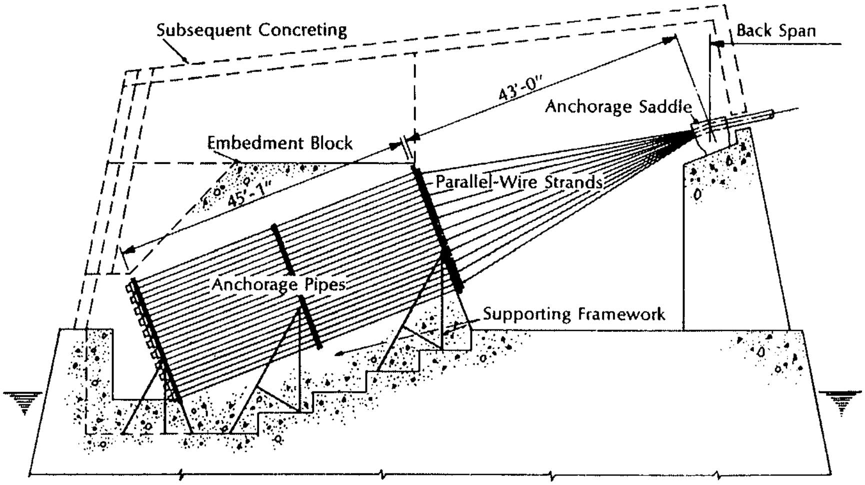


FIGURE 15.33 Anchorage for Newport Bridge.

of the structural rope over the main cable. Because of the bending of the suspender over the main cable, structural rope is used for the suspender, to take advantage of its flexibility. The second basic arrangement for attaching a suspender to a cable band was used for the Hennepin Avenue Bridge (Fig. 15.35). In this case, the suspender is attached to the cable band by standard zinc-poured sockets. Since bending of the suspender is not required, the suspender generally is a structural strand. Properly attached, zinc-poured sockets can develop 100% of the strength of strands and wire rope.

The end fittings or sockets of structural strand or rope are standardized by manufacturers and may be swaged or zinc-poured. These fittings include open or closed sockets of drop-forged or cast steel. Some are illustrated in Fig. 15.36. Fatigue must be considered in designing bridge cables that depend on zinc-poured socketing, particularly if they are subject to a wide range of stress.

The attachments of suspenders to girders depend on the type of girder detail. Generally, the end fitting of a suspender is a swaged or zinc-poured type. Where there are multiple strands or ropes in a suspender, the fitting may be specially made.

Early cable-stayed bridges had stays consisting of parallel structural strands or locked-coil strands. These strands had conventional zinc-poured sockets. Because of concern with the low fatigue strength of structural strand with zinc-poured sockets, a new type of socket, called a HiAm (high-amplitude) socket, was developed in 1968 by Prof. Fritz Leonhardt in conjunction with Bureau BBR Ltd., Zurich. It was intended for use with stays consisting of parallel $^1/_4$-in-diameter prestressing wires that terminate with button heads (ASTM A421 type BA) in an anchor plate in the socket. The anchor socket is filled with steel balls and an epoxy-and-zinc dust binder. This type of anchorage increases the fatigue resistance to about twice that for zinc-poured sockets. The HiAm sockets were used in the United States for the Pasco-Kennewick, Luling, and East Huntington cable-stayed bridges. After those bridges were constructed, seven-wire prestressing strand came into general use, and several types of anchorages were developed to accommodate parallel prestressing strands in cable stays. Today, most highway bridge stays are of the parallel-strand type.

15.10 CORROSION PROTECTION OF CABLES

In the past, the method of protecting the main cables of suspension bridges against corrosion was by coating the galvanized steel with a red lead paste, wrapping the cables with galvanized, annealed wires, and applying a red lead paint. This method has met with a varying degree of success from excellent for the Brooklyn Bridge to poor for the General U. S. Grant Bridge at Portsmouth, Ohio.

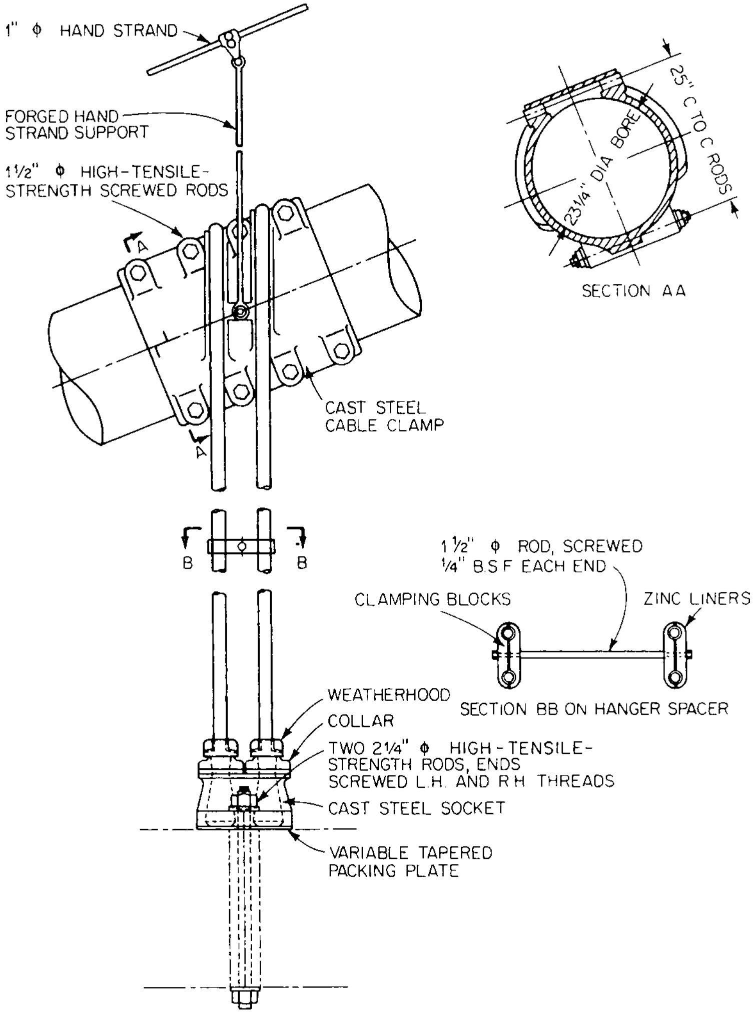


FIGURE 15.34 Cable band and suspender detail used for the Forth Road Bridge. (*Reprinted with permission from Sir Gilbert Roberts, "Forth Road Bridge," Institution of Civil Engineers, London.*)

A potential defect in this system is that, as the cable stretches and shortens under live loads and temperature changes, some separation of adjacent turns of wire wrapping may occur. Depending on the degree of separation, the paint may crack and permit leakage of water and contaminants into the cable.

Alternative protection systems that have been used for some suspension bridges are as follows.

Bidwell Bar Bridge. This 1108-ft-span bridge has 11-in-diameter, parallel structural-strand cables (Fig. 15.37). The protective system consists of the following components: plastic filler pieces, extruded from black polyethylene; a covering of nylon film; a "first-pass" glass-reinforced acrylic resin covering consisting of one layer of glass-fiber mat, two layers of glass cloth, and several coats of acrylic resin; a weather coat of acrylic resin; and a finish coat of acrylic resin containing a sand additive to give the surface a rough texture. This type of covering was developed by Bethlehem Steel in conjunction with DuPont.

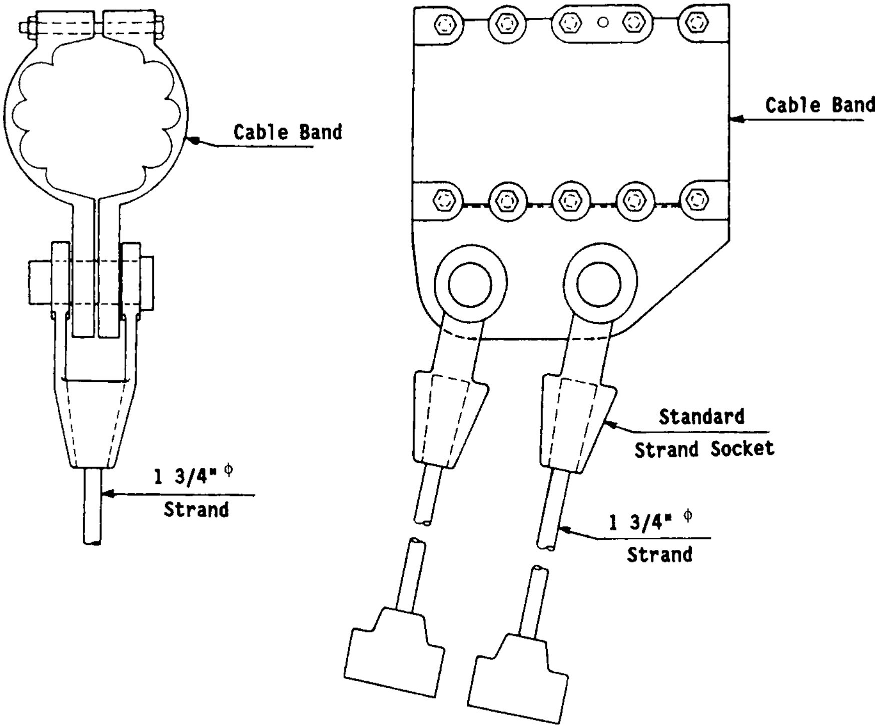


FIGURE 15.35 Cable band and suspender detail used for the Hennepin Avenue Bridge.

Newport (R.I.) Bridge. The protective system for the cables of this bridge is the same as that described for the Bidwell Bar Bridge. However, since the cables consist of parallel wires, the black polyethylene filler pieces were not required.

General U. S. Grant Bridge, Ohio. The protective system comprises spiral-wrapped Neoprene sheet and Hypalon paint, a proprietary system developed by U. S. Steel.

Second Chesapeake Bay Bridge (William Preston Lane, Jr., Memorial). This has the same protective covering as applied to the General U. S. Grant Bridge.

Hennepin Avenue Bridge, Minneapolis, Minnesota. The protective system consists of a wrapped neoprene sheet and hypalon paint system.

The Bidwell Bar Bridge was constructed in 1964 for the California Department of Water Resources. The protective cable covering has been performing satisfactorily. In the early 1970s, some corrosion was discovered at the cable bands, presumably resulting from shrinkage of the covering. Bethlehem Steel corrected the condition by rewrapping a short portion at the cable bands and recaulking. A 1991 inspection indicated no distress in the cable covering.

The similar system applied to the Newport Bridge (installed in 1969) is still performing satisfactorily. A 1980 inspection indicated that some crazing of the top surface had occurred in some areas, but these were superficial and did not extend through the thickness. These areas were patched. There also were some signs of distress at the cable bands, in the caulking groove. As a result of thermal contraction of the covering, the caulking had worked loose (presumably the same condition as that in the Bidwell Bar Bridge). Repairs were made with a more resilient type caulk that accommodates thermal movement.

The system developed by U. S. Steel and applied to the Second Chesapeake Bay Bridge in 1973, the General U. S. Grant Bridge in 1980, and the Hennepin Avenue Bridge in 1990 has been performing satisfactorily. This type of system was also used for rewrapping the Brooklyn Bridge cables in 1986.

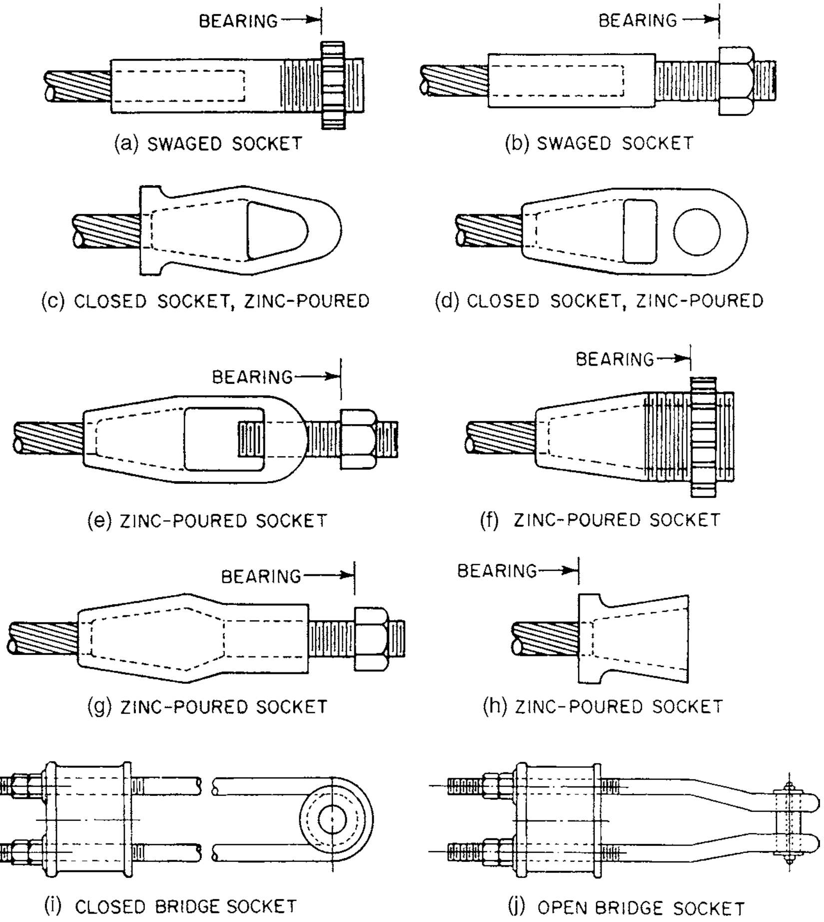


FIGURE 15.36 Types of cable fittings.

Table 15.5 presents a partial listing of suspension bridges with appropriate statistics and the corrosion protection used for the main cables.

An area where the main cable is particularly vulnerable extends from the splay saddle to the eyebars in the anchorage blocks. The only corrosion protection available is the zinc coating of the wires. Depending on environmental conditions in the anchorage blocks, the galvanizing may have a life expectancy on the order of 20 years. Serious corrosion in this area occurred in the Brooklyn, Williamsburg, and Manhattan bridges, requiring corrective measures. In the rehabilitation of one anchorage of the Manhattan Bridge, dehumidification equipment was included to control humidity in the anchorage block.

Suspender Corrosion. Generally, corrosion of suspenders is likely to occur at the anchorage sockets at the stiffening trusses and at retainer castings on top of those trusses. This may be attributed to two possible sources: salt spray from roadway deicing salts, or moisture that enters the interstices of the strand or rope at an upper level and trickles down to the socket or casting.

A 1974 report on the condition of the suspenders of the Golden Gate Bridge revealed that there was considerable reduction in suspender area due to corrosion that occurred as high as 150 ft above the roadway. This could be attributed to saltwater mist or fog.

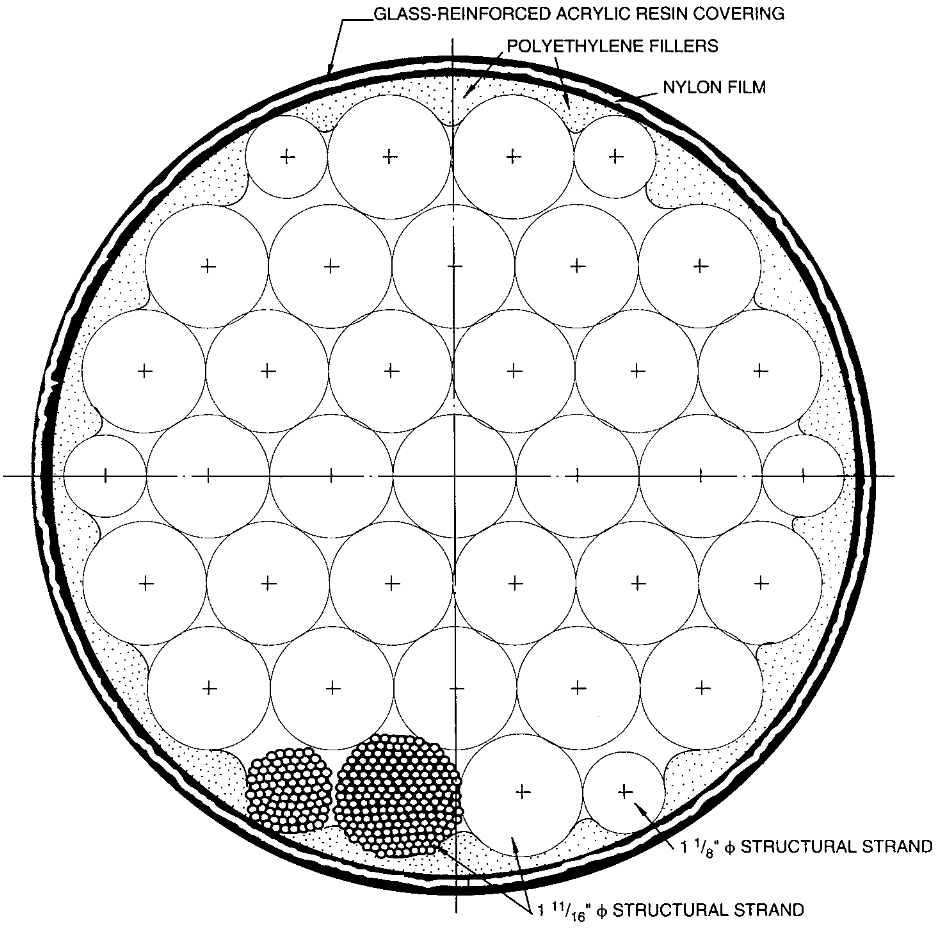


FIGURE 15.37 Cable corrosion-protection system used for the Bidwell Bar Bridge. (*Reprinted with permission from J. L. Durkee, "Advancements in Suspension Bridge Cable Construction," Paper No. 27, Symposium on Suspension Bridges, Lisbon, November 1966.*)

For corrosion protection, U. S. Steel developed a procedure for extruding high-density black polyethylene over strands and rope. In many applications, this jacket also reduces vibration fatigue. For this purpose, particular attention is given to sealing and ends and minimizing the bending of wires at the nose of the socket.

Galvanizing. Wires can be protected against corrosion by galvanizing, a sacrificial coating of zinc that prevents corrosion of the steel so long as the coating is unbroken. Corrosion protection of the individual wires in a structural strand or rope is provided by various thicknesses of zinc coating, depending on the location of the wire in the strand or rope and the degree of corrosive environment expected. The effectiveness of the zinc coating is proportional to its thickness, measured in ounces per square foot of surface area of the uncoated wire. Class A zinc coating varies from 0.40 to 1.00 oz/ft^2, depending on the nominal diameter of the coated wire. A Class B or C coating is, respectively, two or three times as heavy as the Class A coating.

Generally, there are three basic combinations of coating: Class A coating throughout all wires; Class A coating for the inner wires and Class B for the outer wires: and Class A coating for the inner

TABLE 15.5 Cable Construction and Corrosion Protection for Some Suspension Bridges

Name	Location	Year	No. of cables	Cable dia., in	No. of strands	No. of wires or strands	Wire dia., in	Cable construction[1]	Corrosion protection[2]
Brooklyn Bridge	Brooklyn, N.Y.	1883	4	$15\frac{5}{8}$	19	282	0.184[3]	AS	CWR
Williamsburg	New York, N.Y.	1904	4	$18\frac{3}{4}$	37	208	0.192[4]	AS	Note[5]
Manhattan	New York, N.Y.	1910	4	$20\frac{3}{4}$	37	256	0.195	AS	CWR
George Washington	New York, N.Y.	1931	4	26	61	434	0.196	AS	CWR
San Francisco–Oakland Bay	California	1936	2	$28\frac{3}{4}$	37	472	0.195	AS	CWR
Bronx-Whitestone	New York, N.Y.	1939	2	$21\frac{1}{2}$	37	266		AS	CWR
Mackinac Straits	Michigan	1957	2	$24\frac{1}{4}$	37	340	0.196	AS	CWR
Walt Whitman	Philadelphia, Pa.	1957	2	$23\frac{1}{8}$	37	308	0.196	AS	CWR
Throgs Neck	New York, N.Y.	1961	2	23	37	296	0.1875	AS	CWR
Bidwell Bar	State Rt. 62, Feather R., Calif.	1964	2	11	37[6]			PHSS	GRAR
Verrazano Narrows	New York, N.Y.	1964	4	$35\frac{7}{8}$	61	428		AS	CWR
Forth Road	Queensferry, Scotland	1964	2	24	37	314	0.196	AS	CWR
Tagus (Salazar)	Lisbon, Portugal	1966	2	$23\frac{1}{16}$	37	304	0.196	AS	CWR
Severn River	Beachley, England	1966	2	20	19	440	0.196	AS	CWR
Newport	Newport, R.I.	1969	2	$15\frac{1}{4}$	76	61	0.202	FPWS	GRAR
Bosphorus	Istanbul, Turkey	1973	2	23	19	548	0.2	AS	CWR
Humber	England	1980	2	$27\frac{1}{2}$	37	404	0.2	AS	CWR
Gen. U. S. Grant	Portsmouth, Ohio	1927	2	$7\frac{1}{8}$	3	486	0.162[4]	SFPW	CWR
		1940	2	$7\frac{13}{16}$	19[7]			PHSS	CWR
		1980	2		[8]			PHSS	NSHP
Hennepin Ave.	Minneapolis, Minn.	1990	4	$15\frac{3}{8}$	19[9]			PHSS	NSHP

[1] Cable construction: AS = aerial spinning, PHSS = parallel helical structural strand, SFPW = site-fabricated parallel-wire strand, FPWS = factory-fabricated prefabricated parallel-wire strand.

[2] Corrosion protection: CWR = conventional wire wrapping and red lead, GRAR = glass-reinforced acrylic resin, NSHP = Neoprene sheet and Hypalon paint.

[3] Deduced average diameter of the galvanized wire, average bare-wire diameter 0.181 in.

[4] Ungalvanized wires.

[5] Between 1916 and 1922, the original canvas wrapping and steel sheet protection of the cables was removed and replaced by galvanized wrapping wire.

[6] 31 helical structure strands, $1\frac{11}{16}$-in dia., and 6 helical strands, $1\frac{1}{8}$-in dia.

[7] 13 helical structural strands, $1\frac{3}{4}$-in dia., and 6 helical strands, $1\frac{1}{4}$-in dia., Class A coating.

[8] Same configuration as under note 7; i.e., same basic steel area, but changed coating from Class A to Class C.

[9] 13 helical structural strands, $3\frac{3}{8}$-in dia., and 6 helical strands, $2\frac{5}{8}$-in dia.

wires and Class C for the outer wires, depending on the degree of protection desired. Other coating thicknesses and arrangements are possible.

The heavier zinc coatings displace more of the steel area. This necessitates a reduction in rated breaking strength of strand or rope. ASTM A586 and A603 specify minimum breaking strengths required for various sizes of strand or rope in accordance with the three combinations of coating previously described. For other combinations of coating, the manufacturer should be consulted as to minimum breaking strength and modulus of elasticity.

Galvanizing has some disadvantages. Depending on environmental conditions, for example, galvanizing may be expected to last only about 20 years. Also, the possibility that hot-dip galvanizing may cause hydrogen embrittlement is of concern. (There is some indication, however, that, with current technology, the hot-top galvanizing method is not as likely to cause hydrogen embrittlement as previously.) In addition, it may be difficult to meet specifications for a Class C coating with the hot-top method. Furthermore, wire with hot-dip galvanizing may not have the fatigue resistance that wire coated by electrolytic galvanizing has.

Protection of Stays. In early cable-stayed bridges, stays, consisting of locked-coil or structural strands, were protected against corrosion by galvanizing and paint. Nevertheless, extensive corrosion occurred (S. C. Watson and D. G. Strafford, “Cables in Trouble,” *Civil Engineering*, vol 58, no. 4, April 1988, American Society of Civil Engineers). Contemporary stays, in contrast, are similar to external tendons generally used for posttensioned concrete. They consist of prestressing steel, sheathing, corrosion-protection materials, and anchorages.

The Schillerstrasse footbridge in Stuttgart, Germany, completed in 1961, was the first cable-stayed bridge to employ a sheathed and cement-grout-injected stay system. The stays consist of a bundle of parallel prestressing wire encapsulated in a polyethylene (PE) pipe and injected with cement grout. The purpose of the PE sheathing is twofold: to provide a form for the cement grout and to serve as a corrosion barrier. The stays have been inspected on numerous occasions and have shown no signs of corrosion. The first use of this system in the United States was for the Pasco-Kennewick Bridge, completed in 1978. The stays of the bridge were inspected in 1990. After 12 years in service, the exposed wire was as bright and as good as the day it was installed, indicating that with proper care and procedures for installation, cementitious grout can be an effective corrosion inhibitor.

A sheathing of high-density polyethylene (HDPE) pipe is airtight. A $^1/_4$-in thickness provides the same vapor barrier as a 35-ft-thick concrete wall. However, the HDPE pipe must be handled with care. If abused, as in the case of the Luling Bridge (related to excessive grout pressure), the pipe may, in time, develop longitudinal cracks. In addition, the cement-grout column may develop transverse cracks from cyclic tension in the stays, among other reasons. Thus, there is need to prevent direct access to the bare prestressing steel by corrosive agents.

Alternative materials to cement grout or means of providing additional corrosion barriers have been sought to increase corrosion protection of steel stays. Such materials as grease, wax, polymer-cement grout, and polybutadiene polyurethane have been tried with varying degrees of success.

Corrosion protection for stay cables has traditionally been prescribed in design specifications and on design plans, relying on conventional posttensioned concrete practices. Typical methods included steel or HDPE stay pipes, bare 7-wire strands, and cement grout. Later additions included epoxy-coated strand and greased and sheathed strands, adding another protection level to the stay system.

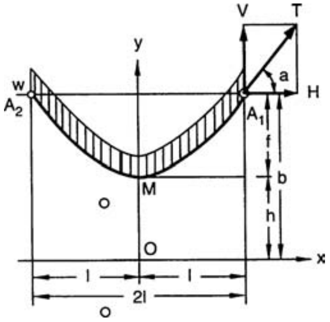
The fourth edition of the *PTI Recommendations for Stay Cable Design, Testing and Installation* introduced a performance standard for stay-cable corrosion protection. Redundant barriers are required, each of which must pass a prescribed corrosion test in order to be qualified. The entire stay system, including anchorage, must then pass a leak test.

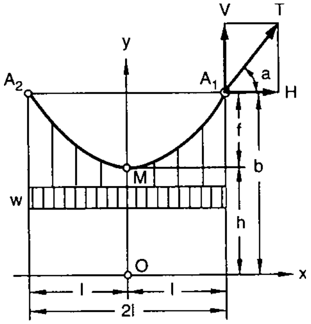
Corrosion-protection systems have progressed differently in the United States and overseas. The major difference is the availability of stay-quality galvanized strand, which to date is not available in the United States. Other elements of protection systems are similar, and include waxed or greased strand, protected by extruded HDPE or polypropylene (PP) sheathing, and HDPE stay pipe. Most stay systems in use today do not include cement grout.

15.11 STATICS OF CABLES

The following summary of elementary statics of cables applies to completely flexible and inextensible cables but includes correction for elastic stretch. The formulas derive from the fundamental differential equation of a cable shape,

TABLE 15.6 Equations for Catenary and Parabolic Cables

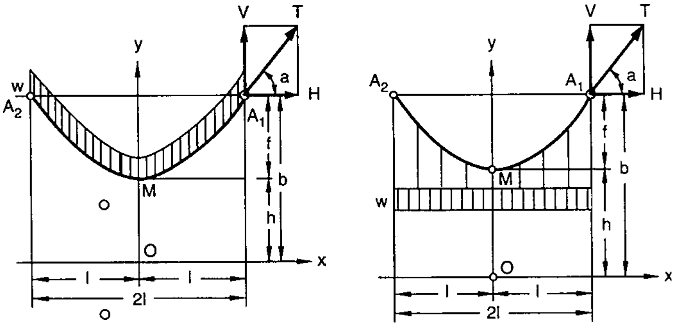




	Catenary	Parabola
Cable ordinate y*	$y=(e^{x/h}+e^{-x/h})h/2$ $=h\cosh x/h$	$y=h+x^2/2h$
Coordinate b of attachment points A_1 and A_2	$b=f+h$ $=h\cosh l/h$	$b=f+h$ $=h+l^2/2h$ $=(l^2+4f^2)/2f$
Sag-span ratio a	$a=f/2l$	$a=f/2l$
Slope y' of cable	$y'=\sinh x/h$	$y'=x/h$
Ordinate h of cable low point	$h=H/w$	$h=H/w$ $=l^2/2f$ $=l/4a$ $=f/8a^2$
Sag f	$f=h(\cosh l/h-1)$	$f=l^2/2h$
Half-length s of cable	$s=\sqrt{b^2-h^2}$ $=\sqrt{f^2+2hf}$ $=\sqrt{2fb-f^2}$	$s=\frac{l}{2h}\sqrt{h^2+l^2}+\frac{h}{2}\times[\log_e(l+\sqrt{h^2+l^2})-\log_e h]$ $=\frac{l}{2h}\sqrt{h^2+l^2}+\frac{h}{2}\sinh^{-1}\frac{l}{h}$ $\approx l\left(1+\frac{8}{3}a^2-\frac{32}{5}\alpha^4+\frac{256}{7}a^6-\cdots\right)$
Angle α of cable at A_1 and A_2	$\tan\alpha=\sin l/h$ $=\frac{1}{h}\sqrt{f^2+2fh}$ $=\frac{1}{b-f}\sqrt{2fb-f^2}$ $=s/h$ $=\frac{1}{h}\sqrt{b^2-h^2}$ $\cos\alpha=h/b$ $=h/(h+f)$ $=(b-f)/b$ $\sin\alpha=\frac{1}{b}\sqrt{2fb-f^2}$ $=s/b$ $=\frac{1}{b}\sqrt{b^2-h^2}$ $=\frac{1}{h+f}\sqrt{f^2+2fh}$	$\tan\alpha=l/h$ $=\sqrt{2f/h}$ $=\sqrt{2f/(b-f)}$ $=2f/l$ $=4a$ $=1/\sqrt{1+16a^2}$ $\cos\alpha=h/\sqrt{h^2+l^2}$ $=h/\sqrt{h^2+2fh}$ $=(b-f)/\sqrt{b^2-f^2}$ $=l/\sqrt{l^2+4f^2}$ $\sin\alpha=\sqrt{2f/(b+f)}$ $=l/\sqrt{h^2+l^2}$ $=\sqrt{2f/(h+2f)}$ $=2f/\sqrt{l^2+4f^2}$ $=4a/\sqrt{1+16a^2}$

(*Continued*)

TABLE 15.6 Equations for Catenary and Parabolic Cables (*Continued*)



	Catenary	Parabola
Vertical component *V* of cable tension	$V = w\sqrt{b^2 - h^2}$ $= w\sqrt{f^2 + 2fh}$ $= w\sqrt{2fb - f^2}$ $= ws$	$V = w\sqrt{2fh}$ $= wl$ $= 4wah$
Horizontal component *H* of cable tension	$H = wh$ $= w(b - f)$	$H = wh$ $= wl^2/2f$ $= wl/4a$ $= wf/8a^2$
Cable tension *T*	$T = wb$	$T = w\sqrt{h^2 + l^2}$ $= w\sqrt{2fh + h^2}$ $= wh\sqrt{1 + 16a^2}$

*Since

$$\cosh \frac{x}{h} = 1 + \frac{(x/h)^2}{2!} + \frac{(x/h)^4}{4!} + \frac{(x/h)^6}{6!} + \cdots$$

the parabolic profile (obtained by dropping the third and subsequent terms) is an approximation for the catenary. The accuracy of this approximation improves as sag *f* becomes smaller.

Source: Adapted from H. Odenhausen, "Statical Principles of the Application of Steel Wire Ropes in Structural Engineering," *Acier-Stahl-Steel,* no. 2, pp. 51–65, 1965.

$$y'' = -\frac{w}{H} \tag{15.2}$$

where y'' = second derivative of the cable ordinate with respect to x
x = distance, measured normal to the cable ordinate, from origin of coordinates to point where y'' is taken
H = horizontal component of cable tension produced by w
w = distributed load, which may vary with x

Two cases are treated: catenary, the shape taken by a cable when the load is uniformly distributed over its length, and parabola, the shape taken by a cable when the load is uniformly distributed over the projection of the span normal to the load.

Table 15.6 lists equations for symmetrical cable. These equations, however, may be extended to asymmetrical cables, as noted later.

The derivation of the equations considered the cable as inextensible. Actually, the tension in the cable stretches it. The stretch, in, of half the cable length may be estimated from

$$\Delta s = \frac{(T + H)s}{2AE} \tag{15.3}$$

where s = half the length of cable, in
T = cable tension, kips, at point of attachment
H = horizontal component of cable tension, kips
A = cross-sectional area of cable, in^2
E = modulus of elasticity of cable steel, ksi

Properties of asymmetrical cables may be obtained by determining first the properties of their component symmetrical elements.

For a parabolic cable (Fig. 15.38), determine point C on the cable, which lies on a horizontal line through a point of attachment. The horizontal distance of C from the support at the cable high point may be computed from $2l_1 - l$, where the cable span $l = l_1 + l_2$, after l_1 has been found from

$$l_1 = \frac{f_1 l}{c}\left(1 \pm \sqrt{1 - \frac{c}{f_1}}\right) \tag{15.4}$$

where l_1, l_2 = horizontal distances from M, the cable low point, to the high and low supports A and B respectively
f_1 = cable sag measured from high point
c = vertical distance between points of support

The portion of the cable between C and the lower support is symmetrical. Its ordinates, slope, length, and cable tension may be computed from the equations in Table 15.6.

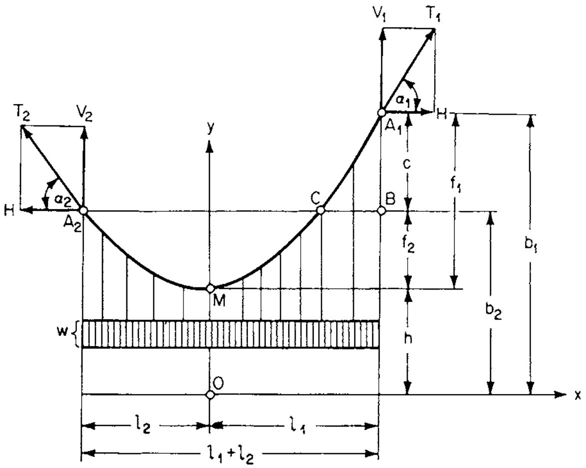


FIGURE 15.38 Cable assumes parabolic shape when subjected to a uniform load acting over its horizontal projection.

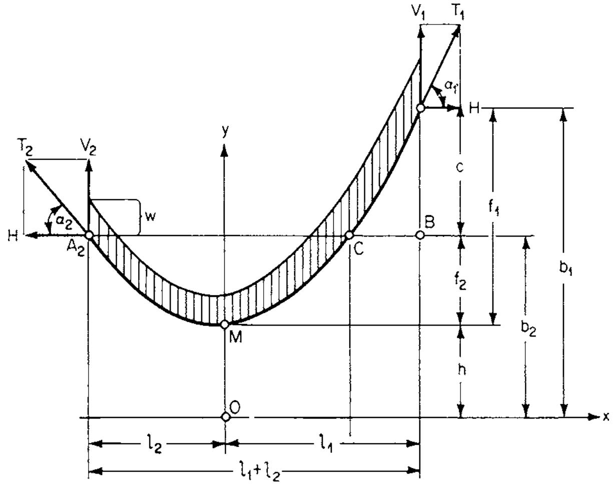


FIGURE 15.39 Cable assumes a catenary shape when subjected to a uniform load acting over its length.

For a catenary cable (Fig. 15.39), point C on a horizontal line through the lower support may be located by stepwise solution of the equation $y = h \cos x/h$ for a symmetrical catenary. An initial solution may be obtained by use of a parabola. Substitution in the exact equation then yields more accurate values. When distances l_1 and l_2 of C from the supports have been determined, the ordinates, slope, length, and cable tension of the symmetrical portion of the cable may be computed.

The portion of the cable from C to the high point is an oblique cable (Fig. 15.40), a special case of the asymmetrical cable. Its properties can be obtained with the equations in Table 15.6 and Eq. (15.4) by treating the oblique cable as part of a symmetrical one.

15.12 SUSPENSION BRIDGE ANALYSIS

Structural analysis of a suspension bridge is that step in the design process whereby, for given structural geometry, materials, and sizes, the moments and shears in stiffening trusses, axial loads in cables and suspenders, and deflections of all elements are determined for given loads and temperature changes. The stress analysis usually is carried out in two broad categories: static and dynamic.

15.12.1 Static Analysis—Elastic Theory

Before the Manhattan Bridge was designed about 1907, suspension bridges were analyzed by the classical theory of structures, the so-called elastic, or first-order, theory of indeterminate analysis. This neglects the deformations of the structural geometry under load in formulation of the equations of equilibrium. The earliest theory was developed by Rankine, who assumed that a stiffening truss distributes the loads uniformly to the cable from which it is suspended. The elastic theory is advantageous because the resulting equations are linear in the loads and internal forces, and linear superposition applies for internal forces caused by different loads. Distortions of the structural geometry under live load, however, can cause a gross overstatement of moments, shears, and deflections calculated by the elastic theory. This theory, therefore, is seldom used, except as a basis for preliminary design or for design of bridges with short spans or rigid stiffening trusses for which large distortions are not possible. (See also Art. 15.12.2.)

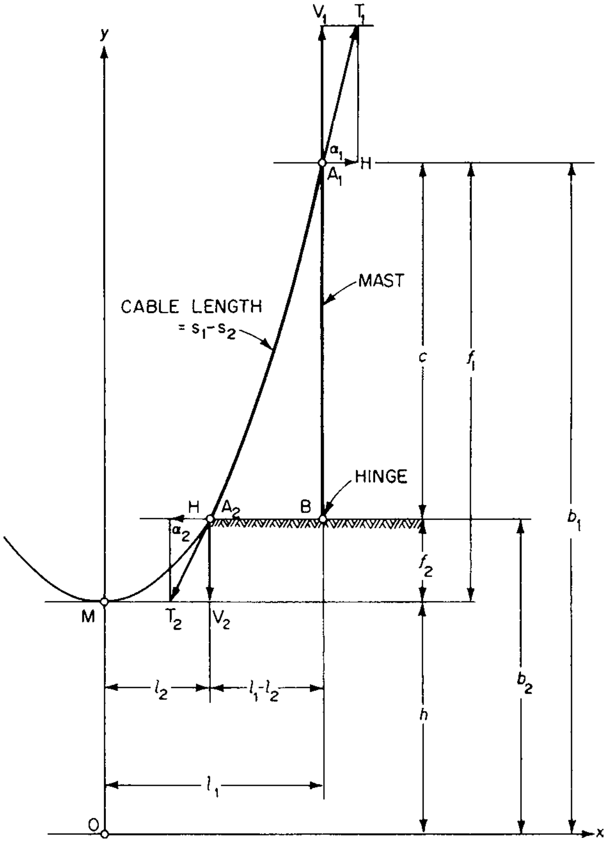


FIGURE 15.40 Part of catenary between low point and a support.

The elastic-theory equations following apply to the structure in Fig. 15.41 with unloaded side spans and pin-ended, main-span stiffening truss. This structure has one redundant, the horizontal component H of cable reaction. An equation for determining H is obtained by making the structure statically determinate by cutting the cable at its low point and applying H there. The gap that is opened at the cut by loads on the stiffening truss must equal the oppositely directed movement at that point produced by H. These deflections can be calculated by the virtual work or dummy-unit-load method, and the equation can readily be solved for H.

For loads,

$$H = \frac{\delta_{ao}}{\delta_{aa}} \tag{15.5}$$

where $\delta_{aa} = \int \frac{M_a^2\, ds}{EI} + \int \frac{N_a^2\, ds}{AE}$

$$\delta_{ao} = \int \frac{M_a M_{ods}}{EI} + \int \frac{N_a N_o\, ds}{AE}$$

M_a = statically determinate moment due to unit horizontal force applied at cut end of cable
M_o = statically determinate moment due to loads

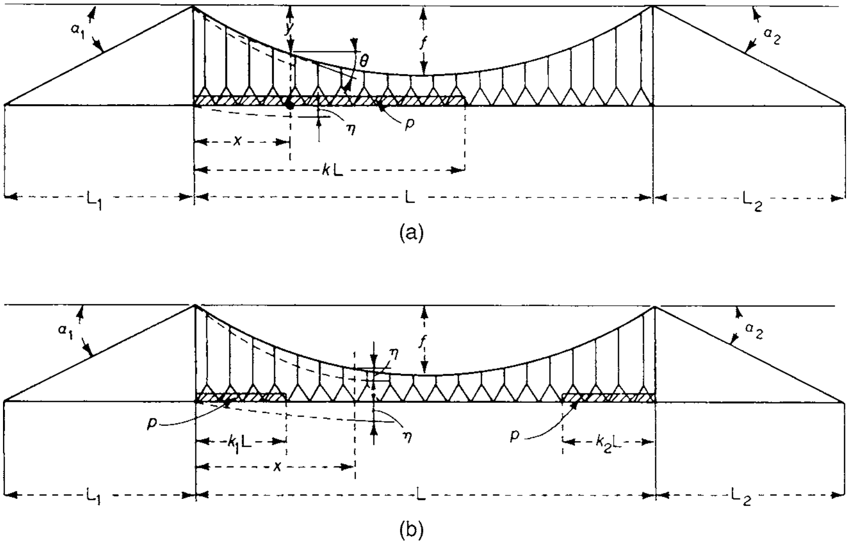


FIGURE 15.41 Suspension bridge with unloaded side spans and pin-ended main-span truss. (*a*) Single uniform load extending from a pylon into the main span. (*b*) Uniform load extending from both pylons into the main span.

N_a = statically determinate axial forces due to unit horizontal force applied at cut end of cable
N_o = statically determinate axial forces due to loads
E = modulus of elasticity of stiffening-truss steel
I = moment of inertia of stiffening truss
A = cross-sectional area of member subjected to axial force

For temperature change,

$$H_t = -\frac{\delta_{at}}{\delta_{aa}} \tag{15.6}$$

where $\delta_{at} = \int \frac{\epsilon_t t \sec\theta}{A_c E_c} ds = \int \frac{ds}{dx} \frac{\epsilon_t t}{A_c E_c} ds$

ϵ_t = coefficient of thermal expansion
t = temperature change
A_c = cross-sectional area of cable
E_c = modulus of elasticity of cable steel

Assumptions. To evaluate Eqs. (15.5) and (15.6), the following conditions are assumed for the structure in Fig. 15.41:

1. The cable takes the shape of a parabola under dead load *w*.
2. Elongation of suspenders and shortening of pylons are so small that they can be neglected.
3. Spacing of suspenders is so small relative to span that the suspenders can be considered a continuous sheet.
4. The horizontal component of cable tension in side spans equals the horizontal component of cable tension in main span. This holds if the cable is fixed to the top of flexible pylons or to a movable saddle atop the pylons.

5. The stiffening truss acts as a beam of constant moment of inertia simply supported at the ends. Under dead load, it is straight horizontal. Usually erected so that it carries none of the dead load, the stiffening truss therefore is stressed only by live load and temperature changes.

Thus, the horizontal component of cable tension due to temperature rise t may be computed from

$$H_t = -\frac{3EI\epsilon_t t L_t}{f^2 NL} \tag{15.7}$$

where f = cable sag

L = length of main span

$$L_t = \int_0^S (ds/dx)ds + L_1 \sec^2 \alpha_1 + L_2 \sec^2 \alpha_2$$

$$\approx L + {}^{16}\!/_{3} a^2 L + L_1 \sec^2 \alpha_1 + L_2 \sec^2 \alpha_2$$

$a = f/L$

S = length of main-span cable

$$N = \frac{8}{5} + \frac{3EI}{A_c E_c f^2}(1 + 8a^2) + \frac{3EIL_t}{A_c E_c L f^2} \sec^3 \alpha_1 + \frac{3EIL_2}{A_c E_c L f^2} \sec^3 \alpha_2$$

L_1, L_2 = lengths of side spans

α_1, α_2 = angle with respect to horizontal of side-span cables

The horizontal component of cable tension due to a uniform live load p extending a distance kL from either end of the main span may be computed from

$$H_p = \frac{pL}{5Na}\left(\frac{5}{2}k^2 - \frac{5}{2}k^4 + k^5\right) \tag{15.8}$$

For maximum cable stress ($k = 1$), the horizontal component due to dead load is

$$H_w = \frac{wL}{8a} \tag{15.9}$$

For live load over the whole span,

$$H_p = \frac{pL}{5Na} \tag{15.10}$$

The sum yields the maximum horizontal component of cable tension:

$$H_{\max} = \frac{L}{a}\left(\frac{w}{8} + \frac{p}{5N}\right) \tag{15.11}$$

For maximum moment at distance x *from pylon:*

A. When $0 \leqq x \leqq NL/4$, solve for k (Fig. 15.41*a*):

$$k + k^2 - k^3 = \frac{NL}{4(L - x)} \tag{15.12}$$

Maximum positive moment with loaded length kL then may be obtained from

$$M_{\max} = \frac{px}{2}(L - x)\left\{1 - \frac{8}{5N}[1 - {}^1\!/_2(1 - k)^4(2 + 3k)]\right\} \tag{15.13a}$$

Maximum negative moment with load length $L - kL$ may be computed from

$$M'_{\max} = -\frac{2}{5}\frac{px(L-x)}{N}(1-k)^4(2+3k) \tag{15.13b}$$

B. When $NL/4 \leqq x \leqq L/2$, solve for k_1 and k_2 (Fig. 15.41*b*):

$$1-2k_1^2+k_1^3 = \frac{NL}{4x} \tag{15.14}$$

$$1-2k_2^2+k_2^3 = \frac{NL}{4(L-x)} \tag{15.15}$$

Maximum negative moment with loaded lengths k_1L and k_2L may be obtained from

$$M = -\frac{2}{5}\frac{px(L-x)}{N}\left[k_1^4(5-3k_1)+k_2^4(5-3k_2)\right] \tag{15.16}$$

Maximum positive moment with loaded length $L - L(k_1 + k_2)$ may be calculated from

$$M_{\max} = \frac{1}{2}px(L-x)\left\{1-\frac{4}{5N}\left[2-k_1^4(5-3k_1)-k_2^4(5-3k_2)\right]\right\} \tag{15.17}$$

For maximum shear at distance x *from pylon:*

A. When $0 \leqq x \leqq (1 - N/4)L/2$, solve for k_o:

$$k_o + k_o^2 - k_o^3 = \frac{NL}{4(L-2x)} \tag{15.18}$$

Maximum positive shear with load between distances x and k_oL from a pylon may be obtained from

$$V_{\max} = V_1 - V_2 \tag{15.19}$$

$$V_1 = \frac{pL}{2}\left(1-\frac{x}{L}\right)^2\left[1-\frac{8}{5N}\left(\frac{1}{2}-\frac{x}{L}\right)\left(2+\frac{4x}{L}+\frac{x^2}{L^2}-2\frac{x^3}{L^3}\right)\right] \tag{15.20a}$$

$$V_2 = \frac{pL}{2}(1-k_o)^2\left[1-\frac{8}{5N}\left(\frac{1}{2}-\frac{x}{L}\right)\left(2+4k_o+k_o^2-2k_o^3\right)\right] \tag{15.20b}$$

B. When $(1 - N/4)L/2 \leqq x \leqq L/2$,

$$V = \frac{pL}{2}\left(1-\frac{x}{L}\right)^2\left[1-\frac{8}{5N}\left(\frac{1}{2}-\frac{x}{L}\right)\left(2+\frac{4x}{L}+\frac{x^2}{L^2}-2\frac{x^3}{L^3}\right)\right] \tag{15.21}$$

(S. P. Timoshenko and D. H. Young, *Theory of Structures*, 2d ed., McGraw-Hill, New York; A. G. Pugsley, *Theory of Suspension Bridges*, Edward Arnold, Ltd., London.)

15.12.2 Static Analysis—Deflection Theory

Distortions of structural geometry of long suspension spans under live load may be very large. As a consequence, the elastic theory (Art. 15.12.1) gives unduly conservative moments, shears, and deflections. For economy, therefore, a deflection theory, also referred to as an exact or second-order theory, that accounts for effects of deformations should be used.

With the notation and assumptions given for the elastic theory in Art. 15.12.1, a differential equation can be written for the structure in Fig. 15.41 to include the vertical deflection η of the cable (and stiffening truss) at any point x. This equation expresses the flexural relationship between the horizontal component of cable tension H under dead and live loads and the stiffening-truss deflection under uniform live load p:

$$EI\eta'''' = p + H_p y'' + H\eta'' \tag{15.22}$$

where each prime represents a differentiation with respect to x.

Equation (15.22) by itself is not sufficient for solution for the two unknowns, η and the horizontal component of cable tension H_p due to live load. (*Note:* H can be expressed in terms of H_p.) Therefore, an additional compatibility equation is necessary. It expresses the cable condition that the total horizontal projection of cable length between anchorages remains unchanged.

The differential equations are not linear, and linear superposition is technically not applicable. This would imply that the use of influence lines for handling moving live loads is not permissible.

In the conventional deflection theory, however, the differential equations are linearized over a small range by assuming that the exponential terms containing H in the solution of the equations are constant during integration (even though that assumption is not valid for a particular loading case). This assumption may be made because, for example, the loading length for maximum moment at a point is not greatly affected by the magnitude of H. With this quasi-linear theory, an average value of H, or two values, $H_{\min}$ and $H_{\max}$, may be used as a basis for drawing linearized influence lines as in first-order theory. With two influence lines (maximum and minimum) thus available, the results can be interpolated for more accurate values of H.

H. Bleich and S. P. Timoshenko suggested that the zero points of the influence lines be determined in this quasi-linear theory to establish the most unfavorable live-loading position. Then, the final results may be calculated by the classical theory with the live load in this position.

Besides the preceding classical differential-equation approach, a trigonometric-series method also is useful. Other advantageous procedures include successive approximation by relaxation theory, simultaneous-linear-equation approach of the flexibility-coefficient methods, elastic-foundation analogy, and analogy of an axially loaded beam.

Much of the literature on classical suspension-bridge theory deals with the effects of minor terms neglected in the assumptions of the deflection theory. S. P. Timoshenko gave an excellent account of the effect of horizontal displacements of the cable, elongation of suspenders, shear deflections, and temperature changes in the cable. (S. P. Timoshenko and D. H . Young, "Theory of Structures," 2d ed., McGraw-Hill, New York.) Other investigators have extended the theory to stiffening trusses that are continuous (such as in the Salazar Bridge) or have variable moments of inertia, widely spaced or inclined suspenders, or multiple main spans.

In general, inclined suspenders can have an important effect on results. Continuous spans are of advantage primarily in short bridges, but the advantage diminishes with long spans. Simple supports are preferred because they avoid settlement problems.

The following treatment of the classical approach is based on A. A. Jakkula's generalization of the work of many investigators. It is restricted here to the case of a suspension bridge with unloaded backstays and a two-hinged stiffening truss (Figs. 15.41 and 15.42). This presentation is useful because it has been extended to configurations with loaded backstays, or other variations of the suspension system, and has been programmed for computers.

Advances in computational methods have prompted several new approaches to analysis of suspension bridges that differ from the classical deflection theory in that they adapt discrete mathematical models to computer programming. For example, if the suspenders are treated as finitely spaced elements (instead of an assumed continuous sheet as in classical methods), the analysis becomes that of an open-panel truss. Solution is required of a set of simultaneous transcendental equations, which are nonlinear because of the effects of distortion. Such solutions involve iterative techniques, the use of the Newton-Raphson method, and other sophisticated mathematical operations. An analogous continuous formulation adapted to computer use has also been proposed.

The solution of the fundamental differential equation [Eq. (15.22)] can be expressed in terms of hyperbolic functions or exponential functions. In the latter form, the solution is

$$\eta = C_1 e^{ax} + C_2 e^{-ax} + \frac{1}{H_w + H_s}\left(M_1 - \frac{p}{a^2}\right) - \frac{H}{H_w + H_s}\left(y - \frac{8f}{a^2 L^2}\right) \tag{15.23}$$

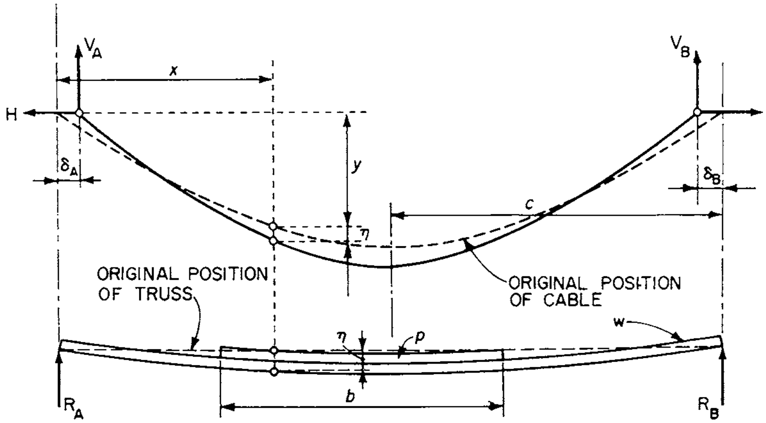


FIGURE 15.42 Original and deflected positions of the main-span cable and stiffening truss for the bridge in Fig. 15.41. Cable and truss have equal deflections at distance *x* from a pylon.

where x = horizontal distance from cable support to point where deflection η is measured
y = vertical distance from cable support to cable at point where deflection η is measured
$e = 2.71828$
a = sag-span ratio f/L
f = cable sag
L = length of main span
H_w = horizontal component of cable tension produced by uniform dead load w
H_s = horizontal component of cable tension produced by all causes other than dead load
M_1 = bending moment in stiffening truss calculated as if the truss were a simple beam independent of the cable
p = total uniform live load per cable

Constants C_1 and C_2 are integration constants to be evaluated, for each load position, from the end conditions and conditions of continuity.

Another method for finding the equation of the deflected truss is to represent deflection by a trigonometric series. If the stiffening truss is considered a free body, it will be in equilibrium under the force system indicated in Fig. 15.43. The truss is acted on by dead load w over the entire span, live load p over any length of span $k_2L - k_1L$, and suspender pull $w + q$, where q is the portion of the uniform live load carried by the cable.

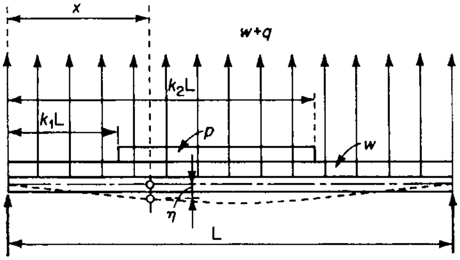


FIGURE 15.43 Forces acting on the truss of Fig. 15.41.

The deflection of the truss can be given by the trigonometric series

$$\eta = \sum_{n=1}^{\infty} a_n \sin \frac{n\,\pi x}{L} = a_1 \sin \frac{\pi x}{L} + a_2 \sin \frac{2\,\pi x}{L} + a_3 \sin \frac{3\,\pi x}{L} + \cdots \tag{15.24}$$

The Fourier coefficients a_n are determined by energy methods:

$$a_n = \frac{(\cos n\,\pi k_1 - \cos n\,\pi k_2)qL/n\pi - (1 - \cos n\,\pi)wL\beta/n\pi}{n^4 EI\pi^4/2L^3 + (1+\beta)H_w\pi^2 n^2/2L} \tag{15.25}$$

where $\beta = H_s/H_w$
E = modulus of elasticity of stiffening-truss steel
I = moment of inertia of stiffening truss

In the following development of the deflection theory, the trigonometric-series solution is used. It converges rapidly and avoids difficulties with the integration constants C_1 and C_2, which hold only for values of x for which M_1 has the same algebraic form. (Thus, the loading in Fig. 15.43 requires evaluation of six constants of integration.)

Equations (15.23) to (15.25) contain the unknown H_s. It must be determined before deflections can be numerically evaluated for any particular case. The energy method may be used for this purpose: the work done by the suspender forces moving through the deflection undergone by the cable is equated to the internal work done by the internal stress in the cable moving through the deformation suffered by the cable.

The force system acting on the cable treated as a free body is shown in Fig. 15.44. Application of the energy equation yields the so-called cable condition:

$$\frac{H_s L_c}{A_c E_c} + \epsilon_t L_t = \frac{8f}{L^2}\int_0^L \eta\, dx - \frac{1+\beta}{2+\beta}\int_0^L \eta''\, \eta\, dx \qquad (15.26)$$

where ϵ_t = coefficient of thermal expansion
t = temperature change
$\beta = H_s/H_w$
A_c = cross-section area of cable
E_c = modulus of elasticity of cable steel

$$L_t = \int_0^S (ds/dx)\, ds + L_1 \sec^2 \alpha_1 + L_2 \sec^2 \alpha_2$$

$$\approx L + {}^{16}/_{3}\alpha^2 L + L_1 \sec^2 \alpha_1 + L_2 \sec^2 \alpha_2$$

L_1, L_2 = lengths of side spans
α_1, α_2 = angle with respect to horizontal of side-span cables

$$L_c = \int_0^S (ds/dx)^2\, ds + L_1 \sec^3 \alpha_1 + L_2 \sec^3 \alpha_2$$

$$\approx L + 8a^2 L + L_1 \sec^3 \alpha_1 + L_2 \sec^3 \alpha_2$$

S = length of main-span cable
$a = f/L$

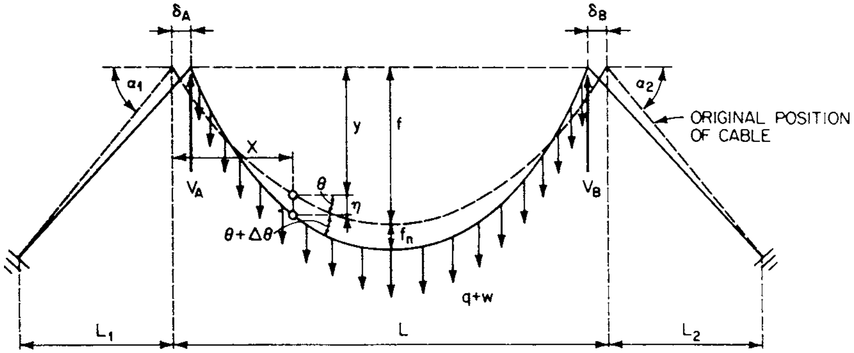


FIGURE 15.44 Forces acting on the truss of Fig. 15.41.

Substitution of Eq. (15.24) in Eq. (15.26) yields the Timoshenko "exact" form of the equation for H_s.

$$\frac{H_s L_c}{A_c E_c} \pm \epsilon_t t L_t - \frac{16f}{L\pi}\left(a_1 + \frac{a_3}{3} + \frac{a_5}{5} + \cdots\right) - \frac{1+\beta}{2+\beta}\left(\frac{\pi^2}{2L}\right)\left[a_1^2 + (2a_2)^2 + (3a_3)^2 + \cdots\right] = 0 \tag{15.27}$$

The last term on the left side of the equation accounts for the actual distribution of live load to the cable. If this term is neglected, the simpler Timoshenko approximate solution is obtained. Direct solution of Eq. (15.27) is possible only by successive approximations.

Successive differentiation of Eq. (15.27) with respect to x yields:

Stiffening-truss angular deflection

$$\phi_x = \eta' = \frac{\pi}{L}\sum_{n=1}^{\infty} n a_n \cos\frac{n\,\pi x}{L} \tag{15.28}$$

Moment

$$M_x = -EI\eta'' = \frac{EI\pi^2}{L^2}\sum_{n=1}^{\infty} n^2 a_n \sin\frac{n\pi x}{L} \tag{15.29}$$

Shear

$$V_x = -EI\eta''' = \frac{EI\pi^3}{L^3}\sum_{n=1}^{\infty} n^3 a_n \cos\frac{n\pi x}{L} \tag{15.30}$$

An alternative form of the equation for H_s, known as the Melan equation, is derived from Eqs. (15.23) and (15.26).

$$H_s^2 \frac{L_c}{A_c E_c} + H_s\left(\frac{H_w L_c}{A_c E_c} \pm \epsilon_t t L_t + \frac{16 f^2}{3L} - \frac{64 f^2 K_2}{L^2}\right) + \left(\frac{pfLK_1}{3} + \epsilon_t t L_t + H_w + \frac{8 fpK_3}{L^2}\right) = 0 \tag{15.31}$$

where $K_1 = k_2^2(4k_2 - 6) - k_1^2(4k_1 - 6)$

$$K_2 = \frac{4 + aL(e^{aL} - e^{-aL}) - 2(e^{aL} - e^{-aL})}{a^3(e^{aL} - e^{-aL})}$$

$$K_3 = \frac{(e^{aL} - 1)(e^{-ak_2L} - e^{-ak_1L}) + (e^{-aL} - 1)(e^{ak_2L} - e^{ak_1L})}{a^3(e^{aL} - e^{-aL})} + \frac{(e^{aL} - e^{-aL})(ak_2L - ak_1L)}{a^3(e^{aL} - e^{-aL})}$$

This form of the equation frequently is useful for determining H_p or H_s directly. Once either has been evaluated, however, the deflections are more readily determined by the series method. Moments are then calculated from

$$M = M_1 - (H_w + H_s)\eta - H_{sy} \tag{15.32}$$

Example. The Ambassador Bridge (Table 15.1), with unloaded backstays and a two-hinged stiffening truss, has the following properties:

$$\begin{array}{ll} f = 205.6 \text{ ft} & A_c = 240.89 \text{ in} \\ L = 1850 \text{ ft} & E_c = 27{,}000 \text{ ksi} \\ L_1 = 984.2 \text{ ft} & E = 30{,}000 \text{ ksi} \\ L_2 = 833.9 \text{ ft} & I = 113.71 \text{ ft}^4 \\ \alpha_1 = 20°32' & \alpha_2 = 24° \end{array}$$

TABLE 15.7 Cable Tension Component H_p, kips

H_p = values from Fig. 15.45, H'_p = check value from Eq. (15.31)

Live loads, kips/ft	$k_1 = 0$ $k_2 = 1$		$k_1 = 0$ $k_2 = 0.5$		$k_1 = 0.25$ $k_2 = 0.75$		$k_1 = 0$ $k_2 = 0.25$		$k_1 = 0.25$ $k_2 = 0.50$		$k_1 = 0.375$ $k_2 = 0.625$	
	H_p	H'_p	H_p	H'_p	H_p	H'_p	H_p	H'_p	H_p	H'_p	H_p	H'_p
0.2	385	386	193	193	269	268	59	59	134	135	144	146*
0.4	770	765	385	383	536	538	117	117	269	268	289	287
0.6	1151	1160*	578	575	803	811*	176	174*	403	403	432	433
0.8	1534	1524	769	770	1069	1079	234	236	537	534	576	576
1.0	1912	1915	961	956	1336	1331	292	293	671	666*	719	720
1.2	2291	2281	1152	1149	1600	1601	351	352	804	804	863	857
1.4	2668	2658	1343	1341	1863	1879	409	413	937	938	1005	1006
1.6	3041	3054	1534	1524*	2127	2139	468	465	1070	1070	1148	1145
1.8	3416	3419	1723	1722	2391	2383	526	526	1203	1203	1291	1286
2.0	3789	3786	1913	1906	2651	2661	584	587	1335	1337	1433	1422
		0.78*		0.65*		1.00*		1.15*		0.75*		1.39*

*Maximum difference, % of H_p.

From the bridge data:

$$w = 6.2 \text{ kips/ft for the east cable}$$

$$H_w = 12{,}920 \text{ kips for the east cable}$$

The structure is analyzed for live loads of 0.2 to 2.0 kips/ft in increments of 0.2. These live loads are placed in various positions: over the entire span, the end half, the center half, the end quarter, the quarter nearest the center, and the center quarter. Analysis is made by both Timoshenko approximate and exact forms of Eq. (15.27).

Approximate Method. Equation (15.27) becomes, when the proper values of the given data are used,

$$H_p = 848.60269a_1 + 282.86755a_3 + 169.72053a_5 \tag{15.33}$$

The coefficients a_1, a_3, and a_5 contain $\beta = H_p/H_w$. So a method of successive approximations must be used. First, a value of H_p or β is assumed. Then, this value is used in Eq. (15.25) to get values of a_1, a_3 and a_5. These, in turn, are substituted in Eq. (15.33) to obtain H_p. This computed value of H_p will not agree with the assumed value unless by accident the correct value of H_p had been guessed.

This procedure is repeated again. Thus, for two assumed values of β, β_1, and β_2 two calculated values of H_p, H_{p1}, and H_{p2} are obtained. On a graph, the straight line $H_p = \beta H_w = 12{,}920\beta$ is drawn (Fig. 15.45), and the points β_1, H_{p1} and $\beta_2 H_{p2}$ plotted. A straight line between these points intersects the line $H_p = 12{,}920\beta$ at the correct value of H and β. (As many as six points were plotted, and always the calculated value of H_p lay on a straight line.)

The preceding procedure was used in calculating 60 values of H_p. Each was checked by finding correct values of a_1, a_3, and a_5 with Eq. (15.31). Values of H_p yielded by both methods are given in Table 15.7. The values of H_p from Fig. 15.45 are the values most nearly correct,

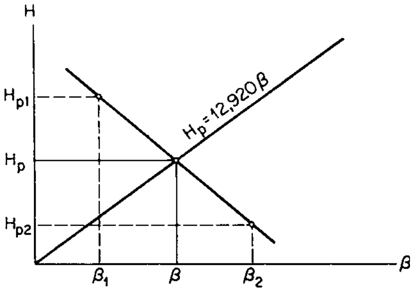


FIGURE 15.45 Chart for linear interpolation of horizontal component H_p of the cable tension.

TABLE 15.8 Deflections, ft, at $x = 0.2L$

Live load, kips/ft	$k_1 = 0$ $k_2 = 1$	$k_1 = 0$ $k_2 = 0.5$	$k_1 = 0.25$ $k_2 = 0.75$	$k_1 = 0$ $k_2 = 0.25$	$k_1 = 0.25$ $k_2 = 0.50$	$k_1 = 0.375$ $k_2 = 0.625$
0.2	0.2744	0.6962	0.0594	0.4122	0.2879	–0.0214
0.4	0.5440	1.3267	0.1229	0.7951	0.5445	–0.0448
0.6	0.8249	2.0758	0.1908	1.1888	0.8622	–0.0646
0.8	1.0842	2.7167	0.2582	1.5838	1.1277	–0.0829
1.0	1.3448	3.2418	0.3170	2.0394	1.3385	–0.1000
1.2	1.6241	3.8637	0.3906	2.3625	1.6768	–0.1211
1.4	1.8926	4.4757	0.4723	2.7514	1.8599	–0.1322
1.6	2.1746	5.0835	0.5435	3.1326	2.1137	–0.1499
1.8	2.4355	5.6719	0.6056	3.5173	2.3668	–0.1634
2.0	2.6870	6.2535	0.6937	4.0276	2.7363	–0.1801

since the check values of H_p often changed several kips when H_p from Fig. 15.45 was changed a few tenths of a kip.

In calculating the deflections, it was found necessary, especially for unsymmetrical loading, to use five terms of Eq. (15.24). Table 15.8 gives deflections, ft. for a typical point.

Timoshenko Exact Method. For the full Eq. (15.27), Table 15.9 gives the values of H_p, kips, for two of the load distributions. The results show that the actual distribution of live load did not increase the value of H_p more than 1%.

(S. O. Asplund, "Structural Mechanics: Classical and Matrix Methods," Prentice-Hall, Inc., Englewood Cliffs, N.J.; E. Egervary, "Bases of a General Theory of Suspension Bridges Using a Matrical Method of Calculation," *International Association of Bridge and Structural Engineers* (*IABSE*) *Publication*, vol. 16, pp. 149–184, 1956; M. Esslinger, "Suspension Bridge Design Calculations by Electronic Computer," *Acier-Stahl-Steel*, no. 5, pp. 223–230, 1962; A. A. Jakkula, "Theory of the Suspension Bridge," *IABSE Publication*, vol. 4, pp. 333–358, 1936; C. P. Kuntz, J. P. Avery, and J. L. Durkee, "Suspension-Bridge Truss Analysis by Electronic Computer," *ASCE Conference on Electronic Computation*, Nov. 20–21, 1958; D. J. Peery, "An Influence-Line Analysis for Suspension Bridges," *ASCE Transactions*, vol. 121, pp. 463–510, 1956; T. J. Poskitt, "Structural Analysis of Suspension Bridges," *ASCE Proceedings*, ST1, February 1966, pp. 49–73; G. C. Priester, "Application of Trigonometric Series to Cable Stress Analysis in Suspension Bridges," *Engineering Research Bulletin* 12, University of Michigan, 1929; A. G. Pugsley, *Theory of Suspension Bridges*, Edward Arnold (Publishers), Ltd., London; A. G. Pugsley, "A Flexibility Coefficient Approach to Suspension

TABLE 15.9 H_p, kips, Obtained by Exact Method [Eq. (15.27)]

Live load, kips/ft	$k_1 = 0$ $k_2 = 0.5$	$k_1 = 0$ $k_2 = 0.25$
0.2	193	59
0.4	386	117
0.6	579	176
0.8	773	235
1.0	966	294
1.2	1,159	353
1.4	1,352	412
1.6	1,545	472
1.8	1,738	531
2.0	1,931	591

Bridge Theory," *Institute of Civil Engineering Proceedings*, vol. 32, 1949; S. A. Saafan, "Theoretical Analysis of Suspension Bridges," *ASCE Proceedings*, ST4, August, 1966, pp. 1–12; S. P. Timoshenko, "The Stiffness of Suspension Bridges," *ASCE Transactions*, vol. 94, pp. 377–405, 1930; S. P. Timoshenko and D. H. Young, *Theory of Structures*, 2d ed., McGraw-Hill, New York.)

15.13 PRELIMINARY SUSPENSION BRIDGE DESIGN

Since suspension bridges are major structures, it is desirable even in preliminary design to proceed into rather detailed refinement of the involved mathematical computations. Often, complete deflection-theory analysis is advisable at that stage. Such refinement is economically feasible with computers. Two procedures for preliminary design are described in the following.

Preliminary design may be started by examining pertinent site factors (clearance requirements, roadway, width, foundation materials, etc.) and studying the details of existing structures of similar proportions and conditions. Table 15.10 gives typical data. Such data should be used with discretion, however, because of major differences in codes with regard to live loads, safety factors, allowable working stresses, and deflections. There also may be significant differences in details, such as roadway structure, which has a major effect on dead loads; as well as different underclearances, lengths of side spans, wind conditions, and other site conditions that influence the weight of steel required. Many published weights per unit area may be misleading because of inclusion of sidewalks, bicycle paths, and other elements in the widths of continental bridges.

Span Ratios. With straight back stays, the ratio of side to main spans may be about 1:4 for economy. For suspended side spans, this ratio may be about 1:2. Physical conditions at the site may, however, dictate the selected span proportions.

Sag. The sag–span ratio is important. It determines the horizontal component of cable force. Also, this ratio affects height of towers, pull on anchorages, and total stiffness of the bridge. For minimum stresses, the ratio should be as large as possible for economy, say 1:8 with suspended side spans, or 1:9 with straight back stays. But the towers may then become high. Several comparative trials should be made. For the Forth Road Bridge, the correct sag–span ratio of 1:11 was thus determined. The general range of this ratio in practice is 1:8 to 1:12, with an average around 1:10.

Truss Depth. Stiffening-truss depths vary from $^1/_{60}$ to $^1/_{70}$ the span. Aerodynamic conditions, however, play a major role in shaping the preliminary design, and some of the criteria given in Art. 15.17 should be studied at this stage.

Other Criteria. Nominal axial stresses in main cables may vary from 80 to 86 ksi. Permissible live-load deflections in practice are seldom specified but usually do not exceed $^1/_{300}$ the span. In Europe, greater reliance is placed on limiting the radius of curvature of the roadway (thus, to 600 or 1000 m); or to limiting the cross slope of the roadway under eccentric load (thus, to about 1%); or to limiting the vertical acceleration under live load (thus, to 0.31 m/s^2).

15.13.1 Preliminary Design by Steinman-Baker Procedure

Analysis by elastic theory is sufficiently accurate for short spans and designs with deep, rigid stiffening systems that limit deflections to small amounts. The simple calculations of elastic theory are also useful, however, for preliminary designs and estimates if tubular percentage corrections are applied, based on experience with the deflection theory.

The corrections depend principally on the magnitude of the dead load and on the flexibility of the structure. The magnitude of the corrections increase with the deflection η and with the horizontal component of cable tension H_w under dead load. They therefore increase with span L and dead load

TABLE 15.10 Details of Major Suspension Bridges*

Item	Verrazano Narrows Bridge	Golden Gate Bridge	Mackinac Bridge	George Washington Bridge	Salazar Bridge (Portugal)	Forth Bridge (Scotland)	Severn Bridge (England–Wales)	Tacoma Narrows Bridge II	San Francisco–Oakland Bay Bridge†	Bronx–Whitestone Bridge	Delaware Memorial Bridge	Walt Whitman Bridge
Length of main span, ft	4,260	4,200	3,800	3,500	3,323	3,300	3,240	2,800	2,310	2,300	2,150	2,000
Length of each side span, ft	1,215	1,125	1,800	610,650	1,586	1,340	1,000	1,100	1,160	735	750	770
Length of suspended structure, ft,	6,690	6,450	7,4000	4,760	6,495	5,980	5,240	5,000	10,450	3,770	3,650	3,540
Length including approach structure, ft	13,700	8,981	19,205	5,800	10,575	8,244	7,640	5,979	43,500	7,995	10,750	11,687
Width of bridge (c to c cables), ft	103	90	68	106	77	78	75	60	66	74	57	79
Number of traffic lanes	12	6	4	14‡	4	4	4	4	9	6	4	7
Height of towers above MHW, ft	690	746	552	595	625	512	470	500	447	377	440	378
Clearance at center above MHW, ft	228	215	148	220‡	246	150	120	187	203	150	175	150
Deepest foundation below MHW, ft	170	115	210	75	260	106	75	224	235	165	115	107
Diameter of cable, in	$35\frac{7}{8}$	36	$24\frac{1}{2}$	36	$23\frac{1}{16}$	24	29	$20\frac{1}{4}$	$28\frac{3}{4}$	22	$19\frac{3}{4}$	$23\frac{1}{8}$
Length of one cable, ft	7,205	7,650	8,683	5,235	7,899	7,000	5,600	5,500	5,080	4,166	4,015	3,845
Number of wires per cable	26,108	27,572	12,580	26,474	11,248	11,618	8,300	8,702	17,464	9,842	8,284	11,396
Total length of wire used, mi	142,500	80,000	41,000	105,000	33,600	30,800	18,000	20,000	70,800	14,800	12,600	16,600
Year of completion	1964	1937	1957	1931	1966	1964	1966	1950	1936	1939	1951	1957

*Courtesy of *Engineering News-Record*.

†Twin spans.

‡With new lower deck.

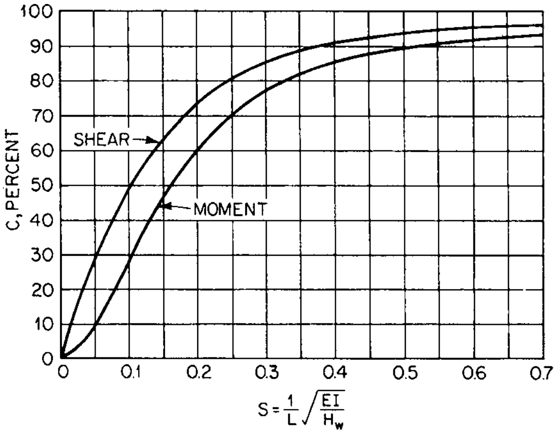
w, while decreasing with truss stiffness *EI* and cable sag *f*. D. B. Steinman expressed this in a simple parameter *S*, the stiffness factor, such that

$$S = \frac{1}{L}\sqrt{\frac{EI}{H_w}} = \frac{1}{L^2}\sqrt{\frac{8fEI}{w}} \tag{15.34}$$

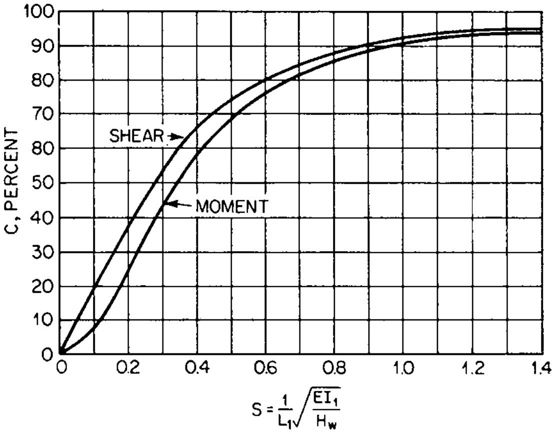
This value is used in the Steinman-Baker charts, Fig. 15.46, to obtain the percentage *C* to be applied to elastic-theory shears and moments to get deflection-theory shears and moments.

Roughly, the percentage reductions from the approximate theory are proportional to 1/*S*, which might be called a flexibility factor; that is, the magnitude of the reduction increases considerably with long spans and heavy dead load, and diminishes with stiffness and sag.

The Steinman-Baker charts are based on the following proportions: side span, one-half the main span; sag–span ratio, 0.1: moment of inertia *I*, constant; cable design stress, 80 ksi; modulus of



(a) FOR MAIN SPAN



(b) FOR SIDE SPAN

FIGURE 15.46 Steinman-Baker correction curves for stresses obtained by elastic theory for suspension bridges. (*Reprinted with permission from D. B. Steinman, "A Practical Treatise on Suspension Bridges," 2d ed., John Wiley & Sons, Inc., New York.*)

elasticity *E*, 29,000 ksi; ratio of dead to live load, 3. Further refinement of *C* for other proportions is suggested as follows:

For unloaded side spans, increase *C* by 2½% of its value.

For sag–span ratio = 0.12 (or 0.08) instead of 0.10, decrease (or increase) *C* by 2% of its value.

For cable stress = 120 (or 40) instead of 80 ksi, increase (or decrease) *C* by ½% of its value.

For $I/I_1 = 0.75$ instead of 1.00, increase *C* by 1½% of its value.

For $L_1/L = 0.25$ instead of 0.50, increase *C* by 2% of its value.

For load ratio $w/p = 5$ instead of 3, add 1% to *C*; for $w/p = 2$, subtract 1% from *C*; for $w/p = 1½$, subtract 2% from C.

In elastic-theory analysis for preliminary design of a bridge with two cable planes, the bridge may be treated as plane frameworks loaded in each plane; that is, the action as a space structure may be disregarded. The alleviating effects of torsion of the stiffening girders, of the unloading action of the cross frames or diaphragms between the girders, and of the participation of the connection of the pylons may all be left for more refined later analysis.

15.13.2 Preliminary Design by Hardesty-Wessman Procedure

Hardesty and Wessman presented an approximate, partly empirical, preliminary design method based on the distortion of an unstiffened cable. The maximum moments at the quarter point and at the center of the main span at constant mean temperature are computed in two major steps:

1. The deflections η' of an unstiffened cable under partial live load, for various ratios of live to dead load *p*/*w*, are obtained from Fig. 15.47. These charts were developed with average live-load lengths from a study of bridges in service and also based on the assumptions that the cable length is unchanged and the pylon tops do not move.
2. Corrections then are made for the effect of adding the stiffening truss (which reduces deflection η'). A trial moment of inertia is used and corrected later if necessary. Equations (15.35) are used for a first estimate of the maximum horizontal components of cable tension $H_w + H_p$. Then, Eqs. (15.36) are used to determine the bending moment M_t induced in the stiffening truss when it is bent to the deflections η' of the unstiffened cable.

Maximum positive moment at a quarter point, with live load at the same end over a length of 0.4*L*, where *L* = main span, ft, is

$$H_w + H_p = \frac{1}{f + n'_c}(0.125wL^2 + 0.040pL^2) \tag{15.35a}$$

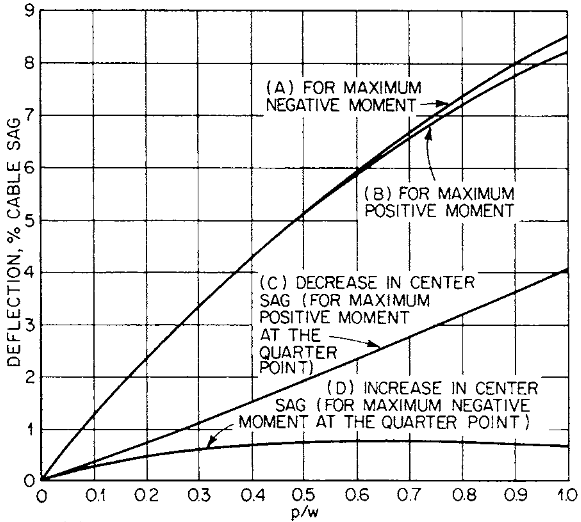
where f = cable sag, ft
η'_c = deflection at center of unstiffened cable, ft
w = uniform dead load
p = uniform live load

Maximum negative moment at a quarter point, with live load at the opposite end over a length of 0.6*L*, is

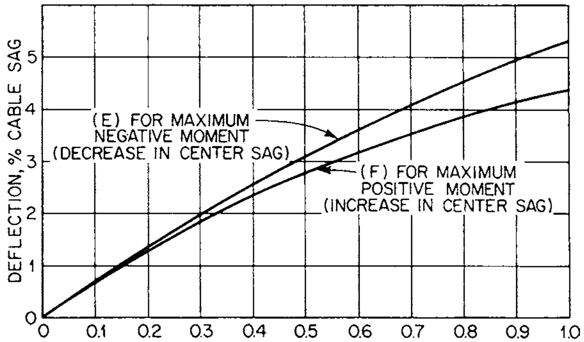
$$H_w + H_p = \frac{1}{f + \eta'_c}(0.125wL^2 + 0.085pL^2) \tag{15.35b}$$

Maximum positive moment at the center, with live load at the center over a length of 0.3*L*, is

$$H_w + H_p = \frac{1}{f + \eta'_c}(0.125wL^2 + 0.0638pL^2) \tag{15.35c}$$



(a) FOR MAXIMUM DEFLECTION AT QUARTER POINT OF SPAN



(b) FOR MAXIMUM DEFLECTION AT CENTER OF SPAN

FIGURE 15.47 Chart gives deflections of unstiffened cables under partial load. (*Reprinted with permission from S. Hardesty and H. E. Wessman, "Preliminary Design of Suspension Bridges,"* ASCE Transactions, *vol. 104, 1939.*)

Maximum negative moment at the center, with live load over a length of $0.35L$ at each end, is

$$H_w + H_p = \frac{1}{f + \eta_c'}(0.125wL^2 + 0.0613pL^2) \tag{15.35d}$$

In each of the preceding equations, the quantity within the parentheses is the bending moment at the center of a simple span due to dead load over the entire span and live load over a part of the span. The deflection η_c' is positive when downward and negative when upward.

Since $H_w = wL^2/8f$ is known, Eqs. (15.35) yield a trial value of H_p, with which the following bending moments in the truss can be computed:

Maximum positive moment at quarter-points:

$$M_t = 47.0\,\frac{EI\eta'}{L^2} \tag{15.36a}$$

where I = moment of inertia of stiffening truss and E = modulus of elasticity of truss steel. Maximum negative moment at quarter points:

Maximum negative moment at quarter-points:

$$M_t = 43.0\,\frac{EI\eta'}{L^2} \tag{15.36b}$$

Maximum positive moment at center:

$$M_t = 65.8\,\frac{EI\eta'}{L^2} \tag{15.36c}$$

Maximum negative moment at center:

$$M_t = 59.2\,\frac{EI\eta'}{L^2} \tag{15.36d}$$

Since the truss is neither infinitely flexible nor infinitely stiff, the cable will be forced back only a part of the distance η'. The moment in the truss is reduced from M_t to

$$M = M_t\,\frac{(H_w + H_p)\eta'}{M_t + (H_w + H_p)\eta'} \tag{15.37}$$

And the deflection is reduced to

$$\eta = \eta'\,\frac{(H_w + H_p)\eta'}{M_t + (H_w + H_p)\eta'} \tag{15.38}$$

Finally, changes in length of cable due to live load or temperature, and the sag changes caused by movement of pylon tops, cable stress, and temperature, are combined in one change in the center sag. These effects are

Change in length of cable due to stress:

Main span:
$$\Delta L_s = \frac{H_p L}{A_c E_c}\left(1 + {}^{16}\!/_{3}a^2\right) \tag{15.39}$$

where $a = f/L$
A_c = cross-sectional area of cable
E_c = modulus of elasticity of cable steel

Unloaded side span:
$$\Delta L_{1s} = \frac{H_p L_1}{A_c E_c}\sec^2 \alpha_1 \tag{15.40}$$

where α_1 = angle side-span cable makes with horizontal.

Change in length of cable due to temperature:

Main span:
$$\Delta L_t = \epsilon_t\,\Delta t L\left(1 + {}^{8}\!/_{3}a^2\right) \tag{15.41}$$

where ϵ = coefficient of expansion and Δt = temperature change.

Unloaded side span: $$\Delta L_{1t} = \epsilon_t\ \Delta t L_1 \sec \alpha_1 \qquad (15.42)$$

Change in sag in main span due to temperature:

$$\Delta f = \frac{15}{16a(5-24a^2)}\Delta L \qquad (15.43)$$

Change in sag due to movement of pylon top:

$$\Delta f = \frac{15-40a^2+288a^4}{16a(5-24a^2)}(2\Delta L_1 \sec \alpha_1) \qquad (15.44)$$

Moment caused by change in sag:

Main span at center: $$M = 9.4\frac{EI}{L^2}\Delta f \qquad (15.45)$$

Main span at quarter-point: $$M = 7.6\frac{EI}{L^2}\Delta f \qquad (15.46)$$

The corrected value of the sag allows a second trial value of H_p to be obtained. Then, the process is repeated.

In applying this method to preliminary design, an arbitrary moment of inertia is selected, based on a tentative chord section and truss depth. The procedure is repeated with other values of I, say one-half and double those in the first analysis. Final selection may be based on limiting values of desired deflection and grade change due to load.

(D. B. Steinman, *A Practical Treatise on Suspension Bridges*, 2d ed., John Wiley & Sons, Inc., New York; S. Hardesty and H. E. Wessman, "Preliminary Design of Suspension Bridges," *Transactions of the American Society of Civil Engineers*, vol. 104, 1939.)

15.14 SELF-ANCHORED SUSPENSION BRIDGES

Self-anchored suspension bridges differ from the type discussed in Arts. 15.12 and 15.13 only in that external anchorages are dispensed with (see Art. 15.3). Unlike the externally anchored type, self-anchored suspension bridges may properly be analyzed by the elastic theory, since the effect of distortions of the structural geometry under live load is practically eliminated. The structure is also not stressed by uniform temperature change of cables and stiffening girders. The analysis is thus simpler. But the favorable reductions of bending moments that occur with externally anchored suspension bridges are lost. Furthermore, the effect of axial load in the stiffening girder must be considered, as well as the effect of girder camber.

For a symmetrical three-span structure with continuous stiffening girders (Fig. 15.48), a plane system (cable, suspenders, and girder) has three redundants. C. H. Gronquist derived in simple form the elastic-theory equations for determining the redundants for a continuous stiffening-truss system. He took into account camber and its action in reducing cable and truss stress by archlike action.

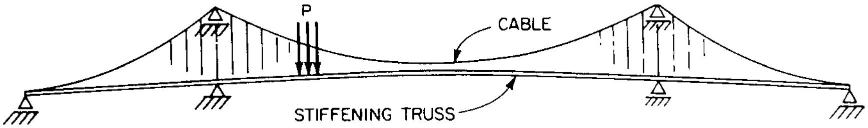


FIGURE 15.48 Self-anchored suspension bridge.

He also demonstrated that the equations for the horizontal component of cable tension H_p under live load for the self-anchored bridge, with girder camber and shortening eliminated, are the same as the elastic-theory equations for an externally anchored suspension bridge.

(C. H. Gronquist, "Simplified Theory of the Self-Anchored Suspension Bridge," *Transactions of the American Society of Civil Engineers*, vol. 107, 1942.)

15.15 CABLE-STAYED BRIDGE ANALYSIS

The static behavior of a cable-stayed girder can best be gauged from the simple, two-span example of Fig. 15.49. The girder is supported by one stay cable in each span, at *E* and *F*, and the pylon is fixed to the girder at the center support *B*. The static system has two internal cable redundants and one external support redundant.

If the cable and pylon were infinitely rigid, the structure would behave as a continuous four-span beam *AC* on five rigid supports *A*, *E*, *B*, *F*, and *C*. The cables are elastic, however, and correspond to springs. The pylon also is elastic, but much stiffer because of its large cross section. If cable stiffness is reduced to zero, the girder assumes the shape of a deflected two-span beam *ABC*.

Cable-stayed bridges of the nineteenth century differed from those of the 1960s in that their stays constituted relatively soft spring supports. Heavy and long, the stays could not be highly stressed. Usually, the cables were installed with significant slack, or sag. Consequently, large deflections occurred under live load as the sag decreased. In contrast, modern cables are made of high-strength steel, are relatively short and taut, and have low weight. Their elastic action may therefore be considered linear, and an equivalent modulus of elasticity may be used [Eq. (15.1)]. The action of such cables then produces something more nearly like a four-span beam for a structure such as the one in Fig. 15.49.

If the pylon were hinged at its base connection with a stayed girder at *B*, rather than fixed, the pylon would act as a pendulum column. This would have an important effect on the stiffness of the system, for the spring support at *E* would become more flexible. The resulting girder deflection might exceed that due to the elastic stretch of the cables. In contrast, the elastic shortening of the pylon has no appreciable effect.

Relative girder stiffness plays a dominant role in the structural action. The stayed girder tends to approach a beam on rigid supports, *A*, *E*, *B*, *F*, *C* as girder stiffness decreases toward zero. With increasing girder stiffness, however, the support of the cables diminishes, and the bridge approaches a girder supported on its piers and abutment, *A*, *B*, *C*.

In a three-span bridge, a side-span cable connected to the abutment furnishes more rigid support to the main span than does a cable attached to some point in the side span. In Fig. 15.49, for example, the support of the load *P* in the position shown would be improved if the cable attachment at *F* were shifted to *C*. This explains why cables from the pylon top to the abutment are structurally more efficient, though not as esthetically pleasing as other arrangements.

The stiffness of the system also depends on whether the cables are fixed at the towers (at *D*, for example, in Fig. 15.49) or whether they run continuously over (or through) the pylons. Some early designs with more than one cable to a pylon from the main span required one of the cables to be fixed to the pylon and the others to be on movable saddle supports. Most contemporary designs fix all the stays to the pylon.

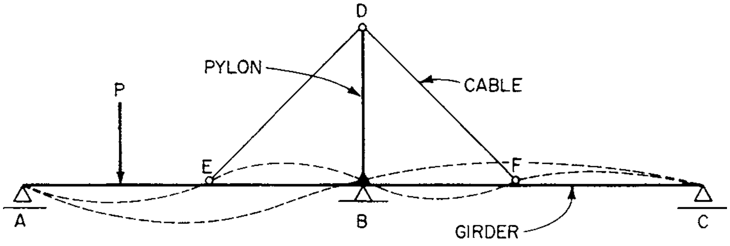


FIGURE 15.49 Dashed lines indicate deflected positions of a cable-stayed girder.

The curves of maximum-minimum girder moments for all load variations usually show a large range of stress. Designs providing for the corresponding normal forces in the girder may require large variations in cross sections. By prestressing the cables or by raising or lowering the support points, it is possible to achieve a more uniform and economical moment capacity. The amount of prestressing to use for this purpose may be calculated by successively applying a unit force in each of the cables and drawing the respective moment diagrams. Then, by trial, the proper multiples of each force are determined so that, when their moments are superimposed on the maximum-minimum moment diagrams, an optimum balance results.

("Guidelines for the Design of Cable-Stayed Bridges," Committee on Cable-Stayed Bridges, American Society of Civil Engineers.)

15.15.1 Static Analysis—Elastic Theory

Cable-stayed bridges may be analyzed by the general method of indeterminate analysis with the equations of virtual work.

The degree of internal redundancy of the system depends on the number of cables, types of connections (fixed or movable) of cables with the pylons, and the nature of the pylon connection at its base with the stayed girder or pier. The girder is usually made continuous over three spans. Figure 15.50 shows the order of redundancy for various single-plane systems of cables.

If the bridge has two planes of cables, two stayed girders, and double pylons, it usually also must be provided with a number of intermediate cross diaphragms in the floor system, each of which is capable of transmitting moment and shear. The bridge may also have cross girders across the top of the pylons. Each of these cross members adds two redundants, to which must be added twice internal redundancy of the single-plane structure, and any additional reactions in excess of those needed for external equilibrium as a space structure. The redundancy of the space structure is very high, usually of the order of 40 to 60. Therefore, the methods of plane statics are normally used, except for large structures.

For a cable-stayed structure such as that illustrated in Fig. 15.51*a*, it is convenient to select as redundants the bending moments in the stayed girder at those points where the cables and pylons support the girder. When these redundants are set equal to zero, an articulated, statistically determinate base system is obtained, Fig. 15.51*b*. When the loads are applied to this choice of base system, the stresses in the cables do not differ greatly from their final values; so the cables may be dimensioned in a preliminary way.

Other approaches are also possible. One is to use the continuous girder itself as a statically indeterminate base system, with the cable forces as redundants. But computation is generally increased.

A third method involves imposition of hinges, for example at *a* and *b* (Fig. 15.52), so placed as to form two coupled symmetrical base systems, each statically indeterminate to the fourth degree. The influence lines for the four indeterminate cable forces of each partial base system are at the same time also the influence lines of the cable forces in the real system. The two redundant moments X_a and X_b are treated as symmetrical and antisymmetrical group loads, $Y = X_a + X_b$ and $Z = X_a - X_b$, to calculate influence lines for the 10-degree indeterminate structure shown. Kern moments are plotted to determine maximum effects of combined bending and axial forces.

A similar concept is illustrated in Fig. 15.53, which shows the application of independent symmetric and antisymmetric group stress relationships to simplify calculations for an 8-degree indeterminate system. Thus, the first redundant group X_1 is the self-stressing of the lowest cables in tension to produce $M_1 = +1$ at supports.

The above procedures also apply to influence-line determinations. Typical influence lines for two bridge types are shown in Fig. 15.54. These demonstrate that the fixed cables have a favorable effect on the girders but induce sizable bending moments in the pylons, as well as differential forces on the saddle bearings.

Note also that the radiating system in Fig. 15.50*c* and *d* generally has more favorable bending moments for long spans than does the harp system of Fig. 15.54. Cable stresses also are somewhat lower for the radiating system, because the steeper cables are more effective. But the concentration of cable forces at the top of the pylon introduces detailing and construction difficulties. When viewed at an angle, the radiating system presents esthetic problems, because of the different intersection

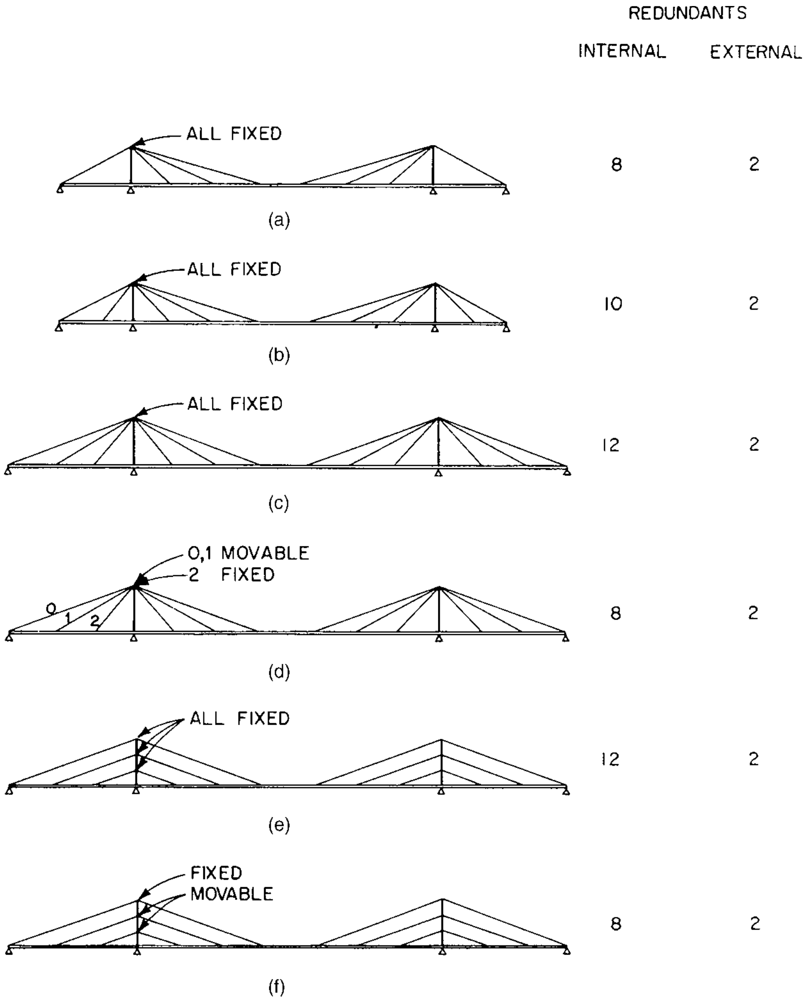


FIGURE 15.50 Number of internal and external redundants for various types of cable-stayed bridges.

angles when the cables are in two planes. Furthermore, fixity of the cables at pylons with the radiating system in Fig. 15.50*c* and *d* produces a wider range of stress than does a movable arrangement. This can adversely influence design for fatigue.

A typical maximum-minimum moment and axial-force diagram for a harp bridge is shown in Fig. 15.55.

The secondary effect of creep of cables (Art. 15.7) can be incorporated into the analysis. The analogy of a beam on elastic supports is changed thereby to that of a beam on linear viscoelastic supports. Better stiffness against creep for cable-stayed bridges than for comparable suspension bridges has been reported. (K. Moser, "Time-Dependent Response of the Suspension and Cable-Stayed Bridges," International Association of Bridge and Structural Engineers, 8th Congress Final Report, 1968, pp. 119–129.)

(W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York.)

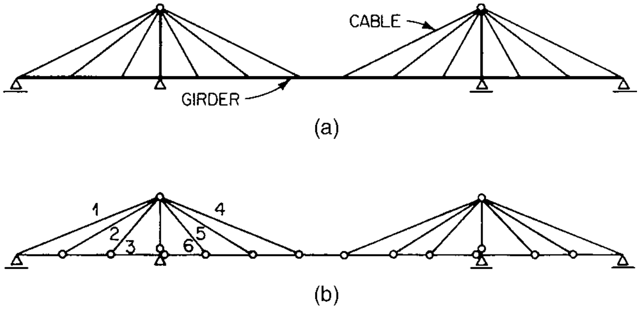


FIGURE 15.51 Cable-stayed bridge with three spans. (*a*) Girder is continuous over the three spans. (*b*) Insertion of hinges in the girder at cable attachments makes system statically determinate.

15.15.2 Static Analysis—Deflection Theory

Distortion of the structural geometry of a cable-stayed bridge under action of loads is considerably less than in comparable suspension bridges. The influence on stresses of distortion of stayed girders is relatively small. In any case, the effect of distortion is to increase stresses, as in arches, rather than the reverse, as in suspension bridges. This effect for the Severn Bridge is 6% for the stayed girder and less than 1% for the cables. Similarly, for the Düsseldorf North Bridge, stress increase due to distortion amounts to 12% for the girders.

The calculations, therefore, most expeditiously take the form of a series of successive corrections to results from first-order theory (Art. 15.15.1). The magnitude of vertical and horizontal displacements of the girder and pylons can be calculated from the first-order theory results. If the cable stress is assumed constant, the vertical and horizontal cable components V and H change by magnitudes ΔV and ΔH by virtue of the new deformed geometry. The first approximate correction determines the effects of these ΔV and ΔH forces on the deformed system, as well as the effects of V and H due to the changed geometry. This process is repeated until convergence, which is fairly rapid.

15.16 PRELIMINARY DESIGN OF CABLE-STAYED BRIDGES

In general, the height of the pylon above deck level in a cable-stayed bridge is about $^1/_5$ to $^1/_6$ the main span. This results in the flattest main span stay having a horizontal angle of 22° to 23°. As stays become flatter than this value, stay efficiency and girder stiffness decline. Depth of stayed girders range from $^1/_{60}$ to $^1/_{80}$ of the main span for box-girder designs, usually 8 to 14 ft. Composite-girder designs have more closely spaced stays, and are typically $^1/_{150}$ to $^1/_{200}$ of the main span, usually 5 to 8 ft in depth.

Span ratios for the conventional three-span cable-stayed structure vary according to use. For rail bridges, a back-span to main-span ratio of 0.40 to 0.42 results in a concentration of back stays (those connected to the rigid back-span pier) to counter the heavy uniform live load in the main span, which gives rise to main-span deflection and back-stay stress range. For highway bridges, back-span to

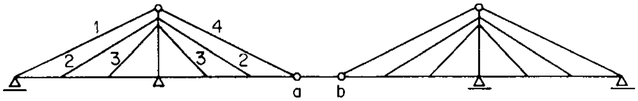


FIGURE 15.52 Hinges at *a* and *b* reduce the number of redundants for a cable-stayed girder continuous over three spans.

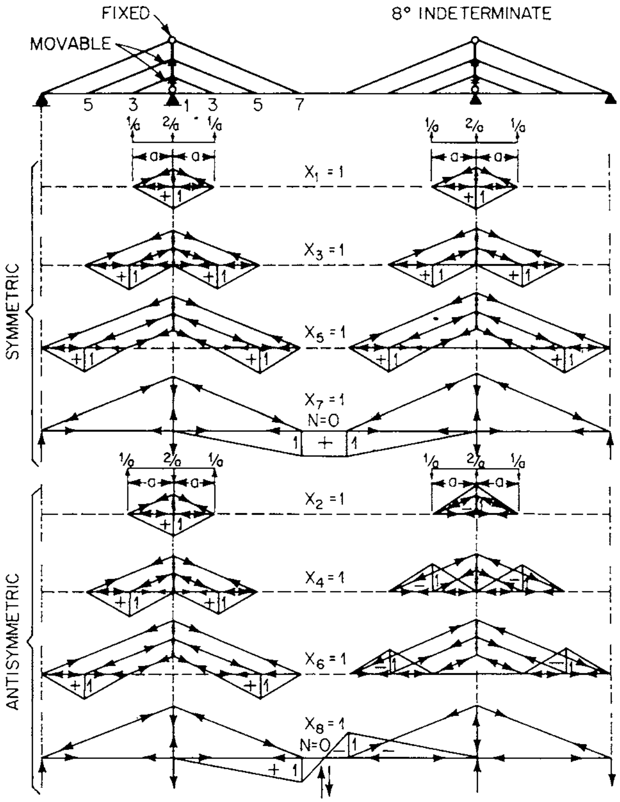


FIGURE 15.53 Forces induced in a cable-stayed bridge by independent symmetric and antisymmetric group loadings. (*Reprinted with permission from O. Braun, "Neues zur Berchnung Statisch Unbestimmter Tragwerke," Stahlbau, vol. 25, 1956.*)

main-span ratios of 0.45 to 0.48 have been used for contemporary composite bridges, and lesser ratios for more classical box girders with fewer stay cables.

Wide box girders are mandatory as stayed girders for single-plane systems, to resist the torsion of eccentric loads. Box girders, even narrow ones, are also desirable for double-plane systems to enable cable connections to be made without eccentricity. Single-web girders, however, if properly braced, may be used.

Since elastic-theory calculations are relatively simple to program for a computer, a formal set may be made for preliminary design after the general structure and components have been sized.

Manual Preliminary Calculations for Cable Stays. Following is a description of a method of manual calculation of reasonable initial values for use as input data for design of a cable-stayed bridge by computer. The manual procedure is not precise but does provide first-trial cable-stay areas. With the analogy of a continuous, elastically supported beam, influence lines for stay forces and bending moments in the stayed girder can be readily determined. From the results, stress variations in the stays and the girder resulting from concentrated loads can be estimated.

If the dead-load cable forces reduce deformations in the girder and pylon at supports to zero, the girder acts as a beam continuous over rigid supports, and the reactions can be computed for

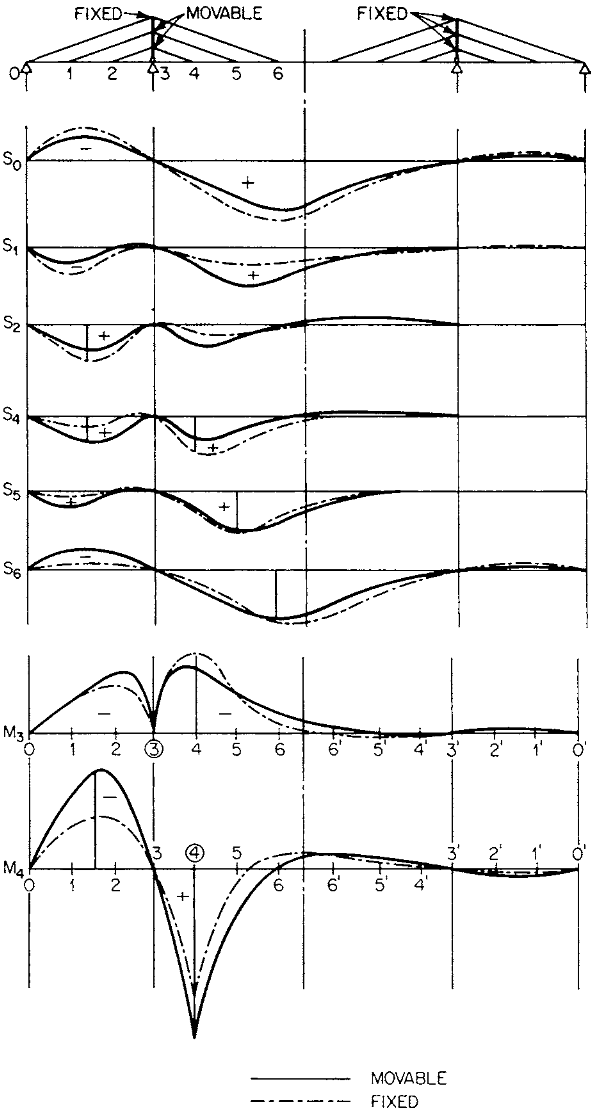


FIGURE 15.54 Typical influence lines for a three-span cable-stayed bridge showing the effects of fixity of cables at the pylons. (*Reprinted with permission from H. Homberg, "Einflusslinien von Schrägseilbruchen," Stahlbau, vol. 24, no. 2, 1995.*)

the continuous beam. Inasmuch as the reactions at those supports equal the vertical components of the stays, the dead-load forces in the stays can be readily calculated. If, in a first-trial approximation, live load is applied to the same system, the forces in the stays (Fig. 15.56) under the total load can be computed from

$$P_i = \frac{R_i}{\sin \alpha_i} \tag{15.47}$$

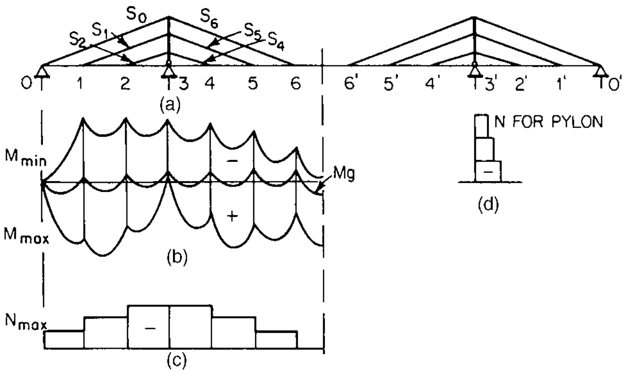


FIGURE 15.55 Typical moment and force diagrams for a cable-stayed bridge. (*a*) Girder is continuous over three spans. (*b*) Maximum and minimum bending moments in the girder. (*c*) Compressive axial forces in the girder. (*d*) Compressive axial forces in a pylon.

where R_i = sum of dead-load and live-load reactions at i and α_i = angle between girder and stay i. Since stay cables usually are designed for service loads, the cross-sectional area of stay i may be determined from

$$A_i = \frac{R_i}{\sigma_a \sin \alpha_i} \tag{15.48}$$

where σ_a = allowable unit stress for the cable steel.

The allowable unit stress for service loads equals $0.45 f_{pu}$, where f_{pu} = the specified minimum tensile strength, ksi, of the steel. For 0.6-in-diameter, seven-wire prestressing strand (ASTM A416), f_{pu} = 270 ksi, and for 1/4-in-diameter ASTM A421 wire, f_{pu} = 240 ksi. Therefore, the allowable stress is 121.5 ksi for strand and 108 ksi for wire.

The reactions may be taken as $R_i = ws$, where w is the uniform load, kips/ft, and s, the distance between stays. At the ends of the girder, however, R_i may have to be determined by other means.

Determination of the force P_o acting on the back-stay cable connected to the abutment (Fig. 15.57) requires that the horizontal force F_h at the top of the pylon be computed first. Maximum force on that cable occurs with dead plus live loads on the center span and dead load only on the side span. If the

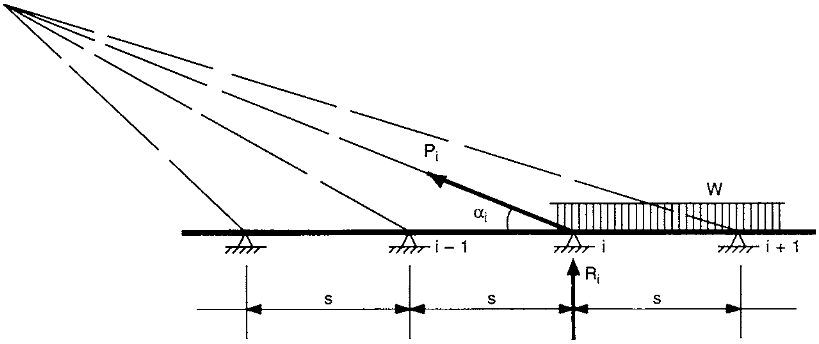


FIGURE 15.56 Cable-stayed girder is supported by cable force P_i at ith point of cable attachment. R_i is the vertical component of P_i.

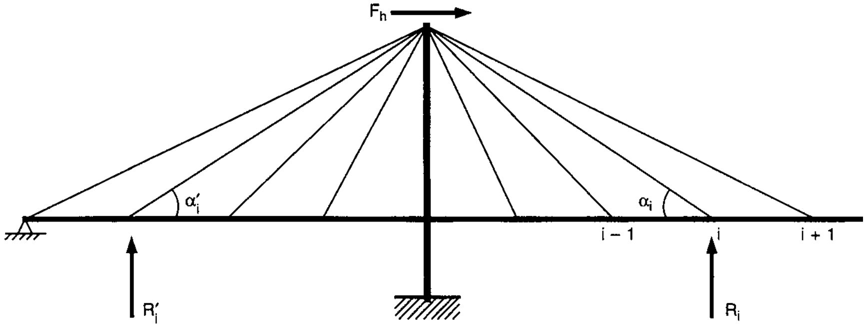


FIGURE 15.57 Cables induce a horizontal force F_h at the top of a pylon.

pylon top is assumed immovable, F_h can be determined from the sum of the forces from all the stays, except the back stay:

$$F_h = \sum \frac{R_i}{\tan \alpha_i} - \sum \frac{R_i'}{\tan \alpha_i'} \tag{15.49}$$

where R_i, R_i' = vertical component of force in the ith stay in the main span and side span, respectively

α_i, α_i' = angle between girder and the ith stay in the main span and side span, respectively

Figure 15.58 shows only the pylon and back-stay cable to the abutment. If, in Fig. 15.58, the change in the angle α_o is assumed to be negligible as F_h deflects the pylon top, the load in the back stay can be determined from

$$P_o = \frac{F_h h_t^3 \cos \alpha_o}{3l_o(E_c I/E_s A_s) + h_t^3 \cos^2 \alpha_o} \tag{15.50}$$

If the bending stiffness $E_c I$ of the pylon is neglected, then the back-stay force is given by

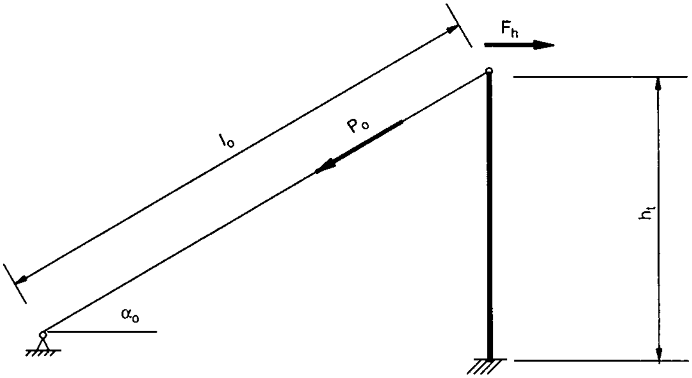


FIGURE 15.58 Cable force P_o in back stay to anchorage and bending stresses in the pylon resist horizontal force F_h at the top of the pylon.

$$P_o = \frac{F_h}{\cos \alpha_o} \tag{15.51}$$

where h_t = height of pylon
l_o = length of back stay
E_c = modulus of elasticity of pylon material
I = moment of inertia of pylon cross section
E_s = modulus of elasticity of cable steel
A_s = cross-sectional area of back-stay cable

For the structure illustrated in Fig. 15.59, values were computed for a few stays from Eqs. (15.47), (15.48), (15.49), and (15.51) and tabulated in Table 15.11*a*. Values for the final design, obtained by computer, are tabulated in Table 15.11*b*.

Inasmuch as cable stays 1, 2, and 3 in Fig. 15.59 are anchored at either side of the anchor pier, they are combined into a single back-stay for purposes of manual calculations. The edge girders of the deck at the anchor pier were deepened in the actual design, but this increase in dead weight was ignored in the manual solution. Further, the simplified manual solution does not take into account other load cases, such as temperature, shrinkage, and creep.

Influence lines for stay forces and girder moments are determined by treating the girder as a continuous, elastically supported beam. From Fig. 15.60, the following relationships are obtained for a unit force at the connection of girder and stay:

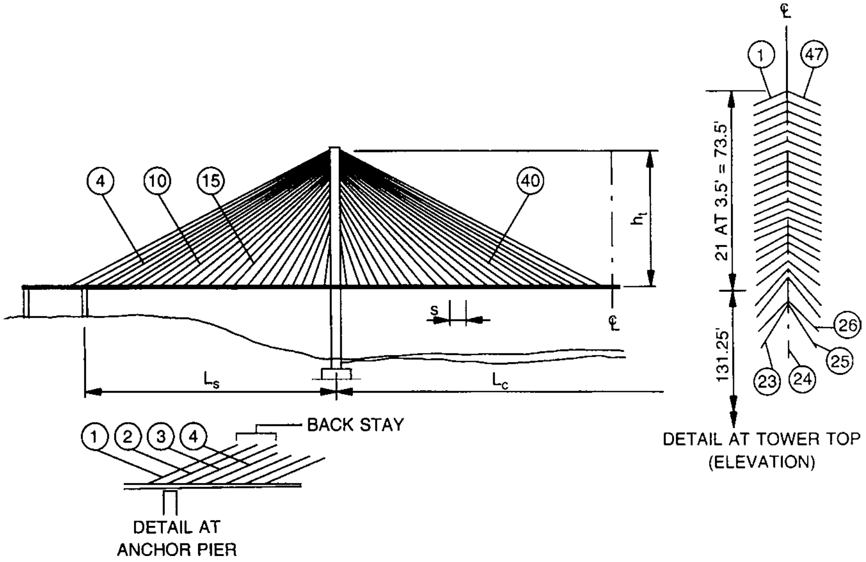


FIGURE 15.59 Half of a three-span cable-stayed bridge. Properties of components are as follows:

Girder		Tower	
Main span L_c	940 ft	Height h_d	204.75 ft
Side span L_b	440 ft	Area A	120 ft^2
Stay spacing s	20 ft	Moment of inertia I	3620 ft^4
Area A	101.4 ft	Elastic modulus E_t	45,000 ksi
Moment of inertia I	48.3 ft^4		
Elastic modulus E_g	47,000 ksi	Stays	
		Elastic modulus E_s	28,000 ksi

(*Reprinted with permission from W. Podolny, Jr., and J. B. Scalzi, "Construction and Design of Cable-Stayed Bridges," 2d ed., John Wiley & Sons, Inc. New York.*)

TABLE 15.11 Comparison of Manual and Computer Solutions for the Stays in Fig. 15.59

	(*a*) According to Eqs. (15.47), (15.48), (15.49), and (15.51)					(*b*) Computer solution			
Stay number	R_{DL}, kips	P_{DL}, kips	R_{DL+LL}, kips	P_{DL+LL}, kips	A, in^2	P_{DL}, kips	P_{DL+LL}, kips[†]	Number of 0.6-in strands[‡]	Strand area, in^2 [‡]
Back stay*	—	2596	—	3969	32.667	2775	3579	136	29.512
4	360	824	400	916	7.539	851	1049	40	8.680
10	360	684	400	760	6.255	695	797	31	6.727
15	360	550	400	611	5.029	558	654	25	5.425
40	360	734	400	815	6.708	756	878	34	7.378

*Stays 1, 2, and 3 combined into one back stay.

[†] Maximum live load.

[‡] Per plane of a two-plane structure.

Source: Reprinted with permission from W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York.

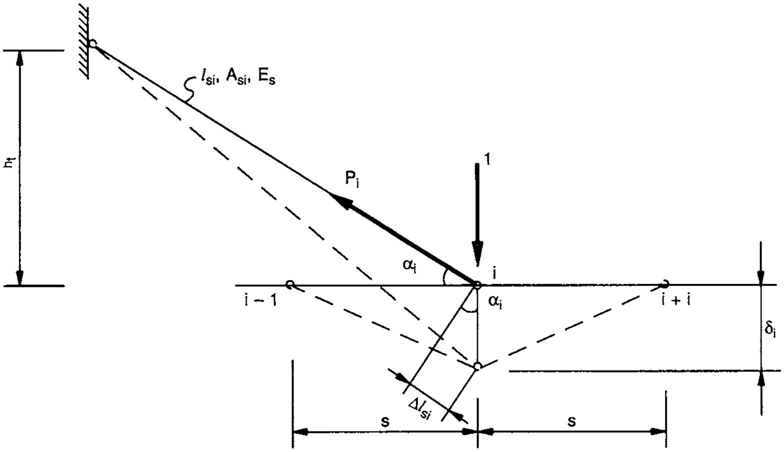


FIGURE 15.60 Unit force applied at point of attachment of *i*th cable stay to girder for determination of spring stiffness.

$$P_i = \frac{1}{\sin \alpha_i} \qquad \Delta I_{si} = \frac{P_i\, l_{si}}{A_{si}E_s} = \delta_i \sin \alpha_i$$

which lead to

$$\delta_i = \frac{l_{si}}{A_{si}E_s \sin^2 \alpha_i}$$

With Eq. (15.48) and $l_{si} = h_t \sin \alpha_i$, the deflection at point i is given by

$$\delta_i = \frac{h_t\, \sigma_a}{R_i E_s \sin^2 \alpha_i} \tag{15.52}$$

With R_i taken as $s(w_{DL} + w_{LL})$, the product of the uniform dead and live loads and the stay spacing s, the spring stiffness of cable stay i is obtained as

$$k_i = \frac{1}{\delta_i s} = \frac{(w_{DL} + w_{LL})E_s \sin^2 \alpha_i}{h_t \sigma_a} \tag{15.53}$$

For a vertical unit force applied on the girder at a distance x from the girder-stay connection, the equation for the cable force P_i becomes

$$P_i = \frac{\xi W s}{2 \sin \alpha_i} \eta_p \tag{15.54}$$

where $\eta_p = e^{-\xi x}(\cos\ \xi x + \sin\ \xi x)$,

$$\xi = \sqrt[4]{\frac{k_i}{4E_c I}} \tag{15.55}$$

The bending moment M_i at point i may be computed from

$$M_i = \frac{W}{4\xi} e^{-\xi x}(\cos \xi x - \sin \xi x) = \frac{W}{4\xi} \eta_m \tag{15.56}$$

where $\eta_m = e^{-\xi x}(\cos \xi x + \sin \xi x)$.

(W. Podolny, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York.)

15.17 AERODYNAMIC ANALYSIS OF CABLE-SUSPENDED BRIDGES

The wind-induced failure on November 7, 1940, of the Tacoma Narrows Bridge in the state of Washington shocked the engineering profession. Many were surprised to learn that failure of bridges as a result of wind action was not unprecedented. During the slightly more than 12 decades prior to the Tacoma Narrows failure, 10 other bridges were severely damaged or destroyed by wind action (Table 15.12). As can be seen from Table 15.12*a*, wind-induced failures have occurred in bridges with spans as short as 245 ft up to 2800 ft. Other "modern" cable-suspended bridges have been observed to have undesirable oscillations due to wind (Table 15.12*b*).

15.17.1 Required Information on Wind at Bridge Site

Prior to undertaking any studies of wind instability for a bridge, engineers should investigate the wind environment at the site of the structure. Required information includes the character of strong

TABLE 15.12 Long-Span Bridges Adversely Affected by Wind

(*a*) Severely damaged or destroyed

Bridge	Location	Designer	Span, ft	Failure date
Dryburgh Abbey	Scotland	John and William Smith	260	1818
Union	England	Sir Samuel Brown	449	1821
Nassau	Germany	Lossen and Wolf	245	1834
Brighton Chain Pier	England	Sir Samuel Brown	255	1836
Montrose	Scotland	Sir Samuel Brown	432	1838
Menai Straits	Wales	Thomas Telford	580	1839
Roche-Bernard	France	Le Blanc	641	1852
Wheeling	USA	Charles Ellet	1010	1854
Niagara–Lewiston	USA	Edward Serrell	1041	1864
Niagara–Clifton	USA	Samuuel Keffer	1260	1889
Tacoma Narrows I	USA	Leon Moisseiff	2800	1940

(*b*) Oscillated violently in wind

Bridge	Location	Year built	Span, ft	Type of stiffening
Fyksesund	Norway	1937	750	Rolled I beam
Golden Gate	USA	1937	4200	Truss
Thousand Island	USA	1938	800	Plate girder
Deer Isle	USA	1939	1080	Plate girder
Bronx–Whitestone	USA	1939	2300	Plate girder
Long's Creek	Canada	1967	713	Plate girder

Source: After F. B. Farquharson et al., "Aerodynamic Stability of Suspension Bridges," University of Washington Bulletin 116, parts I through V, 1949–1954.

wind activity at the site over a period of years. Data are generally obtainable from local weather records and from meteorological records of the U.S. Weather Bureau. However, caution should be used, because these records may have been attained at a point some distance from the site, such as the local airport or federal building. Engineers should also be aware of differences in terrain features between the wind instrumentation site and the structure site that may have an important bearing on data interpretation. Data required are wind velocity, direction, and frequency. From these data, it is possible to predict high wind speeds, expected wind direction, and probability of occurrence.

The aerodynamic forces that wind applies to a bridge depend on the velocity and direction of the wind and on the size, shape, and motion of the bridge. Whether resonance will occur under wind forces depends on the same factors. The amplitude of oscillation that may build up depends on the strength of the wind forces (including their variation with amplitude of bridge oscillation), the energy-storage capacity of the structure, the structural damping, and the duration of a wind capable of exciting motion.

The wind velocity and direction, including vertical angle, can be determined by extended observations at the site. They can be approximated with reasonable conservatism on the basis of a few local observations and extended study of more general data. The choice of the wind conditions for which a given bridge should be designed may always be largely a matter of judgment.

At the start of aerodynamic analysis, the size and shape of the bridge are known. Its energy-storage capacity and its motion, consisting essentially of natural modes of vibration, are determined completely by its mass, mass distribution, and elastic properties and can be computed by reliable methods.

The only unknown element is that factor relating the wind to the bridge section and its motion. This factor cannot, at present, be generalized but is subject to reliable determination in each case. Properties of the bridge, including its elastic forces and its mass and motions (determining its inertial forces), can be computed and reduced to model scale. Then, wind conditions bracketing all probable conditions at the site can be imposed on a section model. The motions of such a dynamic section model in the properly scaled wind should duplicate reliably the motions of a convenient unit length of the bridge. The wind forces and the rate at which they can build up energy of oscillation respond to the changing amplitude of the motion. The rate of energy change can be measured and plotted against amplitude. Thus, the section-model test measures the one unknown factor, which can then be applied by calculation to the variable amplitude of motion along the bridge to predict the full behavior of the structure under the specific wind conditions of the test. These predictions are not precise but are about as accurate as some other features of the structural analysis.

15.17.2 Criteria for Aerodynamic Design

Because the factors relating bridge movement to wind conditions depend on specific site and bridge conditions, detailed criteria for the design of favorable bridge sections cannot be written until a large mass of data applicable to the structure being designed has been accumulated. But, in general, the following criteria for suspension bridges may be used.

A truss-stiffened section is more favorable than a girder-stiffened section.

Deck slots and other devices that tend to break up the uniformity of wind action are likely to be favorable.

The use of two planes of lateral system to form a four-sided stiffening truss is desirable because it can favorably affect torsional motion. Such a design strongly inhibits flutter and also raises the critical velocity of a pure torsional motion.

For a given bridge section, a high natural frequency of vibration is usually favorable:

- For short to moderate spans, a useful increase in frequency, if needed, can be attained by increased truss stiffness. (Although not closely defined, moderate spans may be regarded as including lengths from about 1000 to about 1800 ft.)
- For long spans, it is not economically feasible to obtain any material increase in natural frequency of vertical modes above that inherent in the span and sag of the cable.

- The possibility should be considered that for longer spans in the future, with their unavoidably low natural frequencies, oscillations due to unfavorable aerodynamic characteristics of the cross section may be more prevalent than for bridges of moderate span.

At most bridge sites, the wind may be broken up; that is, it may be nonuniform across the site, unsteady, and turbulent. So a condition that could cause serious oscillation does not continue long enough to build up an objectionable amplitude. However, bear in mind:

- There are undoubtedly sites where the winds from some directions are unusually steady and uniform.
- There are bridge sections on which any wind, over a wide range of velocity, will continue to build up some mode of oscillation.

An increase in stiffness arising from increased weight increases the energy-storage capacity of the structure without increasing the rate at which the wind can contribute energy. The effect is an increase in the time required to build up an objectionable amplitude. This may have a beneficial effect much greater than is suggested by the percentage increase in weight, because of the sharply reduced probability that the wind will continue unchanged for the greater length of time. Increased stiffness may give added structural damping and other favorable results.

Although more specific design criteria than the above cannot be given, it is possible to design a suspension bridge with a high degree of security against aerodynamic forces. This involves calculation of natural modes of motion of the proposed structure, section design with an effort to separate first vertical and torsional modes by at least a factor of 2, performance of dynamic-section-model tests to determine the factors affecting behavior, and application of these factors to the prototype by suitable analysis.

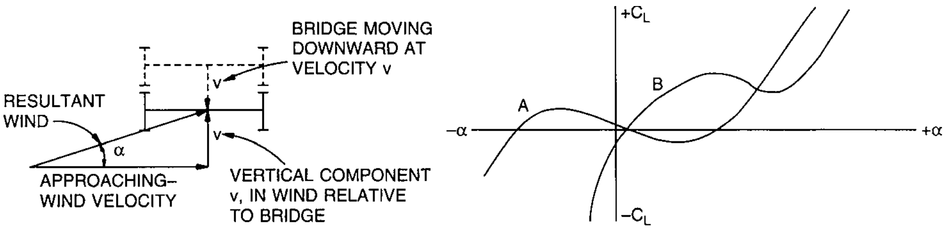
Most long-span bridges built since the Tacoma Bridge failure have followed the above procedures and incorporated special provisions in the design for aerodynamic effects. Designers of these bridges usually have favored stiffening trusses over girders. The second Tacoma Narrows, Forth Road, and Mackinac Straits bridges, for example, incorporate deep stiffening trusses with both top and bottom bracing, constituting a torsion space truss. The Forth Road and Mackinac Straits bridges have slotted decks. The Severn Bridge, however, has a streamlined, closed-box stiffening girder and inclined suspenders. Some designs incorporate longitudinal cable stays, tower stays, or even transverse diagonal stays (Deer Isle Bridge). Some have unloaded backstays. Others endeavor to increase structural damping by frictional or viscous means. *All have included dynamic-model studies as part of the design*.

15.17.3 Wind-Induced Oscillation Theories

Several theories have been advanced as models for mathematical analysis to develop an understanding of the process of wind excitation. Among these are the following.

Negative-Slope Theory. When a bridge is moving downward while a horizontal wind is blowing (Fig. 15.61*a*), the resultant wind is angled upward (positive angle of attack) relative to the bridge. If the lift coefficient C_L, as measured in static tests, shows a variation with wind angle α such as that illustrated by curve *A* in Fig. 15.61*b*, then, for moderate amplitudes, there is a wind force acting downward on the bridge while the bridge is moving downward. The bridge will therefore move to a greater amplitude than it would without this wind force. The motion will, however, be halted and reversed by the action of the elastic forces. Then, the vertical component of the wind also reverses. The angle of attack becomes negative, and the lift becomes positive, tending to increase the amplitude of the rebound. With increasing velocity, the amplitude will increase indefinitely or until the bridge is destroyed. A similar, though more complicated situation, would apply for torsional or twisting motion of the bridge.

Vortex Theory. This attributes aerodynamic excitation to the action of periodic forces having a certain degree of resonance with a natural mode of vibration of the bridge. Vortices, which form around the trailing edge of the airfoil (bridge deck), are shed on alternating sides, giving rise to periodic forces and oscillations transverse to the deck.



(a) (b)

FIGURE 15.61 Wind action on a cable-stayed bridge. (*a*) Downward bridge motion develops upward wind component. (*b*) Lift coefficient C_L depends on angle of attack α of the wind.

Flutter Theory. The phenomenon of flutter, as developed for airfoils of aircraft and applied to suspension-bridge decks, relates to the fact that the airfoil (bridge deck) is supported so that it can move elastically in a vertical direction and in torsion, about a longitudinal axis. Wind causes a lift that acts eccentrically. This causes a twisting moment, which, in turn, alters the angle of attack and increases the lift. The chain reaction becomes catastrophic if the vertical and torsional motions can take place at the same coupled frequency and in appropriate phase relation.

F. Bleich presented tables for calculation of flutter speed v_F for a given bridge, based on flat-plate airfoil flutter theory. These tables are applicable principally to trusses. But the tables are difficult to apply, and there is some uncertainty as to their range of validity.

A. Selberg has presented the following formula for flutter speed:

$$v_F = 0.88\omega_2 b\sqrt{\left[1-\left(\frac{\omega_1}{\omega_2}\right)^2\right]\frac{\sqrt{v}}{\mu}} \qquad (15.57)$$

where v = mass distribution factor for specific section = $2r^2/b^2$ (varies between 0.6 and 1.5, averaging about 1)
$\mu = 2\pi\rho b^2/m$ (ranges between 0.01 and 0.12)
m = mass per unit length
b = half width of bridge
ρ = mass density of air
ω_1 = circular vertical frequency
ω_1 = circular torsional frequency
r = mass radius of gyration

Selberg has also published charts, based on tests, from which it is possible to approximate the critical wind speed for any type of cross section in terms of the flutter speed.

Applicability of Theories. The vortex and flutter theories apply to the behavior of suspension bridges under wind action. Flutter appears dominant for truss-stiffened bridges, whereas vortex action seems to prevail for girder-stiffened bridges. There are mounting indications, however, these are, at best, estimates of aerodynamic behavior. Much work has been done and is being done, particularly in the spectrum approach and the effects of nonuniform, turbulent winds.

15.17.4 Design Indices

Bridge engineers have suggested several criteria for practical design purposes. O. H. Ammann, for example, developed two analytical-empirical indices that were applied in the design of the Verrazano-Narrows Bridge, a vertical-stiffness index and a torsional-stiffness index.

Vertical-Stiffness Index S_v. This is based on the magnitude of the vertical deflection of the suspension system under a static downward load covering one-half the center span. The index includes a correction to allow for the effect of structural damping of the suspended structure and for the effect of different ratios of side span to center span.

$$S_v = \left(8.2\frac{W}{f} + 0.14\frac{I}{L^4}\right)\left(1 - 0.6\frac{L_1}{L}\right) \tag{15.58}$$

where W = weight of bridge, lb/lin ft
f = cable sag, ft
I = moment of inertia of stiffening trusses and continuous stringers, in^2 by ft^2
L = length of center span, in thousands of feet
L_1 = length of side span, in thousands of feet

Torsional-Stiffness Index S_t. This is defined as the maximum intensity of sinusoidal loads, of opposite sign in opposite planes of cables, on the center span and producing 1-ft deflections at quarter points of the main span. This motion simulates deformations similar to those in the first asymmetric mode of torsional oscillations.

$$S_t = \left(\frac{B}{A} + 1\right)\frac{\pi^2}{4}\frac{W}{f} \tag{15.59}$$

where $A = \dfrac{b^2}{2}\dfrac{H_w}{E}$

$$B = \frac{2bd(A_v U_v A_h U_h)}{(b/d)A_v U_v + (d/b)A_h U_h}$$

W = weight of bridge, lb/lin ft
f = cable sag, ft
H_w = horizontal component of cable load due to dead load (half bridge), kips
b = distance between centerlines of cables, or centerlines of pairs of cables, ft
d = vertical distance between top and bottom planes of lateral bracing, ft
E = modulus of elasticity of truss steel, ksf
A_v = area of the diagonals in one panel of vertical truss, ft^2
A_h = area of the diagonals in one panel of horizontal lateral bracing (two members for X or K bracing), ft^2
$U_v = \sin^2 \gamma_v \cos \gamma_v$
$U_h = \sin^2 \gamma_h \cos \gamma_h$
γ_h = angle between diagonals and chord of horizontal truss
γ_v = angle between diagonals and chord of vertical truss

Typical values of these indices are listed in Table 15.13 for several bridges.

Other indices and criteria have been published by D. B. Steinman. In connection with these, Steinman also proposed that, unless aerodynamic stability is otherwise assured, the depth, ft, of stiffening girders and stiffening trusses should be at least $L/120 + (L/1000)^2$, where L is the span, ft. Furthermore, EI of the stiffening system should be at least $bL^4/120\sqrt{f}$, where b is the width, ft, of the bridge and f the cable sag, ft.

15.17.5 Natural Frequencies of Suspension Bridges

Dynamic analyses require knowledge of the natural frequencies of free vibration, modes of motion, energy-storage relationships, magnitude and effects of damping, and other factors.

Two types of vibration must be considered: bending and torsion.

TABLE 15.13 Stiffness Indices and First Asymmetric Mode Natural Frequencies

	Structural parameters									Vertical motions		Torsional motions	
Bridge	L, ft	L_1, ft	f, ft	W, lb/ft	I, $\text{in}^2\ \text{ft}^2$	b, ft	d, ft	A, ft^4	B, ft^4	Stiffness index	Frequency, cycles per min	Stiffness index	Frequency, cycles per min
Verrazano-Narrows Bridge	4,260	1,215	390	36,650	180,000	101.25	24	130.8	144.5	702	6.2	448	11.9
George Washington Bridge, 8-lane single deck complete	3,500	650	319	28,570	168	106	—	—	—	654	6.7	221	8.2
George Washington Bridge, 14-lane double deck complete	3,500	650	326	40,000	66,000	106	30	126.5	163.7	950	6.7	694	13.2
Golden Gate Bridge with upper lateral system only	4,200	1,125	475	21,300	88,000	90	—	—	—	342	5.6	111	7.0
Golden Gate Bridge with double lateral system	4,200	1,125	475	22,800+	88,000	90	25	51.3	75.5	364	5.6	292	11.0
Tacoma Narrows original with 2-lane single deck (very unfavorable aerodynamic characteristics)	2,800	1,100	232	5,700	2,567	39	—	—	—	158	8.0	61	10.0

Source: From M. Brumer, H. Rothman, M. Fiegen, and B. Forsyth, "Verrazano-Narrows Bridge: Design of Superstructure," *Journal of the Construction Division,* vol. 92, no. CO2, March 1966, American Society of Civil Engineers.

Bending. The fundamental differential equation [Eq. (15.22)] and cable condition [Eq. (15.26)] of the suspension bridge in Fig. 15.41 can be transformed into

$$EI\eta'''' - H\eta'' = \omega^2 m\eta + H_p y'' \tag{15.60}$$

$$\frac{H_p L_c}{E_c A_c} + y'' \int_0^L \eta\, dx = 0 \tag{15.61}$$

where ω = circular natural frequency of the bridge
η = deflection of stiffening truss or girder
m = bridge mass = w/g
y = vertical distance from cable to the line through the pylon supports
w = dead load, lb per lin ft
g = acceleration due to gravity = 32.2 ft/s^2

From these equations, the basic Rayleigh energy equation for bending vibrations can be derived:

$$\int EI\eta''^2 dx + H\int \eta'^2 dx + \frac{E_c A_c}{L_c}\left(y''\int \eta\, dx\right)^2 = \omega^2 \int m\eta^2 dx \tag{15.62}$$

Symbols are defined in "Torsion," following. After ω has been determined from this, the natural frequency of the bridge $\omega/2\pi$, Hz, can be computed.

Torsion. The Rayleigh energy equations for torsion are

$$EC_s \int \phi''^2 dx + \left(GI_T + \frac{b^2 H}{2}\right)\int \phi'^2 dx + \frac{E_c A_c}{2L_c}\left(y'' b\int \phi\, dx\right)^2 + EI_y y_M \int \eta''\phi'' dx = \omega^2 I_p \int \phi^2 dx \tag{15.63}$$

$$EI_y y_M \int \phi''\eta'' dx + EI_y \int \eta''^2 dx = \omega^2 m \int \eta^2 dx \tag{15.64}$$

where ϕ = angle of twist, rad
E = modulus of elasticity of stiffening girder, ksf
G = modulus of rigidity of stiffening girder, ksf
I_T = polar moment of inertia of stiffening girder cross section, ft^4
I_p = mass moment of inertia of stiffening girder per unit of length, kips·s^2
I_v = moment of inertia of stiffening girder about its vertical axis, ft^4
C_s = warping resistance of stiffening girder relative to its center of gravity, ft^6
b = horizontal distance between cables, ft
H = horizontal component of cable tension, kips
A_c = cross-sectional area of cable, ft^2
E_c = modulus of elasticity of cable, ksf
$L_c = \int \sec^3 \alpha dx$
α = angle cable makes with horizontal, radians
y_M = ordinate of center of twist relative to the center of gravity of stiffening girder cross, section, ft
ω = circular frequency, rad/sec
$m = m(x)$ = mass of stiffening girder per unit of length, kips·s^2/ft^2

Solution of these equations for the natural frequencies and modes of motion is dependent on the various possible static forms of suspension bridges involved (see Fig. 15.9). Numerous lengthy tabulations of solutions have been published.

15.17.6 Damping

Damping is of great importance in lessening of wind effects. It is responsible for dissipation of energy imparted to a vibrating structure by exciting forces. When damping occurs, one part of the external energy is transformed into molecular energy, and another part is transmitted to surrounding objects or the atmosphere. Damping may be internal, due to elastic hysteresis of the material or plastic yielding and friction in joints, or Coulomb (dry friction), or atmospheric, due to air resistance.

15.17.7 Aerodynamics of Cable-Stayed Bridges

The aerodynamic action of cable-stayed bridges is less severe than that of suspension bridges, because of increased stiffness due to the taut cables and the widespread use of torsion box decks. However, there is a trend towards the use of the composite steel-concrete superstructure girders (Fig. 15.16) for increasingly longer spans and to reduce girder dead weight. This configuration, because of the long spans and decreased mass, can be relatively more sensitive to aerodynamic effects as compared to a torsionally stiff box.

15.17.8 Stability Investigations

It is most important to note that the validation of stability of the completed structure for expected wind speeds at the site is mandatory. However, this does not necessarily imply that the most critical stability condition of the structure occurs when the structure is fully completed. A more dangerous condition may occur during erection, when the joints have not been fully connected and, therefore, full stiffness of the structure has not yet been realized. In the erection stage, the frequencies are lower than in the final condition and the ratio of torsional frequency to flexural frequency may approach unity. Various stages of the partly erected structure may be more critical than the completed bridge. The use of welded components in pylons has contributed to their susceptibility to vibration during erection.

Because no exact analytical procedures are yet available, wind-tunnel tests should be used to evaluate the aerodynamic characteristics of the cross section of a proposed deck girder, pylon, or total bridge. More importantly, the wind-tunnel tests should be used during the design process to evaluate the performance of a number of proposed cross sections for a particular project. In this manner, the wind-tunnel investigations become a part of the design decision process and not a postconstruction corrective action. If the wind-tunnel evaluations are used as an after-the-fact verification and they indicate an instability, there is the distinct risk that a redesign of a retrofit design will be required that will have undesirable ramifications on schedules and availability of funding.

(F. Bleich and L. W. Teller, "Structural Damping in Suspension Bridges," *ASCE Transactions*, vol. 117, pp. 165–203, 1952; F. Bleich, C. B. McCullough, R. Rosecrans, and G. S. Vincent, "The Mathematical Theory of Vibration of Suspension Bridges," Bureau of Public Roads, Government Printing Office, Washington, D.C; F. B. Farquharson, "Wind Forces on Structures Subject to Oscillation," *ASCE Proceedings*, ST4, July, 1958; A. Selberg, "Oscillation and Aerodynamic Stability of Suspension Bridges," *Acta Polytechnia Scandinavica*, Civil Engineering and Construction Series 13, Trondheim, 1961; D. B. Steinman, "Modes and Natural Frequencies of Suspension Bridge Oscillations," *Transactions Engineering Institute of Canada*, vol. 3, no. 2, pp. 74–83, 1959; D. B. Steinman, "Aerodynamic Theory of Bridge Oscillations," *ASCE Transactions*, vol. 115, pp. 1180–1260, 1950; D. B. Steinman, "Rigidity and Aerodynamic Stability of Suspension Bridges," *ASCE Transactions*, vol. 110, pp. 439–580, 1945; "Aerodynamic Stability of Suspension Bridges," 1952 Report of the Advisory Board on the Investigation of Suspension Bridges, *ASCE Transactions*, vol. 120, pp. 721–781, 1955; R. L. Wardlaw, "A Review of the Aerodynamics of Bridge Road Decks and the Role of Wind Tunnel Investigation," U. S. Department of Transportation, Federal Highway Administration, Report No. FHWA-RD-72-76; A. G. Davenport, "Buffeting of a Suspension Bridge by Storm Winds," *ASCE Journal of the Structural Division*, vol. 115, ST3, June 1962; "Guidelines for Design of Cable-Stayed Bridges," ASCE Committee on Cable-Stayed Bridges; W. Podolony, Jr., and J. B. Scalzi, *Construction and Design of Cable-Stayed Bridges*, 2d ed., John Wiley & Sons, Inc., New York; E. Murakami and T. Okubu, "Wind-Resistant Design of a Cable-stayed Bridge," International Association for Bridge and Structural Engineering, Final Report, 8th Congress, New York, September 9–14, 1968.)

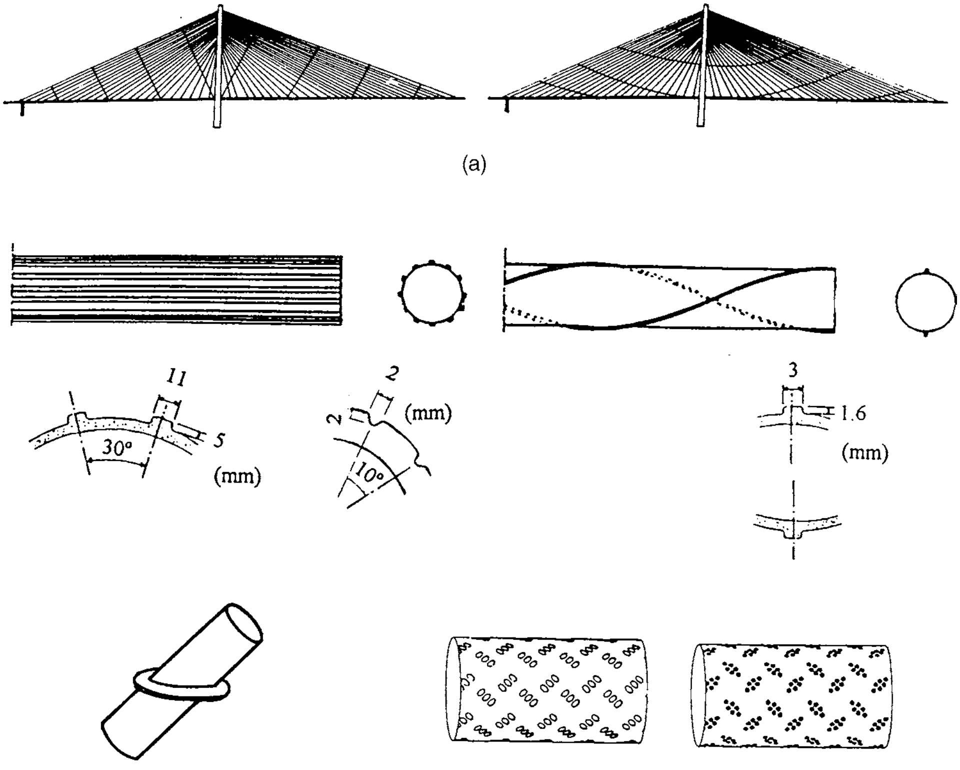
15.17.9 Rain-Wind–Induced Vibration

Well-known mechanisms of cable vibration are vortex and wake galloping. Starting in approximately the mid-1980s, a new phenomenon of cable vibration has been observed that occurs during the simultaneous presence of rain and wind; thus, it is given the name "rain-wind vibration," or rain vibration.

The excitation mechanism is the formation of water rivulets, at the top and bottom, that run down the cable oscillating tangentially as the cables vibrate, thus changing the aerodynamic profile of the cable (or the enclosing HDPE pipe). The formation of the upper rivulet appears to be the more dominant factor in the origin of the rain-wind vibration.

In the current state of the art, three basic methods of rain-wind vibration suppression are being considered or used:

- Rope ties interconnecting the cable stays in the plane of the stays, Fig. 15.62*a*
- Modification of the external surface of the enclosing HDPE pipe, Fig. 15.62*b*
- Providing external damping



(a)

(b)

FIGURE 15.62 Methods of rain-wind vibration suppression. (*a*) Rope ties. (*b*) Modification of external surface.

The interconnection of stays by rope ties produces node points at the point of connection of the secondary tie to the cable stays. The purpose is to shorten the free length of the stay and modify the natural frequency of vibration of the stay. The modification of the surface may be such as protuberances that are axial, helical, elliptical or circular or grooves or dimples. The intent is to discourage the formation of the rivulets and/or its oscillations. Various types of dampers such as viscous, hydraulic, tuned mass, and rubber have also been used to suppress the vibration.

The rain-wind vibration phenomenon has been observed during construction prior to grout injection which then stabilizes after grout injection. This may be as a result of the difference in mass prior to and after grout injection (or not). It also has been noticed that the rain-wind vibration may not manifest itself until some time after completion of the bridge. This may be the result of a transition from initial smoothness of the external pipe to a roughness, sufficient to hold the rivulet, resulting from an environmental or atmospheric degradation of the surface of the pipe.

The interaction of the various parameters in the rain-wind phenomenon is not yet well understood and an optimum solution is not yet available. It should also be noted that under similar conditions of rain and wind, the hangars of arch bridges and suspenders of suspension bridges can also vibrate.

(Hikami, Y., and Shiraishi, N., "Rain-Wind Induced Vibrations of Cable in Cable Stayed Bridges," *Journal of Wind Engineering and Industrial Aerodynamics*, 29 (1988), pp. 409–418, Elsevier Science Publishers B. V., Amsterdam; Matsumoto, M., Shiraishi, N., Kitazawa, M., Knisely, C., Shirato, H., Kim, Y. and Tsujii, M., "Aerodynamic Behavior of Inclined Circular Cylinders—Cable Aerodynamics," *Journal of Wind Engineering* (Japan), no. 37, October 1988, pp. 103–112; Matsumoto, M., Yokoyama, K., Miyata, T., Fujno, Y. and Yamaguchi, H., "Wind-Induced Cable Vibration of Cable-Stayed Bridges in Japan," Proc. of Canada-Japan Workshop on Bridge Aerodynamics, Ottawa, 1989, pp. 101–110; Matsumoto, M., Hikami, Y. and Kitazawa, M., "Cable Vibration and its Aerodynamic/Mechanical Control," Proc. Cable-Stayed and Suspension Bridges, Deauville, France, October 12–15, 1994, vol. 2, pp. 439–452; Miyata, T., Yamada, H. and Hojo, T., "Aerodynamic Response of PE Stay Cables with Pattern-Indented Surface," Proc. Cable-Stayed and Suspension Bridges, Deauville, France, October 12–15, 1994, vol. 2, pp. 515–522.

15.18 SEISMIC ANALYSIS OF CABLE-SUSPENDED STRUCTURES

For short-span structures (under about 500 ft) it is commonly assumed in seismic analysis that the same ground motion acts simultaneously throughout the length of the structure. In other words, the wavelength of the ground waves are long in comparison to the length of the structure. In long-span structures, such as suspension or cable-stayed bridges, however, the structure could be subjected to different motions at each of its foundations. Hence, in assessment of the dynamic response of long structures, the effects of traveling seismic waves should be considered. Seismic disturbances of piers and anchorages may be different at one end of a long bridge than at the other. The character or quality of two or more inputs into the total structure, their similarities, differences, and phasings, should be evaluated in dynamic studies of the bridge response.

Vibrations of cable-stayed bridges, unlike those of suspension bridges, are susceptible to a unique class of vibration problems. Cable-stayed bridge vibrations cannot be categorized as vertical (bending), lateral (sway), and torsional; almost every mode of vibration is instead a three-dimensional motion. Vertical vibrations, for example, are introduced by both longitudinal and lateral shaking in addition to vertical excitation. In addition, an understanding is needed of the multimodal contribution to the final response of the structure and in providing representative values of the response quantities. Also, because of the long spans of such structures, it is necessary to formulate a dynamic response analysis resulting from the multi-support excitation. A three-dimensional analysis of the whole structure and substructure to obtain the natural frequencies and seismic response is advisable. A qualified specialist should be consulted to evaluate the earthquake response of the structure.

("Guide Specifications for Seismic Design of Highway Bridges," American Association of State Highway and Transportation Officials; "Guidelines for the Design of Cable-Stayed Bridges," ASCE Committee on Cable-Stayed Bridges; A. M. Abdel-Ghaffar, and L I. Rubin, "Multiple-Support Excitations of Suspension Bridges," *Journal of the Engineering Mechanics Division*, ASCE, vol. 108, no. EM2, April, 1982; A. M. Abdel-Ghaffar, and L. I. Rubin, "Vertical Seismic Behavior of Suspension Bridges," *The International Journal of Earthquake Engineering and Structural Dynamics*, vol. 11, January–February, 1983; A. M. Abdel-Ghaffar, and L. I. Rubin, "Lateral Earthquake Response of Suspension Bridges," *Journal of the Structural Division*, ASCE, vol. 109, no ST3, March, 1983; A. M. Abdel-Ghaffar, and J. D. Rood, "Simplified Earthquake Analysis of Suspension Bridge Towers," *Journal of the Engineering Mechanics Division*, ASCE, vol. 108, no. EM2, April, 1982.)

15.19 ERECTION OF CABLE-SUSPENDED BRIDGES

The ease of erection of suspension bridges is a major factor in their use for long spans. Once the main cables are in position, they furnish a stable working base or platform from which the deck and stiffening truss sections can be raised from floating barges or other equipment below, without the need for auxiliary falsework. For the Severn Bridge, for example, 60-ft box-girder deck sections were floated to the site and lifted by equipment supported on the cables.

Until the 1960s, the field process of laying the main cables had been by spinning (Art. 15.8.3). (This term is actually a misnomer, for the wires are neither twisted nor braided, but are laid parallel to and against each other.) The procedure (Fig. 15.63) starts with the hanging of a catwalk at each cable location for use in construction of the bridge. An overhead cableway is then installed above each catwalk. Loops of wire (two or four at a time) are carried over the span on a set of grooved spinning wheels. These are hung from an endless hauling rope of the cableway until arrival at the far anchorage. There, the loops are pulled off the spinning wheels manually and placed around a semicircular strand shoe, which connects them by an eyebar or bolt linkage to the anchorage (Fig. 15.28). The wheels then start back to the originating anchorage. At the same time, another set of wheels carrying wires starts out from that anchorage. The loops of wire on the latter set of wheels are also placed manually around a strand shoe at their anchorage destination. Spinning proceeds as the wheels shuttle back and forth across the span. A system of counterweights keeps the wires under continuous tension as they are spun.

The wires that come off the bottom of the wheels (called dead wires) and that are held back by the originating anchorage are laid on the catwalk in the spinning process. The wires passing over the wheels from the unreelers and moving at twice the speed of the wheels, are called *live wires.*

As the wheels pass each group of wire handlers on the catwalks, the dead wires are temporarily clipped down. The live wires pass through small sheaves to keep them in correct order. Each wire is adjusted for level in the main and side spans with come-along winches, to ensure that all wires will have the same sag.

The cable is made of many strands, usually with hundreds of wires per strand (Art. 15.8). All wires from one strand are connected to the same shoe at each anchorage. Thus, there are as many anchorage shoes as strands. At saddles and anchorages, the strands maintain their identity, but throughout the rest of their length, the wires are compacted together by special machines. The cable usually is forced into a circular cross section of tightly bunched parallel wires.

The usual order of erection of suspension bridges is substructure, pylons and anchorages, catwalks, cables, suspenders, stiffening trusses, floor system, cable wrapping, and paving.

Cables are usually coated with a protective compound. The main cables are wrapped with wire by special machines, which apply tension, pack the turns tightly against one another, and at the same time advance along the cable. Several coats of protective material, such as paint, are then applied for alternative wrapping (see Art. 15.10).

Typical cable bands are illustrated in Figs. 15.34 and 15.35. These are usually made of paired, semicylindrical steel castings with clamping bolts, over which the wire-rope or strand suspenders are looped or attached by socket fittings.

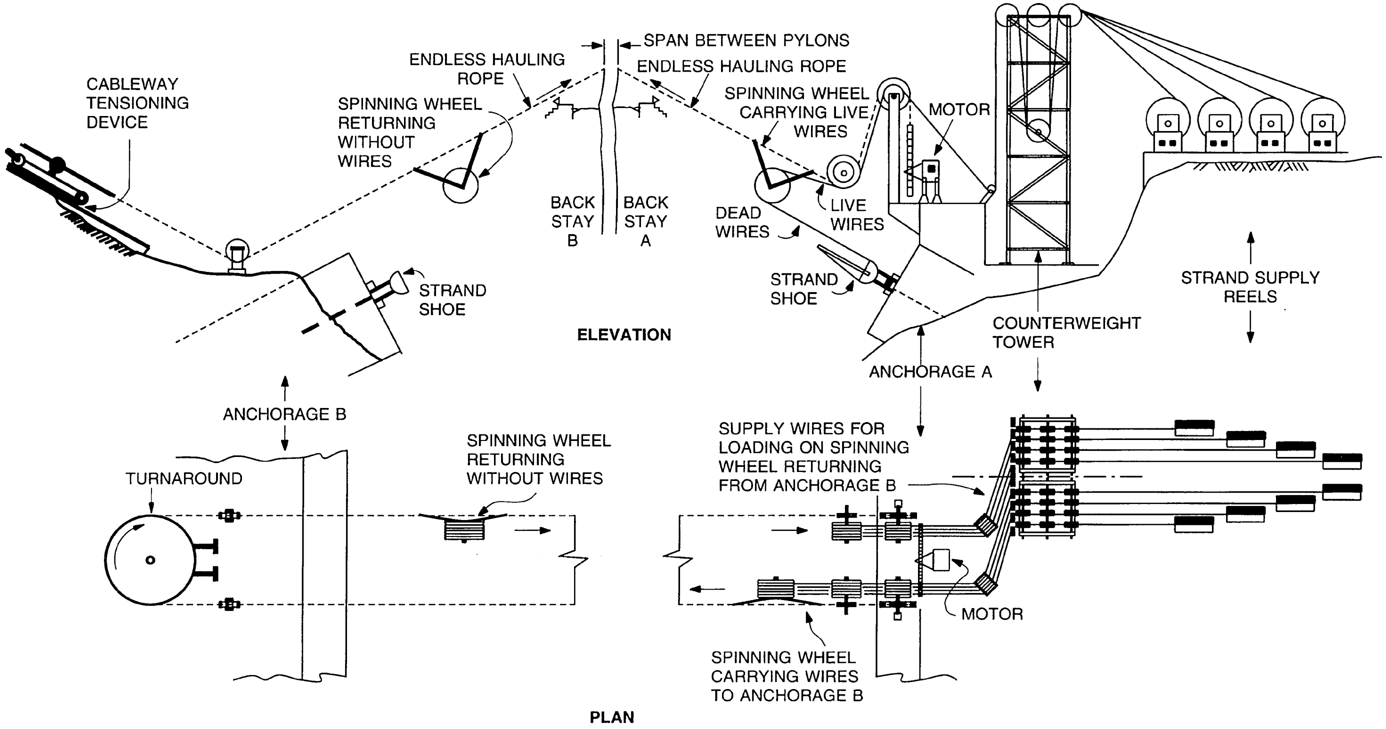


FIGURE 15.63 Scheme for spinning four wires at a time for the cables of the Forth Road Bridge.

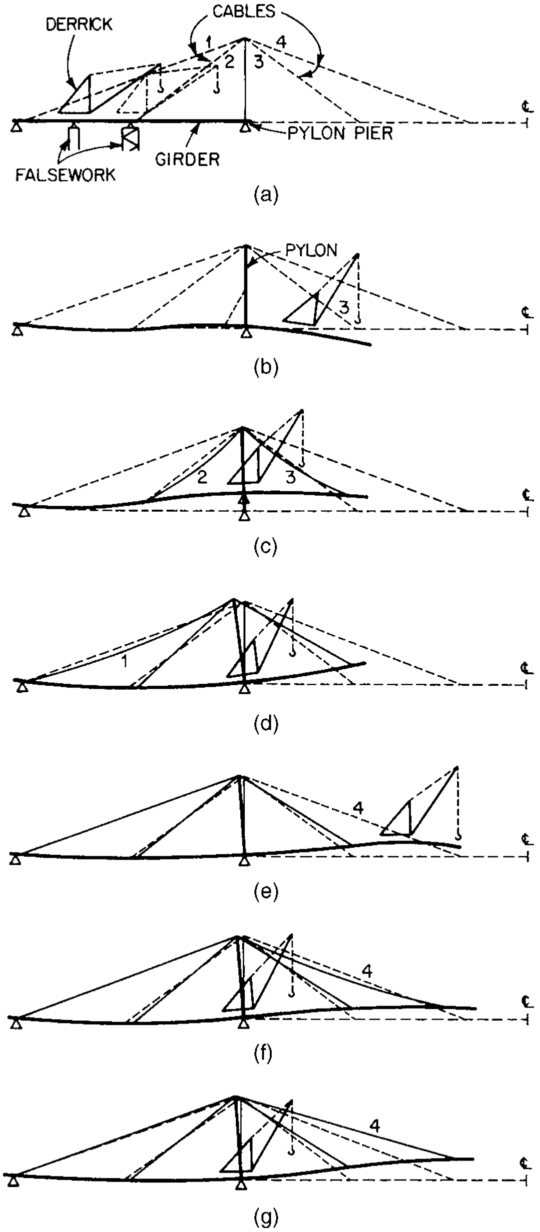
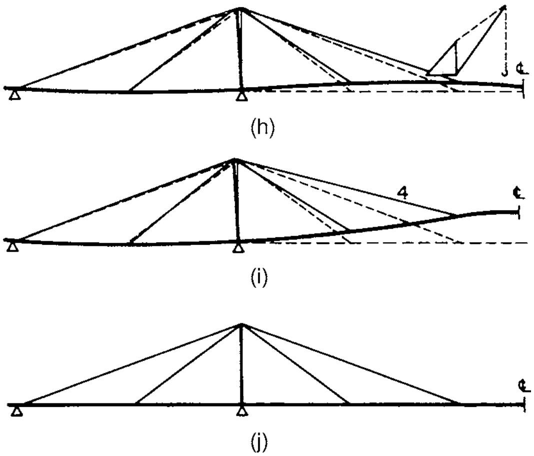


FIGURE 15.64 Erection procedure used for the Strömsund Bridge. (*a*) Girder, supported on falsework, is extended to the pylon pier. (*b*) Girder is cantilevered to the connection of cable 3. (*c*) Derrick is retracted to the pier and the girder is raised, to permit attachment of cables 2 and 3 to the girder. (*d*) Girder is reseated on the pier and cable 1 is attached. (*e*) Girder is cantilevered to the connection of cable 4. (*f*) Derrick is retracted to the pier and cable 4 is connected. (*g*) Preliminary stress is applied to cable 4.



(h)

(i)

(j)

FIGURE 15.64 **(***Continued***)** (*h*) Girder is cantilevered to midspan and spliced to its other half. (*i*) Cable 4 is given its final stress. (*j*) The roadway is paved, and the bridge takes its final position. (*Reprinted with permission from H. J. Ernst, "Montage Eines Seilverspannten Balkens im Grossbrucken-bau," Stahlbau, vol. 25, no. 5, May 1956.*)

Cable-stayed structures are ideally suited for erection by cantilevering into the main span from the piers. Theoretically, erection could be simplified by having temporary erection hinges at the points of cable attachment to the girder, rendering the system statically determinate, then making these hinges continuous after dead load has been applied. The practical implementation of this is difficult, however, because the axial forces in the girder are larger and would have to be concentrated in the hinges. Therefore, construction usually follows conventional tactics of cantilevering the girder continuously and adjusting the cables as necessary to meet the required geometrical and statical constraints. A typical erection sequence is illustrated in Fig. 15.64.

Erection should meet the requirements that, on completion, the girder should follow a prescribed gradient; the cables and pylons should have their true system lengths; the pylons should be vertical, and all movable bearings should be in a neutral position. To accomplish this, all members, before erection, must have a deformed shape the same as, but opposite in direction to, that which they would have under dead load. The girder is accordingly cambered, and also lengthened by the amount of its axial shortening under dead load. The pylons and cable are treated in similar manner.

Erection operations are aided by raising or lowering supports or saddles, to introduce prestress as required. All erection operations should be so planned that the stresses during the erection operations do not exceed those due to dead and live load when the structure is completed; otherwise, loss of economy will result.