CHAPTER 15

Building Connections

15.1 SELECTION OF TYPE OF FASTENER

This chapter is concerned with the actual beam-to-beam and beam-to-column connections commonly used in steel buildings. Under present-day steel specifications, three types of fasteners are permitted for these connections: *welds, unfinished bolts, and high-strength bolts.*

Selection of the type of fastener or fasteners to be used for a particular structure usually involves many factors, including requirements of local building codes, relative economy, preference of designer, availability of good welders, loading conditions (as static or fatigue loadings), preference of fabricator, and equipment available. It is impossible to list a definite set of rules from which the best type of fastener can be selected for any given structure. We can give only a few general statements that may be helpful in making a decision:

1. Unfinished bolts are often economical for light structures subject to small static loads and for secondary members (such as purlins, girts, bracing, etc.) in larger structures.
2. Field bolting is very rapid and involves less skilled labor than welding. The purchase price of high-strength bolts, however, is rather high.
3. If a structure is later to be disassembled, welding probably is ruled out, leaving the job open to selection of bolts.
4. For fatigue loadings, slip-critical high-strength bolts and welds are very good.
5. Notice that special care has to be taken to properly install high-strength, slip-critical bolts.
6. Welding requires the smallest amounts of steel, probably provides the most attractive-looking joints, and also has the widest range of application to different types of connections.
7. When continuous and rigid, fully moment-resisting joints are desired, welding probably will be selected.

8. Welding is almost universally accepted as being satisfactory for shopwork. For fieldwork, it is very popular in most areas of the United States, while in a few others it is stymied by the idea that field inspection is rather questionable.
9. To use welds for very thick members requires a great deal of extra care, and bolted connections may very well be used instead. Furthermore, such bolted connections are far less susceptible to brittle fractures.

15.2 TYPES OF BEAM CONNECTIONS

All connections have some restraint—that is, some resistance to changes of the original angles between intersecting members when loads are applied. Depending on the amount of restraint, the AISC Specification (B3.6) classifies connections as being fully restrained (Type FR) and partially restrained (Type PR). These two types of connections are described in more detail as follows:

1. Type FR connections are commonly referred to as rigid or continuous frame connections. They are assumed to be sufficiently rigid or restrained to keep the original angles between members virtually unchanged under load.
2. Type PR connections are those that have insufficient rigidity to keep the original angles virtually unchanged under load. Included in this classification are simple and semirigid connections, as described in detail in this section.

A *simple connection* is a Type PR connection for which restraint is ignored. It is assumed to be completely flexible and free to rotate, thus having no moment resistance. A *semirigid connection*, or *flexible moment connection*, is also a Type PR connection whose resistance to angle change falls somewhere between the simple and rigid types.

As there are no perfectly rigid connections nor completely flexible ones, all connections really are partly restrained, or PR, to one degree or another. The usual practice in the past was to classify connections based on a ratio of the moment developed for a particular connection to the moment that would theoretically be developed by a completely rigid connection. A rough rule was that simple connections had 0–20 percent rigidity, semirigid connections had 20–90 percent rigidity, and rigid connections had 90–100 percent rigidity. Figure 15.1 shows a set of typical moment-rotation curves for these connections. Notice that the lines are curved because as the moments become larger, the rotations increase at a faster rate.

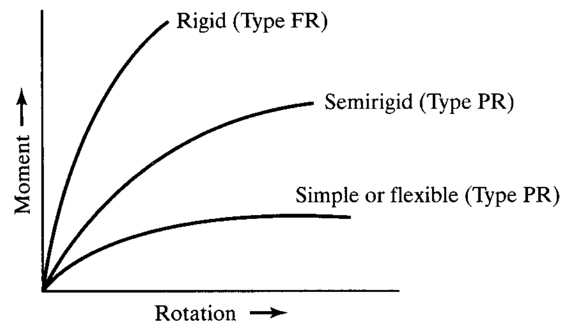

FIGURE 15.1

Typical moment-rotation curves for connections.

During the past few years, quite a few investigators around the world have been trying to develop empirical formulas for describing the rotational characteristics of connections.[1–6] Though they have made some progress, the only accurate method for developing such information today involves the actual fabrication of connections, followed by load testing. It is very difficult to include in a formula the effects of such things as poor fit, improper tightening of bolts, and so on.

Each of these three general types of connections is briefly discussed in this section, with little mention of the specific types of connectors used. The remainder of the chapter is concerned with detailed designs of these connections, using specific types of fasteners. *In this discussion, the author probably overemphasizes the semirigid and rigid type connections, because a very large percentage of the building designs with which the average designer works will be assumed to have simple connections.* A few descriptive comments are given in the paragraphs that follow concerning each of these three types of connections.

Simple connections (Type PR) are quite flexible and are assumed to allow the beam ends to be substantially free to rotate downward under load, as true simple beams should. Although simple connections do have some moment resistance (or resistance to end rotation), it is taken to be negligible, and they are assumed to be able to resist shear only. Several types of simple connections are shown in Fig. 15.2. More detailed descriptions of each of these connections and their assumed behavior under load are given in later sections of this chapter. In this figure, most of the connections are shown as being made entirely with the same type of fastener—that is, all bolted or all welded—while in actual practice two types of fasteners are often used for the same connection. For example, a very common practice is to shop-weld the web angles to the beam web and field-bolt them to the column or girder.

Semirigid connections or flexible moment connections (Type PR) are those that have appreciable resistance to end rotation, thus developing appreciable end moments. In design practice, it is quite common for the designer to assume that all connections are either simple or rigid, with no consideration given to those situations in between, thereby simplifying the analysis. Should he or she make such an assumption for a true semirigid connection, he or she may miss an opportunity for appreciable moment reductions. To understand this possibility, the reader is referred to the moment diagrams shown in Fig. 15.3 for a group of uniformly loaded beams supported with connections having different percentages of rigidity. This figure shows that the maximum moments in a beam vary greatly with different types of end connections. For example, the maximum moment in the semirigid connection of part (d) of the figure is only 50 percent of the

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[1]R. M. Richard, "A Study of Systems Having Conservative Non-Linearity," Ph.D. Thesis, Purdue University, 1961.

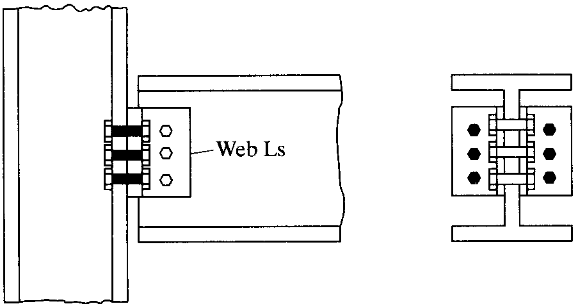
[2]N. Kishi and W. F. Chen, "Data Base of Steel Beam-to-Column Connections," no. CE-STR-86-26, Vols. I and II (West Lafayette, IN: Purdue University, School of Engineering, July 1986).

[3]L. F. Geschwindner, "A Simplified Look at Partially Restrained Beams," *Engineering Journal*, AISC, vol. 28, no. 2 (2nd Quarter, 1991), pp. 73–78.

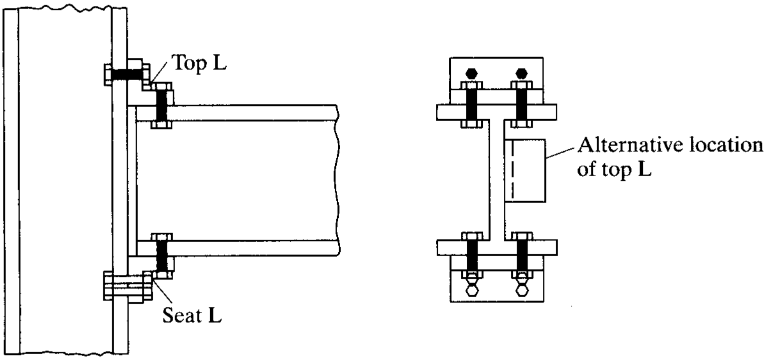
[4]Wai-Fah Chen et al., "Semi-Rigid Connections in Steel Frames," Council on Tall Buildings and Urban Habitat, Committee 43 (McGraw Hill, 1992).

[5]S. E. Kim and W. F. Chen, "Practical Advanced Analysis for Semi-Rigid Frame Design," *Engineering Journal*, AISC, vol. 33, no. 4 (4th Quarter, 1996), pp. 129–141.

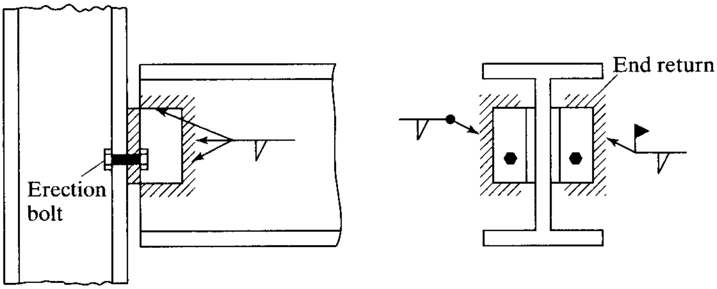
[6]J. E. Christopher and R. Bjorhovde, "Semi-Rigid Frame Design for Practicing Engineers," *Engineering Journal*, AISC, vol. 36, no. 1 (1st Quarter, 1999), pp. 12–28.



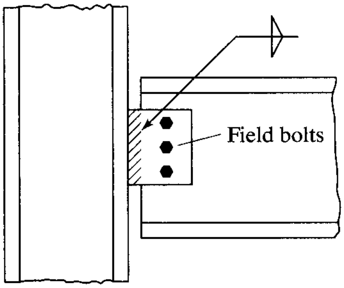
(a) Framed simple connection



(b) Seated simple connection



(c) Framed simple connection



(d) Single-plate or shear tab simple connection

FIGURE 15.2

Some simple connections. Notice how these connections are placed up towards the top flanges so that they provide lateral stability at the compression flanges at the beam supports. (a) Framed simple connection. (b) Seated simple connection. (c) Framed simple connection. (d) Single-plate or shear tab simple connection.

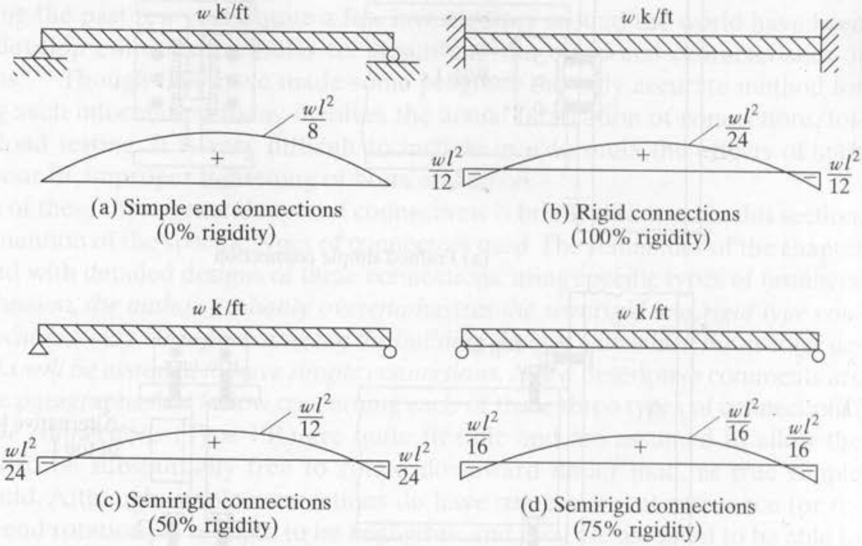


FIGURE 15.3

maximum moment in the simply supported beam of part (a) and only 75 percent of the maximum moment in the rigidly supported beam of part (b).

Actual semirigid connections are used fairly often, but usually no advantage is taken of their moment-reducing possibilities in the calculations. Perhaps one factor that keeps the design professional from taking advantage of them more often is the statement of the AISC Specification (Section B3.6b) that consideration of a connection as being semirigid is permitted only upon presentation of evidence that it is capable of providing a certain percentage of the end restraint furnished by a completely rigid connection. This evidence must consist of documentation in the technical literature, or must be established by analytical or empirical means.

A typical framing angle shop-welded to a beam. It will be field-bolted to another member. (Courtesy of CMC South Carolina Steel.)

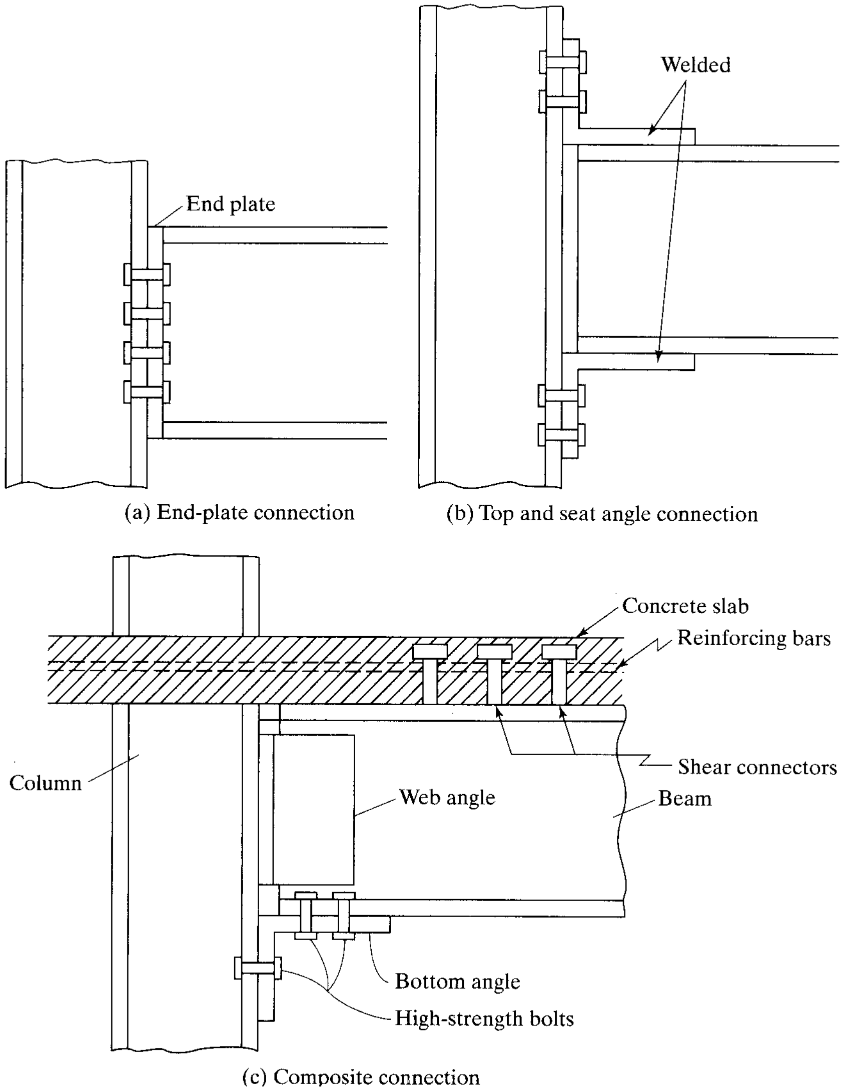


FIGURE 15.4

Some semirigid or flexible moment connections. (Additional types are shown in Part 11 of the AISC Manual.)

Three practical semirigid, or PR, connections capable of providing considerable moment resistance are shown in Fig. 15.4. If the end-plate connection shown in part (a) of the figure is extended above the beam and more bolts are installed, the moment resistance of the connection can be appreciably increased. Part (c) of the figure shows a semirigid connection that is proving to be quite satisfactory for steel-concrete composite floors. Moment resistance in this connection is provided by reinforcing bars placed in the concrete slab above the beam and by the horizontal leg of the seat angle.[7]

[7]D. J. Ammerman and R.T. Leon, "Unbraced Frames with Semirigid Composite Connections," *Engineering Journal*, AISC, vol. 27, no. 1 (1st Quarter 1990), pp. 12–21.

A semirigid beam-to-column connection, Ainsley Building, Miami, FL. (Courtesy of the Lincoln Electric Company.)

Another type of semirigid connection is illustrated in an accompanying photograph from the Lincoln Electric Company.

The use of partially restrained connections with roughly 60 to 75 percent rigidities is gradually increasing. When it becomes possible to accurately predict the percentages of rigidity for various connections, and when better design procedures are available, this type of design will probably become even more common.

Rigid connections (Type FR) are those which theoretically allow no rotation at the beam ends and thus transfer close to 100 percent of the moment of a fixed end. Connections of this type may be used for tall buildings in which wind resistance is developed. The connections provide continuity between the members of the building frame. Several Type FR connections that provide almost 100 percent restraint are shown in Fig. 15.5. It will be noticed in the figure that column web stiffeners may be required for some of these connections to provide sufficient resistance to rotation. The design of these stiffeners is discussed in Section 15.12.[8]

The moment connection shown in part (d) of Fig. 15.5 is rather popular with steel fabricators, and the end-plate connection of part (e) also has been frequently used in recent years.[9]

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[8] J. D. Griffiths "End-Plate Moment Connections—Their Use and Misuse," *Engineering Journal*, AISC, vol. 21, no. 1 (1st Quarter, 1984), pp. 32–34.

[9] AISC, "Seismic Provisions for Structural Steel Buildings," ANSI/AISC 341–05, American Institute of Steel Construction, Inc. (Chicago, IL).

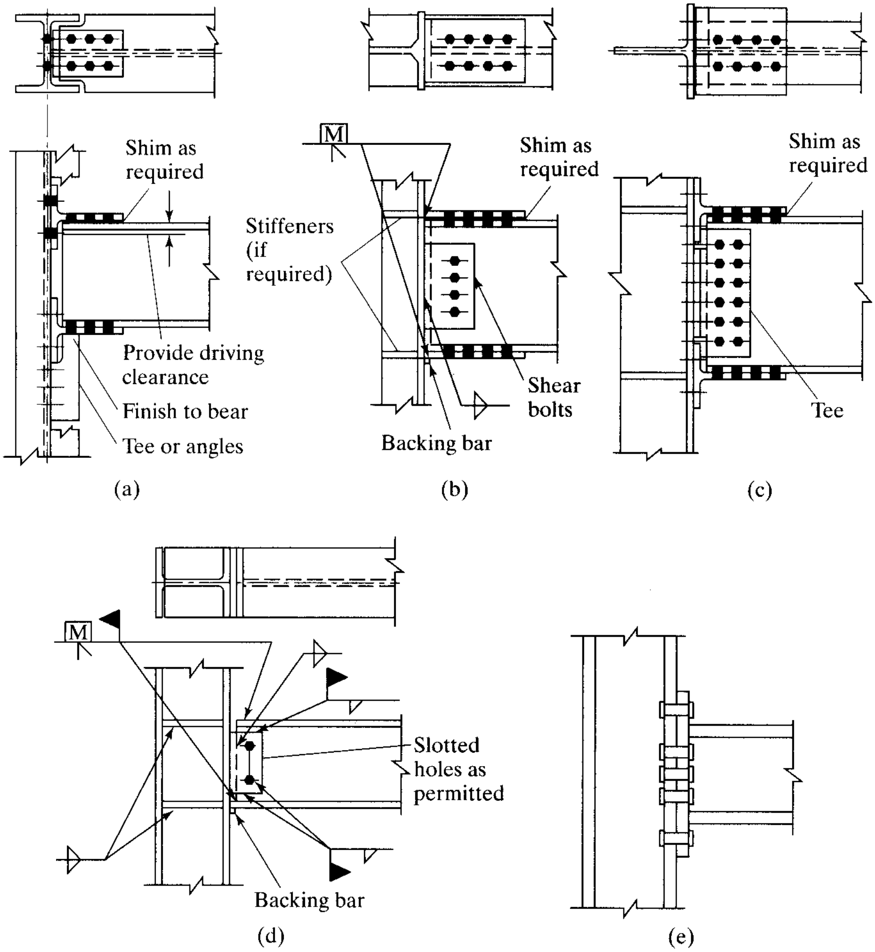
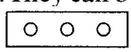
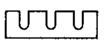


FIGURE 15.5

Moment-resisting connections.

You will note the use of *shims* in parts (a) to (c). Shims are thin strips of steel that are used to adjust the fit at connections. They can be one of two types: conventional shims or finger shims. *Conventional shims*  are installed with the bolts passing through them, while *finger shims*  can be installed after the bolts are in place. You should realize that there is some variation in the depths of beams as they come from the steel mills. (See Table 1-22 in Part 1 of the AISC Manual for permissible tolerances.) To provide for such variations, it is common to make the distance between flange plates or angles larger than the nominal beam depths given in the Manual.[10]

[10]W. T. Segui, *Fundamentals of Structural Steel Design*, 4th ed. (Boston: PWS-Kent, 2007), p. 487.

15.3 STANDARD BOLTED BEAM CONNECTIONS

Several types of standard bolted connections are shown in Fig. 15.6. These connections are usually designed to resist shear only, as testing has proved this practice to be quite satisfactory. Part (a) of the figure shows a connection between beams with the so-called *framed connection*. This type of connection consists of a pair of flexible web angles, probably shop-connected to the web of the supported beam and field-connected to the supporting beam or column. When two beams are being connected, usually it is necessary to keep their top flanges at the same elevation, with the result that the top flange

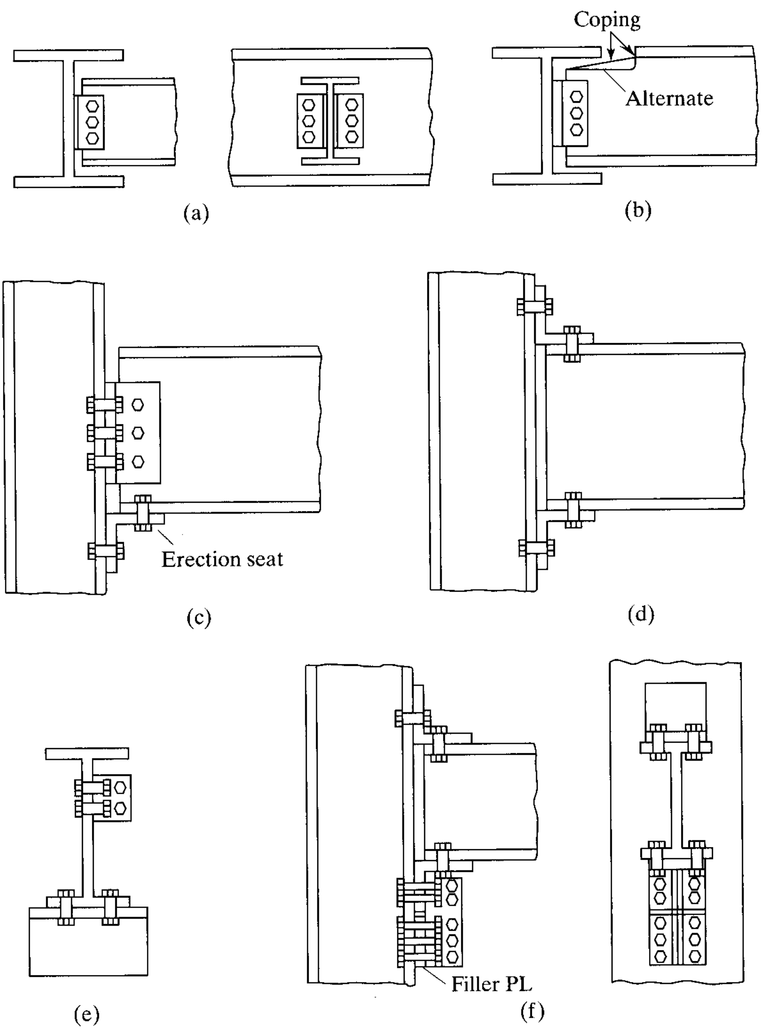


FIGURE 15.6

(a) Framed connection. (b) Framed connection. (c) Framed connection. (d) Seated connection. (e) Seated connection. (f) Seated connection with stiffener angles.

of one will have to be cut back (called *coping*), as shown in part (b) of the figure. For such connections, we must check block shear, as discussed in Section 3.7 of this text. Coping is an expensive process and should be avoided where possible.

Simple connections of beams to columns can be either framed or seated, as shown in Fig. 15.6. In part (c) of the figure, a framed connection is shown in which two web angles are connected to the beam web in the shop, after which bolts are placed through the angles and column in the field. It is often convenient to use an angle called an *erection seat* to support the beam during erection. Such an angle is shown in the figure.

The seated connection has an angle under the beam similar to the erection seat just mentioned, which is shop-connected to the column. In addition, there is another angle—probably on top of the beam—that is field-connected to the beam and column. A seated connection of this type is shown in part (d) of the figure. Should space prove to be limited above the beam, the top angle may be placed in the optional location shown in part (e) of the figure. The top angle at either of the locations mentioned is very helpful in keeping the top flange of the beam from being accidentally twisted out of place during construction.

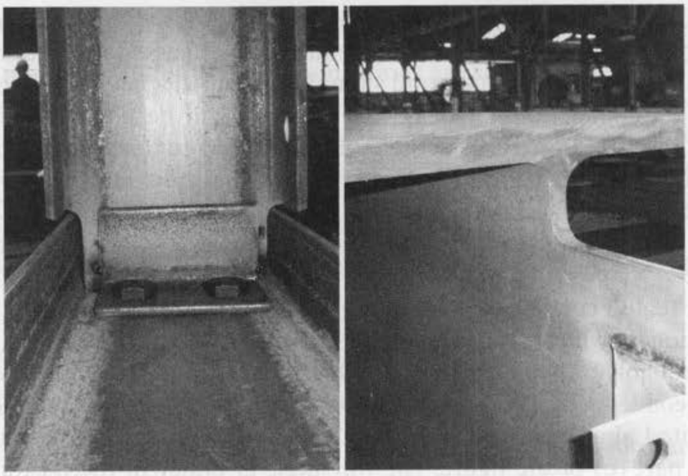
The amount of load that can be supported by the types of connections shown in parts (c), (d), and (e) of Fig. 15.6 is severely limited by the flexibility or bending strength of the horizontal legs of the seat angles. For heavier loads, it is necessary to use stiffened seats, such as the one shown in part (f) of the figure.

The designer selects most of these connections by referring to standard tables. The AISC Manual has excellent tables for selecting bolted or welded beam connections of the types shown in Fig. 15.6. After a rolled-beam section has been selected, it is quite convenient for the designer to refer to these tables and select one of the standard connections, which will be suitable for the vast majority of cases.

In order to make these standard connections have as little moment resistance as possible, the angles used in making up the connections are usually light and flexible. To qualify as simple end supports, the ends of the beams should be as free as possible to rotate downward. Figure 15.7 shows the manner in which framed and seated end connections will theoretically deform as the ends of the beams rotate downward. The designer does not want to do anything that will hamper these deformations if he or she is striving for simple supports.

For the rotations shown in Fig. 15.7 to occur, there must be some deformation of the angles. As a matter of fact, if end slopes of the magnitudes that are computed for simple ends are to occur, the angles will actually bend enough to be stressed beyond their yield points. If this situation occurs, they will be permanently bent and the connections will quite closely approach true simple ends. The student should now see why it is desirable to use rather thin angles and large gages for the bolt spacing if flexible simple end connections are the goal of the designer.

These connections do have some resistance to moment. When the ends of the beam begin to rotate downward, the rotation is certainly resisted to some extent by the tension in the top bolts, even if the angles are quite thin and flexible. Neglecting the moment resistance of these connections will cause conservative beam sizes. If moments of any significance are to be resisted, more rigid-type joints need to be provided than are available with the framed and seated connections.



The beam on the right has a coped top flange, while the beam on the left has both flanges coped because its depth is very close to that of the supporting girder. (Courtesy of CMC South Carolina Steel.)

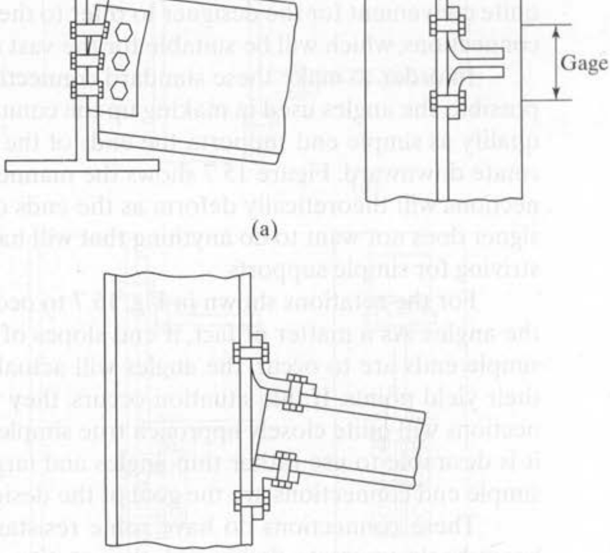
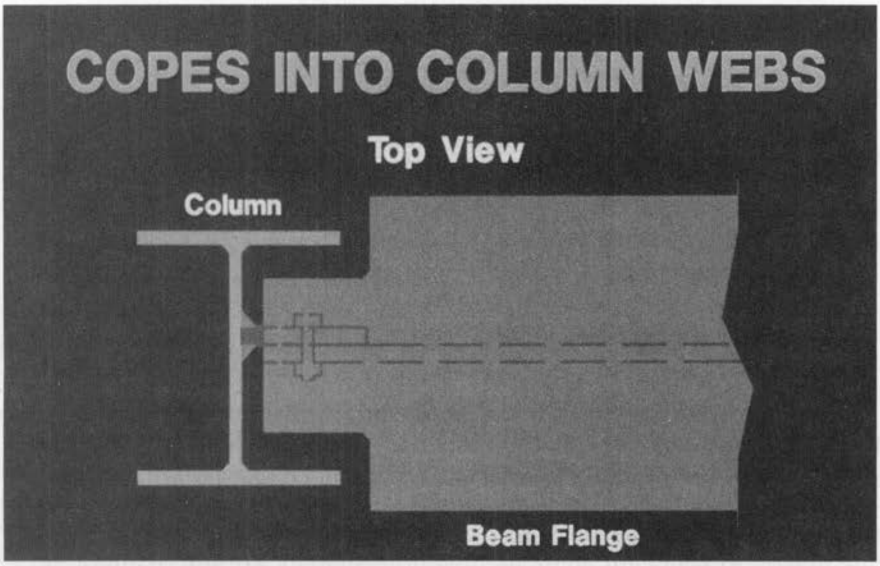


FIGURE 15.7
(a) Bending of framed-beam connection.
(b) Bending of seated-beam connection.



Top view showing a beam framing into a column web where the beam flange is wider than the opening between the column flanges. Thus, both top and bottom flanges are coped by flame-cutting.

15.4 AISC MANUAL STANDARD CONNECTION TABLES

In Part 10 of the AISC Manual, a series of tables is presented that the designer may use to select several different types of standard connections. There are tables for bolted or welded two-angle framed connections, seated beam connections, stiffened seated beam connections, eccentrically loaded connections, single-angle framed connections, and others.

In the next few sections of this chapter (15.5 through 15.8), a few standard connections are selected from the tables in the Manual. The author hopes that these examples will be sufficient to introduce the reader to the AISC tables and to enable him or her to make designs by using the other tables with little difficulty.

Sections 15.9 to 15.11 present design information for some other types of connections.

15.5 DESIGNS OF STANDARD BOLTED FRAMED CONNECTIONS

For small and low-rise buildings (and that means most buildings), simple framed connections of the types previously shown in parts (a) and (b) of Fig. 15.6 are usually used to connect beams to girders or to columns. The angles used are rather thin (1/2 in is the arbitrary maximum thickness used in the AISC Manual), so they will have the necessary flexibility shown in Fig. 15.7. The angles will develop some small moments (supposedly, not more than 20 percent of full fixed-end conditions), but they are neglected in design.

The framing angles extend out from the beam web by 1/2 in, as shown in Fig. 15.8. This protrusion, which is often referred to as the *setback*, is quite useful in fitting members together during steel erection.

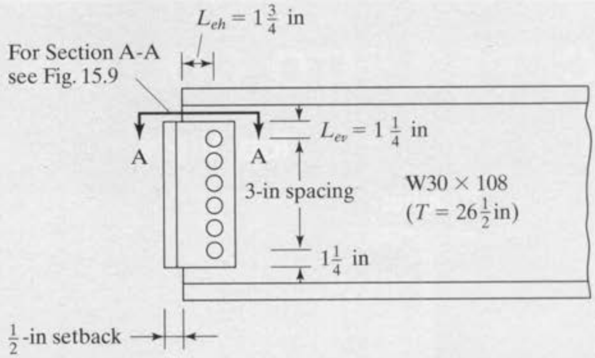


FIGURE 15.8

In this section, several standard bolted framed connections for simple beams are designed, with use of the tables provided in Part 10 of the AISC Manual. In these tables, the following abbreviations for different bolt conditions are used:

1. A325-SC and A490-SC (slip-critical connections)
2. A325-N and A490-N (bearing-type connections with threads included in the shear planes)
3. A325-X and A490-X (bearing-type connections with threads excluded from shear planes)

It is thought that the minimum depth of framing angles should be at least equal to one-half of the distance between the web toes of the beam fillets (called the T distances and given in the properties tables of Part 1 of the Manual). This minimum depth is used so as to provide sufficient stability during steel erection.

Example 15-1 presents the design of standard framing angles for a simply supported beam using bearing-type bolts in standard-size holes. In this example, the design strengths of the bolts and the angles are taken from the appropriate tables.

Example 15-1

Select an all-bolted double-angle framed simple end connection for the uncoped W30 × 108 (t_w = 0.545 in) shown in Fig. 15.8 if R_D = 50 k and R_L = 70 k and if the connection frames into the flange of a W14 × 61 column (t_f = 0.645 in). Assume that F_y = 36 ksi and F_u = 58 ksi for the framing angles and 50 ksi and 65 ksi, respectively, for the beam and column. Use 3/4-in A325-N bolts (bearing-type threads included in shear plane) in standard-size holes.

Solution

LRFD	ASD
$R_u = (1.2)(50) + (1.6)(70) = 172$ k	$R_a = 50 + 70 = 120$ k

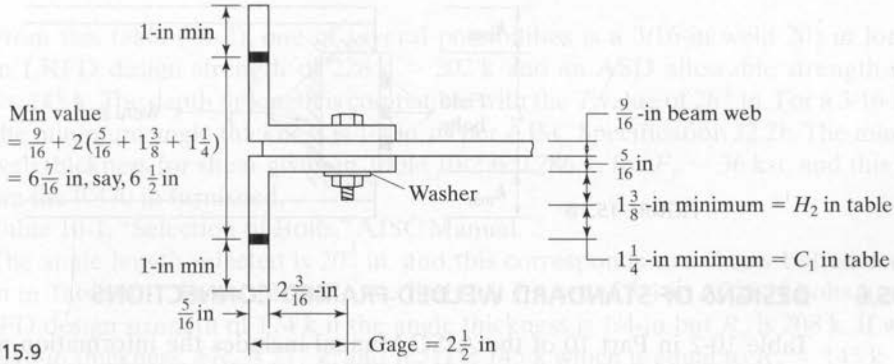


FIGURE 15.9

Using a double-angle connection with $F_y = 36$ ksi and $F_u = 58$ ksi, we look in the tables for the least number of rows of bolts that can be used with a W30 section. It's 5 rows (page 10-20, Part 10 of the AISC Manual), but the connection for 3/4-in A325-N bolts will not support a reaction that large. Thus, we move to a 6-row connection (on page 10-19 in the Manual) and try a connection with an angle thickness of 5/16 in and a length $L = 15 + (2)\left(1\frac{1}{4}\right) = 17.5$ in. (See Fig. 15.8.) This length seems satisfactory compared with the T of 26 1/2 in for this shape that is given in the tables of Part 1 of the Manual.

LRFD	ASD
$\phi R_n = 187\text{ k} > 172\text{ k}$ **OK**	$\frac{R_n}{\Omega} = 124\text{ k} > 120\text{ k}$ **OK**

To select the lengths of the angle legs, it is necessary to study the dimensions given in Fig. 15.9, which is a view along Section A-A in Fig. 15.8. The lower right part of the figure shows the minimum clearances needed for insertion and tightening of the bolts. These are the H_2 and C_1 distances and are obtained for 3/4-in bolts from Table 7.16 in Part 7 of the AISC Manual.

For the legs bolted to the beam web, a $2\frac{1}{2}$-in gage is used. Using a minimum edge distance of 1 in here, we will make these angle legs $3\frac{1}{2}$ in. For the outstanding legs, the minimum gage is $\frac{5}{16} + 1\frac{3}{8} + 1\frac{1}{4} = 2\frac{15}{16}$ in—say, 3 in. We will make this a 4-in angle leg.

Use 2Ls $4 \times 3\frac{1}{2} \times \frac{5}{16} \times 1\text{ ft} - 5\frac{1}{2}$-in **A36**.

From the tables of Part 10 of the AISC Manual, framed bolted connections can be easily selected where one or both flanges of a beam are coped. Coping may substantially reduce the design strength of beams and require the use of larger members or the addition of web reinforcing. Elastic section moduli values for coped sections are provided in Table 9.2 in the Manual.

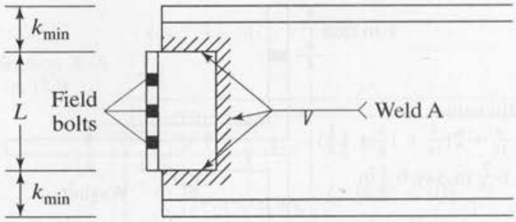


FIGURE 15.10

15.6 DESIGNS OF STANDARD WELDED FRAMED CONNECTIONS

Table 10-2 in Part 10 of the AISC Manual includes the information necessary to use welds instead of bolts as used in Example 15-1. The values in the table are based on E70 electrodes. The table is normally used where the angles are welded to the beams in the shop and then field-bolted to the other member. Should the framing angles be welded to both members, the weld values provided in Table 10-3 in Part 10 of the AISC Manual would be applied.

In the tables, the weld used to connect the angles to the beam web is called Weld A, as shown in Fig. 15.10. If a weld is used to connect the beam to another member, that weld is called Weld B.

For the usual situations, $4 \times 3\frac{1}{2}$-in angles are used with the $3\frac{1}{2}$-in legs connected to the beam webs. The 4-in outstanding legs will usually accommodate the standard gages for the bolts going into the other members. The angle thickness selected equals the weld size plus 1/16 in or the minimum value given in Table 10-1 for the bolts. The angle lengths are the same as those used for the nonstaggered bolt cases (that is, $5\frac{1}{2}$ through $35\frac{1}{2}$ in).

The design strengths of the welds to the beam webs (Weld A) given in Table 10-2 in Part 10 of the AISC Manual were computed by the instantaneous center of rotation method, which we briefly described in Chapter 14. To select a connection of this type, the designer picks a weld size from Table 10-2 and then goes to Table 10-1 to determine the number of bolts required for connection to the other member. This procedure is illustrated in Example 15-2.

Example 15-2

Design a framed beam connection to be welded (SMAW) to a W30 × 90 beam ($t_w = 0.470$ in and $T = 26\frac{1}{2}$ in) and then bolted to another member. The value of the reaction R_D is 75 k, while R_L is 70 k. The steel for the angles is A36, the weld is E70, and the bolts are $\frac{3}{4}$-in A325-N.

Solution. Table 10-2, Weld A Design Strength, AISC Manual.

LRFD	ASD
$R_u = (1.2)(75) + (1.6)(70) = 202$ k	$R_a = 75 + 70 = 145$ k

From this table (10-2), one of several possibilities is a 3/16-in weld $20\frac{1}{2}$ in long with an LRFD design strength of 228 k > 202 k and an ASD allowable strength of 152 k > 145 k. The depth or length is compatible with the T value of $26\frac{1}{2}$ in. For a 3/16-in weld, the minimum angle thickness is 1/4 in, as per AISC Specification J2.2b. The minimum web thickness for shear given in Table 10-2 is 0.286 in for $F_y = 36$ ksi, and this is less than the 0.470 in furnished.

Table 10-1, "Selection of Bolts," AISC Manual.

The angle length selected is $20\frac{1}{2}$ in, and this corresponds to a 7-row bolted connection in Table 10-1. From this table, we find that 7 rows of 3/4-in A325-N bolts have an LRFD design strength of 174 k if the angle thickness is 1/4-in but R_u is 208 k. If we use a 5/16-in thickness, ϕR_n is 217 k; and R_n/Ω is 145 k which is equal to $R_a = 145$ k

Use 2Ls $4 \times 3\frac{1}{2} \times \frac{5}{16} \times 1$ ft $8\frac{1}{2}$ in **A36 steel for LRFD and ASD.**

Table 10-3 in Part 10 of the AISC Manual provides the information necessary for designing all-welded standard framed connections—that is, with Welds A and B as shown in Fig. 15.11. The design strengths for Weld A were determined by the ultimate strength, instantaneous center, or rotation method, while the values for Weld B were determined by the elastic method. A sample design is presented in Example 15-3.

Example 15-3

Select an all-welded double-angle framed simple end connection for fastening a W30 × 108 beam ($t_w = 0.545$ in) to a W14 × 61 column ($t_f = 0.645$ in). Assume that $F_y = 50$ ksi and $F_u = 65$ ksi for the members, with $R_D = 60$ k and $R_L = 80$ k.

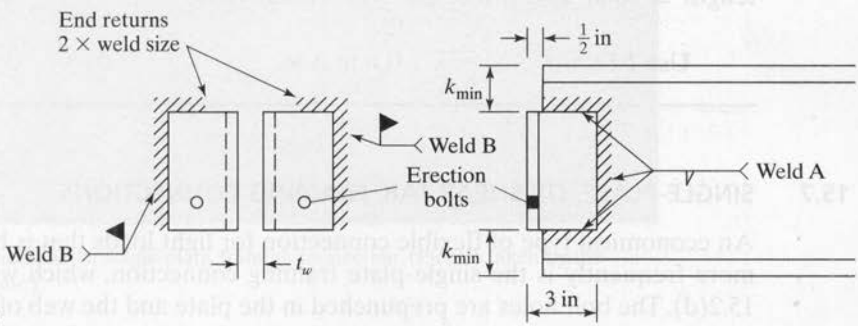


FIGURE 15.11

Solution

LRFD	ASD
$R_u = (1.2)(60) + (1.6)(80) = 200$ k	$R_a = 60 + 80 = 140$ k

Select Weld A for beam

From Table 10-3 in the AISC Manual, one possibility is a 3/16-in weld 20 in long. For a beam with $F_y = 50$ ksi, the minimum web thickness is 0.286 in, which is $< t_w$ of 0.545 in of the beam.

LRFD	ASD
$\phi R_n = 223 \text{ k} > 200 \text{ k OK}$	$\dfrac{R_n}{\Omega} = 149 \text{ k} > 140 \text{ k OK}$

Select Weld B for column

The 1/4-in weld will not provide sufficient capacity. $\therefore$ Use 5/16-in weld. For a column with $F_y = 50$ ksi, the minimum flange t is 0.238 in $< t_f$ of 0.545 in of the column.

LRFD	ASD
$\phi R_n = 226 \text{ k} > 200 \text{ k } \textbf{OK}$	$\dfrac{R_n}{\Omega} = 151 \text{ k} > 140 \text{ k } \textbf{OK}$

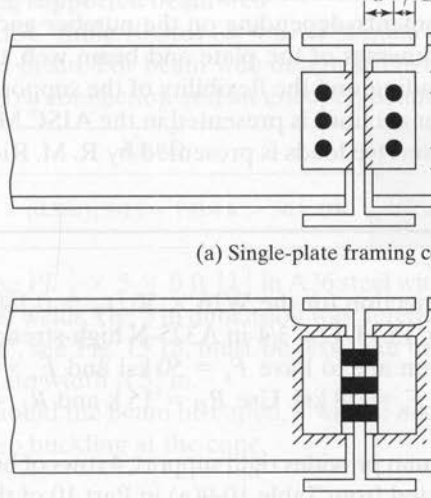
$$\text{Minimum angle thickness} = \frac{5}{16} + \frac{1}{16} = \frac{3}{8} \text{ in.}$$

On page 10-12 in the Manual, the AISC says to use 4×3 angles when angle length ≥ 18 in. Use two angles 3×3 otherwise.

Use 2 Ls $4 \times 3 \times \dfrac{3}{8} \times 1$ ft 8 in **A36**.

15.7 SINGLE-PLATE, OR SHEAR TAB, FRAMING CONNECTIONS

An economical type of flexible connection for light loads that is being used more and more frequently is the single-plate framing connection, which was illustrated in Fig. 15.2(d). The bolt holes are prepunched in the plate and the web of the beam. The plate is then shop-welded to the supporting beam or column; lastly, the beam is bolted to the plate in the field. Steel erectors like this kind of connection because of its simplicity. They are particularly pleased with it when there is a beam connected to each side of a girder, as shown in Fig. 15.12(a). All they have to do is bolt the beam webs to the single plate on each side of the girder. Should web clip angles be used for such a connection, the bolts will have to pass through the angles on each side of the girder, as well as the girder web, as shown in part (b) of the figure. This is a slightly more difficult field

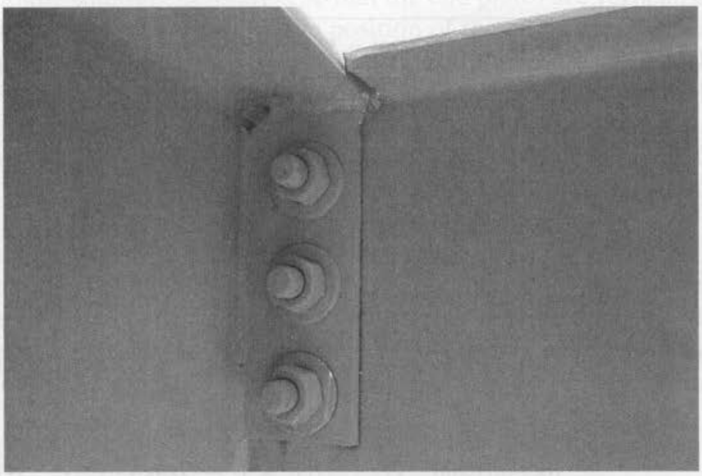


(a) Single-plate framing connection

(b) Simple connection with web angles

FIGURE 15.12

(a) Single-plate framing connection. (b) Simple connection with web angles.



A shear tab, or single-plate, framing connection. (Picture taken by the author.)

operation. Frequently, one side will have one extra bolt row so that each beam can be erected separately.

With the single-plate connection, the reaction or shear load is assumed to be distributed equally among the bolts passing through the beam web. It is also assumed that relatively free rotation occurs between the end of the member and the supporting beam or column. Because of these assumptions, this type of connection often is referred to as a "shear tab" connection. Various studies and tests have shown that these connections can develop some end moments, depending on the number and size of the bolts and their arrangement, the thicknesses of the plate and beam web, the span-to-depth ratio of the beam, the type of loading, and the flexibility of the supporting element.

Table 10-9 for shear tab connections is presented in the AISC Manual. An empirical design procedure that uses service loads is presented by R. M. Richard et al.[11]

Example 15-4

Design a single-plate shear connection for the W16 × 50 ($t_w = 0.380$ in) beam to the W14 × 90 column shown in Fig. 15.13. Use 3/4-in A325-N high-strength bolts and E70 electrodes. The beam and column are to have $F_y = 50$ ksi and $F_u = 65$ ksi, while the plate is to have $F_y = 36$ ksi and $F_u = 58$ ksi. Use $R_D = 15$ k and $R_L = 20$ k.

Solution. Assuming that the column provides rigid support, 4 rows of bolts, a 1/4-in plate, and 3/16-in fillet welds are selected from Table 10-9(a) in Part 10 of the AISC Manual.

LRFD	ASD
$R_U = (1.2)(15) + (1.6)(20) = 50$ k	$R_a = 15 + 20 = 35$ k

LRFD	ASD
$\phi R_n = 52.2$ k > 50 k **OK**	$\dfrac{R_n}{\Omega} = 34.8$ k ≈ 35 k **OK**

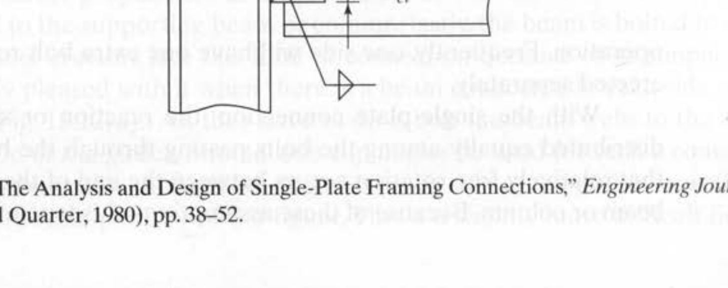


FIGURE 15.13

[11]R. M. Richard et al., "The Analysis and Design of Single-Plate Framing Connections," *Engineering Journal*, AISC, vol. 17, no. 2 (2nd Quarter, 1980), pp. 38–52.

Checking supported beam web

From AISC Table 10-9(a) for 4 rows of bolts with $L_{ev} = 1\frac{1}{4}$ in, and $L_{eh} = 1\frac{3}{4}$ in, and an uncoped beam. For beam web design strength, reference is made to AISC Table 10-1 for a 4-row connection and an uncoped beam.

LRFD	ASD
$\phi R_n = (0.380)(351) = 133.4\text{ k} > 50\text{ k}$ **OK**	$\dfrac{R_n}{\Omega} = (0.380)(234) = 88.9\text{ k} > 35\text{ k}$ **OK**

Use PL $\frac{1}{4} \times 5 \times 0$ ft $11\frac{1}{2}$ in A36 steel with 4 rows of 3/4-in A325 N bolts and 3/16-in E70 fillet welds. The 5 in dimension was determined from page 10-102 in the Manual.

"a", see Fig. 15.13, must be less than or equal to $3\frac{1}{2}$ in. If L_{eh} is $1\frac{3}{4}$ in, then maximum plate width is $5\frac{1}{4}$ in.

Should the beam be coped, it will be necessary to also check flexural yielding and local web buckling at the cope.

15.8 END-PLATE SHEAR CONNECTIONS

Another type of connection is the *end-plate* connection. It consists of a plate shop-welded flush against the end of a beam and field-bolted to a column or another beam. To use this type of connection, it is necessary to carefully control the length of the beam and the squaring of its ends, so that the end plates are vertical. Camber must also be considered as to its effect on the position of the end plate. After a little practice in erecting members with end-plate connections, fabricators seem to like to use them. Nonetheless, there still is trouble in getting the dimensions just right, and end-plate connectors are not as commonly used as the single-plate connectors.

Part (a) of Fig. 15.14 shows an end-plate connection that is satisfactory for PR situations. End-plate connections are illustrated in Fig. 12-6 of the AISC Manual. Should the end plate be extended above and below the beam, as shown in part (b) of Fig. 15.14, appreciable moment resistance will be achieved.

Table 10-4 of the AISC Manual provides tables and a procedure for designing extended end-plate connections. These connections may be designed as FR, for statically loaded structures, for buildings in areas of low seismicity. Their design is described in Part 12 of the Manual.

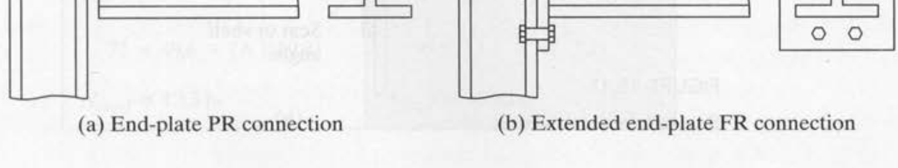


FIGURE 15.14 (a) End-plate PR connection (b) Extended end-plate FR connection

15.9 DESIGNS OF WELDED SEATED BEAM CONNECTIONS

Another type of fairly flexible beam connection is achieved by the use of a beam seat, such as the one shown in Fig. 15.15. Beam seats obviously offer an advantage to the worker performing the steel erection. The connections for these angles may be bolts or welds, but only the welded type is considered here. For such a situation, the seat angle would usually be shop-welded to the column and field-welded to the beam. When welds are used, the seat angles, also called *shelf angles*, may be punched for erection bolts, as shown in the figure. These holes can be slotted, if desired, to permit easy alignment of the members.

A seated connection may be used only when a top angle is used, as shown in Fig. 15.15. This angle provides lateral support for the beam and may be placed on top of the beam, or at the optional location shown on the side of the beam in part (a) of the figure. As the top angle is not usually assumed to resist any of the load, its size probably is selected by judgment. Fairly flexible angles are used that will bend away from the column or girder to which they are connected when the beam tends to rotate downward

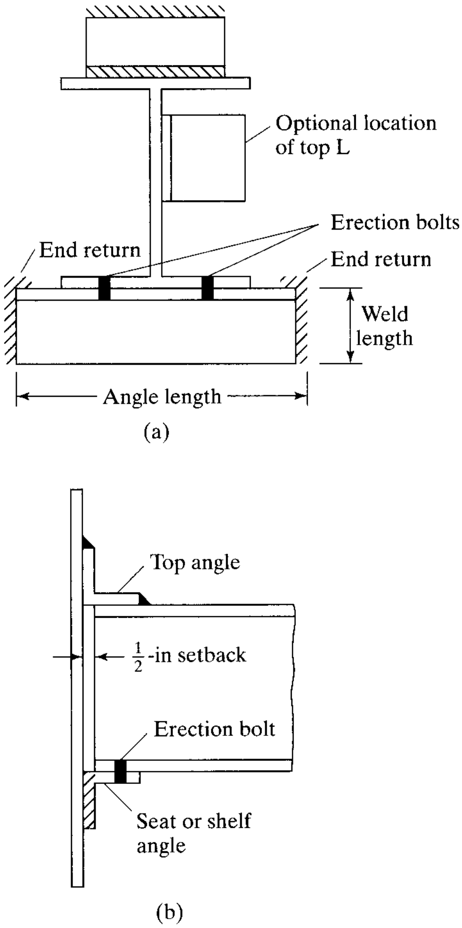


FIGURE 15.15

Seated beam connection.

under load. This desired situation is illustrated in Fig. 15.7(b). A common top angle size selected is 4 × 4 × 1/4.

As will be seen in AISC Tables 10-5 through 10-8, unstiffened seated beam connections of practical sizes can support only fairly light factored loads. For such light loads, two vertical end welds on the seat are sufficient. The top angle is welded only on its toes, so when the beam tends to rotate, this thin flexible angle will be free to pull away from the column.

The seat design strengths given in the AISC tables were developed for seat angles with either 3 1/2- or 4-in outstanding legs. The steel used for the angles is A36 with F_y = 36 ksi and F_u = 58 ksi.

The design strengths in the tables were obtained with consideration given to both shear and flexural yielding of the outstanding leg of the seat angle, and crippling of the beam web as well. The values were calculated on the basis of a 3/4-in setback rather than the nominal 1/2 in used for web framing angles. This larger value was used to provide for possible mill underrun in the beam lengths. Example 15-5 illustrates the use of the Manual tables for designing a welded unstiffened seated beam connection. Other tables are included in the Manual for bolted seated connections, as well as bolted or welded stiffened seated connections.

Example 15-5

Design an all-welded unstiffened seated connection with E70 electrodes to support the reactions $R_D = 20$ k and $R_L = 30$ k from a W24 × 55 beam (d = 23.6 in, t_w = 0.395 in, t_f = 0.505 in, and k = 1.01 in). The connection is to be made to the flange of a W14 × 68 column (t_f = 0.720 in). The angles are A36, while the beam and column have an F_y = 50 ksi and an F_u = 65 ksi.

Solution. Design seat angle and welds.

LRFD	ASD
$R_u = (1.2)(20) + (1.6)(30) = 72$ k	$R_a = 20 + 30 = 50$ k

Checking local web yielding, assuming that N = $3\frac{1}{2}$ in for outstanding leg

Using AISC Equation J10-2 and AISC Table 9-4

LRFD	ASD
$\phi R_1 = 49.6$ and $\phi R_2 = 19.8$	$\dfrac{R_1}{\Omega} = 33.1$ and $\dfrac{R_2}{\Omega} = 13.2$
$\phi R_n = \phi R_1 + N(\phi R_2)$	$\dfrac{R_n}{\Omega} = \dfrac{R_1}{\Omega} + N\left(\dfrac{R_2}{\Omega}\right)$
$72 = 49.6 + (N)(19.8)$	$50 = 33.1 + N(13.2)$
$N_{req'd} = 1.13$ in	$N_{req'd} = 1.28$ in

Checking web crippling

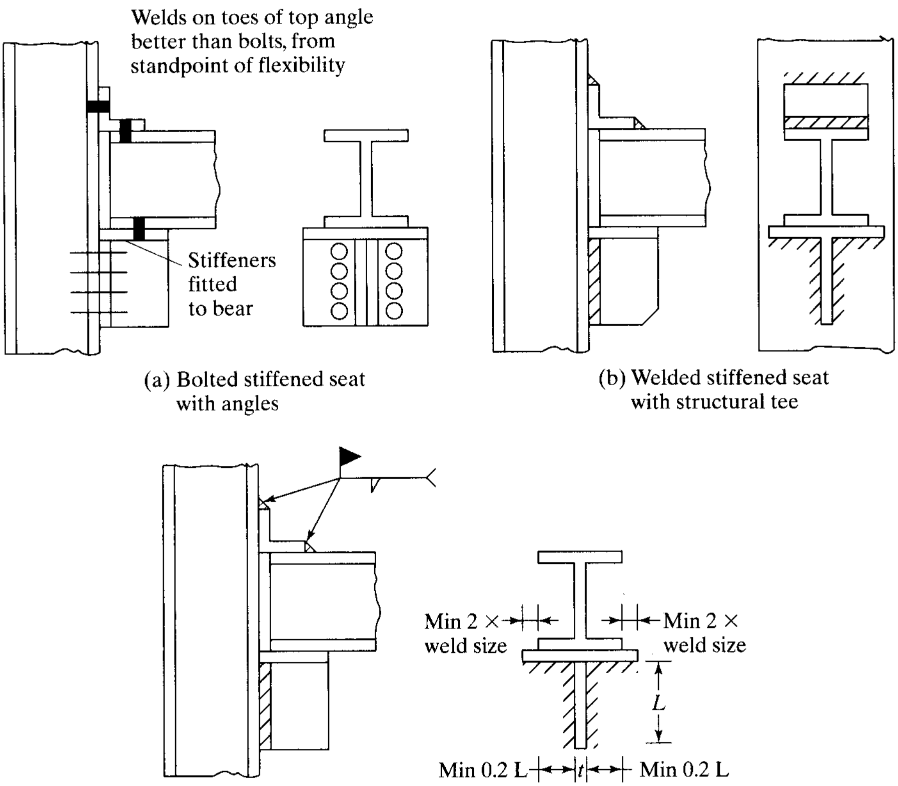
LRFD	ASD
$\phi R_3 = 63.7$ and $\phi R_4 = 5.61$	$\dfrac{R_3}{\Omega} = 42.5$ and $\dfrac{R_4}{\Omega} = 3.74$
$\dfrac{N}{d} = \dfrac{3.5}{23.6} = 0.148 < 0.2$	$\dfrac{N}{d} = \dfrac{3.5}{23.6} = 0.148 < 0.2$
$\therefore$ Use AISC Equation J10-5a	$\therefore$ Use AISC Equation J10-5a
$\phi R_n = \phi R_3 + N(\phi R_4)$	$\dfrac{R_n}{\Omega} = \dfrac{R_3}{\Omega} + N\left(\dfrac{R_4}{\Omega}\right)$
$72 = 63.7 + N(5.61)$	$50 = 42.5 + (N)(3.74)$
$N_{req'd} = 1.48$ in	$N_{req'd} = 2.00$ in

Use 4-in angle leg with 1/2 in nominal setback. $\therefore$ Bearing length, $N = 3.5$ in. Using AISC Table 10-6 for an 8×4 angle, determine t

LRFD	ASD
With $N_{req} = 1.48$ in, say $1\frac{1}{2}$ in	With $N_{req} = 2.00$ in
From Table 10.6 upper portion, an 8 in angle length with 3/4-in thickness will provide:	From Table 10.6 upper portion, an 8 in angle length with 3/4-in thickness will provide:
$\phi R_n = 97.2$ k > 72 k **OK**	$\phi R_n = 38.8$ k > 50 k **N.G.** Increase angle thickness to $\frac{7}{8}$-in.
From Table 10.6 lower portion, an 8×4 angle with a 3/8-in weld will provide:	From Table 10.6 lower portion, an 8×4 angle with a 3/8-in weld will provide:
$\phi R_n = 80.1$ k > 72 k **OK**	$\phi R_n = 53.4$ k > 50 k **OK**
Use L8 × 4 × 3/4 × 0 ft 8 in with 3/8-in weld.	**Use L8 × 4 × 7/8 × 0 ft 8 in with 3/8-in weld.**

15.10 DESIGNS OF STIFFENED SEATED BEAM CONNECTIONS

When beams are supported by seated connections and when the factored reactions become fairly large, it is necessary to stiffen the seats. These larger reactions cause moments in the outstanding or horizontal legs of the seat angles that cannot be supported by standard thickness angles unless they are stiffened in some manner. Typical stiffened seated connections are shown in Fig. 15.6(f) and in Fig. 15.16.



(a) Bolted stiffened seat with angles

(b) Welded stiffened seat with structural tee

(c) Welded two-plate stiffened seat

FIGURE 15.16

(a) Bolted stiffened seat with angles. (b) Welded stiffened seat with structural tee. (c) Welded two-plate stiffened seat.

Stiffened seats may be bolted or welded. Bolted seats may be stiffened with a pair of angles, as shown in part (a) of Fig. 15.16. Structural tee stiffeners either bolted or welded may be used. A welded one is shown in part (b) of the same figure. Welded two-plate stiffeners, such as the one shown in part (c) of the figure, also are commonly used. AISC Table 10-8 provides information for designing stiffened seated connections.

15.11 DESIGNS OF MOMENT-RESISTING FR MOMENT CONNECTIONS

In this section, a brief introduction to moment-resisting connections is presented. It is not the author's intention to describe in detail all of the possible arrangements of bolted and welded moment-resisting connections at this time, nor to provide a complete design. Instead, he attempts to provide the basic theory of transferring shear and moment from a beam to another member. One numerical example is included. This theory is very easy to understand and should enable the reader to design other moment-resisting connections—regardless of their configurations.

One moment-resisting connection that is popular with many fabricators is shown in Fig. 15.17. There, the flanges are groove-welded to the column, while the shear is

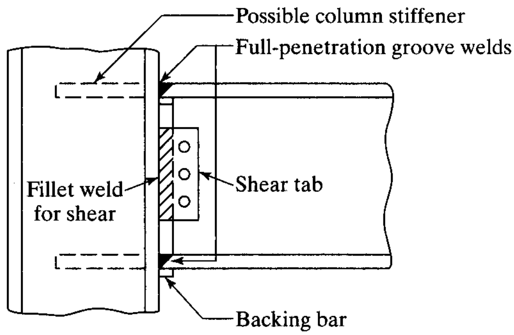


FIGURE 15.17

A moment-resisting connection.

carried separately by a single plate or shear tab connection. (Shear tab connections were previously described in detail in Section 15.7.)

The reader should realize that, at the 1994 Northridge earthquake in California, quite a few brittle fractures were initiated in connections of the type shown in Fig. 15.17. These fractures apparently began at or near the full penetration groove welds between the bottom flanges and the column flanges. Among the factors involved in these failures were notch effects caused by the backup or backing bars, which were commonly left in place. Other factors were welds with porosity and slag inclusions, inconsistent strength and deformation capacities of the steel sections, and so on.

Detailed information on these problems at Northridge was provided in FEMA 267 Report Number SAC-95-02, entitled "Interim Guidelines, Evaluation, Repair, Modification and Design of Steel Moment Frames," dated August 1995. In "Interim Guidelines Advisory No. 1 Supplement to FEMA 267," published in March 1997, several recommendations were presented for correcting the problems. These included the removal of backup bars and weld tabs, the incorporation of full-scale inelastic testing of joints of the types used, and several others.

To design a moment-resisting connection, the first step is to compute the magnitude of the internal compression and tension forces, C and T. These forces are assumed to be concentrated at the centers of the flanges, as shown in Fig. 15.18.

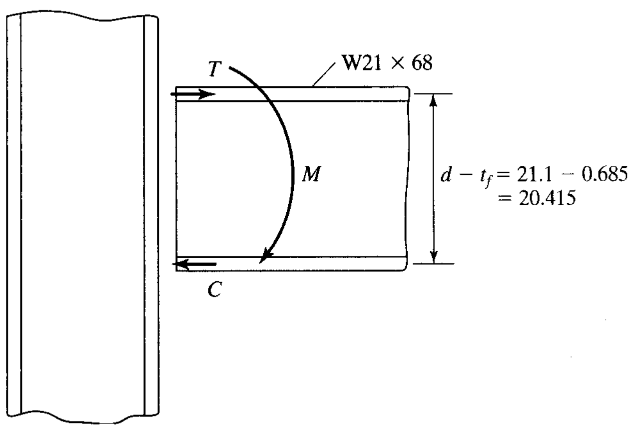


FIGURE 15.18

Moment-resisting couple C or T $(d - t_f) = M$

$$C = T = \frac{M}{d - t_f}$$

Next, the areas of the full-penetration welds up against the column are determined. They equal the magnitude of C or T divided by the design stress of a full penetration groove weld, as provided in Table 14.1 (AISC Table J2.5), with $\phi = 0.9$.

$$\text{Area reqd.} = \frac{C_u \text{ or } T_u}{\phi F_y} \text{ or } \frac{C_a \text{ or } T_a}{F_y/\Omega}$$

With this procedure, it is theoretically possible to have a weld area larger than the cross-sectional area of the flange. It would then be theoretically necessary to use an auxiliary plate on the flange to resist the extra force. (We may just transfer all of the forces with plates on the flanges. Sometimes, the beam flanges are groove-welded flush with the column on one end and connected to the beam on the other end with the auxiliary plates just described. This could help us solve our problems of fit. The forces are transferred from the beam to the plate with fillet welds and from the plate to the column by groove welds.)

Recent research at the University of California and Lehigh University has shown that the full plastic moment capacity of a beam can be developed with full-penetration welds made only to the flanges.

Example 15-6 illustrates the design of a moment-resisting connection with flange full-penetration groove welds. The reader should understand that this example is not quite complete. It is necessary also to design the shear plate, seat angle, or whatever is used to transfer the shear, and to check the column for the concentrated T or C force.

Example 15-6

Design a moment-resisting connection for the W21 × 68 beam shown in Fig. 15.18, with the flanges groove-welded to a column. The beam, which consists of 50 ksi steel, has end reactions $R_D = 20$ k and $R_L = 20$ k, along with moments $M_D = 60$ ft-k and $M_L = 90$ ft-k. Use E70 electrodes.

Solution

Using a W21 × 68 ($d = 21.1$ in, $b_f = 8.27$ in, $t_f = 0.685$ in, $T = 18\frac{3}{8}$ in)

Design of moment welds

LRFD $\phi = 0.90$	ASD $\Omega = 1.67$
$M_u = (1.2)(60) + (1.6)(90) = 216$ ft-k	$M_a = 60 + 90 = 150$ ft-k
$C_u = T_u = \dfrac{(12)(216)}{21.1 - 0.685} = 127$ k	$C_a = T_a = \dfrac{(12)(150)}{21.1 - 0.685} = 88.17$ k
A of groove weld $= \dfrac{127}{(0.9)(50)} = 2.82$ in^2	A of groove weld $= \dfrac{88.17}{50/1.67} = 2.94$ in^2
width reqd $= \dfrac{2.82}{t_f} = \dfrac{2.82}{0.685} = 4.12$ in $< b_f$	width reqd $= \dfrac{2.94}{t_f} = \dfrac{2.94}{0.685} = 4.29$ in $< b_f$
Use 5-in-wide E70 full-penetration groove welds.	**Use 5-in-wide E70 full-penetration groove welds.**

Design of shear welds

Try 1/4-in fillet welds on shear tabs (or on seat angle or on beam web)

$$R_n \text{ of weld per in} = F_{nw}A_{we} = (0.60 \times 70)\left(\frac{1}{4} \times 0.707\right) = 7.42 \text{ k/in}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$R_u = (1.2)(20) + (1.6)(20) = 56$ k	$R_a = 20 + 20 = 40$ k
$\phi R_n = (0.75)(7.42) = 5.56$ k/in	$\frac{R_n}{\Omega} = \frac{7.42}{2.00} = 3.71$ k/in
weld length reqd $= \frac{56}{5.56} = 10.07$ in	weld length $= \frac{40}{3.71} = 10.78$ in
Use $\frac{1}{4}$-in fillet welds $5\frac{1}{2}$ in long each side.	**Use $\frac{1}{4}$-in fillet welds $5\frac{1}{2}$ in long each side.**

Note: The design is incomplete, as it is necessary to design the shear tab, seat L, or whatever is used, and to check the column for the calculated shear force (C or T) from the beam.

Figure 15.19 shows a moment-resisting connection where the C and T forces are carried by cover plates on the top and bottom of a W section. The moment to be resisted is divided by the distance between the centers of gravity of the top and bottom parts of the couple (C and T), and then welds or bolts are selected that will provide the necessary design strengths so determined. Next, a shear tab, a pair of framing angles, or a beam seat is selected to resist the shear force. Finally, as described in the next section, it may be necessary to provide stiffeners for the column web, or to select a larger column section.

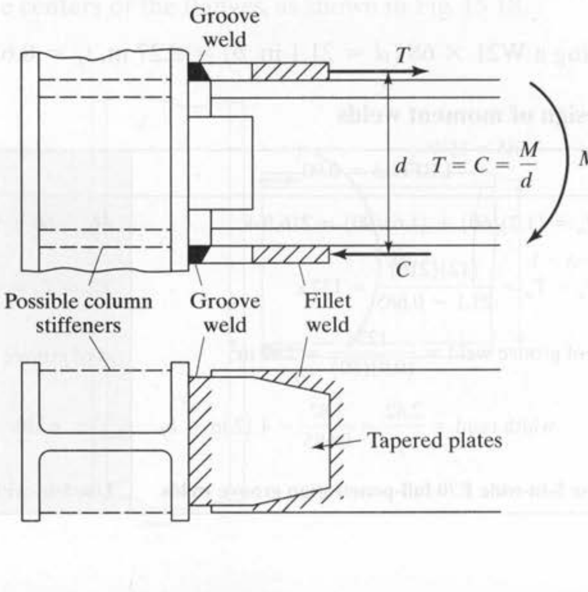


FIGURE 15.19

In this particular connection, the T and C values are transferred by fillet welds into the plates and by groove welds from the plates to the columns. For easier welding, these plates may be tapered as shown in the bottom of the figure. You may have noticed such tapered plates used for facilitating welding in other situations.

For a rigid or continuous connection of the type shown in Fig. 15.19, we must be careful to check the strength of the top and bottom plates. Should the plates be bolted, this check involves the tensile strength of the top plate, including the effect of the bolt holes and block shear. The design compressive strength of the other plate must also be checked.

15.12 COLUMN WEB STIFFENERS

If a column to which a beam is being connected bends appreciably at the connection, the moment resistance of the connection will be reduced, regardless of how good the connection may be. Furthermore, if the top connection plate, in pulling away from the column, tends to bend the column flange as shown in part (a) of Fig. 15.20, the middle part of the weld may be greatly overstressed (like the prying action for bolts discussed in Chapter 13).

When there is a danger of the column flange bending, as described here, we must make sure that the desired moment resistance of the connection is provided. We may do this either by using a heavier column with stiffer flanges or by introducing column web stiffener plates, as shown in part (b) of Fig. 15.20. *It is almost always desirable to use a heavier column, because column web stiffener plates are quite expensive and a nuisance to use.*

Column web stiffener plates are somewhat objectionable to architects, who find it convenient to run pipes and conduits inside their columns; this objection can

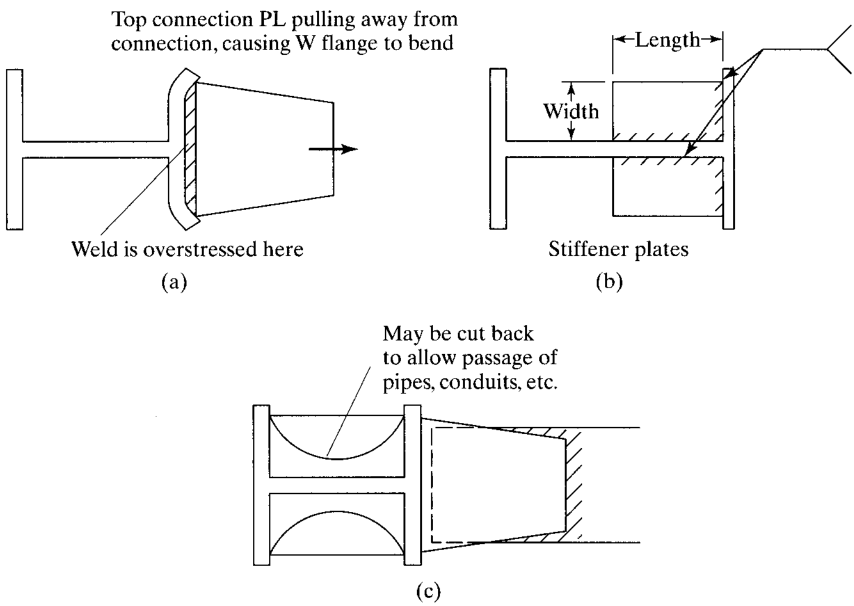


FIGURE 15.20

easily be overcome, however. First, if the connection is to only one column flange, the stiffener does not have to run for more than half the column depth, as shown in part (b) of Fig. 15.20. If connections are made to both column flanges, the column stiffener plates may be cut back to allow the passage of pipes, conduits, etc., as shown in part (c) of the figure.

Should the LRFD or ASD forces applied from the beam flange to the column be greater than any of the values given by the AISC equations for flange local bending, web local yielding, web crippling, and web compression buckling, it will be necessary to use column stiffeners or doubler plates for the column web or to select a column with a thicker flange. The equations for these items were previously presented in Chapter 10 of this text. Their application is again illustrated in the solution of Example 15-7.

The AISC Manual presents a set of suggested rules for the design of column web stiffeners. These are as given in AISC specificaion J10.

1. The width of the stiffener plus one-half of the column web thickness should not be less than one-third the width of the beam flange or of the moment connection plate that applies the concentrated force.
2. The stiffener thickness should not be less than $t_f/2$ or half the thickness of the moment connection plate delivering the concentrated load and not less than the width divided by 16.
3. If there is a moment connection applied to only one flange of the column, the length of the stiffener plate does not have to exceed one-half the column depth.
4. The stiffener plate should be welded to the column web with a sufficient strength to carry the force caused by the unbalanced moment on the opposite sides of the column.

For the column given in Example 15-7, it is necessary to use column web stiffeners or to select a larger column. These alternatives are considered in the solution.

Example 15-7

It is assumed that a particular column is a W12 $\times$ 87 consisting of 50 ksi steel and subjected to $C_D = T_D = 60$ k and $C_L = T_L = 90$ k transferred by an FR-type connection from a W18 $\times$ 46 beam on one side of the column. The connection is located a distance $>d$ from the end of the column. It will be found that this column is not satisfactory to resist these forces. (a) Select a larger W12 column section that will be satisfactory. (b) Using a W12 $\times$ 87 column, design column web stiffeners, including the stiffener connections and using E70 SMAW welds.

Solution

Beam is a W18 $\times$ 46 ($b_f = 6.06$ in, $t_f = 0.605$ in)

Column is a W12 $\times$ 87 ($d = 12.5$ in, $t_w = 0.515$ in, $t_f = 0.810$ in, $k = 1.41$ in)

Checking to see if forces transferred to column are too large.

LRFD	ASD
$C_u = (1.2)(60) + (1.6)(90) = 216\text{ k}$	$C_a = 60 + 90 = 150\text{ k}$

Flange local bending

$$R_n = (6.25)(0.810\text{ in})^2(50\text{ ksi}) = 205\text{ k} \qquad \text{(AISC Equation J10-1)}$$

LRFD $\phi = 0.90$	ASD $\Omega = 1.67$
$\phi R_n = (0.90)(205) = 184.5\text{ k} < 216\text{ k}$ **N.G.**	$\dfrac{R_n}{\Omega} = \dfrac{205}{1.67} = 122.8\text{ k} < 150\text{ k}$ **N.G.**

$\therefore$ Must use a larger column or a pair of transverse stiffeners.

Web local yielding

$$R_n = (5 \times 1.41\text{ in} + 6.06\text{ in})(50\text{ ksi})(0.515\text{ in}) = 337.6\text{ k} \qquad \text{(AISC Equation J10-2)}$$

LRFD $\phi = 1.00$	ASD $\Omega = 1.5$
$\phi R_n = (1.00)(337.6) = 337.6 > 216\text{ k}$ **OK**	$\dfrac{R_n}{\Omega} = \dfrac{337.6}{1.5} = 225.1\text{ k} > 140\text{ k}$ **OK**

Web crippling

$$R_n = (0.80)(0.515\text{ in})^2\left[1 + 3\left(\frac{6.06\text{ in}}{12.5\text{ in}}\right)\left(\frac{0.515\text{ in}}{0.810\text{ in}}\right)^{1.5}\right]\sqrt{\frac{(29 \times 10^3\text{ ksi})(50\text{ ksi})(0.810\text{ in})}{0.515\text{ in}}}$$

$$= 556.7\text{ k} \qquad \text{(AISC Equation J10-4)}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi R_n = (0.75)(556.7) = 417.5\text{ k} > 216\text{ k}$ **OK**	$\dfrac{R_n}{\Omega} = \dfrac{556.7}{2.00} = 278.3\text{ k} > 150\text{ k}$ **OK**

(a) selecting a larger column

Try W12 × 96 ($t_f = 0.900$)

Flange local buckling

$$R_n = (6.25)(0.900\text{ in})^2(50\text{ ksi}) = 253.1\text{ k} \qquad \text{(AISC Equation J10-1)}$$

LRFD $\phi = 0.90$	ASD $\Omega = 1.67$
$\phi R_n = (0.90)(253.1) = 227.8\text{ k} > 216\text{ k}$ **OK**	$\frac{R_n}{\Omega} = \frac{253.1}{1.67} = 151.6\text{ k} > 150\text{ k}$ **OK**

Use W12 × 96 column.

(b) Design of web stiffeners using a W12 × 87 column and the suggested rules presented before this example. The author shows only the LRFD solution for this part of problem.

$$\text{Reqd stiffener area} = \frac{216\text{ k} - 184.5\text{ k}}{50\text{ ksi}} = 0.63\text{ in}^2$$

$$\text{Min width} = \frac{1}{3}b_f - \frac{t_w}{2} = \frac{6.06}{3} - \frac{0.515}{2} = 1.76\text{ in}$$

$$\text{Min } t \text{ of stiffeners} = \frac{0.63\text{ in}^2}{1.76\text{ in}} = 0.358\text{ in } \textbf{say, 3/8 in}$$

$$\text{Reqd width} = \frac{0.63\text{ in}^2}{0.375\text{ in}} = 1.68\text{ in } \textbf{say, 4 in for practical purposes}$$

$$\text{Minimum length} = \frac{d}{2} - t_f = \frac{12.5}{2} - 0.810 = 5.45\text{ in } \textbf{say, 6 in}$$

Design of welds for stiffener plates
Minimum weld size as reqd by AISC Table J-2.4

$$= \frac{3}{16}\text{ in based on the column web } t_w = 0.515\text{ in}$$

$$\text{Reqd length of weld} = \frac{216\text{ k} - 184.5\text{ k}}{(0.75)(0.60 \times 70\text{ ksi})(0.707)\left(\frac{3}{16}\text{ in}\right)} = 7.54\text{ in } \textbf{say, 8 in}$$

15.13 PROBLEMS FOR SOLUTION

For Problems 15-1 through 15-15 use the tables of Part 10 of the AISC Manual.

15-1. Determine the maximum end reaction that can be transferred through the A36 web angle connection shown in the accompanying illustration. Solve by LRFD and ASD. The beam steel is 50 ksi, and the bolts are 3/4-in A325-N and are used in standard-size holes. The beam is connected to the web of a W30 × 90 girder with A36 angles. (*Ans.* 126 k, 83.9 k)

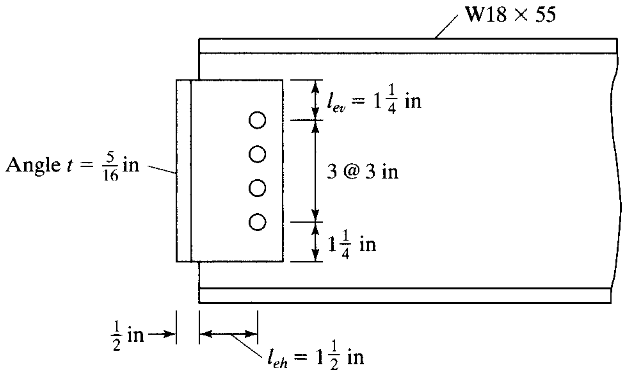


FIGURE P15-1

15-2. Repeat Prob. 15-1 if the bolts are 1-in A325-X.

15-3. Repeat Prob. 15-1 if the bolts are 7/8-in A325-N. (*Ans.* 122 k, 81.6 k).

15-4. Using the AISC Manual, select a pair of bolted standard web angles (LRFD and ASD) for a W33 × 141 connected to the flange of a W14 × 120 column with a dead load service reaction of 75 k and a live load service reaction of 50 k. The bolts are to be 7/8-in A325-N in standard-size holes, and the steel is A36 for the angles and A992 for the W shapes.

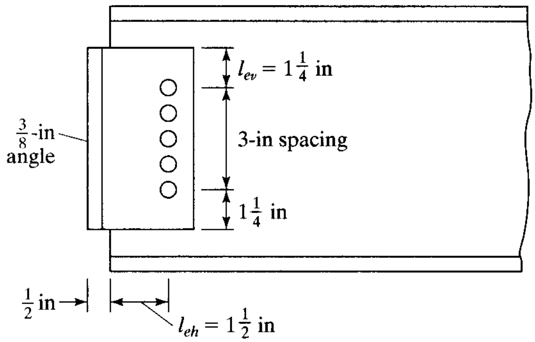


FIGURE P15-4

15-5. Repeat Prob. 15-4 if 1-in A325-N bolts are to be used. (*Ans.* 6 row conn. with 2Ls 4 × $3\frac{1}{2}$ × $\frac{3}{8}$ × 1 ft − $5\frac{1}{2}$ in)

15-6. Repeat Prob. 15-1 if 3/4-in A325 SC Class A bolts are to be used.

15-7. Design framed beam connections for a W27 × 84 connected to a W30 × 116 girder web to support a dead load reaction of 40 k and a live load reaction of 50 k, using the LRFD and ASD methods. The bolts are to be 7/8-in A325 SC Class A in standard-size holes. Angles are A36 steel, while beams are A992. The edge distances and bolt spacings are the same as those shown in the sketch for Prob. 15-4. (*Ans.* 5 row connection with 2Ls 5 × $3\frac{1}{2}$ × $\frac{3}{8}$ × 1 ft − $2\frac{1}{2}$ in)

15-8. Repeat Prob. 15-1 if the bolts are 3/4-in A490-SC Class A and are used in $1\frac{1}{16} \times 1\frac{5}{16}$ in short slots with long axes perpendicular to the transmitted force. Angle t is 1/2 in.

15-9. Design a framed beam connection for a W18 × 50 to support a dead load reaction of 30 k and a live load reaction of 20 k, using the LRFD and ASD methods. The beam's top flange is to be coped for a 2-in depth, and 7/8-in A325-X bolts in standard-size holes are to be used. The beam is connected to a W27 × 146 girder. Connection is A36, while W shapes are A992. (*Ans.* 4 row connection 2Ls 5 × $3\frac{1}{2}$ × $\frac{1}{4}$ × 0 ft − $11\frac{1}{2}$ in)

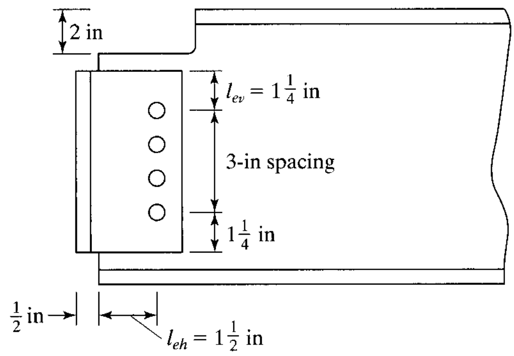


FIGURE P15-9

15-10. Repeat Prob. 15-7 if the dead load reaction is 80 k and the live load reaction is 110 k, and if 1-in A325-N bolts and A572 steel (F_y = 50 ksi and F_u = 65 ksi) are to be used.

15-11. Select a framed beam connection, using LRFD and ASD, for a W33 × 130 beam (A992 steel) with a dead load reaction = 45 k and a live load reaction = 65 k. It is to be connected to the flange of a W36 × 150 column. The A36 web angles are to be welded with E70 electrodes (weld A in the Manual) and are to be field-connected to the girder with 3/4-in A325-N bolts. (*One ans.* 2Ls 4 × $3\frac{1}{2}$ × $\frac{5}{16}$ × 1 ft $5\frac{1}{2}$ in, $\frac{3}{16}$-in weld A and 6-row bolt connection to girder.)

15-12. Repeat Prob. 15-11 using SMAW shop and field welds (welds A and B in the Manual).

15-13. Select an A36 framed beam connection from the AISC Manual (LRFD and ASD) for a W30 × 124 consisting of 50 ksi steel, using SMAW E70 shop and field welds. The dead load reaction is 60 k, while the live load one is 80 k. The beam is to be connected to the flange of a 50 ksi W14 × 145 column. (*Ans.* 2Ls 4 × 3 × $\frac{3}{8}$ × 1 ft − 8 in weld A = $\frac{3}{16}$ in, weld B = $\frac{5}{16}$ in)

15-14. Repeat Prob. 15-13 if R_D = 90 k and R_L = 100 k.

15-15. Select unstiffened A36 seated beam connections (LRFD and ASD) bolted with 7/8-in A325-N bolts in standard-size holes for the following data: Beam is W16 × 67, column is W14 × 82, both consisting of 50 ksi steel, R_D = 25 k, R_L = 30 k, and the column gage is $5\frac{1}{2}$ in. (*Ans.* 1L 6 × 4 × $\frac{3}{4}$ × 0 ft − 8 in)

15-16. Design SMAW welded moment-resisting connections LRFD and ASD for the ends of a W24 × 76 to resist R_D = 30 k, R_L = 60 k, M_D = 60 ft-k, and M_L = 80 ft-k. Use A36 steel and E70 electrodes. Assume that the column flange is 14 in wide. The moment is to be resisted by full-penetration groove welds in the flanges, and the shear is to be resisted by welded clip angles along the web. Assume that the beam was selected for bending with $0.9F_y$.

15-17. The beam shown in the accompanying illustration is assumed to be attached at its ends with moment-resisting connections. Select the beam, assuming full lateral support, and E70 SMAW electrodes. Use a connection of the type used in Prob. 15-16. $F_y = 50$ ksi. Use ASD and LRFD. (*Ans.* W21 × 55, shear welds $10\frac{1}{2}$ in each side LRFD, 11 in ASD.)

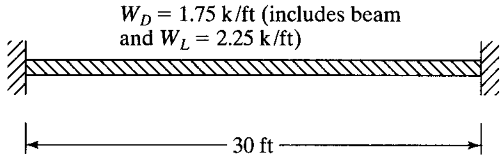


FIGURE P15-17

CHAPTER 16

Composite Beams

16.1 COMPOSITE CONSTRUCTION

When a concrete slab is supported by steel beams, and there is no provision for shear transfer between the two, the result is a noncomposite section. Loads applied to noncomposite sections obviously cause the slabs to deflect along with the beams, resulting in some of the load being carried by the slabs. Unless a great deal of bond exists between the two (as would be the case if the steel beam were completely encased in concrete, or where a system of mechanical steel anchors is provided), the load carried by the slab is small and may be neglected.

For many years, steel beams and reinforced-concrete slabs were used together, with no consideration made for any composite effect. In recent decades, however, it has been shown that a great strengthening effect can be obtained by tying the two together to act as a unit in resisting loads. Steel beams and concrete slabs joined together compositely can often support 33 to 50 percent or more load than could the steel beams alone in noncomposite action.

Composite construction for highway bridges was given the green light by the adoption of the 1944 AASHTO Specifications, which approved the method. Since about 1950, the use of composite bridge floors has rapidly increased, until today they are commonplace all over the United States. In these bridges, the longitudinal shears are transferred from the stringers to the reinforced-concrete slab or deck with steel anchors (described in Section 16.5), causing the slab or deck to assist in carrying the bending moments. This type of section is shown in part (a) of Fig. 16.1.

The first approval for composite building floors was given by the 1952 AISC Specification; today, they are very common. These floors may either be encased in concrete (very rare due to expense) as shown in part (b) of Fig. 16.1, or be nonencased with shear connectors as shown in part (c) of the figure. Almost all composite building floors being built today are of the nonencased type. If the steel sections are encased in concrete, the shear transfer is made by bond and friction between the beam and the

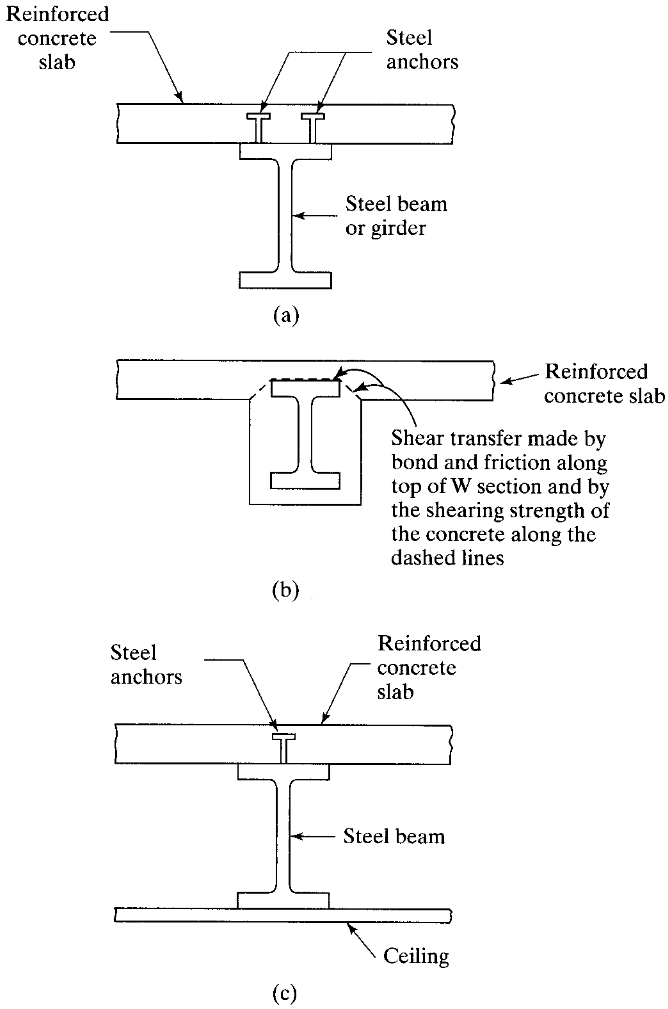


FIGURE 16.1

(a) Composite bridge floor with steel anchors. (b) Encased section for building floors. (c) Building floors with steel anchors.

concrete and by the shearing strength of the concrete along the dashed lines shown in part (b) of Fig. 16.1.

Today, formed steel deck (illustrated in Fig. 16.2) is used for almost all composite building floors. The initial examples in this chapter, however, pertain to the calculations for composite sections where formed steel deck is not used. Sections that make use of formed steel deck are described later in the chapter.

16.2 ADVANTAGES OF COMPOSITE CONSTRUCTION

The floor slab in composite construction acts not only as a slab for resisting the live loads, but also as an integral part of the beam. It actually serves as a large cover plate for the upper flange of the steel beam, appreciably increasing the beam's strength.

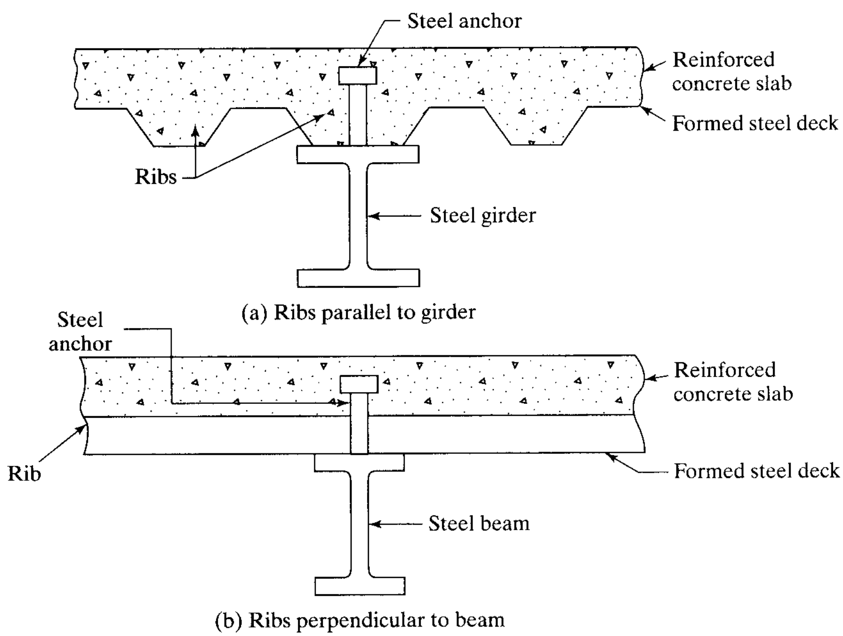


FIGURE 16.2

Composite sections using formed steel deck.

A particular advantage of composite floors is that they make use of concrete's high compressive strength by putting a large part of the slab in compression. At the same time, a larger percentage of the steel is kept in tension (also advantageous) than is normally the case in steel-frame structures. The result is less steel tonnage required for the same loads and spans (or longer spans for the same sections). Composite sections have greater stiffness than noncomposite sections, and they have smaller deflections—perhaps only 20 to 30 percent as large. Furthermore, tests have shown that the ability of a composite structure to take overload is decidedly greater than for a noncomposite structure.

An additional advantage of composite construction is the possibility of having smaller overall floor depths—a fact of particular importance for tall buildings. Smaller floor depths permit reduced building heights, with the consequent advantages of smaller costs for walls, plumbing, wiring, ducts, elevators, and foundations. Another important advantage available with reduced beam depths is a saving in fireproofing costs, because a coat of fireproofing material is provided on smaller and shallower steel shapes.

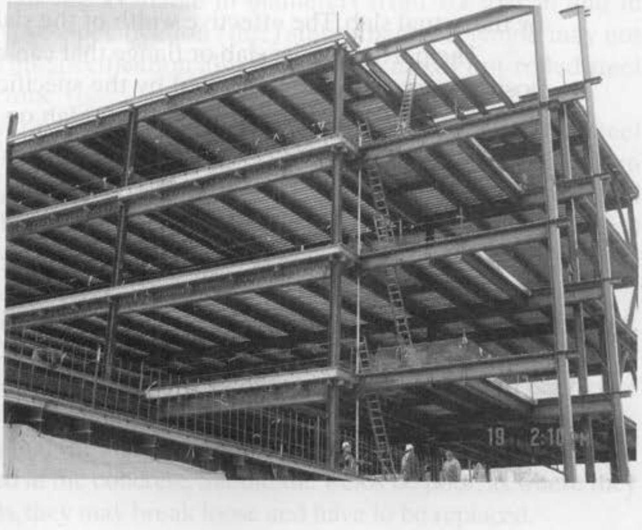
It occasionally is necessary to increase the load-carrying capacity of an existing floor system. Often, this can be handled quite easily for composite floors by welding cover plates onto the bottom flanges of the beams.

A disadvantage for composite construction is the cost of furnishing and installing the steel anchors. This extra cost usually will exceed the cost reductions mentioned when spans are short and lightly loaded.

16.3 DISCUSSION OF SHORING

After the steel beams are erected, the concrete slab is placed on them. The formwork, wet concrete, and other construction loads must therefore be supported by the beams or by temporary shoring. Should no shoring be used, the steel beams must support all of these loads as well as their own weights. Most specifications say that after the concrete has gained 75 percent of its 28-day strength, the section has become composite and all loads applied thereafter may be considered to be supported by the composite section. When shoring is used, it supports the wet concrete and the other construction loads. It does not really support the weight of the steel beams unless they are given an initial upward deflection (which probably is impractical). When the shoring is removed (after the concrete gains at least 75 percent of its 28-day strength), the weight of the slab is transferred to the composite section, not just to the steel beams. The student can see that if shoring is used, it will be possible to use lighter, and thus cheaper, steel beams. The question then arises, "Will the savings in steel cost be greater than the extra cost of shoring?" The answer probably is no. The usual decision is to use heavier steel beams and to do without shoring for several reasons, including the following:

1. Apart from reasons of economy, the use of shoring is a tricky operation, particularly where settlement of the shoring is possible, as is often the case in bridge construction.
2. Both theory and load tests indicate that the ultimate strengths of composite sections of the same sizes are the same, whether shoring is used or not. If lighter steel beams are selected for a particular span because shoring is used, the result is a smaller ultimate strength.
3. Another disadvantage of shoring is that after the concrete hardens and the shoring is removed, the slab will participate in composite action in supporting the dead loads. The slab will be placed in compression by these long-term loads and will have substantial creep and shrinkage parallel to the beams. The result will be a great decrease in the stress in the slab, with a corresponding increase in the steel stresses.



Floor framing for Glen Oaks School, Bellerose, NY. (Courtesy of CMC South Carolina Steel.)

The probable consequence is that most of the dead load will be supported by the steel beams anyway, and composite action will really apply only to the live loads, as though shoring had not been used.

4. Also, in shored construction cracks occur over the steel girders, necessitating the use of reinforcing bars. In fact, we should use reinforcing over the girders in unshored construction, too. Although cracks will be smaller there, they are going to be present nonetheless, and we need to keep them as small as possible.

Nevertheless, shored construction does present some advantages compared with unshored construction. First, deflections are smaller because they are all based on the properties of the composite section. (In other words, the initial wet concrete loads are not applied to the steel beams alone, but rather to the whole composite section.) Second, it is not necessary to make a strength check for the steel beams for this wet load condition. This is sometimes quite important for situations in which we have low ratios of live to dead loads.

The deflections of unshored floors due to the wet concrete sometimes can be quite large. If the beams are not cambered, additional concrete (perhaps as much as 10 percent or more) will be used to even up the floors. If, on the other hand, too much camber is specified, we may end up with slabs that are too thin in those areas where wet concrete deflections aren't as large as the camber.

16.4 EFFECTIVE FLANGE WIDTHS

There is a problem involved in estimating how much of the slab acts as part of the beam. Should the beams be rather closely spaced, the bending stresses in the slab will be fairly uniformly distributed across the compression zone. If, however, the distances between beams are large, bending stresses will vary quite a bit nonlinearly across the flange. The further a particular part of the slab or flange is away from the steel beam, the smaller will be its bending stress. Specifications attempt to handle this problem by replacing the actual slab with a narrower or effective slab that has a constant stress. This equivalent slab is deemed to support the same total compression as is supported by the actual slab. The effective width of the slab b_e is shown in Fig. 16.3.

The portion of the slab or flange that can be considered to participate in the composite beam action is controlled by the specifications. AISC Specification I3.1a states that the effective width of the concrete slab on each side of the beam center line shall

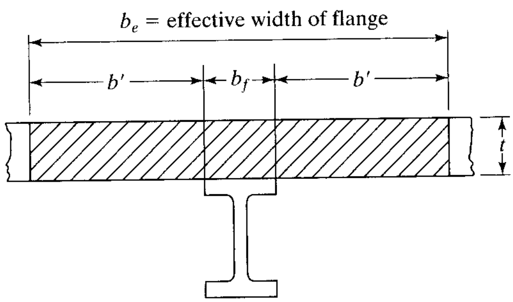


FIGURE 16.3

not exceed the least of the values to follow. The following set of rules applies, whether the slab exists on one or both sides of the beam:

1. One-eighth of the span of the beam measured center-to-center of supports for both simple and continuous spans.
2. One-half of the distance from the beam center line to the center line of the adjacent beam.
3. The distance from the beam center line to the edge of the slab.

The AASHTO requirements for determining effective flange widths are somewhat different. The maximum total flange width may not exceed one-fourth of the beam span, twelve times the least thickness of the slab, or the distance center-to-center of the beams. Should the slab exist on only one side of the beam, its effective width may not exceed one-twelfth of the beam span, six times the slab thickness, or one-half of the distance from the center line of the beam to the center line of the adjacent beam.

16.5 SHEAR TRANSFER

The concrete slabs may rest directly on top of the steel beams, or the beams may be completely encased in concrete for fireproofing purposes. This latter case, however, is very expensive and thus is rarely used. The longitudinal shear can be transferred between the two by bond and shear (and possibly some type of shear reinforcing), if needed, when the beams are encased. When not encased, mechanical connectors must transfer the load. Fireproofing is not necessary for bridges, and the slab is placed on top of the steel beams. Bridges are subject to heavy impactive loads, and the bond between the beams and the deck, which is easily broken, is considered negligible. For this reason, steel anchors are designed to resist all of the shear between bridge slabs and beams.

Various types of steel anchors have been tried, including spiral bars, channels, zees, angles, and studs. Several of these types of connectors are shown in Fig. 16.4. Economic considerations have usually led to the use of round studs welded to the top flanges of the beams. These studs are available in diameters from 1/2 to 1 in and in lengths from 2 to 8 in, but the AISC Specification (I8.2) states that their length may not be less than 4 stud diameters. This specification also permits the use of hot-rolled steel channels, but not spiral connectors.

The studs actually consist of rounded steel bars welded on one end to the steel beams. The other end is upset or headed to prevent vertical separation of the slab from the beam. These studs can be quickly attached to the steel beams through the steel decks with stud-welding guns by semiskilled workers. The AISC Commentary (I3.2d) describes special procedures needed for 16-gage and thicker decks and for decks with heavy galvanized coatings (>1.25 ounces per sq ft).

A rather interesting practical method is used by many engineers in the field to check the adequacy of the welds used to connect the studs to the steel beams. They take a 5 or 6 lb hammer and hit occasional studs a sufficient number of times to cause them to bend over roughly 25 or 30 degrees. If the studs don't break loose during this hammering, the welds are considered to be satisfactory and the studs are left in their bent positions, which is OK because they will later be encased in the concrete. Should the welds be poor, as where they were made during wet conditions, they may break loose and have to be replaced.

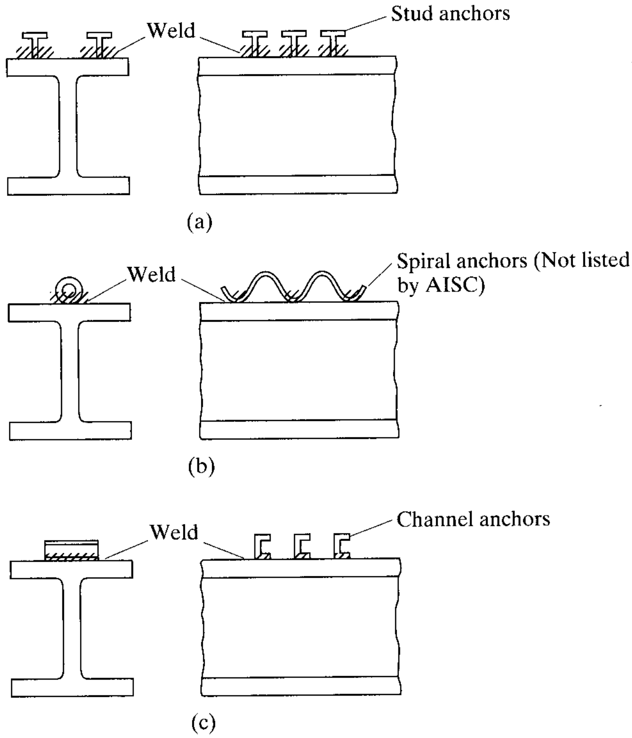


FIGURE 16.4
Steel anchors.

Shop installation of steel anchors initially is more economical, but there is a growing tendency to use field installation. There are two major reasons for this trend: The anchors may easily be damaged during transportation and setting of the beams, and they serve as a hindrance to the workers walking along the top flanges during the early phases of construction.

When a composite beam is being tested, failure will probably occur with a crushing of the concrete. It seems reasonable to assume that at that time the concrete and steel will both have reached a plastic condition.

For the discussion to follow, reference is made to Fig. 16.5. Should the plastic neutral axis (PNA) fall in the slab, the maximum horizontal shear (or horizontal force on the plane between the concrete and the steel) is said to be A_sF_y; and if the plastic neutral axis is in the steel section, the maximum horizontal shear is considered to equal $0.85\,f'_cA_c$, where A_c is the effective area of the concrete slab. (For the student unfamiliar with the strength design theory for reinforced concrete, the average stress at failure on the compression side of a reinforced concrete beam is usually assumed to be $0.85\,f'_c$.)

From this information, expressions for the shear to be taken by the anchors can be determined: The AISC (I3.2d) says that, for composite action, the total horizontal shear between the points of maximum positive moment and zero moment is to be taken as the least of the following, where ΣQ_n is the total nominal strength of the steel anchors provided,

a. For concrete crushing

$$V' = 0.85\,f'_cA_c \qquad \text{(AISC Equation I3-1a)}$$

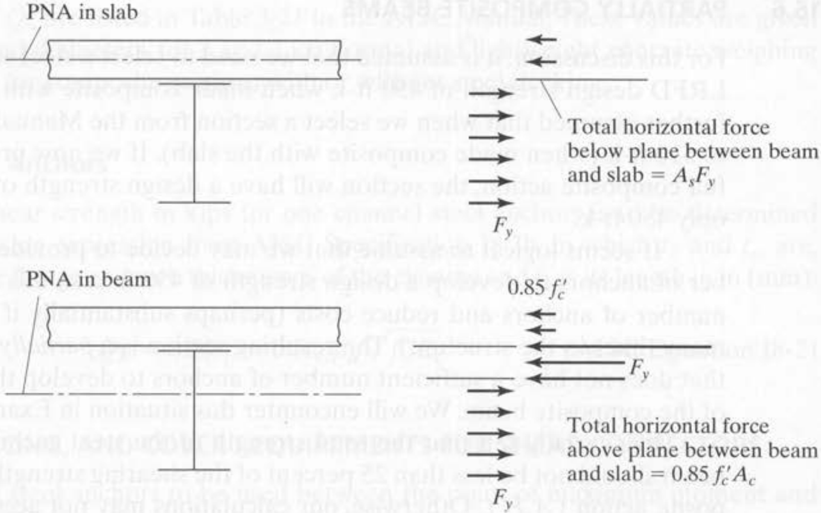


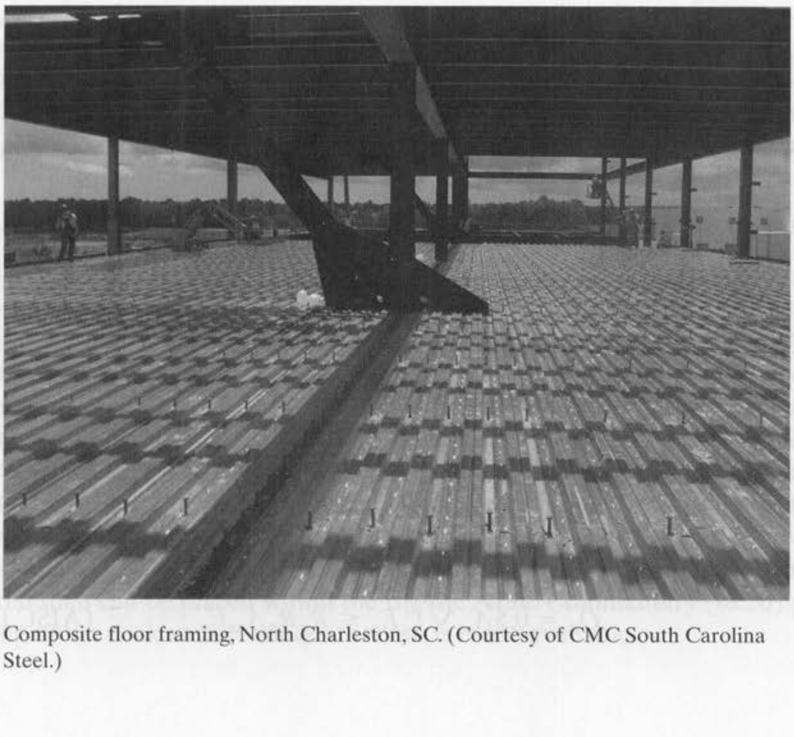
FIGURE 16.5

b. For tensile yielding of the steel section (for hybrid beams, this yield force must be calculated separately for each of the components of the cross section)

$$V' = F_y A_s \qquad \text{(AISC Equation I3-1b)}$$

c. For strength of steel anchors

$$V' = \Sigma Q_n \qquad \text{(AISC Equation I3-1c)}$$



Composite floor framing, North Charleston, SC. (Courtesy of CMC South Carolina Steel.)

16.6 PARTIALLY COMPOSITE BEAMS

For this discussion, it is assumed that we need to select a steel section that will have an LRFD design strength of 450 ft-k when made composite with the concrete slab. It is further assumed that when we select a section from the Manual, it has an $\phi_b M_n$ equal to 510 ft-k (when made composite with the slab). If we now provide steel anchors for full composite action, the section will have a design strength of 510 ft-k. But we need only 450 ft-k.

It seems logical to assume that we may decide to provide only a sufficient number of anchors to develop a design strength of 450 ft-k. In this way we can reduce the number of anchors and reduce costs (perhaps substantially if we repeat this section many times in the structure). The resulting section is a *partially composite section*, one that does not have a sufficient number of anchors to develop the full flexural strength of the composite beam. We will encounter this situation in Examples 16-3 and 16-4.

It is usually felt that the total strength of the steel anchors used in a particular beam should not be less than 25 percent of the shearing strength required for full composite action (A_sF_y). Otherwise, our calculations may not accurately depict the stiffness and strength of a composite section.

16.7 STRENGTH OF STEEL ANCHORS

For composite sections, it is permissible to use normal weight stone concrete (made with aggregates conforming to ASTM C33) or lightweight concrete weighing not less than 90 lb/ft^3 (made with rotary kiln-produced aggregates conforming to ASTM C330).

The AISC Specification provides strength values for headed steel studs not less than 4 diameters in length after installation and for hot-rolled steel channels. *They do not, however, give resistance factors for the strength of steel anchors.* This is because they feel that the factor used for determining the flexural strength of the concrete is sufficient to account for variations in concrete strength, including those variations that are associated with steel anchors.

16.7.1 Steel Headed Stud Anchors

The nominal shear strength in kips of one stud steel anchors embedded in a solid concrete slab is to be determined with an expression from AISC Specification I8.2a. In this expression, A_{sa} is the cross-sectional area of the shank of the anchors in square inches and f'_c is the specified compressive stress of the concrete in ksi. E_c is the modulus of elasticity of the concrete in ksi (MPa) and equals $w^{1.5}\sqrt{f'_c}$ in which w is the unit weight of the concrete in lb/ft^3, while F_u is the specified minimum tensile strength of the steel stud in ksi (MPa). R_g is a coefficient used to account for the group effect of the anchors, while R_p is the position effect of the anchors. Values of these latter two factors are given in AISC Specification I8.2a. Here is the expression for the normal shear strength:

$$Q_n = 0.5A_{sa}\sqrt{f'_cE_c} \leq R_gR_pA_{sa}F_u \qquad \text{(AISC Equation I8-1)}$$

Values of Q_n are listed in Table 3-21 in the AISC Manual. These values are given for different stud diameters, for 3 and 4 ksi normal and lightweight concrete weighing 110 lbs/ft^3, and for composite sections with or without steel decking.

16.7.2 Steel Channel Anchors

The nominal shear strength in kips for one channel steel anchors is to be determined from the following expression from AISC Specification I8.2b, in which t_f and t_w are, respectively, the flange and web thicknesses of the channel and l_a is its length in in (mm):

$$Q_n = 0.3(t_f + 0.5t_w)l_a\sqrt{f'_c E_c} \qquad \text{(AISC Equation I8-2)}$$

16.8 NUMBER, SPACING, AND COVER REQUIREMENTS FOR SHEAR CONNECTORS

The number of steel anchors to be used between the point of maximum moment and each adjacent point of zero moment equals the horizontal force to be resisted divided by the nominal strength of one anchor Q_n.

16.8.1 Spacing of Anchors

Tests of composite beams with steel anchors spaced uniformly, and of composite beams with the same number of anchors spaced in variation with the statical shear, show little difference as to ultimate strengths and deflections at working loads. This situation prevails as long as the total number of anchors is sufficient to develop the shear on both sides of the point of maximum moment. As a result, the AISC Specification (I8.2c and I8.2d) permits uniform spacings of anchors on each side of the maximum moment points. However, the number of anchors placed between a concentrated load and the nearest point of zero moment must be sufficient to develop the maximum moment at the concentrated load.

16.8.2 Maximum and Minimum Spacings

Except for formed steel decks, the minimum center-to-center spacing of steel anchors along the longitudinal axes of composite beams permitted by AISC Specification (I8.2d) is 6 diameters, while the minimum value transverse to the longitudinal axis is 4 diameters. Within the ribs of formed steel decks, the minimum permissible spacing is 4 diameters in any direction. The maximum spacing may not exceed 8 times the total slab thickness, or 36 in.

When the flanges of steel beams are rather narrow, it may be difficult to achieve the minimum transverse spacings described here. For such situations, the studs may be staggered. Figure 16.6 shows possible arrangements.

If the deck ribs are parallel to the axis of the steel beam and more anchors are required than can be placed within the rib, the AISC Commentary (I8.2d) permits the splitting of the deck so that adequate room is made available.

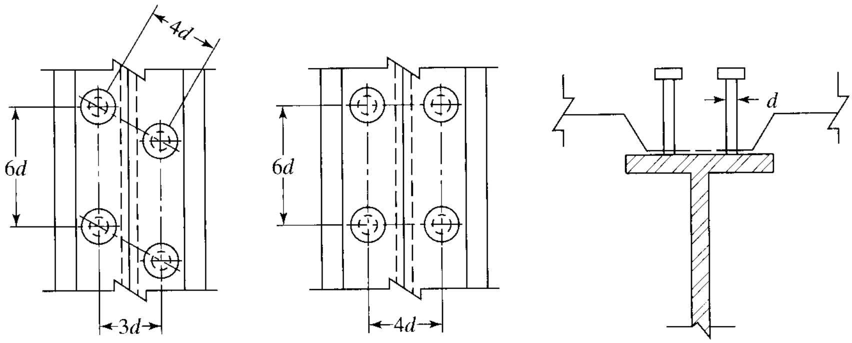


FIGURE 16.6
Anchor arrangements.

Steel anchors must be capable of resisting both horizontal and vertical movement, because there is a tendency for the slab and beam to separate vertically, as well as to slip horizontally. The upset heads of stud steel anchors help to prevent vertical separation.

16.8.3 Cover Requirements

The AISC Specification (I8.2d) requires that there be at least 1 in of lateral concrete cover provided for steel anchors. This rule does not apply to anchors used within the ribs of formed steel decks, because tests have shown that strengths are not reduced, even when studs are placed as close as possible to the ribs.

When studs are not placed directly over beam webs, they have a tendency to tear out of the beam flanges before their full shear capacity is reached. To keep this situation from occurring, the AISC Specification (I8.1) requires that the diameter of the studs may not be greater than 2.5 times the flange thickness of the beam to which they are welded, unless they are located over the web.

When formed steel deck is used, the steel beam must be connected to the concrete slab with steel headed stud anchors with diameters not larger than 3/4 in. These may be welded through the deck or directly to the steel beam. After their installation, they must extend for at least 1 1/2 in above the top of the steel deck, and the concrete slab thickness above the steel deck may not be less than 2 in (AISC Specification I3.2c(1)).

16.8.4 Strong and Weak Positions for Steel Headed Stud Anchors

If we examine Table 3-21 in the AISC Manual, which provides the calculated strengths of steel headed studs for different situations, we will see that if formed steel decks are used with their ribs placed perpendicular to the longitudinal direction of the steel beam, the studs are referred to as being either *strong* or *weak*. Most composite steel floor decks today have a stiffening rib in the middle of the deck flutes. This means that the shear studs will have to be placed on one side or the other of the ribs. Figure C-I8.1 of the AISC Specification Commentary shows strong and weak positions for placing the studs. The strong position is on the side away from the direction from which the shear is applied. Making sure that the studs are placed in the strong positions in the field is not an easy task, and some designers assume conservatively that the studs will always be placed in the weak positions, with their smaller shear strengths, as shown in AISC Table 3-21.

16.9 MOMENT CAPACITY OF COMPOSITE SECTIONS

The nominal flexural strength of a composite beam in the positive moment region may be controlled by the plastic strength of the section, by the strength of the concrete slab, or by the strength of the steel anchors. Furthermore, if the web is very slender and if a large portion of the web is in compression, web buckling may limit the nominal strength of the member.

Little research has been done on the subject of web buckling for composite sections, and for this reason the AISC Specification (I3.2) has conservatively applied the same rules to composite section webs as to plain steel webs. The positive nominal flexural strength, M_n, of a composite section is to be determined, assuming a plastic stress distribution if $h/t_w \leq 3.76\sqrt{E/F_{yf}}$. In this expression, h is the distance between the web toes of the fillet (that is, $d - 2k$), t_w is the web thickness, and F_{yf} is the yield stress of the beam flange. All of the rolled W, S, M, HP, and C shapes in the Manual meet this requirement for F_y values up to 65 ksi. (For built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used.)

If h/t_w is greater than $3.76\sqrt{E/F_{yf}}$, the value of M_n with $\phi_b = 0.90$ and $\Omega = 1.67$ is to be determined by superimposing the elastic stresses. The effects of shoring must be considered for these calculations.

The nominal moment capacity of composite sections as determined by load tests can be estimated very accurately with the plastic theory. With this theory, the steel section at failure is assumed to be fully yielded, and the part of the concrete slab on the compression side of the neutral axis is assumed to be stressed to $0.85\,f'_c$. If any part of the slab is on the tensile side of the neutral axis, it is assumed to be cracked and incapable of carrying stress.

The plastic neutral axis (PNA) may fall in the slab or in the flange of the steel section or in its web. Each of these cases is discussed in this section.

16.9.1 Neutral Axis in Concrete Slab

The concrete slab compression stresses vary somewhat from the PNA out to the top of the slab. For convenience in calculations, however, they are assumed to be uniform, with a value of $0.85\,f'_c$ over an area of depth a and width b_e, determined as described in Section 16.4. (This distribution is selected to provide a stress block having the same total compression C and the same center of gravity for the total force as we have in the actual slab.)

The value of a can be determined from the following expression, where the total tension in the steel section is set equal to the total compression in the slab:

$$A_sF_y = 0.85f'_cab_e$$

$$a = \frac{A_sF_y}{0.85f'_cb_e}$$

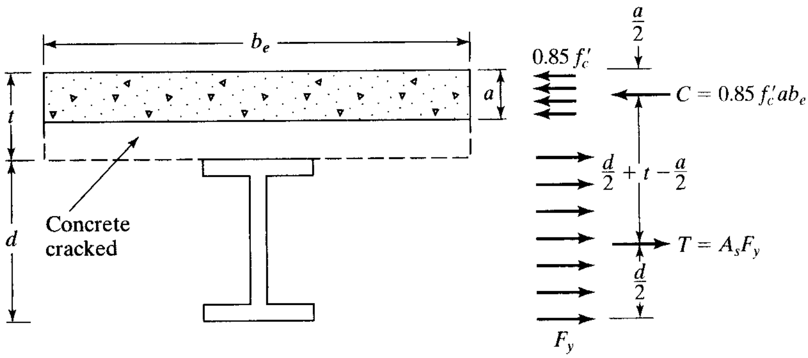


FIGURE 16.7
Plastic neutral axis (PNA) in the slab.

If a is equal to or less than the slab thickness, the PNA will fall in the slab and the nominal or plastic moment capacity of the composite section may be written as the total tension T or the total compression C times the distance between their centers of gravity. Reference is made here to Fig. 16.7.

Example 16-1 illustrates the calculation of $\phi_b M_p = \phi_b M_n$ and $\frac{M_n}{\Omega}$ for a composite section where the PNA falls within the slab.

Example 16-1

Compute $\phi_b M_n$ and $\frac{M_n}{\Omega_b}$ for the composite section shown in Fig. 16.8 if $f'_c = 4$ ksi and $F_y = 50$ ksi.

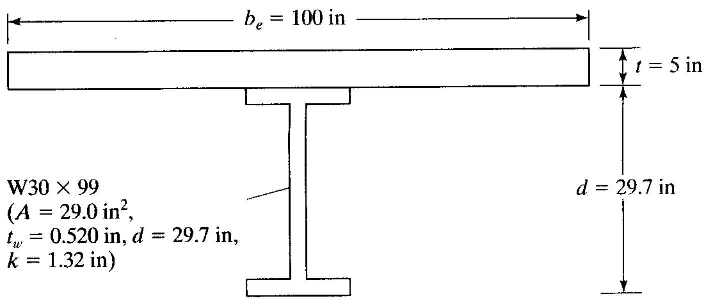


FIGURE 16.8

Solution. Determining M_n

$$h = d - 2k = 29.7 \text{ in} - (2)(1.32 \text{ in}) = 27.06 \text{ in}$$

$$\frac{h}{t_w} = \frac{27.06 \text{ in}}{0.520 \text{ in}} = 52.04 < 3.76\sqrt{\frac{E}{F_{yf}}} = 3.76\sqrt{\frac{29 \times 10^3}{50}} = 90.55$$

$\therefore$ OK to determine M_n from the plastic stress distribution on the composite section for the limit state of yielding (plastic moment).

Locate PNA

$$a = \frac{A_s F_y}{0.85 f'_c b_e} = \frac{(29.0 \text{ in}^2)(50 \text{ ksi})}{(0.85)(4 \text{ ksi})(100 \text{ in})} = 4.26 \text{ in} < 5 \text{ in} \quad \therefore \text{ PNA is in slab}$$

$$\therefore M_n = M_p = A_s F_y\left(\frac{d}{2} + t - \frac{a}{2}\right)$$

$$= (29.0 \text{ in}^2)(50 \text{ ksi})\left(\frac{29.7 \text{ in}}{2} + 5 \text{ in} - \frac{4.26 \text{ in}}{2}\right)$$

$$= 25{,}694 \text{ in-k} = 2141.2 \text{ ft-k}$$

LRFD $\phi_b = 0.90$	ASD $\Omega_b = 1.67$
$\phi_b M_n = (0.90)(2141.2) = 1927.1$ ft-k	$\frac{M_n}{\Omega_b} = \frac{2141.2}{1.67} = 1282.2$ ft-k

Note: If the reader refers to Part 3 of the AISC Manual, he or she can determine the ϕM_n and $\frac{M_n}{\Omega}$ values for this composite beam; see Fig. 16.9. To use the Manual composite tables, we assume that the PNA is located at the top of the steel flange (TFL) or down in the steel shape. In the AISC tables, Y1 represents the distance from the PNA to the top of the beam flange while Y2 represents the distance from the centroid of the effective concrete flange force to the top flange of the beam ($Y_{con} - a/2$).

The reader should realize that the PNA locations will be different for LRFD and ASD if the live load over the dead load is not equal to 3. The AISC has selected the PNA locations that require the most shear transfer. The result is a slight variation in the calculated moment values and Table 3-19 values.

With the PNA for the preceding example being located at the top of the beam flange, from page 3-170 of the Manual, with $Y2 = 5 - 4.28/2 = 2.86$ in and $Y1 = 0$, and for a W30 × 99, the value of $\phi M_n = \phi_b M_p$ by interpolation is 1926 ft-k.

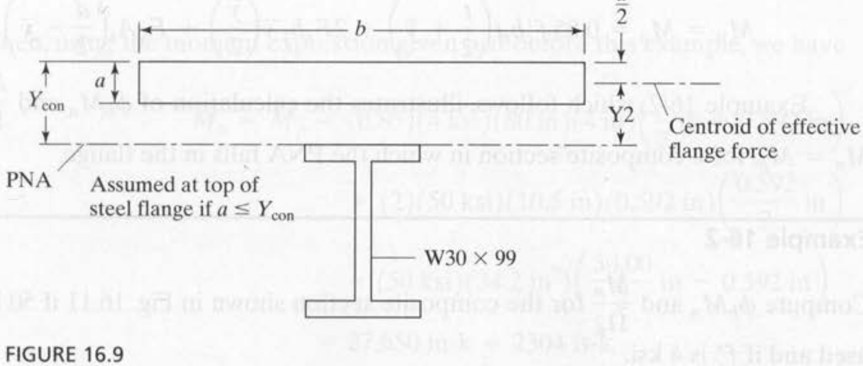


FIGURE 16.9

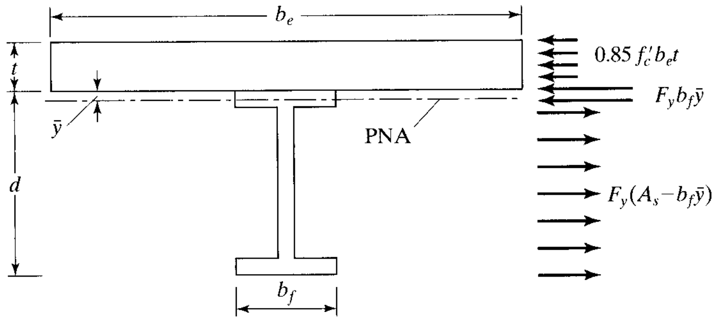


FIGURE 16.10
Composite section with PNA in steel flange.

16.9.2 Neutral Axis in Top Flange of Steel Beam

If a is calculated as previously described and is greater than the slab thickness t, the PNA will fall down in the steel section. If this happens, it will be necessary to find out whether the PNA is in the flange or below the flange. Suppose we assume that it's at the base of the flange. We can calculate the total compressive force C above the PNA $= 0.85 f'_c b_e t + A_f F_y$, where A_f is the area of the flange, and the total tensile force below $T = F_y(A_s - A_f)$. If $C > T$, the PNA will be in the flange. If $C < T$, the PNA is below the flange.

Assuming that we find that the PNA is in the flange, we can determine its location, letting $\bar{y}$ be the distance to the PNA measured from the top of the top flange, by equating C and T as follows:

$$0.85 f'_c b_e t + F_y b_f \bar{y} = F_y A_s - F_y b_f \bar{y}$$

From this, $\bar{y}$ is

$$\bar{y} = \frac{F_y A_s - 0.85 f'_c b_e t}{2 F_y b_f}$$

Then the nominal or plastic moment capacity of the section can be determined from the expression to follow, with reference being made to Fig. 16.10. Taking moments about the PNA, we get

$$M_p = M_n = 0.85 f'_c b_e t\left(\frac{t}{2} + \bar{y}\right) + 2F_y b_f \bar{y}\left(\frac{\bar{y}}{2}\right) + F_y A_s\left(\frac{d}{2} - \bar{y}\right)$$

Example 16-2, which follows, illustrates the calculation of $\phi_b M_n$ and $\frac{M_n}{\Omega_b}$, where $M_n = M_p$, for a composite section in which the PNA falls in the flange.

Example 16-2

Compute $\phi_b M_n$ and $\frac{M_n}{\Omega_b}$ for the composite section shown in Fig. 16.11 if 50 ksi steel is used and if f'_c is 4 ksi.

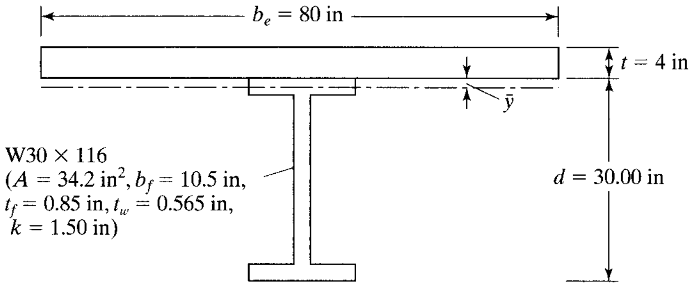


FIGURE 16.11

Solution. Determining M_n

$$h = d - 2k = 30.00 - (2)(1.50) = 27.00 \text{ in}$$

$$\frac{h}{t_w} = \frac{27.00 \text{ in}}{0.565 \text{ in}} = 47.78 < 3.76\sqrt{\frac{29 \times 10^3}{50}} = 90.55$$

$$\therefore M_n = M_p$$

Is PNA located at top of steel flange?

$$a = \frac{A_s f_y}{0.85 f'_c b_e} = \frac{(34.2 \text{ in}^2)(50 \text{ ksi})}{(0.85)(4 \text{ ksi})(80 \text{ in})} = 6.29 \text{ in} > 4.00 \text{ in}$$

$\therefore$ PNA is located down in steel section.

Is PNA in flange or in web? Here, we assume it is at base of steel flange.

$$C = 0.85 f'_c b_e t + F_y b_f t_f = (0.85)(4 \text{ ksi})(80 \text{ in})(4 \text{ in}) + (50 \text{ ksi})(10.5 \text{ in})(0.850 \text{ in}) = 1534 \text{ k}$$

$$T = F_y(A_s - b_f t_f) = (50 \text{ ksi})(34.2 \text{ in}^2 - 10.5 \text{ in} \times 0.850 \text{ in}) = 1264 \text{ k}$$

Since $C > T$, the PNA falls in the steel flange and can be located as follows:

$$\bar{y} = \frac{F_y A_s - 0.85 f'_c b_e t}{2 F_y b_f} = \frac{(50 \text{ ksi})(34.2 \text{ in}^2) - (0.85)(4 \text{ ksi})(80 \text{ in})(4 \text{ in})}{(2)(50 \text{ ksi})(10.5 \text{ in})} = 0.592 \text{ in}$$

Then, using the moment expression given just before this example, we have

$$\begin{aligned} M_n = M_p &= (0.85)(4 \text{ ksi})(80 \text{ in})(4 \text{ in})\left(\frac{4}{2} \text{ in} + 0.592 \text{ in}\right) \\ &+ (2)(50 \text{ ksi})(10.5 \text{ in})(0.592 \text{ in})\left(\frac{0.592}{2} \text{ in}\right) \\ &+ (50 \text{ ksi})(34.2 \text{ in}^2)\left(\frac{30.00}{2} \text{ in} - 0.592 \text{ in}\right) \\ &= 27{,}650 \text{ in-k} = 2304 \text{ ft-k} \end{aligned}$$

LRFD $\phi_b = 0.90$	ASD $\Omega_b = 1.67$
$\phi_b M_n = (0.90)(2304) = 2074$ ft-k	$\dfrac{M_n}{\Omega_b} = \dfrac{2304}{1.67} = 1380$ ft-k

By interpolation in AISC Table 3-19, page 3-170, with Y1 = 0.592 in and Y2 = 2.0 in, we get $2110 - \left(\dfrac{0.592 - 0.425}{0.638 - 0.425}\right)(2110 - 2060) = 2070$ ft-k for LRFD, and in a similar manner for ASD, we get 1376.5 ft-k.

If we have a partially composite section with ΣQ_n less than $A_s F_y$, the PNA will be down in the shape; if in the flange, the value of $\phi_b M_n$ can be determined with the equation used in Example 16-2. In the composite design tables presented in the Manual, values of ΣQ_n and $\phi_b M_n$ are shown for seven different PNA locations—top of the flange, quarter points in the flange, bottom of the flange, and two points down in the web. Straight-line interpolation may be used for numbers in between the tabulated values.

16.9.3 Neutral Axis in Web of Steel Section

If for a particular composite section we find that a is larger than the slab thickness, and if we then assume that the PNA is located at the bottom of the steel flange and we calculate C and T and find T is larger than C, the PNA will fall in the web. We can go through calculations similar to the ones we used for the case in which the PNA was located in the flange. Space is not taken to show such calculations, because the Composite Design tables in Part 3 of the Manual cover most common cases.

16.10 DEFLECTIONS

Deflections for composite beams may be calculated by the same methods used for other types of beams. The student must be careful to compute deflections for the various types of loads separately. For example, there are dead loads applied to the steel section alone (if no shoring is used), dead loads applied to the composite section, and live loads applied to the composite section.

The long-term creep effect in the concrete in compression causes deflections to increase with time. These increases, however, are usually not considered significant for the average composite beam. This is usually true, unless long spans and large permanent live loads are involved (AISC Commentary I3.2.4).

Should lightweight concrete be used, the actual modulus of elasticity of that concrete E_c (which may be rather small) should be used in calculating the transformed section moment of inertia I_{tr} for deflection computations. For stress calculations, we use E_c for normal-weight concrete.

Generally speaking, shear deflections are neglected, although on occasion they can be quite large.[1] The steel beams can be cambered for all or some portion of deflec-

[1] L. S. Beedle et al., *Structural Steel Design* (New York: Ronald Press, 1964), p. 452.

tions. It may be feasible in some situations to make a floor slab a little thicker in the middle than on the edges to compensate for deflections.

The designer may want to control vibrations in composite floors subject to pedestrian traffic or other moving loads. This may be the case where we have large open floor areas with no damping furnished by partitions, as in shopping malls. For such cases, dynamic analyses should be made.[2]

When the AISC Specification is used to select steel beams for composite sections, the results often will be some rather small steel beams and thus some quite shallow floors. Such floors, when unshored, frequently will have large deflections when the concrete is placed. For this reason, designers will often require cambering of the beams. Other alternatives include the selection of larger beams or the use of shoring.[3,4]

The beams selected must, of course, have sufficient $\phi_b M_n$ or $\frac{M_n}{\Omega}$ values to support themselves and the wet concrete. Nevertheless, their sizes are often dictated more by wet concrete deflections than by moment considerations. It is considered to be good practice to limit these deflections to maximum values, of about 2 1/2 in. Larger deflections than this value tend to cause problems with the proper placement of the concrete.

An alternative solution for these problems involves the use of partly restrained or semirigid PR connections (discussed in Chapter 15). When these connections are used, the midspan deflections and moments are appreciably reduced, enabling us to use smaller girders. Furthermore, there are reductions in the annoying vibrations that are a problem in shallow composite floors.

When semirigid PR connections are used, negative moments will develop at the supports. In Section 16.12 of this text, it is shown that the AISC Specification permits the use of negative design moment strength for composite floors, provided that certain requirements are met as to shear connectors and development of slab reinforcing in the negative moment region.

For unshored composite construction, the final deflections will equal the initial deflections caused by the wet concrete calculated with the moments of inertia of the steel beams, plus the deflections due to the loads applied after the concrete hardens, calculated with the moments of inertia of the composite sections. Should shored construction be used, all deflections will be calculated with moments of inertia of the composite sections. These latter moments of inertia, which are referred to as lower bound moments of inertia, are discussed later in the next section of this chapter.

16.11 DESIGN OF COMPOSITE SECTIONS

Composite construction is of particular advantage economically when loads are heavy, spans are long, and beams are spaced at fairly large intervals. For steel building frames, composite construction is economical for spans varying roughly from 25 to 50 ft, with particular advantage in the longer spans. For bridges, simple spans have been economically

[2]Thomas M. Murray, "Design to Prevent Floor Vibrations," *Engineering Journal*, AISC, vol. 12, no. 3 (3rd Quarter, 1975), pp. 82–87.

[3]R. Leon, "Composite Semi-Rigid Connections," *Modern Steel Construction*, vol. 32, no. 9 (AISC, Chicago: October 1992), pp. 18–23.

[4]"Innovative Design Cuts Costs," *Modern Steel Construction*, vol. 33, no. 4 (AISC, Chicago: April 1993), pp. 18–21.

constructed up to approximately 120 ft—and continuous spans 50 or 60 ft longer. Composite bridges are generally economical for simple spans greater than about 40 ft and for continuous spans greater than about 60 ft.

Occasionally, cover plates are welded to the bottom flanges of steel beams, with improved economy. One can see that with the slab acting as part of the beam, there is a very large compressive area available and that by adding cover plates to the tensile flange, a slightly better balance is obtained.

In tall buildings where headroom is a problem, it is desirable to use the minimum possible overall floor thicknesses. For buildings, minimum depth-span ratios of approximately 1/24 are recommended if the loads are fairly static and 1/20 if the loads are of such a nature as to cause appreciable vibration. The thicknesses of the floor slabs are known (from the concrete design), and the depths of the steel beams can be fairly well estimated from these ratios.

Before we attempt some composite designs, several additional points relating to lateral bracing, shoring, estimated steel beam weights, and lower bound moments of inertia are discussed in the paragraphs to follow.

16.11.1 Lateral Bracing

After the concrete slab hardens, it will provide sufficient lateral bracing for the compression flange of the steel beam. However, during the construction phase before the concrete hardens, lateral bracing may be insufficient and its design strength may have to be reduced, depending on the estimated unbraced length. When steel-formed decking or concrete forms are attached to the beam's compression flange, they usually will provide sufficient lateral bracing. The designer must very carefully consider lateral bracing for fully encased beams.

16.11.2 Beams with Shoring

If beams are shored during construction, we will assume that all loads are resisted by the composite section after the shoring is removed.

16.11.3 Beams without Shoring

If temporary shoring is not used during construction, the steel beam alone must be able to support all the loads before the concrete is sufficiently hardened to provide composite action.

Without shoring, the wet concrete loads tend to cause large beam deflections, which may lead us to build thicker slabs where the beam deflections are larger. This situation can be counteracted by cambering the beams.

The AISC Specification does not provide any extra margin against yield stresses occurring in beams during construction of unshored composite floors. Assuming that satisfactory lateral bracing is provided, the Specification (F2) states that the maximum factored moment may not exceed 0.90 F_yZ. The 0.90 in effect limits the maximum factored moment to a value about equal to the yield moment F_yS.

To calculate the moment to be resisted during construction, it makes sense to count the wet concrete as a live load and also to include some extra live load (perhaps 20 psf) to account for construction activities.

16.11.4 Estimated Steel Beam Weight

As illustrated in Example 16-3, it sometimes may be useful to make an estimate of the weight of the steel beam. The third edition of the LRFD Manual provided the following empirical formula for this purpose in its Part 5 (page 5-26):

$$\text{Estimated beam weight} = \left[\frac{12M_u}{(d/2 + Y_{\text{con}} - a/2)\phi F_y}\right]3.4$$

Here,

M_u = required flexural strength of composite section, ft-k
d = nominal steel beam depth, in
Y_{con} = distance from top of steel beam to top of concrete slab, in
a = effective concrete slab thickness, in (which can be conservatively estimated as somewhere in the range of about 2 in)
ϕ = 0.85

16.11.5 Lower Bound Moment of Inertia

To calculate the service load deflections for composite sections, a table of lower bound moment of inertia values is presented in Part 3 of the Manual (Table 3-20). These values are computed from the area of the steel beam and an equivalent concrete area of $\Sigma Q_n/F_y$. The remainder of the concrete flange is not used in these calculations. This means that if we have partially composite sections, the value of the lower bound moment of inertia will reflect this situation because ΣQ_n will be smaller. The lower bound moment of inertia is computed with the expression that follows. See Fig. 16.12 which is Figure 3-5 from Part 3 of the AISC Manual. Terms are defined in AISC commentary I3.

$$I_{LB} = I_s + A_s(Y_{ENA} - d_3)^2 + \left(\frac{\Sigma Q_n}{F_y}\right)(2d_3 + d_1 - Y_{ENA})^2$$

(AISC Commentary Equation C-I3-1)

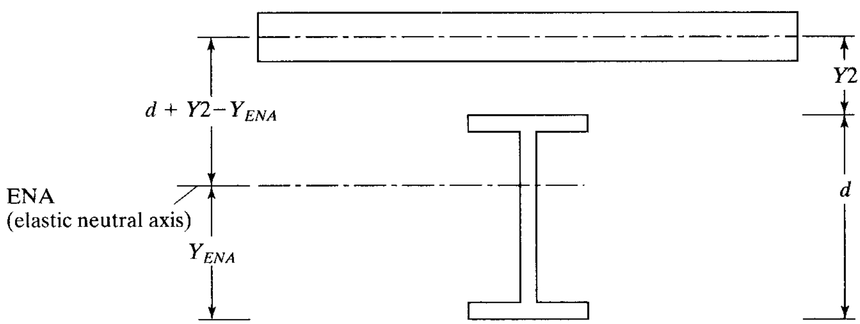


FIGURE 16.12

Here,

I_{LB} = lower bound moment of inertia, in^4
I_s = moment of inertia of steel section, in^4
d_1 = distance from the compression force in the concrete to the top of the steel section, in
d_3 = distance from the resultant steel tension force for full section tension yield to the top of the steel, in
Y_{ENA} = the distance from the bottom of the beam to the elastic neutral axis (ENA), in

$$= \left[\left(A_s d_3 + \left(\frac{\Sigma Q_n}{F_y}\right)(2d_3 + d_1)\right) \Big/ \left(A_s + \frac{\Sigma Q_n}{F_y}\right)\right] \quad \text{(C-I3-2)}$$

16.11.6 Extra Reinforcing

For building design calculations, the spans often are considered to be simply supported, but the steel beams generally do not have perfectly simple ends. The result is that some negative moment may occur at the beam ends, with possible cracking of the slab above. To prevent or minimize cracking, some extra steel can be placed in the top of the slab, extending 2 or 3 ft out into the slab. The amount of steel added is in addition to that needed to meet the temperature and shrinkage requirements specified by the American Concrete Institute.[5]

16.11.7 Example Problems

Examples 16-3 and 16-4 illustrate the designs of two unshored composite sections.

Example 16-3

Beams 10 ft on center with 36-ft simple spans are to be selected to support a 4-in-deep lightweight concrete slab on a 3-in-deep formed steel deck with no shoring. The ribs for the steel deck, which are perpendicular to the beam center lines, have average widths of 6 in. If the service dead load (including the beam weight) is to be 0.78 k/ft of length of the beams and the service live load is 2 k/ft, (a) select the beams, (b) determine the number of 3/4-in-diameter headed studs required, (c) compute the service live load deflection, and (d) check the beam shear. Other data are as follows: 50 ksi steel, f_c = 4 ksi, and concrete weight 110 lb/ft^3.

Solution. Loads and moments

[5] *Building Code Requirements for Reinforced Concrete*, ACI std. 318-05 (Detroit: American Concrete Institute, 2005), Section 7.12.

LRFD	ASD
$w_u = (1.2)(0.78) + (1.6)(2.0) = 4.14$ k/ft	$w_a = 0.78 + 2.0 = 2.78$ k/ft
$M_u = \dfrac{(4.14)(36)^2}{8} = 670.7$ ft-k	$M_a = \dfrac{(2.78)(36)^2}{8} = 450.4$ ft-k

Effective flange width b_e

$$b_e = (2)\left(\tfrac{1}{8} \times 36 \times 12\right) = 108 \text{ in} \leftarrow$$
$$b_e = (2)(5 \times 12) = 120 \text{ in}$$

a. Select W section

Y_{con} = distance from top of slab to top of steel flange $= 4 + 3 = 7$ in

Assume $a = 2$ in < 4 in slab thickness (It's usually quite small, particularly for relatively light sections.)

Y1 is distance from PNA to top flange $= 0$ in

Y2 is the distance from the center of gravity of the concrete flange force to the top flange of the beam $= 7 - a/2 = 7 - 2/2 = 6$ in

Looking through the composite tables of the Manual, with $M_u = 670.7$ ft-k, Y1 = 0, and Y2 = 6 in, we can see that several W 18s (46 lb, 50 lb, and 55 lb) seem reasonable.

Try W18 × 46 ($A = 13.5 \text{ in}^2$, $I_x = 712 \text{ in}^4$);

Check deflection due to wet concrete plus beam wt

$$w = \left(\frac{4}{12}\right)(110)(10) + 46 = 413 \text{ lbs/ft}$$

$$M = \frac{(0.413)(36)^2}{8} = 66.9 \text{ ft-k}$$

$C_1 = 161$ from Fig. 3-2 in AISC Manual

$$\Delta = \frac{ML^2}{C_1 I_x} = \frac{(66.9)(36)^2}{(161)(712)} = 0.76 \text{ in} < 2.5 \text{ in} \quad \textbf{OK}$$

Assume $\Sigma Q_n = A_s F_y = (13.5)(50) = 675$ k ($\Sigma Q_n = 677$ k from AISC Table 3-19)

$$\text{Thus, } a \text{ required} = \frac{\Sigma Q_n}{0.85 f'_c b_e} = \frac{675}{(0.85)(4)(108)} = 1.84 \text{ in} < 4 \text{ in}$$

$$\text{Y1} = 0$$
$$\text{Y2} = 7.00 - \frac{1.84}{2} = 6.08 \text{ in}$$

$\phi_b M_n$ from Manual, by interpolation,

$$= 763 + \left(\frac{0.08}{0.50}\right)(788 - 763) = 767 \text{ ft-k} > 670.7 \text{ ft-k} \quad \textbf{OK}$$

This moment for which ΣQ_n is 677 k is somewhat on the high side. With this same W18 × 46, we can go to the case in AISC Table 3-19 where Y1 is the largest possible, to provide a $\phi_b M_n$ of about 671 ft-k with Y2 = approximately 6 in. The result will be that ΣQ_n will be smaller, and fewer shear connectors will be necessary. This will occur when Y1 = 0.303 in and ΣQ_n = 494 ft-k (see AISC Table 3-19) with Y2 = 6 in.

$$a_{reqd} = \frac{\Sigma Q_n}{0.85 f'_c b_e} = \frac{494}{(0.85)(4)(108)} = 1.35 \text{ in}$$

$$Y2 = 7.00 - \frac{1.35}{2} = 6.33 \text{ in}$$

LRFD	ASD
$\phi_b M_n = 678 + \left(\frac{0.33}{0.50}\right)(697 - 678) = 690.5$ ft-k	$\frac{M_n}{\Omega} = 451 + \left(\frac{0.33}{0.50}\right)(464 - 451)$
> 670.7 ft-k **OK**	$= 459.6$ ft-k > 450.4 ft-k

Use W18 × 46, F_y = 50 ksi.

b. Design of steel headed stud anchors

The strengths (or Q_n values) of individual studs are provided in AISC Table 3-21. The author selected from this table a value of 21.2 k for 3/4-in headed studs enclosed in 4 ksi lightweight concrete weighing 110 lbs/ft^3. To obtain this value, he also made the assumptions that only one stud would be placed in each rib and that the studs would be in the strong position described in the AISC Commentary Section I8.2a.

$$\text{Number of anchors required} = \frac{\Sigma Q_n}{Q_n} = \frac{494}{21.2} = 23.3$$

Use 24 3/4-in studs on each side of point of maximum moment (which is L_c here).

c. Compute LL deflection

Assume maximum permissible LL deflection

$$\frac{1}{360}\text{ span} = \left(\frac{1}{360}\right)(12 \times 36) = 1.2 \text{ in}$$

$$C_1 = 161 \text{ from Fig. 3-2 in AISC Manual}$$

$$M_L = \frac{(2.0 \text{ k/ft})(36 \text{ ft})^2}{8} = 324 \text{ ft-k}$$

I_{LB} = lower bound moment of inertia from AISC Table 3-20 using straight-line interpolation

$$= 2000 + \left(\frac{0.33}{0.50}\right)(2090 - 2000) = 2059 \text{ in}^4$$

$$\Delta_L = \frac{ML^2}{C_1 I_{LB}} = \frac{(324)(36)^2}{(161)(2059)} = 1.27 \text{ in} > 1.2 \text{ in } \textbf{A little high}$$

d. Check beam shear for steel section

LRFD	ASD
$V_u = \dfrac{4.14(36)}{2} = 74.5\text{ k}$	$V_a = \dfrac{2.78(36)}{2} = 50.0\text{ k}$
$\phi V_n = 195$ k from Table 3-2 in Manual **OK**	$\dfrac{V_n}{\Omega} = 130$ k from Table 3-2 in Manual **OK**

Use W18 × 46 with forty-eight 3/4-in headed studs.

Example 16-4

Using the same data as for Example 16-3, except that $w_L = 1.2$ k/ft and $f'_c = 3$ ksi, perform the following tasks:

a. Select steel beam for composite action.
b. If the studs are placed in the weak position with no more than one stud per rib, determine the number of 3/4-in headed studs required.
c. Check the beam strength before the concrete hardens.
d. Compute service load deflection before concrete hardens. Assume a construction live load of 20 psf.
e. Determine the service live load deflection after composite action is available.
f. Check shear.
g. Select a steel section to carry all the loads if no shear connectors are used, and compute its service live load deflection.

Solution

LRFD	ASD
$w_u = (1.2)(0.78) + (1.6)(1.2) = 2.86\text{ k/ft}$	$w_a = 0.78 + 1.2 = 1.98\text{ k/ft}$
$M_u = \dfrac{(2.86)(36)^2}{8} = 463.3\text{ ft-k}$	$M_a = \dfrac{(1.98)(36)^2}{8} = 320.8\text{ ft-k}$

a. Select W section

$$Y_{\text{con}} = 4 + 3 = 7\text{ in}$$

Assume $a = 2$ in

$$Y1 = 0$$

$$Y2 = 7 - \frac{2}{2} = 6\text{ in}$$

Try W16 × 31 ($A = 9.13\text{ in}^2$, $d = 15.9$ in, $t_w = 0.275$ in)

Assume $\Sigma Q_n = (9.13)(50) = 456.5$ k or 456 k from AISC Manual Table 3-19

$$a = \frac{456.5}{(0.85)(3)(108)} = 1.66 \text{ in} < 4 \text{ in}$$

$$Y2 = 7 - \frac{1.66}{2} = 6.17 \text{ in}$$

$\phi_b M_n$ from AISC Table 3-19 by interpolation

$$= 477 + \left(\frac{0.17}{0.50}\right)(494 - 477) = 482.8 \text{ ft-k} > 463.3 \text{ ft-k} \quad \textbf{OK}$$

b. Design of studs

Q_n from AISC Table 3-21 = 17.2 k

We cannot go down in the W16 × 31 values to reduce, because $\phi_b M_n$ values are insufficient

$$\Sigma Q_n = 456.5 \text{ kips}$$

$$\text{Number of stud anchors required} = \frac{456.5}{17.2} = 26.5$$

Use twenty-seven 3/4-in stud anchors each side of ℄.

c. Check strength of W section before concrete hardens.

Assume that the wet concrete is a LL during construction, and also add a 20 psf construction live load.

Concrete wt = wt of slab + wt of ribs

$$= \left(\frac{4}{12}\right)(10)(110) + [(3)(6)/144](110)(10) = 504 \text{ lbs/ft}$$

Other dead loads = Deck wt (assume = 2 psf) + Beam wt

$$= 2(10) + 31 = 51 \text{ lbs/ft}$$

LRFD	ASD
$w_u = (1.2)(0.051) + (1.6)(0.020 \times 10 + 0.504) = 1.19$ k/ft	$w_a = 0.051 + 0.704 = 0.76$ k/ft
$M_u = \dfrac{(1.19)(36)^2}{8} = 192.8$ ft-k	$M_a = \dfrac{(0.76)(36)^2}{8} = 123.1$ ft-k
Assume metal deck provides lateral bracing	$\dfrac{M_n}{\Omega} = 135$ ft-k from AISC Table 3-2
$\phi M_n = 203$ ft-k from AISC Table 3-2 > 192.8 ft-k **OK**	> 123.1 ft-k **OK**

(d) Service load deflection before concrete hardens. I_x for W16 × 31 = 375 in^4 (not lower bound I)

Use $w_D = 0.76$ k/ft

$$M_D = \frac{(0.76)(36)^2}{8} = 123.1 \text{ ft-k}$$

$$\Delta_{DL} = \frac{(123.1)(36)^2}{(161)(375)} = 2.64 \text{ in} > 2.50 \text{ in}$$

(We might camber beam for this deflection and/or use PR connections.)

e. Service LL deflection after composite action is available

$$M_L = \frac{(1.2)(36)^2}{8} = 194.4 \text{ ft-k}$$

Lower bound I from AISC Table 3-20 with Y1 = 0 and Y2 = 6.17 in

$$I = 1260 + \left(\frac{0.17}{0.50}\right)(1320 - 1260) = 1284 \text{ in}^4$$

$$\Delta_L = \frac{(194.4)(36)^2}{(161)(1284)} = 1.22 \text{ in} > \frac{L}{360} = 1.2 \text{ in (Probably OK)}$$

f. Check shear

LRFD	ASD
$w_u = (1.2)(0.78) + (1.6)(1.2) = 2.86$ k/ft	$w_a = 0.78 + 1.2 = 1.98$ k/ft
$V_u = \frac{(2.86)(36)}{2} = 51.5$ k	$V_a = \frac{(1.98)(36)}{2} = 35.64$ k
$\phi V_n = 131$ k from AISC Table 3-2 > 51.4 k **OK**	$\frac{V_n}{\Omega} = 87.3$ k from AISC Table 3-2 > 35.64 k **OK**

g. Selecting steel section, no composite action

LRFD	ASD
$M_u = 463.3$ ft-k	$M_a = 320.8$ ft-k
Select W21 × 55 from AISC Table 3-2	Select W24 × 55 from AISC Table 3-2
$\phi M_n = 473$ ft-k > 463.3 ft-k	$\frac{M_n}{\Omega} = 334$ ft-k > 320.8 ft-k

I_x for W21 × 55 = 1140 in^4

M_L = 194.4 ft-k from part e of this problem

$$\text{Service live load deflection} = \frac{(194.4)(36)^2}{(161)(1140)} = 1.37 \text{ in} > \frac{L}{360} = 1.2 \text{ in}$$

$$\text{Min } I_x \text{ to limit deflection to 1.2 in} = \left(\frac{1.37}{1.2}\right)(1140) = 1302 \text{ in}^4$$

(A W24 × 55 would be required by Table 3-3 in the Manual to provide such an I_x, or a W21 × 62 to keep depth approximately the same.)

Continuous-welded plate girders in Henry Jefferson County, IA. (Courtesy of the Lincoln Electric Company.)

16.12 CONTINUOUS COMPOSITE SECTIONS

The AISC Specification (I3.2b) permits the use of continuous composite sections. The flexural strength of a composite section in a negative moment region may be considered to equal $\phi_b M_n$ for the steel section alone, or it may be based upon the plastic strength of a composite section considered to be made up of the steel beam and the longitudinal reinforcement in the slab. For this latter method to be used, the following conditions must be met:

1. The steel section must be compact and adequately braced.
2. The slab must be connected to the steel beams in the negative moment region with shear connectors.
3. The longitudinal reinforcing in the slab parallel to the steel beam and within the effective width of the slab must have adequate development lengths. (*Development length* is a term used in reinforced-concrete design and refers to the length that reinforcing bars have to be extended or embedded in the concrete in order to properly anchor them or develop their stresses by means of bond between the bars and the concrete.)

For a particular beam, the total horizontal shear force between the point of zero moment and the point of maximum negative moments is to be taken as the smaller of $A_{sr}F_{ysr}$ and ΣQ_n, where A_{sr} is the cross-sectional area of the properly developed reinforcing and F_{ysr} is the yield stress of the bars. The plastic stress distribution for negative moment in a composite section is illustrated in Fig. 16.13.

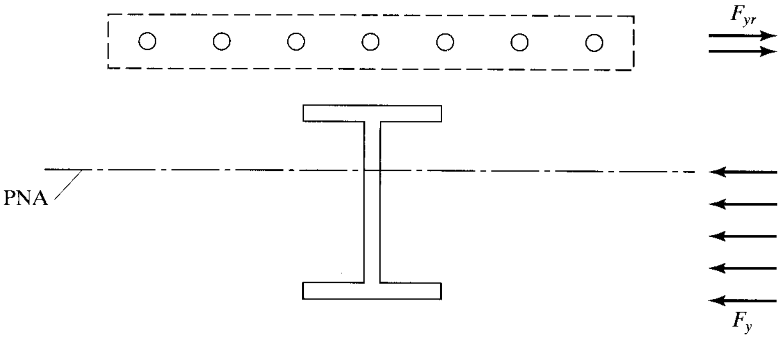


FIGURE 16.13
Stress distribution in negative moment range.

16.13 DESIGN OF CONCRETE-ENCASED SECTIONS

For fireproofing purposes, it is possible to completely encase in concrete the steel beams used for building floors. This practice definitely is not economical, because lightweight spray-on fire protection is so much cheaper. Furthermore, encased beams may increase the floor system dead load by as much as 15 percent.

For the rare situation in which encased beams are used, steel anchors shall be provided.

The nominal flexual strength, M_n, shall be determined using one of several methods, see AISC Specification I3.3.

1. With one method, the design strength of the encased section may be based on the plastic moment capacity $\phi_b M_p$ or $\dfrac{M_p}{\Omega_b}$ of the steel section alone, with $\phi_b = 0.90$ and $\Omega_b = 1.67$.
2. By another method, the design strength is based on the first yield of the tension flange, assuming composite action between the concrete, which is in compression, and the steel section. Again, $\phi_b = 0.90$ and $\Omega_b = 1.67$.

If the second method is used and we have unshored construction, the stresses in the steel section caused by the wet concrete and other construction loads are calculated. Then the stresses in the composite section caused by loads applied after the concrete hardens are computed. These stresses are superimposed on the first set of stresses. If we have shored construction, all of the loads may be assumed to be supported by the composite section and the stresses computed accordingly. For stress calculations, the properties of a composite section are computed by the transformed area method. In this method, the cross-sectional area of one of the two materials is replaced or transformed into an equivalent area of the other. For composite design, it is customary to replace the concrete with an equivalent area of steel, whereas the reverse procedure is used in the working stress design method for reinforced-concrete design.

In the transformed area procedure, the concrete and steel are assumed to be bonded tightly together so that their strains will be the same at equal distances from the neutral axis. The unit stress in either material can then be said to equal its strain times its modulus of elasticity (ϵE_c for the concrete or ϵE_s for the steel). The unit stress in the steel is then $\epsilon E_s/\epsilon E_c = E_s/E_c$ times as great as the corresponding unit stress in the concrete. The E_s/E_c ratio is referred to as the modular ratio n; therefore, n in^2 of concrete are required to resist the same total stress as 1 in^2 of steel; and the cross-sectional area of the slab (A_c) is replaced with a transformed or equivalent area of steel equal to A_c/n.

The American Concrete Institute (ACI) Building Code states that the following expression may be used for calculating the modulus of elasticity of concrete weighing from 90 to 155 lb/ft^3:

$$E_c = w_c^{1.5} 33\sqrt{f'_c}$$

In this expression, w_c is the weight of the concrete in pounds per cubic foot, and f'_c is the 28-day compressive strength in pounds per square inch.

> In SI units with w_c varying from 1500 to 2500 kg/m^3 and with
> f'_c in N/mm^2 or MP_a $E_c = w_c^{1.5}(0.043)\sqrt{f'_c}$

There are no slenderness limitations required by the AISC Specification for either of the two methods, because the encasement is effective in preventing both local and lateral buckling.

In Example 16-5, which follows, stresses are computed by the elastic theory, assuming composite action as described for the second method. Notice that the author has divided the effective width of the slab by n to transform the concrete slab into an equivalent area of steel.

Example 16-5

Review the encased beam section shown in Fig. 16.14 if no shoring is used and the following data are assumed:

$$\text{Simple span} = 36 \text{ ft}$$

$$\text{Service dead load} = 0.50 \text{ k/ft before concrete hardens plus an additional } 0.25 \text{ k/ft after concrete hardens}$$

$$\text{Construction live loads} = 0.2 \text{ k/ft}$$

$$\text{Service live load} = 1.0 \text{ k/ft after concrete hardens}$$

$$\text{Effective flange width } b_e = 60 \text{ in and } n = 9$$

$$F_y = 50 \text{ ksi}$$

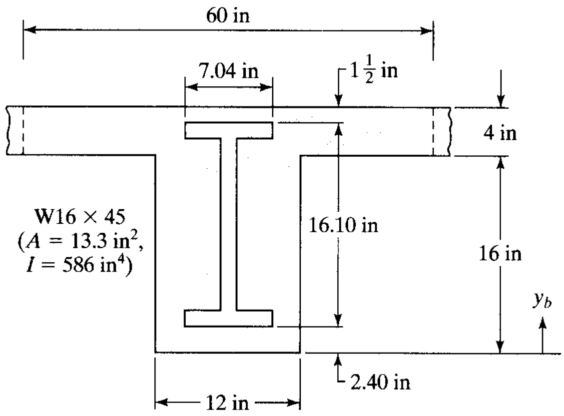


FIGURE 16.14

Solution. Calculated Properties of Composite Section: Neglecting concrete area below the slab.

$$A = 13.3 \text{ in}^2 + \frac{(4 \text{ in})(60 \text{ in})}{9} = 39.96 \text{ in}^2$$

$$y_b = \frac{(13.3 \text{ in}^2)(10.45 \text{ in}) + (26.66 \text{ in}^2)(18 \text{ in})}{39.96} = 15.50 \text{ in}$$

$$I = 586 \text{ in}^4 + (13.3 \text{ in}^2)(5.05 \text{ in})^2 + \left(\tfrac{1}{12}\right)\left(\tfrac{60}{9} \text{ in}\right)(4 \text{ in})^3 + (26.66 \text{ in}^2)(2.5 \text{ in})^2$$
$$= 1127 \text{ in}^4$$

Stresses before concrete hardens

Assume that wet concrete is a live load

$$w_u = (1.6)(0.5 \text{ k/ft} + 0.2 \text{ k/ft}) = 1.12 \text{ k/ft}$$

$$M_u = \frac{(1.12 \text{ k/ft})(36 \text{ ft})^2}{8} = 181.4 \text{ ft-k}$$

Assume beam only properties

$$f_t = \frac{(12 \text{ in/ft})(181.4 \text{ ft-k})(8.05 \text{ in})}{586 \text{ in}^4} = 29.90 \text{ ksi}$$

$$< \phi_b F_y = (0.9)(50) = 45 \text{ ksi} \qquad \text{(OK)}$$

Stresses after concrete hardens

$$w_u = (1.2)(0.25 \text{ k/ft}) + (1.6)(1.0 \text{ k/ft}) = 1.9 \text{ k/ft}$$

$$M_u = \frac{(1.9 \text{ k/ft})(36 \text{ ft})^2}{8} = 307.8 \text{ ft-k}$$

$$f_t = \frac{(12 \text{ in/ft})(307.8 \text{ ft-k})(15.50 \text{ in} - 2.40 \text{ in})}{1127 \text{ in}^4} = 42.93 \text{ ksi}$$

Total $f_t = 29.90 + 42.93 = 72.83 \text{ ksi} > 0.9F_y = 45 \text{ ksi}$ (NG)

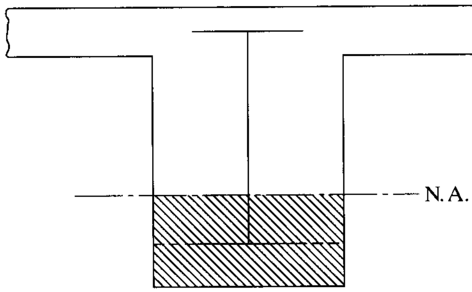


FIGURE 16.15

Composite section subjected to negative moment.

For buildings, continuous composite construction with encased sections is permissible. For continuous construction, the positive moments are handled exactly as has been illustrated by the preceding example. For negative moments, however, the transformed section is taken as shown in Fig. 16.15. The crosshatched area represents the concrete in compression, and all concrete on the tensile side of the neutral axis (that is, above the neutral axis) is neglected.

16.14 PROBLEMS FOR SOLUTION

Use LRFD and ASD methods for Problems 16-1 through 16-19.

16-1. Determine $\phi_b M_n$ and $\dfrac{M_n}{\Omega_b}$ for the section shown, assuming that sufficient steel anchors are provided to ensure full composite section. Solve by using the procedure presented in Section 16.9 and check answers with tables in the Manual. $F_y = 50$ ksi, $f'_c = 3$ ksi. (*Ans.* 366.3 ft-k, LRFD; 243.7 ft-k, ASD)

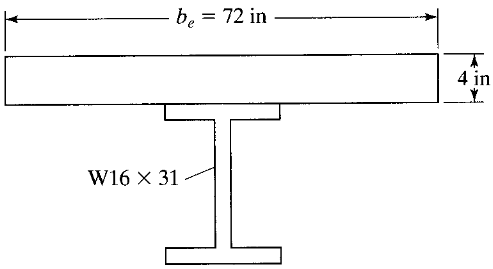


FIGURE P16-1

16-2. Repeat Prob. 16-1 if a W18 × 55 is used.

16-3. Repeat Prob. 16-2, using tables in Manual if it is considered to be partially composite and if ΣQ_n is 454 k. (*Ans.* 637.8 ft-k, 424.2 ft-k)

16-4. Determine $\phi_b M_n$ and $\dfrac{M_n}{\Omega_b}$ for the section shown, if 50 ksi steel and sufficient steel anchors are used to guarantee full composite action. Use formulas and check with Manual. $f'_c = 4$ ksi.

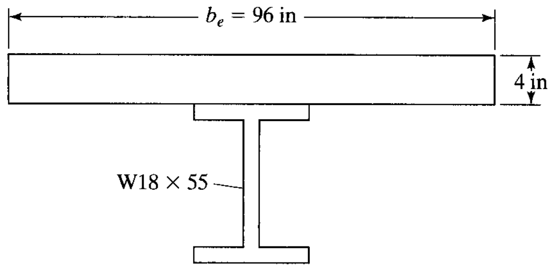


FIGURE P16-4

16-5. Repeat Prob. 16-4 if a W16 × 36 is used. (*Ans.* 442.7 ft-k, 294.5 ft-k)

16-6. Compute $\phi_b M_n$ and $\frac{M_n}{\Omega_b}$ for the composite section shown, if 50 ksi steel is used and sufficient steel anchors are used to provide full composite action. A 3-in concrete slab is supported by 2-in-deep composite metal deck ribs perpendicular to beam; f'_c = 4 ksi. Check answers with Manual.

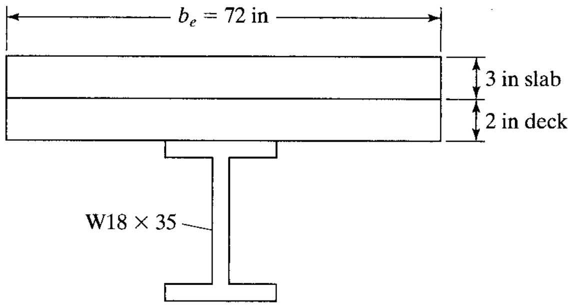


FIGURE P16-6

16-7. Repeat Prob. 16-6 using Manual tables if ΣQ_n of the anchors is 387 k. (*Ans.* 462.9 ft-k, 308.2 ft-k)

16-8. Using the Composite Design Tables of the AISC Manual, 50 ksi steel, a 145 lb/ft^3 concrete slab with f'_c = 4 ksi and shored construction, select the steel section, design 3/4-in headed studs, calculate live load service deflection, and check the shear if the service live load is 100 psf. Refer to the accompanying figure.

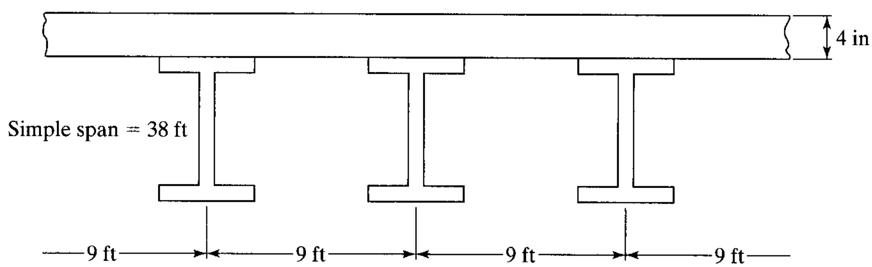


FIGURE P16-8

16-9. Repeat Prob. 16-8 if span is 32 ft and live load is 80 psf. (*Ans.* For LRFD and ASD W14 × 22 with 19 studs)

16-10. For Prob. 16-9, calculate the deflection during construction for wet concrete, plus 20 psf live load for construction activities.

16-11. Select a 50 ksi section to support a service dead load of 200 psf and a service live load of 100 psf. The beams are to have 37.5-ft simple spans and are to be spaced 8 ft 6 in on center. Construction is shored, concrete weighs 110 lb/ft^3, f'_c is 3.5 ksi, and a metal deck with ribs perpendicular to the steel beams is used together with a 4-in concrete slab. The ribs are 3-in deep and have average widths of 6 in. Design 3/4-in headed studs and calculate live load deflection. (*Ans.* For LRFD W18 × 46 with 61 studs)

16-12. Using the AISC Manual and 50 ksi steel, design a nonencased unshored composite section for the simple span beams shown in the accompanying figure if a 4-in concrete slab (145 lb/ft^3) with $f'_c = 4$ ksi is used. The total service dead load, including the steel beam, is 0.6 k/ft of length of the beam, and the service live load is 1.25 k/ft. Assume construction $LL = 20$ psf and $L_b = 0$.

a. Select the beams.
b. Determine the number of 3/4-in-diameter headed studs required.
c. Compute the service live load deflection.
d. Check the beam shear.

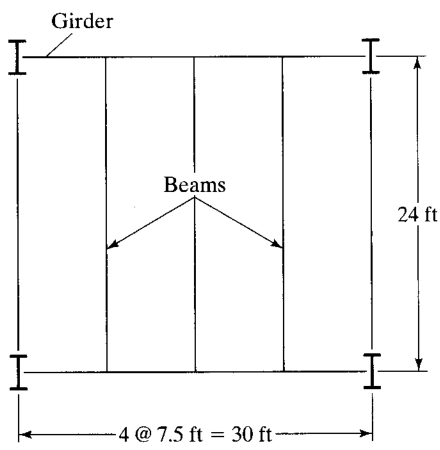


FIGURE P16-12

16-13. Repeat Prob. 16-12 if span is 28 ft and live load is 1 k/ft (*Ans.* W14 × 22 with 22 3/4-in studs)

16-14. 50 ksi beams 9 ft on center and spanning 40 ft are to be selected to support a 4-in-deep lightweight concrete slab ($f'_c = 4$ ksi, weight 110 lb/ft^3) on a 3-in-deep formed steel deck without shoring. The ribs for the steel deck, which are perpendicular to the steel beams, have average widths of 6 in. If the total service dead load, including the beam weight, is to be 0.80 k/ft of length of the beams, and the service live load is 1.25 k/ft, (a) select the beams, (b) determine the number of 3/4-in-diameter headed studs required, (c) compute the service live load deflection, and (d) check the beam shear.

16-15. Repeat Prob. 16-14 if spans are 45 ft. (*Ans.* W21 × 44 with 68 studs LRFD)

16-16. Repeat Prob. 16-14 if spans are 32 ft.

16-17. Repeat Prob. 16-16 if spans are 34 ft and the live load is 2 k/ft. (*Ans.* For LRFD W18 × 40 with 60 3/4-in studs)

16-18. Using the same data as for Prob. 16-14, except that unshored construction is to be used for a 45 ft span, perform the following tasks:

a. Select the steel beam.
b. If the stud reduction factor for metal decks is 1.0, determine the number of 3/4-in headed studs required assuming deck ribs are perpendicular to the beams.
c. Check the beam strength before the concrete hardens.
d. Compute service load deflection before the concrete hardens, assuming a construction live load of 25 psf.
e. Determine the service load deflection after composite action is available.
f. Check shear.

16-19. Repeat Prob. 16-18 if span is 35 ft and live load is 1.60 k/ft. (*Ans.* W18 × 35 with 54 studs LRFD)

16-20. Using the transformed area method, compute the stresses in the encased section shown in the accompanying illustration if no shoring is used. The section is assumed to be used for a simple span of 30 ft and to have a service dead uniform load of 30 psf applied after composite action develops and a service live uniform load of 120 psf. Assume $n = 9$, $F_y = 50$ ksi, $f'_c = 4$ ksi, and concrete weighing 150 lb/ft^3.

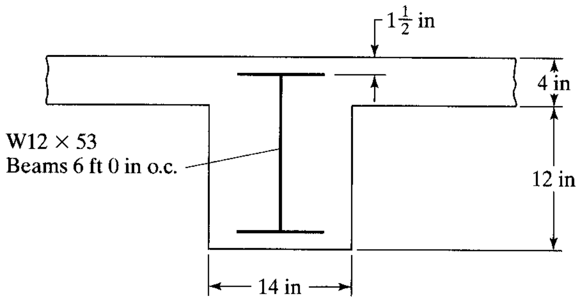


FIGURE P16-20

C H A P T E R 1 7

Composite Columns

17.1 INTRODUCTION

Composite columns are constructed with rolled or built-up steel shapes encased in concrete, or with concrete placed inside HSS or pipe sections. The resulting members are able to support significantly higher loads than reinforced-concrete columns of the same sizes.

Several composite columns are shown in Fig. 17.1. In part (a) of the figure, a *W* shape embedded in concrete is shown. The cross sections, which usually are square or rectangular, have one or more longitudinal bars placed in each corner. In addition, lateral ties are wrapped around the longitudinal bars at frequent vertical intervals. Ties are effective in increasing column strengths. They prevent the longitudinal bars from being displaced during construction, and they resist the tendency of these

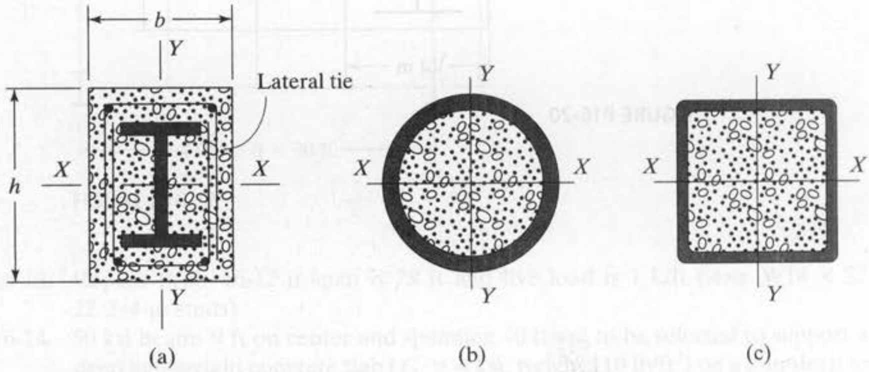


FIGURE 17.1

Composite columns.

same bars to buckle outward under load, which would cause breaking or spalling off of the outer concrete cover. Notice that these ties are all open and U-shaped. Otherwise they could not be installed, because the steel column shapes always will have been erected at an earlier time. In parts (b) and (c) of the figure, steel pipe and HSS sections filled with concrete are shown.

17.2 ADVANTAGES OF COMPOSITE COLUMNS

For a number of decades, structural steel shapes have been used in combination with plain or reinforced concrete. Originally, the encasing concrete was used to provide only fire and corrosion protection for the steel, with no consideration given to its strengthening effects. More recently, however, the development and increasing popularity of composite frame construction has encouraged designers to include the strength of the concrete in their calculations.[1,2]

Composite columns may be practically used for low-rise and high-rise buildings. For the low-rise warehouses, parking garages, and so on, the steel columns are often encased in concrete for the sake of appearance or for protection from fire, corrosion, and (in garages) vehicles. If we are going to encase the steel in concrete anyway, we may as well take advantage of the concrete and use smaller steel shapes.

For high-rise buildings, the sizes of composite columns often are considerably smaller than is required for reinforced-concrete columns to support the same loads. The results with composite designs are appreciable savings of valuable floor space. Closely spaced composite steel-concrete columns connected with spandrel beams may be used around the outsides of high-rise buildings to resist lateral loads by the tubular concept (described in Chapter 19). Very large composite columns are sometimes placed on the corners of high-rise buildings to increase lateral resisting moments. Also, steel sections embedded within reinforced-concrete shear walls may be used in the central core of high-rise buildings. This also ensures a greater degree of precision in the construction of the core.

With composite construction, the bare steel sections support the initial loads, including the weights of the structure, the gravity, lateral loads occurring during construction, and the concrete later cast around the *W* shapes or inside the tube shapes. The concrete and steel are combined in such a way that the advantages of both materials are used in the composite sections. For instance, the reinforced concrete enables the building frame to more easily limit swaying or lateral deflections. At the same time, the light weight and strength of the steel shapes permit the use of smaller and lighter foundations.[3]

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[1]D. Belford, "Composite Steel Concrete Building Frame," *Civil Engineering* (New York: ASCE, July 1972), pp. 61–65.

[2]Fazlur R. Kahn, "Recent Structural Systems in Steel for High Rise Buildings," BCSA Conference on Steel in Architecture, (London, November 24–26, 1969).

[3]L. G. Griffis, "Design of Encased *W*-shape Composite Columns," Proceedings 1988 National Steel Construction Conference (AISC, Chicago, June 8–11, 1988), pp. 20-1–20-28.

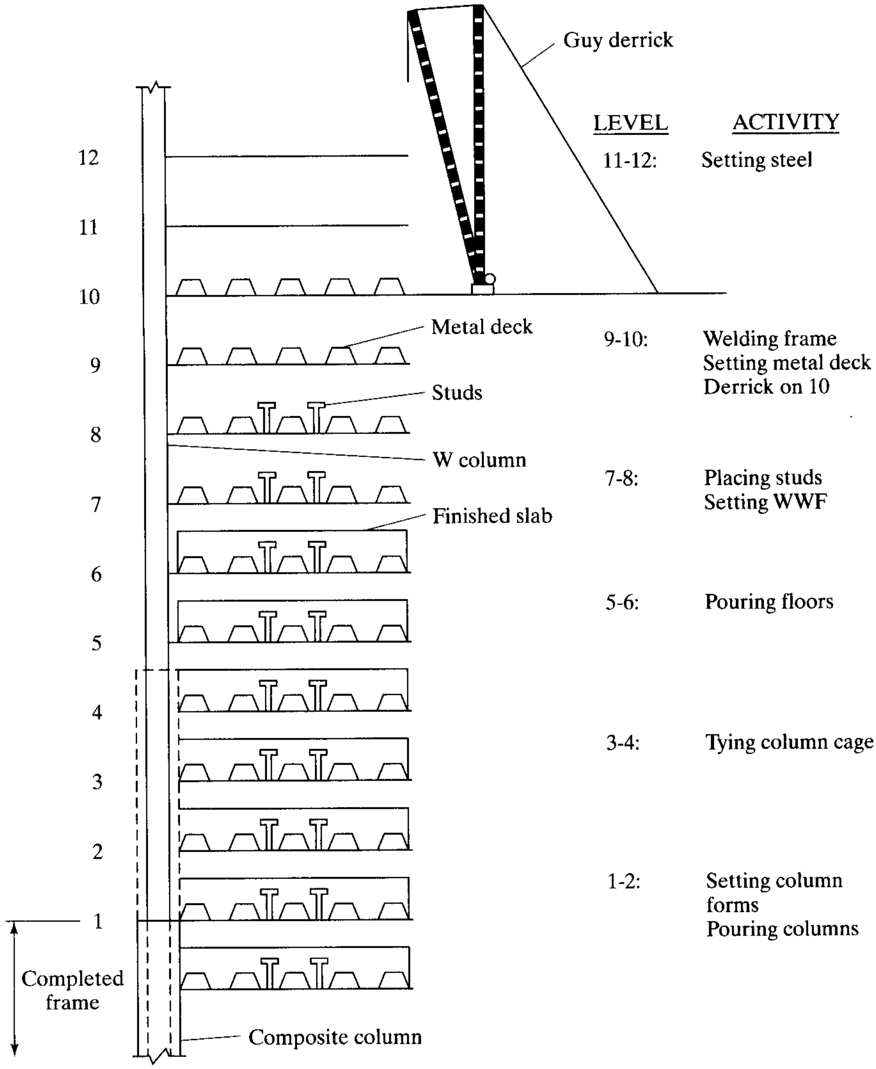

FIGURE 17.2

Sequence of construction operations for a composite building frame. (Courtesy of AISC.)

Composite high-rise structures are erected in a rather efficient manner. There is quite a vertical spread of construction going on at any one time, with numerous trades working simultaneously. This situation, which is pictured in Fig. 17.2, is briefly described here.[4]

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[4]Griffis, op.cit.

1. One group of workers may be erecting the steel beams and columns for one or two stories on top of the frame.
2. Two or three stories below, another group will be setting the metal decking for the floors.
3. A few stories below that, another group may be placing the concrete for the floor slabs.
4. This continues as we go down the building, with one group tying the column reinforcing bars in cages, while below them others are placing the column forms, placing the column concrete, and so on.

17.3 DISADVANTAGES OF COMPOSITE COLUMNS

As described in the preceding section, composite columns have several important advantages. They also have a few disadvantages. One particular problem with their use in high-rise buildings is the difficulty of controlling their rates and amounts of shortening in relation to shear walls and, perhaps, adjacent plain steel columns. The accurate estimation of these items is made quite difficult by the different types and stages of construction activities going on simultaneously over a large number of building stories.

If composite columns are used around the outside of a high-rise building, and plain steel sections are used in the building core (or if we have shear walls), the creep in the composite sections can be a problem. The results may be concrete floors that are not very level. Some erectors make very careful elevation measurements at column splices and then try to make appropriate adjustments with steel shims to try to even out the differences between measured elevations and computed elevations.

Another problem with composite columns is the lack of knowledge available concerning the mechanical bond between the concrete and the steel shapes. This is particularly important for the transfer of moments through beam–column joints. It is feared that if large cyclical strain reversals were to occur at such a joint (as in a seismic area), there could be a severe breakdown of the joint.[5]

17.4 LATERAL BRACING

Resistance to lateral loads for the usual structural steel or reinforced-concrete high-rise building is provided as the floors are being constructed. For instance, diagonal bracing or moment-resisting joints may be provided for each floor as a structural steel building frame is being constructed. In a similar manner, the needed lateral strength of a reinforced-concrete frame may be provided by the moment resistance obtained with monolithic construction of its members and/or by shear walls.

For composite construction, the desired lateral strength of a building is not obtained until the concrete has been placed around or inside the erected steel members and has sufficiently hardened. This situation is probably being achieved 10 to 18 stories behind the steel erection (see Fig. 17.2).

[5]Griffis, op.cit.

As we have mentioned, the steel fabricator is used to erecting a steel frame and providing the necessary wind bracing as the floors are erected. The steel frames used for high-rise composite buildings, however, do not usually have such bracing, and the frames will not have the desired lateral strength. This strength will be achieved only after the concrete is placed and cured for many building stories. Thus, the engineer of record for a composite high-rise building must clearly state the lateral force conditions and what is to be done about them during erection.[6]

17.5 SPECIFICATIONS FOR COMPOSITE COLUMNS

Composite columns theoretically can be constructed with cross sections that are square, rectangular, round, triangular, or any other shape. Practically, however, they usually are square or rectangular, with one reinforcing bar in each column corner. This arrangement enables us to use reasonably simple connections from the exterior spandrel beams and floor beams to the steel shapes in the columns, without unduly interfering with the vertical reinforcing.

The AISC Specification does not provide detailed requirements for reinforcing bar spacings, splices, and so on. Therefore, it seems logical that the requirements in this regard of the ACI 318 Code[7] should be followed for situations not clearly covered by the AISC Specification.

Sections I1 and I2 of the AISC Specification provide detailed requirements pertaining to cross-sectional areas of steel shapes, concrete strengths, tie areas, and spacings for the vertical reinforcing bars, and so on. This information is listed and briefly discussed in the paragraphs to follow.

For encased composite columns

1. *The total cross-sectional area of the steel section or sections may not be less than 1 percent of the gross column area.* If the steel percentage is less than 1 percent, the member is classified as a reinforced-concrete column, and its design must adhere to the *Building Code Requirements for Reinforced Concrete* of the American Concrete Institute.
2. *Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals.*

   Where lateral ties are used, a minimum of either a No. 3 bar spaced at a maximum of 12 in on center, or a No. 4 bar or larger spaced at a maximum of 16 in on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

   Maximum spacing of lateral ties shall not exceed 0.5 times the least column dimension.
3. The minimum reinforcing ratio for this steel is $\rho_{sr} = A_{sr}/A_g = 0.004$

   where A_{sr} = area of continuous reinforcing bars, in^2
   A_g = gross area of composite member, in^2

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[6]Griffis, op. cit.
[7]American Concrete Institute, *Building Code Requirements for Reinforced Concrete*, ACI 318-08 (Detroit: 2008).

4. *It is necessary to use steel anchors to resist the shear force in Section I4.1 of the AISC Specification. Steel anchors utilized to transfer longitudinal shear shall be distributed within the* load introduction length, *which shall not exceed a distance of two times the minimum transverse dimension of the encased composite member above and below the load transfer region. Anchors utilized to transfer longitudinal shear shall be placed on at least two faces of the steel shape in a generally symmetric configuration about the steel shape axes.*
   Steel anchor spacing, both within and outside of the load introduction length, shall conform to Section I8.3e.
5. *Should two or more steel shapes be used in the composite section, they must be connected together with lacing, tie plates, batten plates, or similar components.* Their purpose is to prevent the buckling of individual shapes before the concrete sets.
6. *There must be at least 1.5 in clear cover of concrete outside of any steel (ties or longitudinal bars).* The cover is needed for the protection of the steel from fire or corrosion. The amount of longitudinal and transverse reinforcing required in the encasement is thought to be sufficient to prevent severe spalling of the concrete surface from occurring during a fire.
7. *The specified compression strength of the concrete f_c' must be at least 3 ksi (21 MPa), but not more than 10 ksi if normal-weight concrete is used. For lightweight concrete, it may not be less than 3 ksi or more than 6 ksi.* The upper limits are provided, because sufficient test data are not available for composite columns with higher-strength concretes at this time. The lower limits of 3 ksi were specified for the purpose of ensuring the use of good-quality, but readily available, concrete and for the purpose of making sure that adequate quality control is used. This might not be the case if a lower grade of concrete were specified. The upper limit of 10 ksi for normal-weight concrete was specified because of the lack of data available for higher-strength concretes and because of the changes in behavior that have been observed in such concretes. The upper limit of 6 ksi for lightweight concrete is to ensure the use of readily available material. Higher-strength concretes may be used for calculating the modulus of elasticity for stiffness calculations, but may not be used for strength calculations, unless such use is justified by testing and analysis.
8. *The yield stresses of the steel sections and reinforcing bars used may not be greater than 75 ksi (525 MPa)*, unless higher strengths are justified by testing and analysis.

The original reason for limiting the value of F_y is given here. One major objective in composite design is the prevention of local buckling of the longitudinal reinforcing bars and the contained steel section. To achieve this objective, the covering concrete must not be allowed to break or spall. It was assumed by the writers of previous LRFD Specifications that such concrete was in danger of breaking or spalling if its strain reached 0.0018. If we take this strain and multiply it by F_s, we get $(0.0018)(29{,}000) \approx 55$ ksi. Hence, that value was specified as the maximum useable yield stress.

Recent research has shown that, due to concrete confinement effects, the 55 ksi value is conservative, and it has been raised to 75 ksi in the specification.

For filled composite columns

1. The cross-sectional area of the HSS section must make up no less than 1 percent of the total composite member cross section.
2. Filled composite columns are classified as compact, non-compact, or slender (AISC I1.4). They are compact if the width-to-thickness ratio does not exceed λp. If the ratio exceeds λp but does not exceed λ_r the section is non-compact. If the ratio exceeds λ_r the section is slender. The maximum permitted width-to-thickness ratios for both rectangular HSS (b/t) and round filled sections (D/t) are specified in Table I1.1A in the AISC Specification.

17.6 AXIAL DESIGN STRENGTHS OF COMPOSITE COLUMNS

If a composite column were perfectly axially loaded and fully braced laterally, its nominal strength would equal the sum of the axial strengths of the steel shape, the concrete, and the reinforcing bars, as given by

$$P_{no} = A_s F_y + A_{sr} F_{ysr} + 0.85 f'_c A_c \qquad \text{(AISC Equation I2-4)}$$

in which

A_s = area of steel section, in^2
A_{sr} = area of continuous reinforcing bars, in^2
F_{ysr} = specified minimum yield strength of reinforcing bars, ksi
A_c = area of concrete, in^2

Unfortunately, these ideal conditions are not present in practical composite columns. The contribution of each component of a composite column to its overall strength is difficult, if not impossible, to determine. The amount of flexural concrete cracking varies throughout the height of the column. The concrete is not nearly as homogeneous as the steel, and furthermore, the modulus of elasticity of the concrete varies with time and under the action of long-term or sustained loads. The effective lengths of composite columns in the rigid monolithic structures in which they are frequently used cannot be determined very well. The contribution of the concrete to the total stiffness of a composite column varies, depending on whether it is placed inside a tube or it is on the outside of a W section where its stiffness contribution is less.

The preceding paragraph presented some of the reasons it is difficult to develop a useful theoretical formula for the design of composite columns. As a result, one set of empirical equations is presented by the AISC Specification for concrete-encased sections (AISC I2.1), and another set is presented for concrete-filled sections (AISC I2.2).

Concrete-encased sections

In the expressions to follow, the following terms are used:

P_{no} = nominal compressive strength of the column without consideration of its length

$$= A_s F_y + A_{sr} F_{ysr} + 0.85 A_c f'_c \qquad \text{(AISC Equation I2-4)}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \qquad \text{(AISC Equation I2-7)}$$

I_s = moment of inertia of steel shape, in^4

I_{sr} = moment of inertia of reinforcing bars, in^4

EI_{eff} = effective stiffness of composite column, kip-in^2

$$= E_sI_s + 0.5E_sI_{sr} + C_1E_cI_c \qquad \text{(AISC Equation I2-6)}$$

P_e = elastic buckling load, kips

$$= \frac{\pi^2(EI_{eff})}{(KL)^2} \qquad \text{(AISC Equation I2-5)}$$

The available compressive strength ϕ_cP_n, with $\phi_c = 0.75$, and the allowable compressive strength P_n/Ω_c, with $\Omega_c = 2.00$, of doubly symmetric axially loaded encased composite columns are to be determined with the following expressions:

When $\dfrac{P_{no}}{P_e} \leq 2.25$

$$P_n = P_{no}\left[0.658^{\left(\frac{P_{no}}{P_e}\right)}\right] \qquad \text{(AISC Equation I2-2)}$$

When $\dfrac{P_{no}}{P_e} > 2.25$

$$P_n = 0.877P_e \qquad \text{(AISC Equation I2-3)}$$

Concrete-filled composite columns

(a) For compact sections:

$$P_{no} = P_p \qquad \text{(AISC Equation I2-9a)}$$

$$P_p = A_sF_y + C_2 f'_c\left[A_c + A_{sr}\left(\frac{E_s}{E_c}\right)\right] \qquad \text{(AISC Equation I2-9b)}$$

C_2 = 0.85 for rectangular sections and 0.95 for circular ones

For all sections:

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c \qquad \text{(AISC Equation I2-12)}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \qquad \text{(AISC Equation I2-13)}$$

P_e and P_n are determined with AISC Equations I2-2, I2-3 and I2-5, as in concrete-encased sections.

(b) For non-compact sections:

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2 \qquad \text{(AISC Equation I2-9c)}$$

λ, λ_p, λ_r are slenderless ratio from Table I1.1a

P_p is from Equation I2-9b

$$P_y = A_sF_y + 0.7f'_c\left(A_c + A_{sr}\left(\frac{E_s}{E_c}\right)\right) \qquad \text{(AISC Equation I2-9d)}$$

(c) For slender sections:

$$P_{no} = A_sF_{cr} + 0.7f'_c\left[A_c + A_{sr}\left(\frac{E_s}{E_c}\right)\right] \qquad \text{(AISC Equation I2-9e)}$$

For rectangular-filled sections: $F_{cr} = \dfrac{9E_s}{(b/t)^2}$ (AISC Equation I2-10)

or

For round-filled sections: $F_{cr} = \dfrac{0.72F_y}{\left[\left(\frac{D}{t}\right)\left(\frac{F_y}{E_s}\right)\right]^{0.2}}$ (AISC Equation I2-11)

Example 17-1

Compute the values of ϕ_cP_n and P_n/Ω_c, for the axially loaded encased composite column shown in Fig. 17.3 if KL = 12.0 ft, F_y = 50 ksi, and f'_c = 3.5 ksi. The concrete weighs 145 lb/ft^3.

Solution

Using a W12 × 72(A_s = 21.1 in^2, I_{sx} = 597 in^4, I_{sy} = 195 in^4)

$$A_c = (20\text{ in})(20\text{ in}) - 21.1\text{ in}^2 - (4)(1.0\text{ in}^2) = 374.9\text{ in}^2$$

$$P_{no} = A_sF_y + A_{sr}F_{ysr} + 0.85A_cf'_c \qquad \text{(AISC Equation I2-4)}$$

$$= (21.1\text{ in}^2)(50\text{ ksi}) + (4.0\text{ in}^2)(60\text{ ksi}) + (0.85)(374.9\text{ in})(3.5\text{ ksi}) = 2410\text{ k}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \qquad \text{(AISC Equation I2-7)}$$

$$= 0.1 + 2\left(\frac{21.1}{374.9 + 21.1}\right) = 0.2066 < 0.3 \qquad \textbf{OK}$$

$$E_c = w_c^{1.5}\sqrt{f'_c} = 145^{1.5}\sqrt{3.5} = 3.267 \times 10^3\text{ ksi}$$

$$I_c = \left(\frac{1}{12}\right)(20)(20)^3 - 195 = 13{,}138\text{ in}^4$$

$$EI_{eff} = E_sI_s + 0.5E_sI_{sr} + C_1E_cI_c \qquad \text{(AISC Equation I2-6)}$$

$$= (29 \times 10^3)(195) + (0.5)(29 \times 10^3)(4 \times 1.0 \times 7.5^2)$$
$$+ (0.2066)(3.267 \times 10^3)(13{,}138) = 17.785 \times 10^6\text{ k-in}^2$$

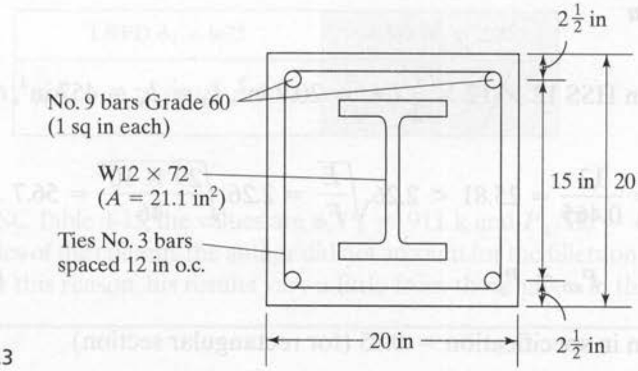


FIGURE 17.3

$$P_e = \frac{(\pi^2)(EI_{eff})}{(KL)^2} \qquad \text{(AISC Equation I2-5)}$$

$$= \frac{(\pi)^2(17.785 \times 10^6)}{(12 \times 12)^2} = 8465 \text{ k}$$

$$\frac{P_{no}}{P_e} = \frac{2410}{8465} = 0.28 \leq 2.25$$

$\therefore$ Must use AISC Equation I2-2 for P_n.

$$P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] = 2410\left[0.658^{\frac{2410}{8465}}\right] = 2139 \text{ k}$$

LRFD $\phi_c = 0.75$	ASD $\Omega_c = 2.00$
$\phi_c P_n = (0.75)(2139) = 1604$ k	$\dfrac{P_n}{\Omega_c} = \dfrac{2139}{2.00} = 1070$ k

Note: The W12 × 72 alone has $\phi_c P_n = 807$ k and $P_n/\Omega_c = 537$ k.

Example 17-2

Determine the LRFD design strength $\phi_c P_n$ and the ASD allowable strength P_n/Ω_c of a concrete-filled 46 ksi HSS 12 × 12 × 1/2 section filled with 4 ksi concrete weighing 145 lb/ft^3. $(KL)_x = (KL)_y = 16$ ft.

Solution

Using an HSS $12 \times 12 \times \frac{1}{2}$ ($A_s = 20.9$ in^2, $I_x = I_y = 457$ in^4, $t = 0.465$ in)

$$\frac{b}{t} = \frac{12}{0.465} = 25.81 < 2.26\sqrt{\frac{E}{F_y}} = 2.26\sqrt{\frac{29 \times 10^3}{46}} = 56.7 \therefore \text{ Compact section}$$

$$P_{no} = P_p \qquad \text{(AISC Equation I2-9a)}$$

C_2 given in specification $= 0.85$ (for rectangular section)

$$A_c = (12)(12) - (20.9) = 123.1 \text{ in}^2$$

$$P_p = A_s F_y + C_2 f'_c \left[A_c + A_{sr}\left(\frac{E_s}{E_c}\right)\right] \qquad \text{(AISC Equation I2-9b)}$$

$$= (20.9)(46) + 0.85(4)\,[123.1 + 0] = 1380 \text{ k} = P_p = P_{no}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \qquad \text{(AISC Equation I2-13)}$$

$$= 0.6 + 2\left(\frac{20.9}{(12 \times 12) + 20.9}\right) = 0.85 < 0.9 \qquad \textbf{OK}$$

$$E_c = w_c^{1.5}\sqrt{f'_c} = (145)^{1.5}\sqrt{4} = 3.492 \times 10^3 \text{ ksi}$$

$$I_c = \left(\frac{1}{12}\right)(12)(12)^3 - 457 = 1271 \text{ in}^4$$

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \qquad \text{(AISC Equation I2-12)}$$

$$= (29 \times 10^3)(457) + 0 + (0.85)(3.492 \times 10^3)(1271)$$

$$= 17.026 \times 10^6 \text{ k-in}^2$$

$$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} \qquad \text{(AISC Equation I2-5)}$$

$$= \frac{(\pi^2)(17.026 \times 10^6)}{(12 \times 16)^2} = 4558 \text{ k}$$

$$\frac{P_{no}}{P_e} = \frac{1380}{4558} = 0.30 \leq 2.25$$

$\therefore$ Use AISC Equation I2-2.

$$P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] = 1380\left[0.658^{\frac{1380}{4558}}\right] = 1216 \text{ k}$$

LRFD $\phi_c = 0.75$	ASD $\Omega_c = 2.00$
$\phi_c P_n = (0.75)(1216) = 912$ k	$\dfrac{P_n}{\Omega_c} = \dfrac{1216}{2.00} = 608$ k

From AISC Table 4-15, the values are $\phi_c P_n = 911$ k and $P_n/\Omega_c = 607$ k. In computing the properties of the column, the author did not account for the fillets on the inside corners of the HSS. For this reason, his results vary a little from those given in the AISC tables.

17.7 SHEAR STRENGTH OF COMPOSITE COLUMNS

Section I4 of the AISC Specification states that the shear strength of composite columns may be calculated based on one of the following:

1. available shear strength of the steel section alone per AISC Spec. Chapter G.
2. available shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone per ACI 318 with $\phi_v = 0.75$ (LRFD) or $\Omega_v = 2.00$ (ASD).
3. nominal shear strength of the steel section per AISC Spec. Chapter G plus the nominal strength of the reinforcing steel per ACI 318 with $\phi_v = 0.75$ (LRFD) or $\Omega_v = 2.00$ (ASD).

The shear strength for method 2 is determined using ACI 318 Chapter 11 equation:

$$V_n = V_c + V_s$$

where $V_c = 2\sqrt{f'_c}\, bd$

$$V_s = A_{st} F_{yt} \frac{d}{s}$$

The shear strength for method 3 is determined using the following expression:

$$V_n = 0.6 F_y A_w + A_{st} F_{yt} \frac{d}{s}$$

where A_w represents the steel section area. For square and rectangular HSS sections and for box sections, it equals $2ht$ (AISC Specification G5), where h is the clear distance between the member flanges less the inside corner radius on each side. Should this radius not be available, the designer may assume that h equals the outside dimension minus three times the flange thickness t.

Example 17-3 presents the calculation of the shear strength of an HSS section filled with concrete.

Example 17-3

It is assumed that the HSS 12 × 12 × 1/2 column of Example 17-2 is filled with 4 ksi concrete and is subjected to the end shear forces $V_D = 50$ k and $V_L = 100$ k. Does the member possess sufficient strength to resist these forces if $F_y = 46$ ksi?

Solution

Calculating required shear strength using method 1.

LRFD	ASD
$V_u = (1.2)(50) + (1.6)(100) = 220$ k	$V_a = 50 + 100 = 150$ k

Using an HSS 12 × 12 × $\frac{1}{2}$ (d = 12.00 in, t_w = 0.465 in)

$$h = d - 3t = 12.00 - (3)(0.465) = 10.605 \text{ in}$$

$$A_w = 2ht = (2)(10.605)(0.465) = 9.86 \text{ in}^2$$

$$V_n = 0.6F_y A_w = (0.6)(46)(9.86) = 272 \text{ k}$$

LRFD $\phi_v = 0.90$	ASD $\Omega_v = 1.67$
$\phi_v V_n = (0.90)(272) = 244.8$ k > 220 k **OK**	$\frac{V_n}{\Omega_v} = \frac{272}{1.67} = 162.9$ k > 150 k **OK**

17.8 LRFD AND ASD TABLES

In Part 4 of the Manual, a series of tables is presented for HSS sections and steel pipe sections filled with concrete. These tables, numbered 4-13 to 4-20, are set up in exactly the same fashion as the tables for axially loaded plain steel columns, which are also presented in Section 4 of the Manual. The axial strengths are given with respect to the minor axis for a range of $(KL)_y$ values.

Included are values for composite HSS square and rectangular sections ($F_y = 46$ ksi), for round HSS sections ($F_y = 42$ ksi), and for steel pipe sections ($F_y = 35$ ksi). The tables cover steel sections filled with 4 and 5 ksi concretes. For other grades of concrete, and for steel shapes made composite with encasing concrete, the formulas presented earlier in this chapter may be used to determine $\phi_c P_n$ values.

Examples 17-4 and 17-5 show how the tables can be used to directly determine design strengths for square and rectangular composite HSS sections.

Example 17-4

Determine the LRFD design strength and the ASD allowable strength of a 46 ksi HSS 10 × 10 × 3/8 filled with 4 ksi concrete if $(KL)_x = (KL)_y = 15$ ft.

Solution. From Table 4-15 in the Manual, for $(KL)_y = 15$ ft.

LRFD	ASD
$\phi_c P_n = 575$ k	$\dfrac{P_n}{\Omega_c} = 383$ k

Example 17-5

Determine $\phi_c P_n$ and $\dfrac{P_n}{\Omega_c}$ for a concrete-filled HSS 20 × 12 × 5/8 ($F_y = 46$ ksi) if $f'_c = 5$ ksi, $(KL)_x = 24$ ft, and $(KL)_y = 12$ ft.

Solution. From the AISC Manual, Table 4-14, we obtain $\dfrac{r_{mx}}{r_{my}} = 1.54$. Then, determining the controlling unbraced length yields

$$(KL)_y = 12 \text{ ft}$$

$$(KL)_{y\,\text{EQUIV}} = \frac{(KL)_x}{r_{mx}/r_{my}} = \frac{24}{1.54} = 15.58 \text{ ft} > 12 \text{ ft}$$

$$\therefore (KL)_y = 15.58 \text{ ft controls}$$

By interpolation, values to follow are found.

LRFD	ASD
$\phi_c P_n = 1652.6$ k	$\dfrac{P_n}{\Omega_c} = 1098.4$ k

17.9 LOAD TRANSFER AT FOOTINGS AND OTHER CONNECTIONS

A small steel base plate usually is provided at the base of a composite column. Its purpose is to accommodate the anchor bolts needed to anchor the embedded steel shape to the footing for the loads occurring during the erection of the structure before the encasing concrete hardens and composite action is developed. This plate should be sufficiently small to be out of the way of the dowels needed for the reinforced-concrete part of the column.[8]

The AISC Specification does not provide details for the design of these dowels, but a procedure similar to the one provided by Section 10.14 of the ACI 318 Code seems to be a good one. If the column P_u exceeds $\phi_c(0.85 f_c' A_1)(\sqrt{A_2/A_1})$ the excess load should be resisted by dowels. If P_u does not exceed the equation above, it would appear that no dowels are needed. For such a situation, the ACI Code (Section 15.8.2.1) states that a minimum area of dowels equal to 0.005 times the cross-sectional area of the column must be used, and those dowels may not be larger than No. 11 bars. This diameter requirement ensures sufficient tying together of the column and footing over the whole contact area. The use of a very few large dowels spaced far apart might not do this very well.

17.10 TENSILE STRENGTH OF COMPOSITE COLUMNS

The tensile design strength, or the allowable design, strength of composite sections may be needed when uplift forces are present and perhaps for some beam column interaction situations. The AISC Specification (I2.1c and I2.2c) provides the nominal tensile strength for such situations by the following expression, for which $\phi_t = 0.90$ and $\Omega_t = 1.67$:

$$P_n = A_s F_y + A_{sr} F_{ysr} \qquad \text{(AISC Equations I2-8 and I2-14)}$$

17.11 AXIAL LOAD AND BENDING

To determine the required strengths for composite columns subject to both axial load and bending, it is necessary (as for steel beam columns) to include second-order effects in the analysis. The AISC Specification does not provide specific equations to assess the available strength of such members. Section I5 of the Specification does provide information with which interaction curves for the forces may be constructed, much as they are done in reinforced concrete design. Furthermore, a suggested procedure for doing this is presented in Section I5 of the AISC Commentary, and a numerical example is provided on the CD enclosed with the Manual.

17.12 PROBLEMS FOR SOLUTION

For all problems, use 145 lb/ft^3 *concrete and 50 ksi steel shapes.*

17-1 to 17-3. *Using the AISC equations, compute* $\phi_c P_n$ *and* P_n/Ω_c *for each encased concrete* (5 ksi) *section shown.* $F_y = 50$ ksi, $F_{yr} = 60$ ksi, $w_c = 145$ lbs/ft^3.

[8]Griffis, op. cit.

17-1.

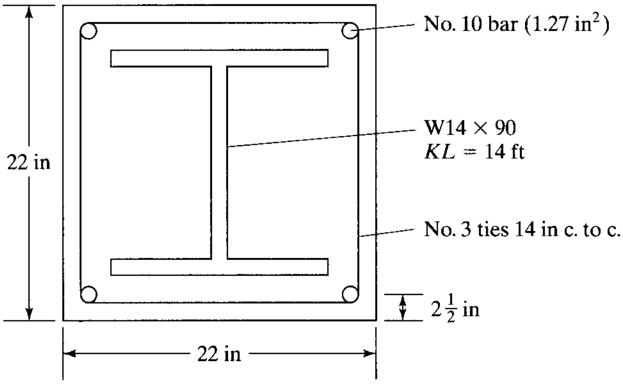


FIGURE P17-1 (*Ans.* 2343 k, 1562 k)

17-2.

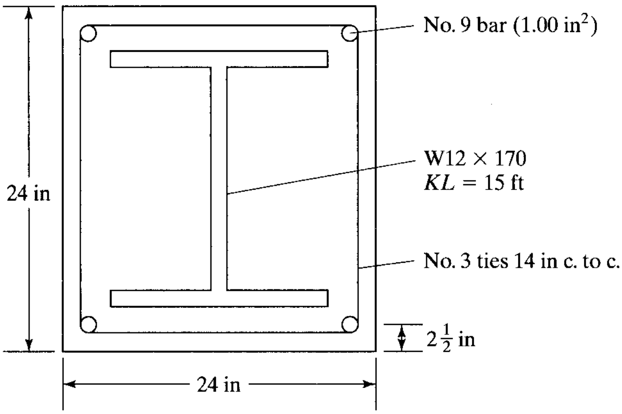


FIGURE P17-2

17-3.

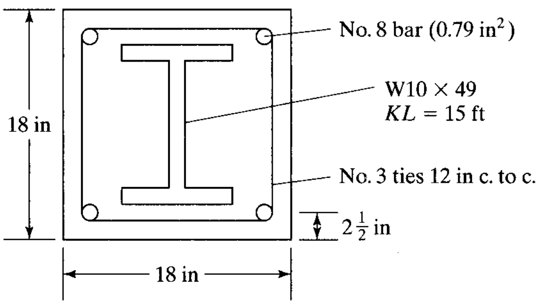


FIGURE P17-3 (*Ans.* 1262.6 k, 841.7 k)

17-4 to 17-6. *Using the appropriate AISC equations, determine $\phi_c P_n$ and P_n/Ω_c for each of the given sections, which are filled with* 145 lb/ft^3 *concrete.*

17-4. An HSS 14 × 14 × 1/2, $F_y = 46$ ksi, $f_c' = 4$ ksi, and $(KL)_x = (KL)_y = 14$ ft.

17-5. An HSS 14 × 10 × 1/2, $F_y = 46$ ksi, $f_c' = 4$ ksi, and $(KL)_x = (KL)_y = 12$ ft. (*Ans.* 930.4 k, 620.2 k)

17-6. A pipe 12 std, $F_y = 35$ ksi, $f_c' = 5$ ksi, and $(KL)_x = (KL)_y = 15$ ft.

17-7. Repeat the following problems, using the tables in Part 4 of the AISC Manual:

(a) Problem 17-4 (*Ans.* 1200 k, 797 k)
(b) Problem 17-5 (*Ans.* 928 k, 619 k)
(c) Problem 17-6 (*Ans.* 671 k, 448 k)

17-8. Select the lightest available concrete-filled round HSS column to support $P_D = 80$ k and $P_L = 120$ k. $F_y = 42$ ksi. $f_c' = 4$ ksi. $KL = 16$ ft.

CHAPTER 18

Cover-Plated Beams and Built-up Girders

18.1 COVER-PLATED BEAMS

Should the largest available W section be insufficient to support the loads anticipated for a certain span, several possible alternatives may be taken. Perhaps the most economical solution involves the use of a higher-strength steel W section. If this is not feasible, we may make use of one of the following: (1) two or more regular W sections side-by-side (an expensive solution), (2) a cover-plated beam, (3) a built-up girder, or (4) a steel truss. This section discusses the cover-plated beams, while the remainder of the chapter is concerned with built-up girders.

In addition to being practical for cases in which the moments to be resisted are slightly in excess of those that can be supported by the deepest W sections, there are other useful applications for cover-plated beams. On some occasions, the total depth may be so limited that the resisting moments of W sections of the specified depth are too small. For instance, the architect may show a certain maximum depth for beams in his or her drawings for a building. In a bridge, beam depths may be limited by clearance requirements. Cover-plated beams frequently will be a very satisfactory solution for situations like these. Furthermore, there may be economical uses for cover-plated beams where the depth is not limited and where there are standard W sections available to support the loads. A smaller W section than required by the maximum moment can be selected and have cover plates attached to its flanges. These plates can be cut off where the moments are smaller, with resulting saving of steel. Applications of this type are quite common for continuous beams.

Should the depth be fixed and a cover-plated beam seem to be a feasible solution, the usual procedure will be to select a standard section with a depth that leaves room for top and bottom cover plates. Then the cover plate sizes can be selected.

For this discussion, reference is made to Fig. 18.1. For the derivation to follow, Z is the plastic modulus for the entire built-up section, Z_W is the plastic modulus for the

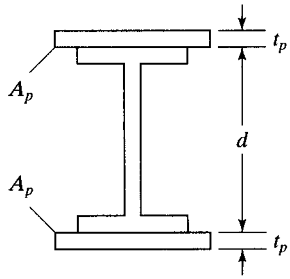


FIGURE 18.1

W section, d is the depth of the W section, t_p is the thickness of one cover plate, and A_p is the area of one cover plate.

The expressions to follow are written for LRFD design. Similar expressions could be developed for ASD. In Example 18-1, the author selects a cover-plated beam and then computes its LRFD design strength and its ASD allowable strength.

An expression for the required area of one flange cover plate can be developed as follows:

$$Z_{\text{reqd}} = \frac{M_u}{\phi_b F_y}$$

The total Z of the built-up section must at least equal the Z required. It will be furnished by the W shape and the cover plates as follows:

$$Z_{\text{reqd}} = Z_W + Z_{\text{plates}}$$

$$= Z_W + 2A_p\left(\frac{d}{2} + \frac{t_p}{2}\right)$$

$$A_p = \frac{Z_{\text{reqd}} - Z_W}{d + t_p}$$

Example 18-1 illustrates the design of a cover-plated beam. Quite a few satisfactory solutions involving different W sections and varying-size cover plates are available, other than the one made in this problem.

Example 18-1

Select a beam limited to a maximum depth of 29.50 in for the loads and span of Fig. 18.2. A 50 ksi steel is used, and the beam is assumed to have full lateral bracing for its compression flange.

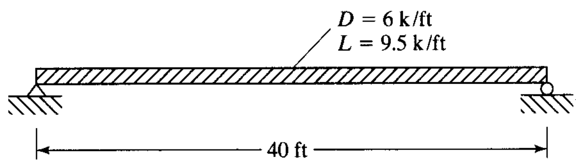


FIGURE 18.2

Solution

Assume beam wt = 350 lb/ft

LRFD	ASD
$w_u = (1.2)(6.35) + (1.6)(9.5) = 22.82$ k/ft	$w_a = 6.35 + 9.5 = 15.85$ k/ft
$M_u = \dfrac{(22.82)(40)^2}{8} = 4564$ ft-k	$M_a = \dfrac{(15.85)(40)^2}{8} = 3170$ ft-k

$$Z_{\text{reqd}} = \frac{(12)(4564)}{(0.9)(50)} = 1217 \text{ in}^3$$

The only W sections listed in the Manual with depths $\overline{<}$ 29.50 in and Z values $\overline{>}$ 1217 in^3 are the impractical, extremely heavy and expensive, W14 × 605, W14 × 665, and W14 × 730. As a result, the author decided to use a lighter W section with cover plates. He assumes the plates are each 1 in thick.

Try W27 × 146 (d = 27.4 in, Z_x = 464 in^3, b_f = 14.0 in)

Total depth = 27.4 + (2)(1.00) = 29.4 in < 29.5 in **OK**

Area of 1 cover plate for each flange

$$A_p = \frac{Z_{\text{reqd}} - Z_w}{d + t_p} = \frac{1217 - 464}{27.4 + 1.00} = 26.51 \text{ in}^2$$

Try a 1 × 28 in **cover plate each flange**

$$Z_{\text{furnished}} = 464 + (1)(28)(2)\left(\frac{27.4}{2} + 0.5\right)$$

$$= 1259.2 \text{ in}^3 > 1217 \text{ in}^3 \quad \textbf{OK}$$

$$M_n = \frac{F_y Z}{12} = \frac{(50)(1259.2)}{12} = 5246.7 \text{ ft-k}$$

LRFD $\phi_b = 0.9$	ASD $\Omega_b = 1.67$
$\phi_b M_n = (0.9)(5246.7) = 4722$ ft-k > 4564 ft-k	$\dfrac{M_n}{\Omega_b} = \dfrac{5246.7}{1.67} = 3142$ ft-k < 3170 ft-k
OK	**Not quite**

A check of the b/t ratios for the plates, web, and flanges show them to be satisfactory.

Use W27 × 146 with one 1 × 28 in each flange for LRFD (slightly larger plate needed for ASD).

$$\text{Wt of steel for LRFD design} = 146 + \left(\frac{2 \times 1 \times 28}{144}\right)(490) = 337 \text{ lb/ft} < \text{estimated}$$

350 lb/ft **OK**

18.2 BUILT-UP GIRDERS

Built-up I-shaped girders, frequently called plate girders, are made up with plates and perhaps with rolled sections. They usually have design moment strengths somewhere between those of rolled beams and steel trusses. Several possible arrangements are shown in Fig. 18.3. Rather obsolete bolted girders are shown in parts (a) and (b) of the figure, while several welded types are shown in parts (c) through (f). *Since nearly all built-up girders constructed today are welded (although they may make use of bolted field splices), this chapter is devoted almost exclusively to welded girders.*

The welded girder of part (d) of Fig. 18.3 is arranged to reduce overhead welding, compared with the girder of part (c), but in so doing may be creating a slightly worse corrosion situation if the girder is exposed to the weather. A box girder, illustrated in part (g), occasionally is used where moments are large and depths are quite limited. Box girders also have great resistance to torsion and lateral buckling. In addition, they make very efficient curved members because of their high torsional strengths.

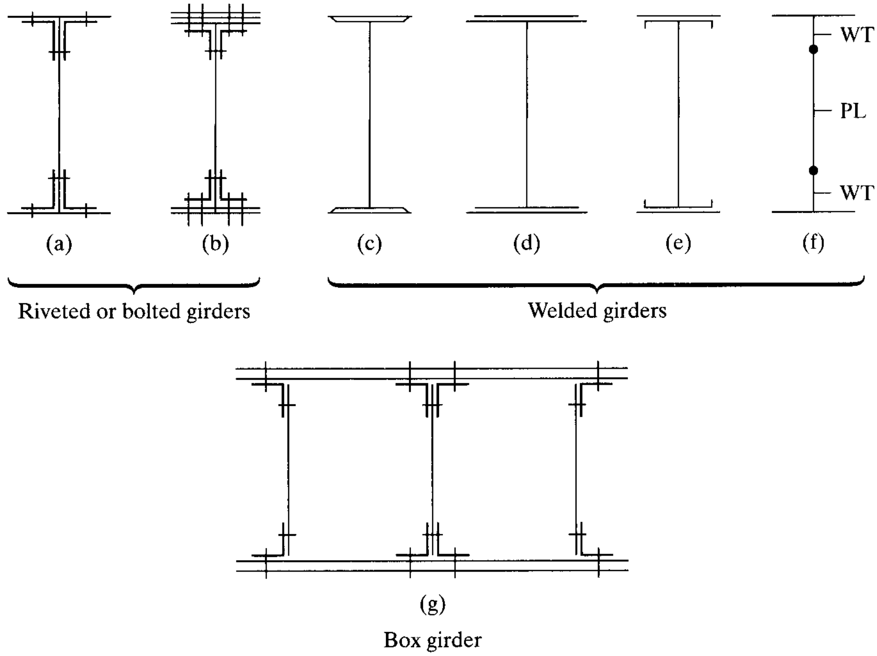


FIGURE 18.3

Built-up girders.

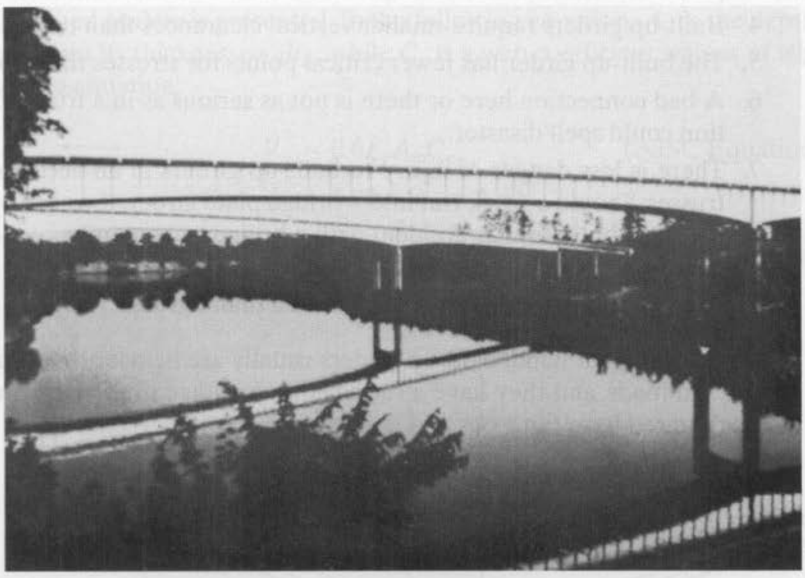
Plates and shapes can be arranged to form built-up girders of almost any reasonable proportions. This fact may seem to give them a great advantage for all situations, but for the smaller sizes the advantage usually is canceled out by the higher fabrication costs. For example, it is possible to replace a W36 with a built-up girder roughly twice as deep that will require considerably less steel and will have much smaller deflections; however, the higher fabrication costs will almost always rule out such a possibility.

Most steel highway bridges built today for spans of less than about 80 ft are steel-beam bridges. For longer spans, the built-up girder begins to compete very well economically. Where loads are extremely large, such as for railroad bridges, built-up girders are competitive for spans as small as 45 or 50 ft.

The upper economical limits of built-up girder spans depend on several factors, including whether the bridge is simple or continuous, whether a highway or railroad bridge is involved, and what is the largest section that can be shipped in one piece.

Generally speaking, built-up girders are very economical for railroad bridges in spans of 50 to 130 ft (15 to 40 m), and for highway bridges in spans of 80 to 150 ft (24 to 46 m). However, they are often very competitive for much longer spans, particularly when continuous. In fact, they are actually common for 200-ft (61-m) spans and have been used for many spans in excess of 400 ft (122 m). The main span of the continuous Bonn-Beuel built-up girder bridge over the Rhine River in Germany is 643 ft.

Built-up girders are not only used for bridges. They also are fairly common in various types of buildings to support heavy concentrated loads. Frequently, a large ballroom or dining room with no interfering columns is desired on a lower floor of a multistory building. Such a situation is shown in Fig. 18.4. The girder shown must support some tremendous column loads for many stories above. The usual building girder



Buffalo Bayou Bridge, Houston, TX—a 270-ft span. (Courtesy of the Lincoln Electric Company.)

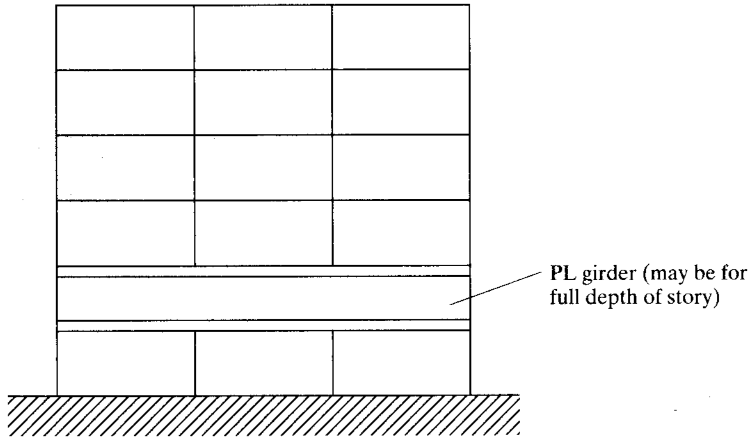


FIGURE 18.4

of this type is simple to analyze because it probably does not have moving loads, although some building girders may be called upon to support traveling cranes.

The usual practical alternative to built-up girders in the spans for which they are economical is the truss. In general, plate girders have the following advantages, particularly compared with trusses:

1. The pound price for fabrication is lower than for trusses, but it is higher than for rolled beam sections.
2. Erection is cheaper and faster than for trusses.
3. Due to the compactness of built-up girders, vibration and impact are not serious problems.
4. Built-up girders require smaller vertical clearances than trusses.
5. The built-up girder has fewer critical points for stresses than do trusses.
6. A bad connection here or there is not as serious as in a truss, where such a situation could spell disaster.
7. There is less danger of injury to built-up girders in an accident, compared with trusses. Should a truck run into a bridge plate girder, it would probably just bend it a little, but a similar accident with a bridge truss member could cause a broken member and, perhaps, failure.
8. A built-up girder is more easily painted than a truss.

On the other hand, built-up girders usually are heavier than trusses for the same spans and loads, and they have a further disadvantage in the large number of connections required between webs and flanges.

18.3 BUILT-UP GIRDER PROPORTIONS

18.3.1 Depth

The depths of built-up girders vary from about 1/6 to 1/15 of their spans, with average values of 1/10 to 1/12, depending on the particular conditions of the job. One condition that may limit the proportions of the girder is the largest size that can be fabricated in the shop and shipped to the job. There may be a transportation problem such as clearance requirements that limit maximum depths to 10 or 12 ft along the shipping route.

Shallow girders probably will be used when loads are light, and the deeper ones when very large concentrated loads need to be supported, as from the columns in a tall building. If there are no depth restrictions for a particular girder, it will probably pay for the designer to make rough designs and corresponding cost estimates to arrive at a depth decision. (Computer solutions will be very helpful in preparing these alternative designs.)

18.3.2 Web Size

After the total girder depth is estimated, the general proportions of the girder can be established from the maximum shear and the maximum moment. As previously described for I-shaped sections in Section 10.2, the web of a beam carries nearly all of the shearing stress; this shearing stress is assumed by the AISC Specification to be uniformly distributed throughout the web. The web depth can be closely estimated by taking the total girder depth and subtracting a reasonable value for the depths of the flanges (roughly 1 to 2 in each). The web depths usually are selected to the nearest even inch, because these plates are not stocked in fractional dimensions.

As a plate girder bends, its curvature creates vertical compression in the web, as illustrated in Fig. 18.5. This is due to the downward vertical component of the compression flange bending stress and the upward vertical component of the tension flange bending stress.

The web must have sufficient vertical buckling strength to withstand the squeezing effect shown in Fig. 18.5. This problem is handled in Section G of the AISC Specification. There, the nominal shearing strength of the webs of stiffened or unstiffened built-up I-shaped girders is presented. In the following equation, A_w is the depth of the girder web times its thickness $= dt_w$, while C_v is a web coefficient, values of which are given after the equation:

$$V_n = 0.6F_yA_wC_v \qquad \text{(AISC Equation G2-1)}$$

$$\phi_v = 0.90 \quad \Omega_v = 1.67$$

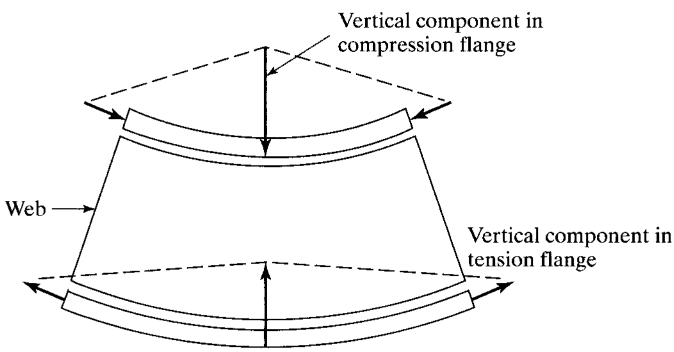


FIGURE 18.5

Squeezing of plate girder web.

Values of C_v are as follows:

1. For $h/t_w \leq 1.10\sqrt{k_v\, {}^{E}\!/_{F_y}}$

$$C_v = 1.0 \qquad \text{(AISC Equation G2-3)}$$

2. For $1.10\sqrt{k_v\, {}^{E}\!/_{F_y}} < \dfrac{h}{t_w} \leq 1.37\sqrt{k_v\, {}^{E}\!/_{F_y}}$

$$C_v = \frac{1.10\sqrt{k_v\, {}^{E}\!/_{F_y}}}{{}^{h}\!/_{t_w}} \qquad \text{(AISC Equation G2-4)}$$

3. For ${}^{h}\!/_{t_w} > 1.37\sqrt{k_v\, {}^{E}\!/_{F_y}}$

$$C_v = \frac{1.51Ek_v}{({}^{h}\!/_{t_w})^2 F_y} \qquad \text{(AISC Equation G2-5)}$$

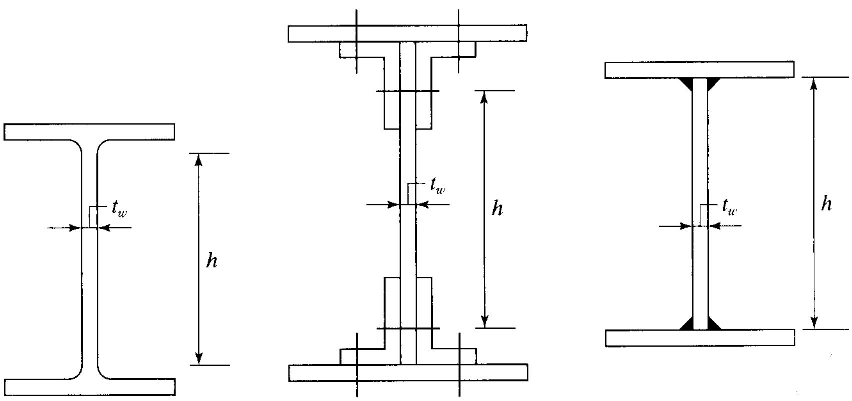
In the preceding C_v expressions, h is equal to (a) the clear distance between flanges less two times the fillet or corner radius for rolled shapes, (b) the distance between adjacent lines of fasteners for built-up sections, or (c) the clear distance between flanges for built-up sections when welds are used. These values are illustrated in Fig. 18.6.

The term k_v is a web plate-buckling coefficient $= 5 + \dfrac{5}{({}^{a}\!/_{h})^2}$ except that it is to be 5.0 when $a/h > 3.0$ or $> \left[\dfrac{260}{({}^{h}\!/_{t_w})}\right]^2$.

From a corrosion standpoint, the usual practice is to use some absolute minimum web thickness. For bridge girders, 3/8 in is a common minimum, while 1/4 or 5/16 in are the minimum values used for the more sheltered building girders.

18.3.3 Flange Size

After the web dimensions are selected, the next step is to select an area of the flange so that it will not be overloaded in bending. The total bending strength of a plate girder



(a) Rolled section

(b) Bolted built-up section (seldom used today)

(c) Welded built-up section

FIGURE 18.6

equals the bending strength of the flange plus the bending strength of the web. As almost all of the bending strength is provided by the flange, an approximate expression can be developed to estimate the flange area as follows:

$$Z_{\text{reqd}} = \frac{M_u}{\phi_b F_y}$$

$$Z_{\text{furnished}} = 2A_f\left(\frac{h + t_f}{2}\right) + (2)\left(\frac{h}{2}\right)(t_w)\left(\frac{h}{4}\right)$$

Equating $Z_{\text{reqd}} = Z_{\text{furnished}}$ and solving for A_f

$$\frac{M_u}{\phi_b F_y} = A_f(h + t_f) + (2)\left(\frac{h}{2}\right)(t_w)\left(\frac{h}{4}\right)$$

$$A_f = \frac{M_u}{\phi_b F_y(h + t_f)} - \frac{t_w h^2}{4(h + t_f)}$$

In Example 18-2, the web and flanges are proportioned for a built-up I-shaped girder such that transverse stiffeners are not required for the web.

Example 18-2

Select trial proportions for a 60-in deep welded built-up I-shaped section with a 70 ft simple span to support a service dead load (not including the beam weight) of 1.1 k/ft and a service live load of 3 k/ft. The A36 section will be assumed to have full lateral bracing for its compression flange, and an unstiffened web is to be used.

Solution

Trial proportions

Try 60 in depth $\approx l/14$

$$\text{Estimated beam weight: 2 Flanges} = 2(1.0)(15) = 30 \text{ in}^2$$

$$\text{Web} = (0.75)(58) = 43.5 \text{ in}^2$$

$$A \text{ total} = 73.5 \text{ in}^2$$

$$\text{wt (plf)} = \frac{73.5 \text{ in}^2}{144 \text{ in}^2/\text{ft}^2}\left(490 \frac{\text{lbs}}{\text{ft}^3}\right) = 250.1 \text{ plf}$$

Assume beam wt $= 250$ lbs/ft

Maximum moment and shear

LRFD	ASD
$w_u = (1.2)(1.1 + 0.250) + (1.6)(3) = 6.42$ k/ft	$w_a = 1.1 + 0.250 + 3 = 4.35$ k/ft
$R_u = \left(\frac{70}{2}\right)(6.42) = 224.7$ k	$R_a = \left(\frac{70}{2}\right)(4.35) = 152.2$ k
$M_u = \frac{(6.42)(70)^2}{8} = 3932$ ft-k	$M_a = \frac{(4.35)(70)^2}{8} = 2664$ ft-k

Design for a *compact web* and *flange*

$$Z_{\text{reqd}} = \frac{M_u}{\phi F_y} = \frac{(12)(3932)}{(0.9)(36)} = 1456 \text{ in}^3$$

Trial web size

For web to be compact by AISC Table B4.1 $\frac{h}{t_w}$ must be $\leq 3.76\sqrt{\frac{E}{F_y}}$

$$= 3.76\sqrt{\frac{29 \times 10^3}{36}} = 106.7 \qquad \text{(Case 15 AISC Table B4.1b)}$$

Assuming h to be 60 in $- 2(1.0 \text{ in}) = 58$ in

$$Min\, t_w = \frac{58}{106.7} = 0.544 \text{ in, Say, } \frac{9}{16} \text{ in } (0.563 \text{ in})$$

Try $\frac{9}{16} \times 58$ **web**

$$\frac{h}{t_w} = \frac{58}{\frac{9}{16}} = 103.1$$

$$\text{Since } 103.1 \text{ is } > 2.46\sqrt{\frac{E}{F_y}} = 2.46\sqrt{\frac{29 \times 10^3}{36}} = 69.82$$

transverse stiffeners may be required, states AISC Specification G2.2.

But the same specification states that stiffeners are not required if the necessary shear strength for the web is less than or equal to its available shear strength, as stipulated by AISC Specification G2.1, using $k_v = 5.0$.

$$\frac{h}{t_w} = 103.1 > 1.37\sqrt{\frac{k_v E}{F_y}} = 1.37\sqrt{\frac{(5)(29 \times 10^3)}{36}}$$

$$= 86.94$$

$$\therefore C_v = \frac{1.51Ek_v}{\left(\frac{h}{t_w}\right)^2 F_y} = \frac{(1.51)(29 \times 10^3)(5.0)}{(103.1)^2(36)} = 0.572$$

$$V_n = 0.6F_yA_wC_v = (0.6)(36)\left(58 \times \frac{9}{16}\right)(0.572)$$
$$= 403.1 \text{ k}$$

Available shear strength without stiffeners

LRFD $\phi_v = 0.90$	ASD $\Omega_v = 1.67$
$\phi_v V_n = (0.90)(403.1) = 362.8 \text{ k}$	$\frac{V_n}{\Omega_v} = \frac{403.1}{1.67} = 241.4 \text{ k}$
$> 224.7 \text{ k}$	$> 152.2 \text{ k}$
$\therefore$ **Stiffeners are not required.**	$\therefore$ **Stiffeners are not required.**

Trial flange size

Assume 1-in plates ($t_f = 1.0$ in)

$$A_f = \frac{M_u}{\phi_b F_y(h + t_f)} - \frac{t_w h^2}{4(h + t_f)}$$

$$= \frac{(12)(3932)}{(0.9)(36)(58 + 1)} - \frac{\left(\frac{9}{16}\right)(58)^2}{4(58 + 1)} = 16.66 \text{ in}^2$$

Try 1 × 18 plate each flange. Are they compact by AISC Table B4.1b (Case 11)?

$$\frac{b_f}{2t_f} = \frac{18.00}{(2)(1.00)} = 9.00 < 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.79 \text{ (Yes, is compact)}$$

Check Z of section

$$Z = (2)\left(\frac{58}{2}\right)\left(\frac{9}{16}\right)\left(\frac{58}{4}\right) + (2)(1 \times 18)\left(\frac{58}{2} + \frac{1}{2}\right)$$
$$= 1535 \text{ in}^3 > 1456 \text{ in}^3 \quad \textbf{OK}$$

Check girder wt

$$\text{wt} = \frac{\left(\frac{9}{16}\right)(58) + (2)(1 \times 18)}{144}(490) = 233.5 \text{ lb} < 250 \text{ lb estimated} \quad \textbf{OK}$$

Trial Section $\frac{9}{16} \times 58$ web with 1×18 PL each flange. (See Fig. 18.7.)

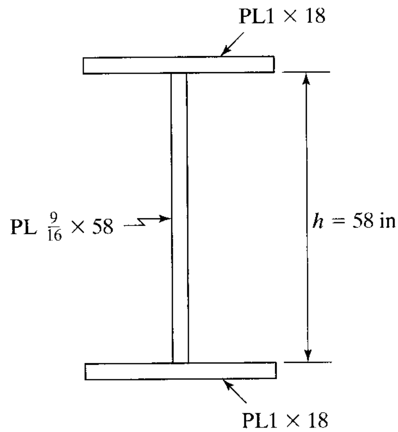


FIGURE 18.7

Trial proportions for girder of Example 18.2.

18.4 FLEXURAL STRENGTH

The *nominal* flexural strength, M_n, of a plate girder bent about its major axis is based on one of the limit states as defined in Chapter F of the AISC Specification, Section F2 to F5. These limit states include yielding (Y), lateral-torsional buckling (LTB), compression flange local buckling (FLB), compression flange yielding (CFY), and tension flange yielding (TFY). This strength, M_n, is the lowest value obtained according to these limit states. The application of the limit states defined in F2 to F5 are based on whether plate girder has compact, non-compact, or slender flanges and webs as defined in Specification Section B4.1 for flexure and the unbraced length of the compression flange, L_b. Table F1.1 in the specification summarizes the application of the Chapter F sections.

In Example 18-2, the doubly symmetrical I-shaped plate girder was proportioned so that both the flanges and web were compact. With this condition, Section F2 was applicable and the limit states of yielding (Y) and lateral-torsional buckling (LTB) would need to be checked to determine M_n. In the example, the member was assumed to have full lateral bracing for its compression flange. Therefore, $L_b = 0$ and the limit state of lateral-torsional buckling does not apply. The yielding limit state was used to determine the *nominal* flexural strength.

Section F3 applies to doubly symmetrical I-shaped members having compact webs and non-compact or slender flanges. The *nominal* flexural strength, M_n, shall be the lower value obtained from the limit states of LTB and FLB. Section F4 applies to doubly symmetrical I-shaped members with non-compact webs and singly symmetrical I-shaped members with compact or non-compact webs. The *nominal* flexural strength, M_n, shall be the lowest value obtained from the limit states of CFY, LTB, FLB and TFY. Section F5 applies to doubly symmetric and singly symmetric I-shaped members with slender webs. The *nominal* flexural strength, M_n, shall be the lowest value obtained from the limit states of CFY, LTB, FLB and TFY.

The *design* flexural strength, $\Phi_b M_n$, and the *allowable* flexural strength, M_n/Ω_b, shall be determined using $\Phi_b = 0.90$ (LRFD) and $\Omega_b = 1.67$ (ASD).

Example 18-3

Determine the *design* flexural strength, $\Phi_b M_n$, and the *allowable* flexural strength, M_n/Ω_b, of the following welded I-shaped plate girder. The flanges are 1 1/4 in $\times$ 15 in, the web is 1/4 in $\times$ 50 in, and the member is uniformly loaded and simply-supported. Use A36 steel and assume the girder has continuous bracing for its compression flange.

Solution

Determine if flange is compact, non-compact, or slender? *Case 11.* Table B4.1b

$$\frac{b}{t_f} = \frac{b_f/2}{t_f} = \frac{15/2}{1.25} = 6.0 < 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.79$$

Compact—Flange

Determine if web is compact, non-compact, or slender? *Case 15.* Table B4.1b

$$\frac{h}{t_w} = \frac{50}{0.25} = 200 > 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 161.78$$

Slender—Web

$\therefore$ (F5) Doubly symmetric section with *slender web* and *compact flange* bent about their major axis.

$\phi_b M_n$ is lowest value of Y, LTB, FLB, TFY

$\boxed{\text{LTB}}$ – Since $L_b = 0$, limit state of LTB does not apply.

$\boxed{\text{FLB}}$ – Since flange is compact, limit state of FLB does not apply

$\boxed{\text{TFY}}$ – Since member is symmetric about x–x axis $S_{xt} = S_{xc}$, limit state of TFY does not apply.

$\boxed{\text{Y}}$ – Compression flange yielding.

$$M_n = R_{pg} F_y S_{xc} \qquad \text{(AISC Equation F5-1)}$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{50(1/4)}{15(1.25)} \qquad \text{(AISC Equation F4-12)}$$

$$a_w = 0.667 < 10 \text{ (upper limit)}$$

$$R_{pg} = 1 - \frac{a_w}{1200 + 300\,a_w}\left[\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right] \leq 1.0 \qquad \text{(AISC Equation F5-6)}$$

$$R_{pg} = 1 - \frac{0.667}{1200 + 300(0.667)}\left[200 - 5.7\sqrt{\frac{29{,}000}{36}}\right] \leq 1.0$$

$$R_{pg} = 0.982$$

$$F_y = 36 \text{ ksi}$$

$$I_{xc} = \frac{1}{12}\left(\frac{1}{4}\right)(50)^3 + 2\left(\frac{1}{12}\right)(15)(1.25)^3 + 2(1.25)(15)(25.625)^2$$

$$I_{xc} = 27{,}233 \text{ in}^4$$

$$S_{xc} = \frac{I}{c} = \frac{27{,}233}{26.25} = 1037.4 \text{ in}^3$$

$$M_n = R_{pg}F_yS_{xc} = \frac{(0.982)(36 \text{ ksi})(1037.4 \text{ in}^3)}{12 \text{ in/ft}}$$

$$M_n = 3056 \text{ ft-k}$$

LRFD $\phi = 0.90$	ASD $\Omega = 1.67$
$\phi M_n = 0.9(3056)$ ft-k	$M_n/\Omega = 3056$ ft-k/1.67
$\phi M_n = 2750$ ft-k	$M_n/\Omega = 1830$ ft-k

Example 18-4

Determine the *design* flexural strength, $\Phi_b M_n$, and the *allowable* flexural strength, M_n/Ω_b, of the following welded I-shaped plate girder. The flanges are 1 in × 24 in, the web is 5/16 in × 45 in, and the member is uniformly loaded and has a simply-supported 100 ft. span. Use A36 steel and the unbraced length of the compression flange is 20 ft.

Solution

Determine if flange is compact, non-compact, or slender? *Case 11.* Table B4.1b

$$\frac{b}{t_f} = \frac{b_f/2}{t_f} = \frac{24/2}{1} = 12.00 > 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.79$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{45/0.3125}} = 0.333$$

$$\frac{b}{t_f} = 12.00 < 0.95\sqrt{\frac{k_cE}{F_y}} = 0.95\sqrt{\frac{0.333(29{,}000)}{36}} = 15.56$$

Non-Compact—Flange

Determine if web is compact, non-compact, or slender? *Case 15.* Table B4.1b

$$\frac{h}{t_w} = \frac{45}{0.3125} = 144 > 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000}{36}} = 106.72$$

$$< 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 161.78$$

Non-Compact—Web

∴ (F4) Doubly symmetric I-shaped members with *non-compact webs* bent about their major axis.

$\phi_b M_n$ is lowest value of Y, LTB, FLB, TFY

$\boxed{\text{TFY}}$ — Since member is symmetric about x-x axis, $S_{xt} = S_{xc}$, limit state of TFY does not apply.

$\boxed{\text{Y}}$ — Compression flange yielding

$$M_n = R_{pc} F_y S_{xc} \qquad \text{(AISC Equation F4-1)}$$

$$\text{Since } \frac{h}{t_w} = 144 \geq \lambda_{pw} = 106.72 = 3.76\sqrt{\frac{E}{F_y}}$$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yc}} \qquad \text{(AISC Equation F4-9b)}$$

$$\frac{M_p}{M_{yc}} = \frac{Z}{S}$$

where: $Z = 2(1)(24)(22.5 + 0.5) + 0.3125(2)(22.5)(11.25)$

$Z = 1262 \text{ in}^3$

$$S = \frac{2\left(\frac{1}{12}\right)(24)(1)^3 + \left(\frac{1}{12}\right)(0.3125)(45)^3 + 2(24)(1)(22.5 + 0.5)^2}{23.5}$$

$S = 1182 \text{ in}^3$

$$\frac{M_p}{M_{yc}} = \frac{Z}{S} = \frac{1262}{1182} = 1.068$$

$$R_{pc} = \left[1.068 - (1.068-1)\left(\frac{144 - 106.72}{161.78 - 106.72}\right)\right] \leq 1.068$$

$$R_{pc} = 1.022$$

$$M_n = \frac{(1.022)(36\text{ ksi})(1182\text{ in}^3)}{12\text{ in /ft}} = 3624\text{ ft-k} \quad \boxed{\text{Y}}$$

LTB — Lateral Torsional Buckling check unbraced length of 20 ft,

$L_b = 20$ ft or 240 in

$$L_p = 1.1\, r_t\sqrt{\frac{E}{F_y}} \qquad \text{(AISC Equation F4-7)}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_o}{d} + \frac{1}{6}a_w\frac{h^2}{h_o d}\right)}} \qquad \text{(AISC Equation F4-11)}$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{45(0.3125)}{24(1.0)} = 0.586 \qquad \text{(AISC Equation F4-12)}$$

$b_{fc} = 24$ in, $h_o = 46$ in, $d = 47$ in, $h = 45$ in

$$r_t = \frac{24}{\sqrt{12\left(\frac{46}{47} + \frac{1}{6}(0.586)\left(\frac{45^2}{46(47)}\right)\right)}} = 6.70\text{ in}$$

$$L_p = 1.1(6.70\text{ in})\sqrt{\frac{29{,}000}{36}} = 209.1\text{ in} = 17.42\text{ ft}$$

$$L_r = 1.95\, r_t\frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \qquad \text{(AISC Equation F4-8)}$$

Since $\frac{S_{xt}}{S_{xc}} = 1.0 \geq 0.7$

$\therefore F_L = 0.7\, F_y = 0.7\,(36) = 25.2$ ksi (AISC Equation F4-6a)

$$J = \sum\frac{1}{3}bt^3 = 2\left(\frac{1}{3}\right)(24)(1)^3 + \left(\frac{1}{3}\right)(45)(0.3125)^3 = 16.46\text{ in}^3$$

$$L_r = 1.95(6.70)\frac{29{,}000}{25.2}\sqrt{\frac{16.46}{1182(46)} + \sqrt{\left(\frac{16.46}{1182(46)}\right)^2 + 6.76\left(\frac{25.2}{29{,}000}\right)^2}}$$

$$L_r = 764.0\text{ in} = 63.67\text{ ft}$$

$$M_n = C_b\left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\left[\frac{L_b - L_p}{L_r - L_p}\right]\right] \leq R_{pc}M_{yc} \text{ (AISC Equation F4-2)}$$

$$R_{pc}M_{yc} = \frac{1.022(36)(1182)}{12} = 3624 \text{ ft-k}$$

$$M_n = 1.0\left[3624 - \left(3624 - \frac{25.2(1182)}{12}\right)\left[\frac{20 - 17.42}{63.67 - 17.42}\right]\right] \leq 3624$$

$$M_n = 3560 \text{ ft-k } \boxed{\text{LTB}}$$

$\boxed{\text{FLB}}$ – Flange Local Buckling

$$M_n = \left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\left[\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right]\right] \text{ (AISC Equation F4-12)}$$

For sections with non-compact flanges

$$\lambda = \frac{b_f/2}{t_f} = 12.00 \quad \lambda_{pf} = 10.79 = 0.38\sqrt{\frac{E}{F_y}}$$

$$\lambda_{rf} = 15.56 = 0.95\sqrt{\frac{kE}{F_y}}$$

$$M_n = \left[3624 - \left(3624 - \frac{25.2(1182)}{12}\right)\left(\frac{12.00 - 10.79}{15.56 - 10.79}\right)\right]$$

$$M_n = 3334 \text{ ft-k } \boxed{\text{FLB}}$$

M_n is controlled by least of Y, LTB, FLB

$$\therefore M_n = 3334 \text{ ft-k } \boxed{\text{FLB}}$$

LRFD $\phi = 0.90$	ASD $\Omega = 1.67$
$\phi M_n = 0.9\ (3334 \text{ ft-k})$	$M_n/\Omega = 3334 \text{ ft-k}/1.67$
$\phi M_n = 3001 \text{ ft-k}$	$M_n/\Omega = 1996 \text{ ft-k}$

18.5 TENSION FIELD ACTION

The AISC Specification for built-up I-shaped girders permits their design on the basis of postbuckling strength. Designs on this basis provide a more realistic idea of the actual strength of a girder. (Such designs, however, do not necessarily result in better economy, because stiffeners are required.) Should a girder be loaded until initial buckling occurs, it will not then collapse, because of a phenomenon known as *tension field action*.

The panels of a built-up I-shaped girder located between suitably designed vertical stiffeners will resist much larger shear forces than the theoretical buckling strength of the girder's web. Once the shearing forces reach the theoretical buckling strength of the web, the girder will be displaced by a small and insignificant amount.

If transverse stiffeners have been properly designed, membrane forces or diagonal tension fields will develop in the web between the stiffeners, as illustrated in Fig. 18.8. These diagonal tension stresses are caused by those shearing forces which are larger than the shearing forces theoretically required to buckle the web. For these excess forces, the girder will behave much like a Pratt truss, with parts of the web acting as tension diagonals and with the stiffeners acting as compression verticals, as shown in Fig. 18.8.

Students who have studied reinforced-concrete design will notice that tension field action is somewhat like the behavior of reinforced-concrete beams with web reinforcing (according to the Ritter-Morsch theory), as the beams resist shearing forces. Actually, there the beam behavior, according to this theory, is rather like a Warren truss with the concrete "diagonals" being in compression and the web reinforcing serving as tension verticals.

The stiffeners of the built-up I-girders keep the flanges from coming together, and the flanges keep the stiffeners from coming together. The intermediate stiffeners, which before initial buckling were assumed to resist no load, will after buckling resist compressive loads (or will serve as the compression verticals of a truss) due to diagonal tension. The result is that a plate-girder web probably can resist loads equal to two or three times those present at initial buckling before complete collapse will occur.

Until the web buckles initially, deflections are relatively small. However, after initial buckling, the girder's stiffness decreases considerably, and deflections may increase to several times the values estimated by the usual deflection theory.

The estimated total or ultimate shear that a panel (a part of the girder between a pair of stiffeners) can withstand equals the shear initially causing web buckling plus the shear that can be resisted by tension field action. The amount of tension field action is dependent on the proportions of the panels.

The capability necessary for the girders to develop tension field action is based on the ability of the stiffeners to resist compression from both sides of a panel. You can see that there is a panel on only one side of an end panel, so tension field action is not to be considered in such panels. Also, it's not allowed when the panels have very large aspect ratios. The aspect ratio, a, is the ratio of the clear distance between stiffeners in a panel to the height of the panel. According to AISC Specification G3.1,

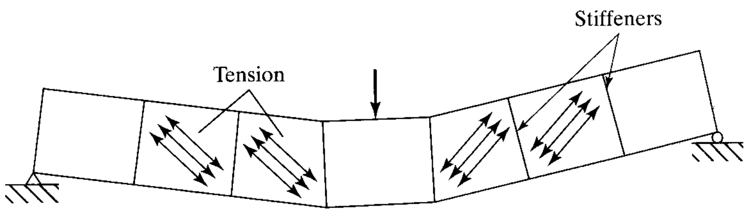


FIGURE 18.8

Tension field action in plate-girder web. (Note that end panels cannot develop tension field action.)

the a/h ratios may not be larger than 3.0 or $\left[\dfrac{260}{h/t_w}\right]^2$. Tension field action also may not be considered if $\left(\dfrac{2A_w}{A_{fc} + A_{ft}}\right) > 2.5$ or if h/b_{fc} or $h/b_{ft} > 6.0$. (Here, A_{fc} and A_{ft} are the areas of the compression and tension flanges, respectively, while b_{fc} and b_{ft} are the widths of those same flanges.

Example 18-5

The built-up A36 I-shaped girder shown in Fig. 18.9 has been selected for a 65-ft simple span to support the loads $w_D = 1.1$ k/ft (not including the beam weight) and $w_L = 2$ k/ft. Select transverse stiffeners as needed.

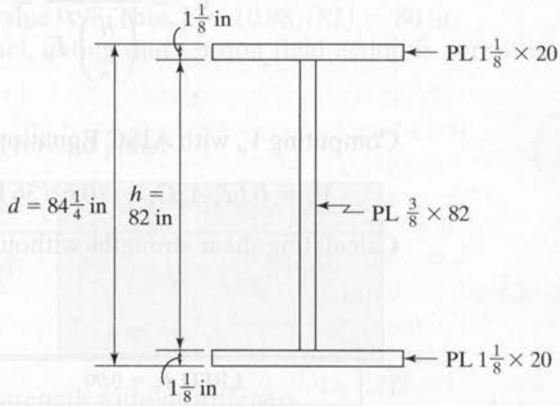


FIGURE 18.9

Computing girder weight

$$A = (2)\left(1\frac{1}{8}\text{ in}\right)(20\text{ in}) + \left(\frac{3}{8}\text{ in}\right)(82\text{ in}) = 75.75\text{ in}^2$$

$$\text{wt per ft} = \left(\frac{75.75\text{ in}^2}{144\text{ in}^2/\text{ft}^2}\right)\left(490\text{ lb/ft}^3\right) = 258\text{ lb/ft}$$

Computing the required shear strength at the support

LRFD	ASD
$w_u = (1.2)(1.1 + 0.258) + (1.6)(2) = 4.83$ k/ft	$w_a = 1.1 + 0.258 + 2 = 3.358$ k/ft
$R_u = \left(\frac{65}{2}\right)(4.83) = 156.98$ k	$R_a = \left(\frac{65}{2}\right)(3.358) = 109.14$ k

Are stiffeners needed?

$$A_w = dt_w = (84.25 \text{ in})\left(\frac{3}{8} \text{ in}\right) = 31.59 \text{ in}^2$$

$$\frac{h}{t_w} = \frac{82}{0.375} = 219 < 260 \therefore k_v = 5.0, \text{ says AISC Section G2.1b.}$$

C_v from same AISC section

$$219 > 1.37\sqrt{k_v E/F_y} = 1.37\sqrt{\frac{(5)(29 \times 10^3)}{36}} = 86.95$$

$\therefore$ Must use AISC Equation G2-5.

$$C_v = \frac{1.51 E k_v}{\left(\frac{h}{t_w}\right)^2 F_y} = \frac{(1.51)(29 \times 10^3)(5)}{(219)^2(36)} = 0.1268$$

Computing V_n with AISC Equation G2-1

$$V_n = 0.6 F_y A_w C_v = (0.6)(36 \text{ k/in}^2)(31.59 \text{ in}^2)(0.1268) = 86.52 \text{ k}$$

Calculating shear strengths without stiffeners

LRFD $\phi_v = 0.90$	ASD $\Omega_v = 1.67$
$\phi_v V_n = (0.90)(86.52) = 77.87$ k	$\frac{V_n}{\Omega_v} = \frac{86.52}{1.67} = 51.81$ k
< 156.98 k $\therefore$ **Stiffeners are required.**	< 109.14 k $\therefore$ **Stiffeners are required.**

Can we use tensile field action? (AISC Specification G3)

a. Not in end panels with transverse stiffeners.

b. Not in members where $\frac{a}{h} > 3.0$ or $\left[260\left(\frac{h}{t_w}\right)\right]^2$.

c. Not if $\left(\frac{2A_w}{A_{fc} + A_{ft}}\right) > 2.5$. Here, A_{fc} = the area of compression flange and A_{ft} = the area of the tension flange.

d. Not if $\frac{h}{b_{fc}}$ or $\frac{h}{b_{ft}} > 6.0$.

Select stiffener spacing for end panel.

By (a), tension field action may not be used

LRFD	ASD
$\dfrac{\phi V_n}{A_w} = \dfrac{V_u}{A_w} = \dfrac{156.98}{31.59} = 4.97 \text{ ksi}$	$\dfrac{V_n}{\Omega_c A_w} = \dfrac{V_a}{A_w} = \dfrac{109.14}{31.59} = 3.45 \text{ ksi}$

Referring to AISC Table 3-16a, which provides the available shear stress (tension field action not included). Entering left margin with $h/t_w = 219$ and moving horizontally from that value to the $\phi V_n/A_w$ curve $= 4.97$ ksi (actually interpolating between the curves in table). There, move down vertically to base and read 1.00. This is the a/h value that can be used.

$\therefore$ a = (1.00)(82) = 82 in.

Using the ASD values $h/t_w = 219$ and $V_n/\Omega_v A_w = 3.45$ ksi and entering AISC Table 3-16a, we read at the base the value 0.98. Thus, $a = (0.98)(82) = 80$ in.

Select stiffeners for second panel, noting that tension field action is permitted, since it's not an end panel.

Required shear strength needed for 2nd panel

LRFD (82 in out in span)	ASD (80 in out in span)
$V_u = 156.98 - \left(\dfrac{82}{12}\right)(4.83)$	$V_a = 109.14 - \left(\dfrac{80}{12}\right)(3.358)$
$= 123.97$ k	$= 86.75$ k

Computing the available shear strength without stiffeners

LRFD $\phi_v = 0.90$	ASD $\Omega_v = 1.67$
$\phi_v v_n = (0.90)(86.52) = 77.87 \text{ k}$	$\dfrac{v_n}{\Omega_v} = \dfrac{86.52}{1.67} = 51.81 \text{ k}$
< 123.97 k	< 86.75 k
∴ More stiffeners reqd.	**∴ More stiffeners reqd.**
$\dfrac{\phi V_n}{A_w} = \dfrac{123.97}{31.59} = 3.92 \text{ ksi}$	$\dfrac{V_n}{\Omega_v A_w} = \dfrac{V_a}{A_w} = \dfrac{86.75}{31.59} = 2.75 \text{ ksi}$

For LRFD with $\phi_v V_n/A_w = 3.92$ ksi and $h/t_w = 219$, we use Table 3-16b, tension field action is included. The stress does not intersect the h/t_w value, so we read the maximum value $a/h = 1.4$. This is obtained by moving horizontally from $h/t_w = 219$ then pivoting on the bold line and moving down vertically to the base and read 1.40.

$$a = (1.4)(82) = 114.8 \text{ in}$$

ASD results are the same with $a = 114.8$ in.

18.6 DESIGN OF STIFFENERS

As previously indicated, it usually is necessary to stiffen the high thin webs of built-up girders to keep them from buckling. If the girders are bolted, the stiffeners probably will consist of a pair of angles bolted to the girder webs. If the girders are welded (the normal situation), the stiffeners probably will consist of a pair of plates welded to the girder webs. Figure 18.10 illustrates these types of stiffeners.

Stiffeners are divided into two groups: *bearing stiffeners*, which transfer heavy reactions or concentrated loads to the full depth of the web, and *intermediate* or *nonbearing stiffeners*, which are placed at various intervals along the web to prevent buckling due to diagonal compression. An additional purpose of bearing stiffeners is the transfer of heavy loads to plate girder webs without putting all the loads on the flange connections.

As described in Sections G2.2 and J10.8 and the corresponding commentaries for those sections in the AISC Specification, transverse stiffeners may be either single or double. They do not have to be connected to the flanges, except in the following situations:

1. Where bearing strength is needed to transmit concentrated loads or reactions.
2. Where single stiffeners are used and the flange of the girder consists of a rectangular plate. For such a situation, the stiffener must be attached to the flange to resist any possible uplift tendency that may be caused by torsion in the flange.
3. Where lateral bracing is attached to a stiffener or stiffeners. Such a stiffener must be connected to the compression flange with a strength sufficient to transmit no less than 1 percent of the total flange stress, unless the flange is composed only of angles.

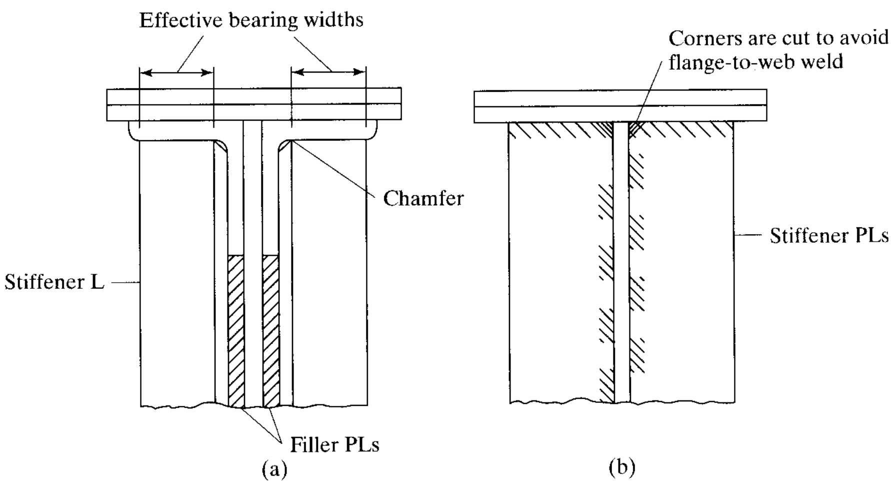


FIGURE 18.10

(a) Angle-bearing stiffeners. (b) Plate-bearing stiffeners.

Welds used to attach stiffeners to girder webs must be terminated at distances not less than 4 times the web thickness nor more than 6 times the web thickness from the near toe of the flange to the web weld.

If bolts are used to connect stiffeners to the web, they may not be spaced farther apart than 12 inches. If intermittent fillet welds are used, the clear distance between them may not exceed the lesser of 16 times the web thickness or 10 inches.

Stiffeners that are located at concentrated loads or reactions have some special requirements because of the possibility of web crippling or compression buckling of the web. In these situations, the stiffeners need to be designed as columns. Should the load or reaction be tensile, it will be necessary to weld the stiffeners to the loaded flange. If the force is compressive, the stiffener may either bear against the loaded flange or be welded to it.

18.6.1 Bearing Stiffeners

Bearing stiffeners are placed in pairs on the webs of built-up girders at unframed girder ends and where required for concentrated loads. They should fit tightly against the flanges being loaded and should extend out toward the edges of the flange plates or angles as far as possible. If the load normal to the girder flange is tensile, the stiffeners must be welded to the loaded flange. If the load is compressive, it is necessary to obtain a snug fit—that is, a good bearing between the flange and the stiffeners. To accomplish this goal, the stiffeners may be welded to the flange or the outstanding legs of the stiffeners may be milled.

A bearing stiffener is a special type of column that is difficult to accurately analyze, because it must support the load in conjunction with the web. The amount of support provided by these two elements is difficult to estimate. The AISC Specification (J10.8) states that the factored load, or reaction, may not exceed the design strength of a column consisting of the stiffener effective area plus a portion of the web equal to $12t_w$ at girder ends and $25t_w$ at interior concentrated loads. Only the part of the stiffeners outside of the fillets of the flange angles or the parts outside of the flange to web welds (see Fig. 18.10) are to be considered effective to support the bearing loads. The effective length of these *bearing stiffener columns* is assumed by the AISC Specification (J10.8) to be equal to 0.75 h. The welds made to the girder flange must be designed for the difference between the required strength and the applicable limit state strength. Several recommentations for the details of bearing stiffeners are given in AISC Commentary J10.8.

At an unframed girder end, an end-bearing stiffener is required if the factored reaction R_u is larger than ϕR_n or if $R_a > R_n/\Omega$ If an interior load or reaction is larger than the same values, an interior bearing stiffener is needed.

If the concentrated force is applied at a distance from the member end greater than the member depth d,

$$R_n = (5k + l_b)F_{yw}t_w \qquad \text{(AISC Equation J10-2)}$$

If the concentrated force is applied at a distance from the member end less than or equal to the member depth d,

$$R_n = (2.5k + l_b)F_{yw}t_w \qquad \text{(AISC Equation J10-3)}$$

In these expressions,

F_{yw} = specified minimum yield stress of web, ksi
k = distance from outer face of flange to web toe of fillet, in
l_b = length of bearing (not less than k for end beam reactions), in
t_w = web thickness, in
$\phi = 1.0$ and $\Omega = 1.50$

18.6.2 Intermediate Stiffeners

Intermediate or non-bearing stiffeners are also called stability or transverse intermediate stiffeners. These stiffeners are not required by the AISC Specification Section G2.2 where $h/t_w \leq 2.46\sqrt{\frac{E}{F_{yw}}}$ or where the available strength provided in accordance with Section G2.1 for $k_v = 5$ is greater than the required shear strength.

If $1.10\sqrt{\dfrac{k_v E}{F_y}} < \dfrac{h}{t_w} \leq 1.37\sqrt{\dfrac{k_v E}{F_y}}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad \text{(AISC Equation G2-4)}$$

If $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_v E}{F_y}}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \qquad \text{(AISC Equation G2-5)}$$

The AISC Specification imposes some arbitrary limits on panel aspect ratios (a/h) for plate girders, even where shear stresses are small. The purpose of these limitations is to facilitate the handling of girders during fabrication and erection.

$$V_n = 0.6F_y A_w C_v \qquad \text{(AISC Equation G2-1)}$$

You should note here that, as the intermediate stiffener spacing is decreased, C_v will become larger, as will the shear capacity of the girder.

The AISC Specification (G2.2) states that the moment of inertia of a transverse intermediate stiffener about an axis at the center of the girder web if a pair of stiffeners is used, or about the face in contact with the web when single stiffeners are used, may not be less than

$$I_{st} \text{ minimum} \geq bt_w^3 j \quad \text{where } b \text{ is the smaller of } a \text{ and } h$$

AISC Specification G3 states that transverse stiffeners subject to tension field action must meet the following limitations in which $(b/t)_{st}$ is the width thickness ratio of the stiffener:

$$\left(\frac{b}{t}\right)_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}} \qquad \text{(AISC Equation G3-3)}$$

$$I_{st} \geq I_{st1} + (I_{st2} - I_{st1})\left[\frac{V_r - V_{c1}}{V_{c2} - V_{c1}}\right] \qquad \text{(AISC Equation G3-4)}$$

I_{st} is the moment of inertia of the transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners. I_{st1} is the minimum moment of inertia of transverse stiffeners required for development of the web shear buckling resistance in Section G2.2. I_{st2} is the *minimum* moment of inertia of the transverse stiffeners required for development of the full web shear buckling plus the web tension field resistance, $V_r = V_{c2}$.

$$I_{st2} = \frac{h^4 \rho_{st}^{\ 1.3}}{40}\left[\frac{F_{yw}}{E}\right]^{1.5} \qquad \text{(AISC Equation G3-5)}$$

V_r is the larger of the required shear strengths in the adjacent web panels. V_{c1} is the smaller of the available shear strength in the adjacent web panels with V_n as defined in Section G2.1 V_{c2} is the smaller of the available shear strength in the adjacent web panels with V_n as defined in Section G 3.2. ρ_{st} is the larger of F_{yw}/F_{yst} and 1.0.

Equation G3-4 is the same requirement as specified in AASHTO (2007).[1]

18.6.3 Longitudinal Stiffeners

Longitudinal stiffeners, though not as effective as transverse ones, frequently are used for bridge plate girders, because many designers feel they are more attractive. As such stiffeners are seldom used for plate girders in buildings, they are not covered in this textbook.

Highway bypass bridge, Stroudsburg, PA. (Courtesy of Bethlehem Steel Corporation.)

[1]AASHTO LRFD Bridge Design Specifications, 2007 (Washington, DC: American Association of State Highway and Transportation Officials).

Example 18-6

Design bearing and intermediate stiffeners for the plate girder of Example 18-5, which is not framed at its ends.

Solution

a. Are end bearing stiffeners required?

Assume point bearing (conservative) of the end reaction (that is, $l_b = 0$) and assume a 5/16-in web-to-flange fillet weld.

Check local web yielding

$$k = \text{distance from outer edge of flange to web end of fillet weld}$$

$$= 1\frac{1}{8} + \frac{5}{16} = 1.44 \text{ in}$$

$$R_n = (2.5\text{ k} + l_b)F_y t_w \qquad \text{(AISC Equation J10-3)}$$

$$= (2.5 \times 1.44 + 0)(36)\left(\frac{3}{8}\right) = 48.6 \text{ k}$$

LRFD $\phi = 1.00$	ASD $\Omega = 1.50$
$\phi R_n = (1.00)(48.6) = 48.6 \text{ k} < 156.98 \text{ k}$	$\frac{R_n}{\Omega} = \frac{48.6}{1.50} = 32.4 \text{ k} < 109.14 \text{ k}$
$\therefore$ End bearing stiffener is reqd.	$\therefore$ End bearing stiffener is reqd.

If local web yielding check had been satisfactory, it would have been necessary to also check the web crippling criteria set forth in AISC Section J10.3 before we could definitely say that end bearing stiffeners were not required.

b. Design of end-bearing stiffeners

Try two plate stiffeners $\frac{5}{8} \times 9$, as shown in Fig. 18.11.

Check width–thickness ratio (AISC Table B4.1a).

$$\frac{9.00}{0.875} = 10.29 < 0.56\sqrt{\frac{29{,}000}{36}} = 15.89 \qquad \textbf{OK}$$

Check column strength of stiffener, which is shown crosshatched in Fig. 18.11.

$I \approx \left(\frac{1}{12}\right)\left(\frac{5}{8}\right)(18.375)^3 = 323 \text{ in}^4$ (neglecting the almost infinitesimal contribution of the web)

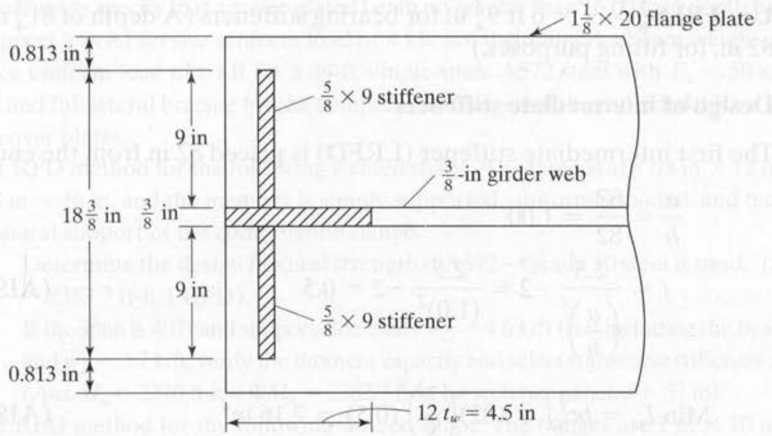


FIGURE 18.11
End bearing stiffeners.

$$A \text{ of column} = (2)\left(\frac{5}{8}\right)(9) + (4.5)\left(\frac{3}{8}\right) = 12.94 \text{ in}^2$$

$$r = \sqrt{\frac{323}{12.94}} = 5.00 \text{ in}$$

$$kL = (0.75)(82) = 61.5 \text{ in}$$

$$\frac{kL}{r} = \frac{61.5}{5.00} = 12.30$$

$$\phi_c F_{cr} = 32.17 \text{ ksi and } \frac{F_{cr}}{\Omega} = 21.39 \text{ ksi from AISC Table 4-22}$$

LRFD	ASD
$\phi_c P_n = (32.17)(12.94) = 416.3 \text{ k} > 156.98 \text{ k}$ **OK**	$\frac{P_n}{\Omega_c} = (21.39)(12.94) = 276.8 \text{ k} > 109.14 \text{ k}$ **OK**

Check bearing criterion

$$P_n = 1.8\, F_y A_{pd} = (1.8)(36)(2)(9 - 0.5)\left(\frac{5}{8}\right) = 688.5 \text{ k}$$

LRFD $\phi = 0.75$	ASD $\Omega = 2.00$
$\phi P_n = (0.75)(688.5) = 516.4 \text{ k} > 156.98 \text{ k}$ **OK**	$\frac{P_n}{\Omega} = \frac{688.5}{2.00} = 344.2 \text{ k} > 109.14 \text{ k}$ **OK**

Use 2 PLS $\frac{5}{8} \times 9 \times 6$ ft $9\frac{3}{4}$ in for bearing stiffeners (A depth of $81\frac{3}{4}$ in is used instead of 82 in, for fitting purposes.)

Design of intermediate stiffeners

The first intermediate stiffener (LRFD) is placed 82 in from the end.

$$\frac{a}{h} = \frac{82}{82} = 1.00$$

$$j = \frac{2.5}{\left(\frac{a}{h}\right)^2} - 2 = \frac{2.5}{(1.0)^2} - 2 = 0.5 \qquad \text{(AISC Equation G2-8)}$$

$$\text{Min } I_{st} = bt_{w}^{3}j = (82)\left(\frac{3}{8}\right)^3(0.5) = 2.16 \text{ in}^4 \qquad \text{(AISC Equation G2-7)}$$

There are many possible satisfactory stiffener sizes, but only two sizes are tried here.

Try $\frac{1}{4} \times 6$ single plate stiffener

$$I_{st} = \left(\frac{1}{12}\right)\left(\frac{1}{4}\right)(6)^3 = 4.5 \text{ in}^4 > 2.16 \text{ in}^4$$

Try a pair of $\frac{1}{4} \times 4$ plate stiffeners

$$I_{st} = \left(\frac{1}{12}\right)\left(\frac{1}{4}\right)\left(2 \times 4 + \frac{3}{8}\right)^3 = 12.24 \text{ in}^4 > 2.16 \text{ in}^4$$

use $\frac{1}{4} \times 6$ single plate stiffener

18.7 PROBLEMS FOR SOLUTION

All problems are to be solved by both the LRFD and ASD procedures, UNO.

18-1. Select a cover-plated W section limited to a maximum depth of 21.00 in to support the service loads shown in the accompanying figure. Use A36 steel and assume the beam has full lateral bracing for its compression flange. (*Ans.* one solution W18 × 86 with PL7/8 × 14 each flange LRFD, W18 × 86 with PL7/8 × 16, ASD)

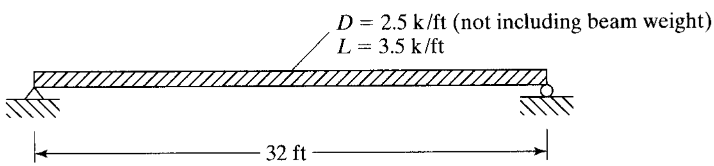


FIGURE P18-1

18-2. The architects specify that a cover-plated beam no greater than 16.00 in in depth be designed to support a dead service uniform load of 4 klf, not including the beam weight, and a live service uniform load of 6 klf for a 24-ft simple span. A572 steel with $F_y = 50$ ksi is to be used, and full lateral bracing for the compression flange is to be provided. Use a W14 × 53 plus cover plates.

18-3. Use LRFD method for the following welded shape: The flanges are 7/8 in × 12 in, the web is 3/8 in × 46 in, and the member is simply supported, uniformly loaded, and has continuous lateral support of the compression flange.

(a) Determine the design flexural strength if A572 – Grade 50 steel is used. (*Ans.* $\Phi M_n = 2383.7$ ft-k, LRFD)

(b) If the span is 40 ft and supports the loads $w_D = 4.6$ k/ft (not including the beam weight) and $w_L = 3.7$ k/ft, verify the moment capacity and select transverse stiffeners as needed. (*Ans.* $M_u = 2320$ ft-k $< \Phi M_n = 2383.7$ ft-k; 1st stiffener panel, $a = 57$ in)

18-4. Use LRFD method for the following welded shape: The flanges are 1 in × 10 in, the web is 5/16 in × 50 in, and the member is simply supported, uniformly loaded. Lateral support of the compression flange is provided at the ends and at the midspan.

(a) Determine the design flexural strength if A572 – Grade 50 steel is used.

(b) If the span is 34 ft and supports the loads $w_D = 4.5$ k/ft (not including the beam weight) and $w_L = 4.5$ k/ft, verify the moment capacity and select transverse stiffeners as needed.

18-5. Design a 48-in-deep welded built-up wide-flange section with no intermediate stiffeners for a 50-ft simple span to support a service dead load of 1 klf (not including the beam weight) and a service live load of 1.8 klf. The section is to be framed between columns and is to have full lateral bracing for its compression flange. The design is to be made with A36 steel. (*Ans.* One solution $\frac{7}{16} \times 46$ -in web, $\frac{5}{8} \times 10$ in flange plates)

18-6. Repeat Prob. 18-5 if the span is to be 60 ft and $F_y = 50$ ksi.

18-7. Design, using the LRFD method, a simply supported plate girder to span 50 ft and support the service loads shown in the figure below. The maximum permissible depth of the girder is 58 in. Use A36 steel and E70XX electrodes and assume that the girder has continuous lateral support of the compression flange. The ends have a bearing-type connection (with the bearing length, $l_b = 6$ in). Use 16 in wide flanges and a 5/16 in thick web. Also, select transverse stiffeners as needed. Design end bearing stiffener, first intermediate stiffener, the welded connection of the web to the flange, and if a bearing stiffener is required at the concentrated loads. (*Ans.* 1 1/2 × 16 flanges, 5/16 × 55 web; $M_u = 3414$ ft-k $< \Phi M_n = 3943$ ft-k; 1st stiffener panel, $a = 35$ in)

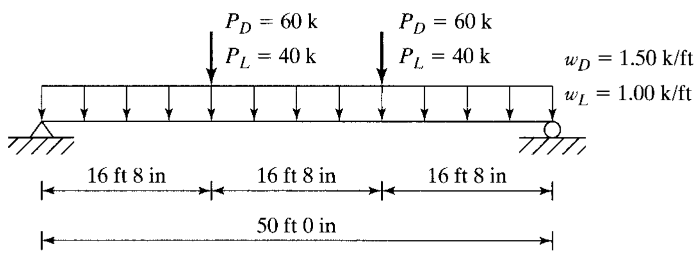


FIGURE P18-7

CHAPTER 19

Design of Steel Buildings

19.1 INTRODUCTION TO LOW-RISE BUILDINGS

The material in this section and the next pertains to the design of low-rise steel buildings up to several stories in height, while Sections 19.3 to 19.10 provide information concerning common types of building floors, and Section 19.11 presents common types of roof construction. Sections 19.12 and 19.13 are concerned with walls, partitions, and fireproofing. The sections thereafter present general information relating to high-rise or multistory buildings.

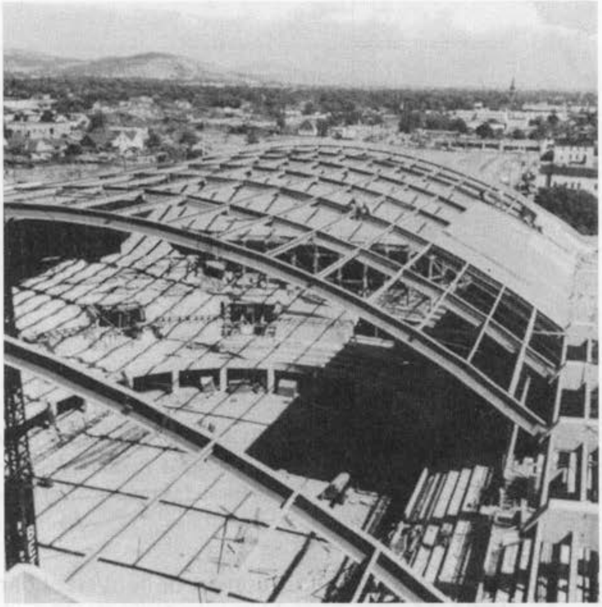
The low-rise buildings considered include apartment houses, office buildings, warehouses, schools, and institutional buildings that are not very tall with respect to their least lateral dimensions.

19.2 TYPES OF STEEL FRAMES USED FOR BUILDINGS

Steel buildings usually are classified as being in one of four groups according to their type of construction: *bearing-wall* construction, *skeleton* construction, *long-span* construction, and *combination steel and concrete framing*. More than one of these construction types can be used in the same building. We discuss each of these types briefly in the paragraphs that follow.

19.2.1 Bearing-Wall Construction

Bearing-wall construction is the most common type of single-story light commercial construction. The ends of beams or joists or light trusses are supported by the walls that transfer the loads to the foundation. The old practice was to rapidly thicken load-bearing walls as buildings became taller. For instance, the wall on the top floor of a building might be one or two bricks thick, while the lower walls might be increased by one brick thickness for each story as we come down the building. As a result, this type of construction was usually thought to have an upper economical limit of about two or three stories, although



Coliseum in Spokane, WA. (Courtesy of Bethlehem Steel Corporation.)

some load-bearing buildings were much higher. The tallest load-bearing building built in the United States in the nineteenth century was the 17-story Monadnock building in Chicago. This building, completed in 1891, had 72-in-thick walls on its first floor. A great deal of research has been conducted for load-bearing construction in recent decades, and it has been discovered that thin load-bearing walls may be quite economical for many buildings up to 10, 20, or even more stories.

The average engineer is not very well versed in the subject of bearing-wall construction, with the result that he or she may often specify complete steel or reinforced-concrete frames where bearing-wall construction might have been just as satisfactory and more economical. Bearing-wall construction is not very resistant to seismic loadings, and has an erection disadvantage for buildings of more than one story. For such cases, it is necessary to place the steel floor beams and trusses floor by floor as the masons complete their work below, thus requiring alternation of the masons and ironworkers.

Bearing plates usually are necessary under the ends of the beams, or light trusses, that are supported by the masonry walls, because of the relatively low bearing strength of the masonry. Although, theoretically, the beam flanges may on many occasions provide sufficient bearing without bearing plates, plates are almost always used—particularly where the members are so large and heavy that they must be set by a steel erector. The plates usually are shipped loose and set in the walls by the masons. Setting them in their correct positions and at the correct elevation is a critical part of the construction. Should they not be properly set, there will be some delay in correcting their positions. If a steel erector is used, he or she probably will have to make an extra trip to the job.

When the ends of a beam are enclosed in a masonry wall, some type of wall anchor is desirable to prevent the beam from moving longitudinally with respect to the

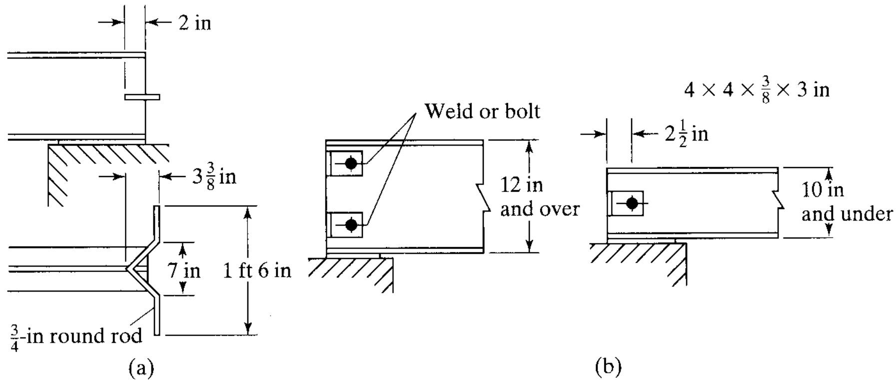


FIGURE 19.1

(a) Government anchor. (b) Angle wall anchors. From American Institute Steel Construction, *Manual of Steel Construction Load & Resistance Factor Design*, 2d ed. (Chicago: AISC, 1994), p. 12–24. "Copyright © American Institute of Steel Construction. Reprinted with permission. All rights reserved."

wall. The usual anchors consist of bent steel bars passing through beam webs. These so-called *government anchors* are shown in Fig. 19.1(a). Occasionally, clip angles attached to the web are used instead of government anchors. These are shown in Fig. 19.1(b). Should longitudinal loads of considerable magnitude be anticipated, regular vertical anchor bolts may be used at the beam ends.

For small commercial and industrial buildings, bearing-wall construction is quite economical when the clear spans are not greater than roughly 35 or 40 ft. If the clear spans are much greater, it becomes necessary to thicken the wall and use pilasters to ensure stability. For these cases, it may often be more economical to use intermediate columns if permissible.

19.2.2 Skeleton Construction

In skeleton construction, the loads are transmitted to the foundations by a framework of steel beams and columns. The floor slabs, partitions, exterior walls, and so on, all are supported by the frame. This type of framing, which can be erected to tremendous heights, often is referred to as beam-and-column construction.

In beam-and-column construction, the frame usually consists of columns spaced 20, 25, or 30 ft apart, with beams and girders framed into them from both directions at each floor level. One very common method of arranging the members is shown in Fig. 19.2. The more heavily loaded girders are placed in the short direction between the columns, while the comparatively lightly loaded beams are framed between the girders in the long direction. It is common practice to orient columns in a way that will minimize eccentric loading. Many engineers will orient columns so that the girders will frame into the web and beams will frame into the flange as shown for the interior bay in Fig. 19.2. With various types of floor construction, other arrangements of beams and girders may be used.

For skeleton framing, the walls can be supported by the steel frame and generally are referred to as *nonbearing* or *curtain walls*. The beams supporting the exterior walls

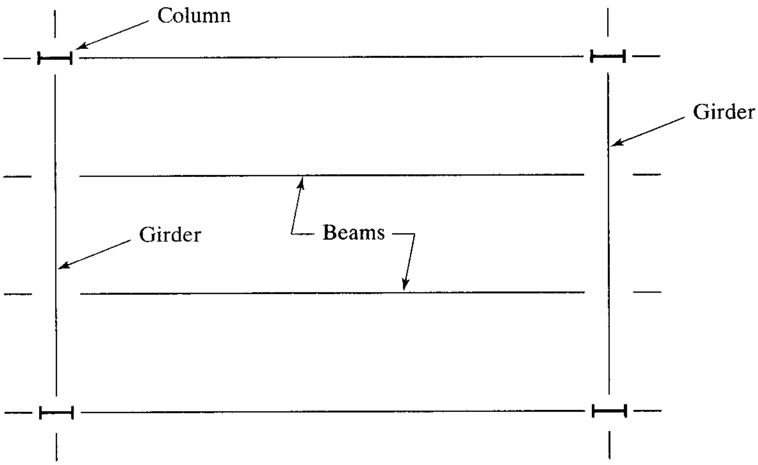


FIGURE 19.2

Beam-and-column construction.

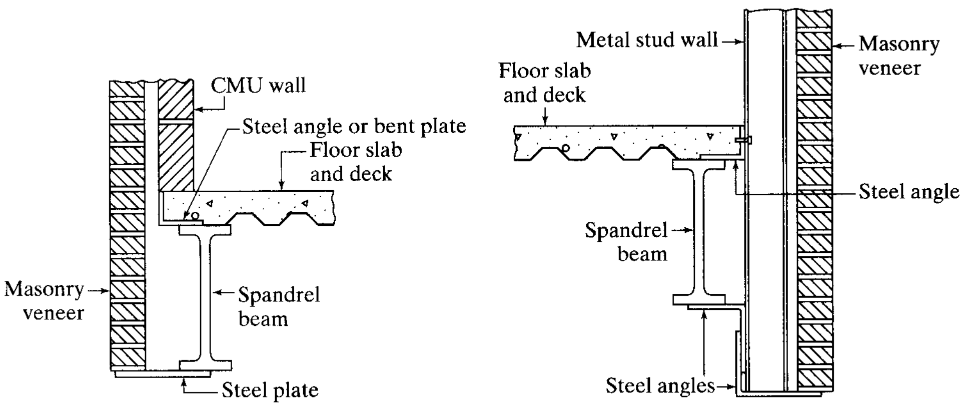


FIGURE 19.3

Spandrel beams.

are called *spandrel beams*. These beams, illustrated in Fig. 19.3, can be placed so that they will serve as the lintels for the windows or other openings.

19.2.3 Long-Span Steel Structures

When it becomes necessary to use very large spans between columns—such as for gymnasiums, auditoriums, theaters, hangars, or hotel ballrooms—the usual skeleton construction may not be sufficient. Should the ordinary rolled *W* sections be insufficient, it may be necessary to use cover-plated beams, built-up I-shaped girders, box girders, large trusses, arches, rigid frames, and the like. When depth is limited, cover-plated beams, plate girders, or box girders may be called upon to do the job. Should

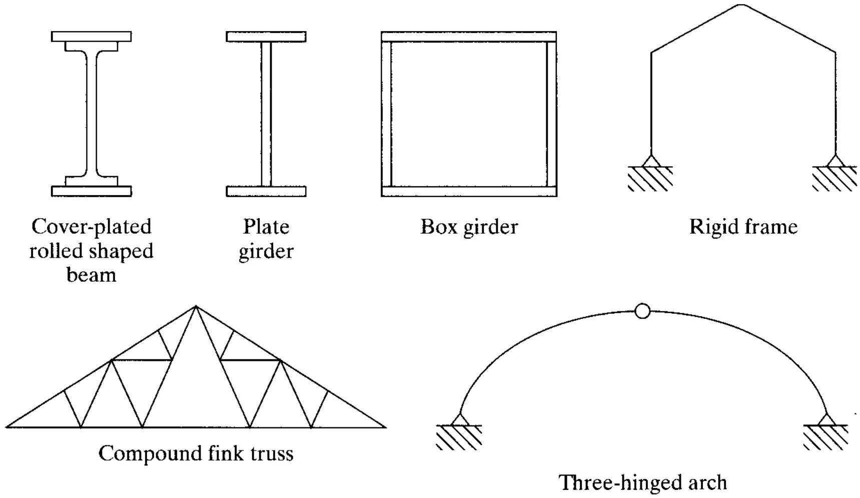


FIGURE 19.4

Long-span structures.

depth not be so critical, trusses may be satisfactory. For very large spans, arches and rigid frames often are used. These various types of structures are referred to as *long-span structures*. Figure 19.4 shows a few of these types of structures.

19.2.4 Combination Steel and Concrete Framing

A large percentage of the buildings erected today make use of a combination of reinforced concrete and structural steel. When reinforced-concrete columns are used in very tall buildings, they are rather large on the lower floors and take up considerable space. Steel column shapes surrounded by and bonded to reinforced concrete may be used and are referred to as *encased* or *composite* columns. Composite columns consisting of HSS members filled with concrete (called filled composite columns) may also be used.

19.3 COMMON TYPES OF FLOOR CONSTRUCTION

Concrete floor slabs of one type or another are used almost universally for steel-frame buildings. They are strong and have excellent fire ratings and good acoustic ratings. On the other hand, appreciable time and expense are required to provide the formwork necessary for most slabs. Concrete floors are heavy, they must include some type of reinforcing bars or mesh, and there may be a problem involved in making them watertight. The following are some of the types of concrete floors used today for steel-frame buildings:

1. Concrete slabs supported with open-web steel joists (Section 19.4)
2. One-way and two-way reinforced-concrete slabs supported on steel beams (Section 19.5)

3. Concrete slab and steel beam composite floors (Section 19.6)
4. Concrete-pan floors (Section 19.7)
5. Steel-decking floors (Section 19.8)
6. Flat slab floors (Section 19.9)
7. Precast concrete slab floors (Section 19.10)

Among the several factors to be considered in selecting the type of floor system to be used for a particular building are loads to be supported; fire rating desired; sound and heat transmission; dead weight of floor; ceiling situation below (to be flat or to have beams exposed); facility of floor for locating conduits, pipes, wiring, and so on; appearance; maintenance required; time required to construct; and depth available for floor.

You can get a lot of information about these and other construction practices by referring to various engineering magazines and catalogs, particularly *Sweet's Catalog File*, published by McGraw-Hill Information Systems Company. The author cannot make too strong a recommendation for these books to help the student see the tremendous amount of data available. The sections to follow present brief descriptions of the floor systems mentioned in this section, along with some discussions of their advantages and chief uses.

19.4 CONCRETE SLABS ON OPEN-WEB STEEL JOISTS

Perhaps the most common type of floor slab in use for small steel-frame buildings is the slab supported by open-web steel joists. The joists are small parallel chord trusses whose members usually are made from bars (hence, the common name *bar joist*) or small angles or other rolled shapes. Steel forms or decks are usually attached to the joists by welding or self-drilling or self-tapping screws; then concrete slabs are poured on top. This is one of the lightest types of concrete floors and also one of the most economical. A sketch of an open-web joist floor is shown in Fig. 19.5.

Open-web joists are particularly well suited to building floors with relatively light loads and for structures where there is not too much vibration. They have been used a great deal for fairly tall buildings, but generally speaking, they are better suited for shorter buildings. Open-web joists are very satisfactory for supporting floor and roof slabs for schools, apartment houses, hotels, office buildings, restaurant buildings, and similar low-level structures. These joists generally are not suitable for supporting concentrated loads, however, unless they are especially detailed to carry such loads.

Open-web joists must be braced laterally to prevent them from twisting or buckling and to keep the floors from becoming too springy. Lateral support is provided by *bridging*, which consists of continuous horizontal rods fastened to the top and bottom chords of the joists, or of diagonal cross bracing. Bridging is desirably used at spaces not exceeding 7 ft on centers.

Open-web joists are easy to handle, and they are quickly erected. If desired, a ceiling can be attached to the bottom or suspended from the joists. The open spaces in the webs are well suited for placing conduits, ducts, wiring, piping, and so on. The joists should be either welded to supporting steel beams, or well-anchored in the masonry

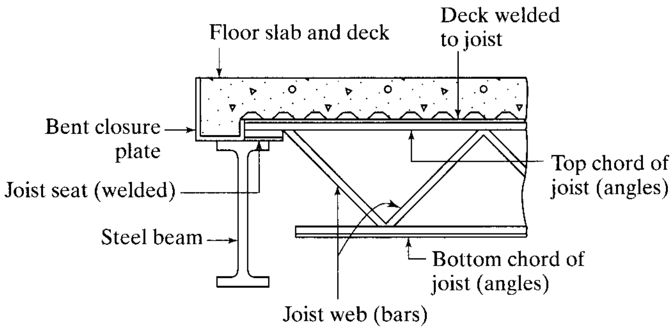


FIGURE 19.5

Open-web joists.

walls. When concrete slabs are placed on top of the joists, they are usually from 2 to 2 1/2 in thick. Nearly all of the many concrete slabs cast in place or precast on the market today can be used successfully on top of open-web joists.

Detailed information concerning open-web joists is not included in the AISC Manual. Such data can be obtained from a publication by the Steel Joist Institute, entitled *Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders.*[1] The load tables and specifications are based on an allowable stress method (ASD) and load and resistance factor design (LRFD) method.

Three categories of joists are presented in *Standard Specifications: Open-Web Joists* (called the K-series), the *Longspan Steel Joists* (LH-series), and the *Deep Longspan Steel Joists* (DLH-series). These three types of joists are designed as simply supported trusses which themselves support uniform loads on their top chords. Should concentrated loads need to be supported, a special analysis should be requested from the manufacturer.

One page of the Steel Joist Institute's Standard ASD Load Table for the K-series joists is presented as Table 19.1 of this chapter. As an illustration of the use of this table, look across the top headings of the columns until you reach the 12K3 joists. This member has a nominal depth of 12 in and an approximate weight of 5.7 lb/ft. If we now move vertically down the column under the 12K3 heading until we reach a 20-ft span, as shown on the left side of the table, we can see that this joist can support a total service load equal to 302 lb/ft. The number given below the 302 is 177 lb/ft. It represents the live uniform load that will cause this joist to have an approximate deflection equal to 1/360th of the span. The joists given in the Institute tables run all the way from the 8K1 to the 72DLH19. This latter joist will support a total load of 497 lb/ft for a 144 ft span.

The Steel Joist Institute actually presents a fourth category of joists called *joist girders*. These are quite large joists that may be used to support open-web joists.

[1]Myrtle Beach, SC: Steel Joist Institute, 2005.

TABLE 19.1 Standard Load Table Open Web Steel Joists, K-Series Based on a Maximum Allowable Tensile Stress of 30,000 psi

Joist Designation	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (in)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt. (lb/ft)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
Span (ft) ↓ 8	550 550															
9	550 550															
10	550 480	550 550														
11	532 377	550 542														
12	444 288	550 455	550 550	550 550	550 550											
13	377 225	479 363	550 510	550 510	550 510											
14	324 179	412 289	500 425	550 463	550 463	550 550	550 550	550 550	550 550							
15	281 145	358 234	434 344	543 428	550 434	511 475	550 507	550 507	550 507							
16	246 119	313 192	380 282	476 351	550 396	448 390	550 467	550 467	550 467	550 550	550 550	550 550	550 550	550 550	550 550	550 550
17		277 159	336 234	420 291	550 366	395 324	495 404	550 443	550 443	512 488	550 526	550 526	550 526	550 526	550 526	550 526
18		246 134	299 197	374 245	507 317	352 272	441 339	530 397	550 408	456 409	508 456	550 490	550 490	550 490	550 490	550 490
19		221 113	268 167	335 207	454 269	315 230	395 287	475 336	550 383	408 347	455 386	547 452	550 455	550 455	550 455	550 455
20		199 97	241 142	302 177	409 230	284 197	356 246	428 287	525 347	368 297	410 330	493 386	550 426	550 426	550 426	550 426
21			218 123	273 153	370 198	257 170	322 212	388 248	475 299	333 255	371 285	447 333	503 373	548 405	550 406	550 406
22			199 106	249 132	337 172	234 147	293 184	353 215	432 259	303 222	337 247	406 289	458 323	498 351	550 385	550 385
23			181 93	227 116	308 150	214 128	268 160	322 188	395 226	277 194	308 216	371 252	418 282	455 307	507 339	550 363
24			166 81	208 101	282 132	196 113	245 141	295 165	362 199	254 170	283 189	340 221	384 248	418 269	465 298	550 346
25						180 100	226 124	272 145	334 175	234 150	260 167	313 195	353 219	384 238	428 263	514 311
26						166 88	209 110	251 129	308 156	216 133	240 148	289 173	326 194	355 211	395 233	474 276
27						154 79	193 98	233 115	285 139	200 119	223 132	268 155	302 173	329 188	366 208	439 246
28						143 70	180 88	216 103	265 124	186 106	207 118	249 138	281 155	306 168	340 186	408 220
29										173 95	193 106	232 124	261 139	285 151	317 167	380 198
30										161 86	180 96	216 112	244 126	266 137	296 151	355 178
31										151 78	168 87	203 101	228 114	249 124	277 137	332 161
32										142 71	158 79	190 92	214 103	233 112	259 124	311 147

Short-span joists in the Univac Center of Sperry-Rand Corporation, Blue Bell, PA. (Courtesy of Bethlehem Steel Corporation.)

19.5 ONE-WAY AND TWO-WAY REINFORCED-CONCRETE SLABS

19.5.1 One-Way Slabs

A very large number of concrete floor slabs in old office and industrial buildings consisted of one-way slabs about 4 in thick, supported by steel beams 6 to 8 ft on centers. These floors were often referred to as *concrete arch* floors, because, at one time, brick or tile floors were constructed in approximately the same shape—that is, in the shape of arches with flat tops.

A one-way slab is shown in Fig. 19.6. The slab spans in the short direction shown by the arrows in the figure. One-way slabs usually are used when the long direction is two or more times the short direction. In such cases, the short span is so much stiffer than the long span that almost all of the load is carried by the short span. The short direction is the main direction of bending and will be the direction of the main reinforcing bars in the concrete, but temperature-and-shrinkage steel is needed in the other direction.

A typical cross section of a one-way slab floor with supporting steel beams is shown in Fig. 19.7. When steel beams or joists are used to support reinforced-concrete floors, it may be necessary to encase them in concrete or other materials to provide the required fire rating. Such a situation is very expensive.

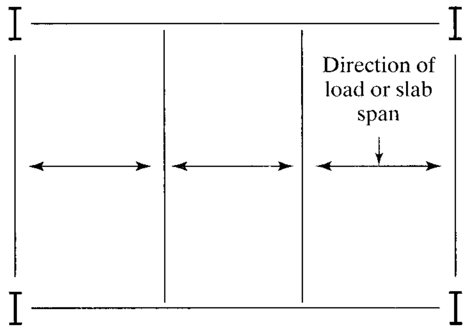


FIGURE 19.6
One-way slab.

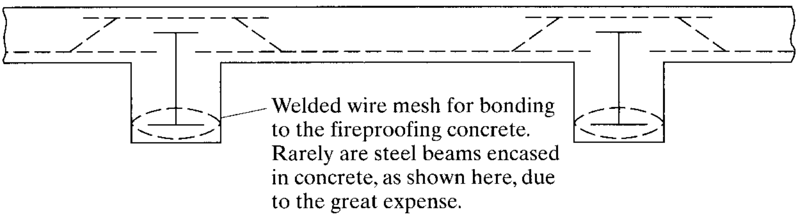


FIGURE 19.7
Cross section of one-way slab floor.

It may be necessary to leave steel lath protruding from the bottom flanges or soffits of the beam for the purpose of attaching plastered ceilings. Should such ceilings be required to cover the beam stems, this floor system will lose a great deal of its economy.

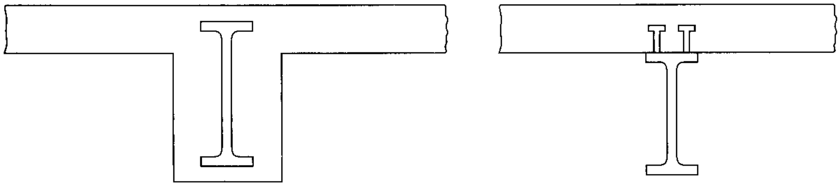
One-way slabs have an advantage when it comes to formwork, because the forms can be supported entirely by the steel beams, with no vertical shoring needed. They have a disadvantage in that they are much heavier than most of the newer lightweight floor systems. The result is that they are not used as often as formerly for lightly loaded floors. Should, however, a rigid floor, a floor to support heavy loads, or a durable floor be needed, the one-way slab may be the appropriate selection.

19.5.2 Two-Way Concrete Slabs

The two-way concrete slab is used when the slabs are square or nearly so, with supporting beams under all four edges. The main reinforcing runs in both directions. Other characteristics are similar to those of the one-way slab.

19.6 COMPOSITE FLOORS

Composite floors, previously discussed in Chapter 16, have steel beams (rolled sections, cover-plated beams, or built-up members) bonded together with concrete slabs in such a manner that the two act as a unit in resisting the total loads that the beam sections would otherwise have to resist alone. The steel beams can be smaller when composite floors are used, because the slabs act as part of the beams.



(a) (b)

FIGURE 19.8

Composite floors. (a) Steel beam encased in concrete (very expensive). (b) Steel beam bonded to concrete slab with steel anchors.

A particular advantage of composite floors is that they utilize concrete's high compressive strength by keeping all or nearly all of the concrete in compression, and at the same time stress a larger percentage of the steel in tension than is normally the case with steel-frame structures. The result is less steel tonnage in the structure. A further advantage of composite floors is an appreciable reduction in total floor thickness, which is particularly important in taller buildings.

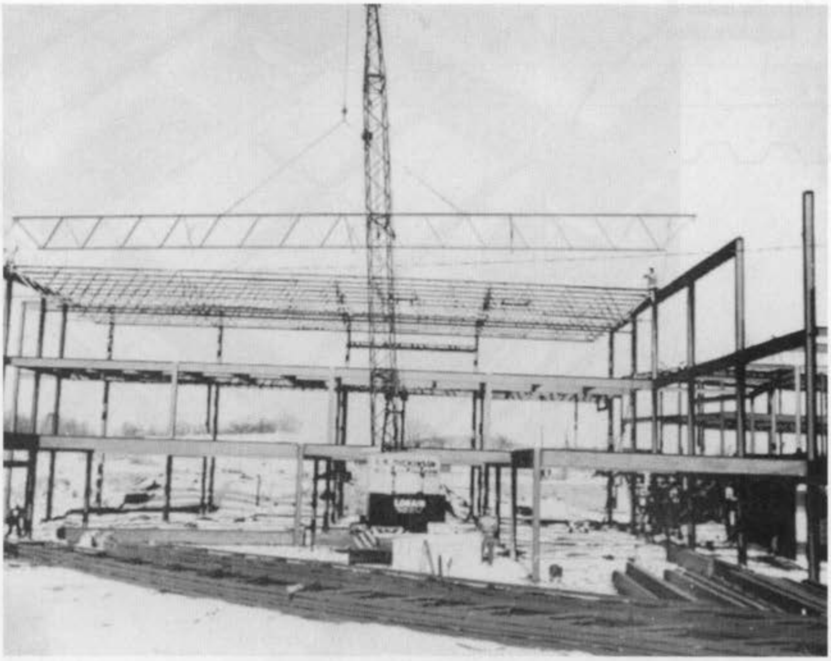
Two types of composite floor systems are shown in Fig. 19.8. The steel beam can be completely encased in the concrete and the horizontal shear transferred by friction and bond (plus some shear reinforcement if necessary). *This type of composite floor is usually quite uneconomical.* The usual type of composite floor is shown in part (b), where the steel beam is bonded to the concrete slab with some type of steel anchors. Various types of steel anchors have been used during the past few decades, including spiral bars, channels, angles, studs, and so on, but economic considerations usually lead to the use of round studs welded to the top flanges of the beams in place of the other types mentioned. Typical studs are 1/2 to 3/4 in in diameter and 2 to 4 in in length.

Cover plates may occasionally be welded to the bottom flanges of the rolled steel sections used for composite floors. The student can see that, with the slab acting as a part of the beam, there is quite a large area available on the compressive side of the beam. By adding plates to the tensile flange, a little better balance is obtained.

19.7 CONCRETE-PAN FLOORS

There are several types of pan floors that are constructed by placing concrete in removable pan molds. (Some special light corrugated pans also are available that can be left in place.) Rows of the pans are arranged on wooden floor forms, and the concrete is placed over the top of them, producing a floor cross section such as the one shown in Fig. 19.9. Joists are formed between the pans, giving a tee-beam-type floor.

These floors, which are suitable for fairly heavy loads, are appreciably lighter than the one-way and two-way concrete slab floors. They require a good deal of formwork, including appreciable shoring underneath the stems. Labor is thus higher than for many floors, but savings due to weight reduction and the reuse of standard-size pans may make pan floor design economically competitive. If suspended ceilings are required, pan floors will have a decided economic disadvantage.



The 104-ft joists for the Bethlehem Catholic High School, Bethlehem, PA. (Courtesy of Bethlehem Steel Corporation.)

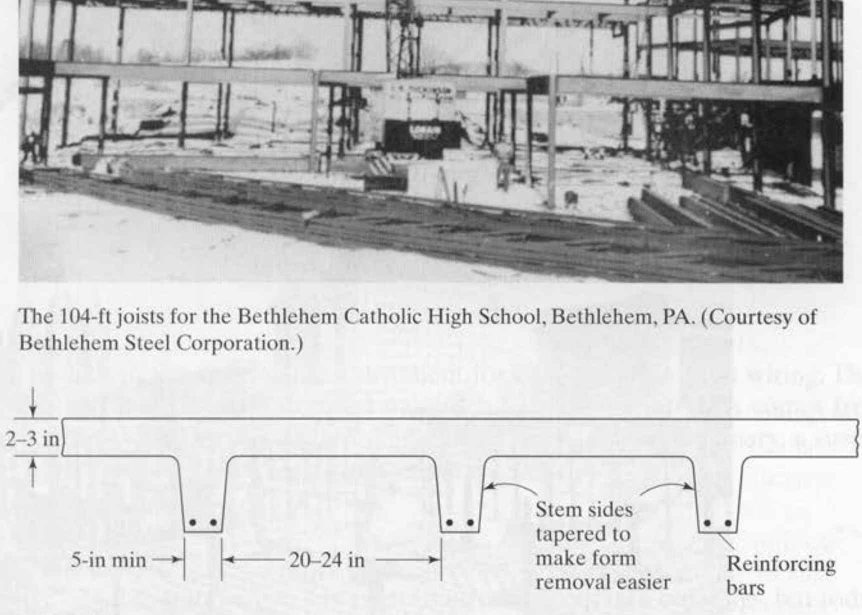


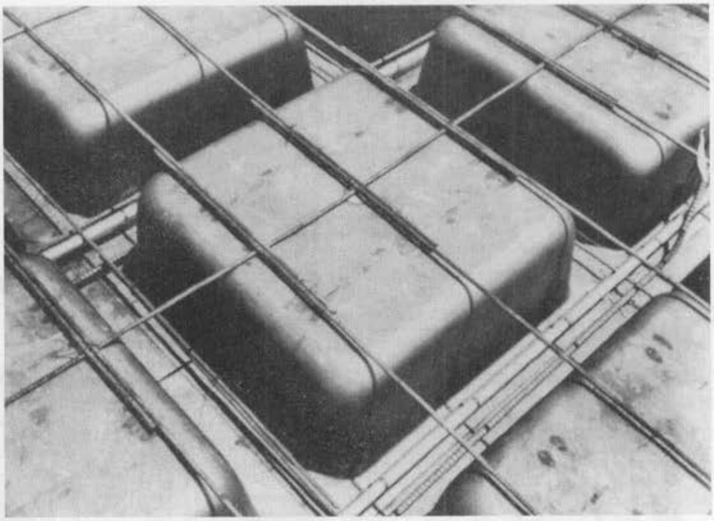
FIGURE 19.9

Concrete-pan floor.

Two-way construction is available—that is, with ribs or stems running in both directions. Pans with closed ends are used, and the result is a waffle-type floor. This type of floor usually is used when the floor panels are square or nearly so. Two-way construction can be obtained for reasonably economical prices and yields a very attractive ceiling below with fairly good acoustical properties.

19.8 STEEL FLOOR DECK

Typical cross sections of steel-decking floors are shown in Fig. 19.10; several other variations are available. *Today, formed steel decking with a concrete topping is by far the most common type of floor system used for office and apartment buildings*. It also is popular for hotels and other buildings where the loads are not very large.



One-piece metal dome pans. (Courtesy of Gateway Erectors, Inc.)

Temple Plaza parking facility, Salt Lake City, UT. (Courtesy of Ceco Steel Products Corporation.)

A particular advantage of steel-decking floors is that the decking immediately forms a working platform. The corrugated steel floor decks are quite strong. Due to the strength of the decking, the concrete does not have to be particularly strong, which permits the use of lightweight concrete, often as thin as 2 or 2 1/2 in, depending on the space of the supporting beams or joists.

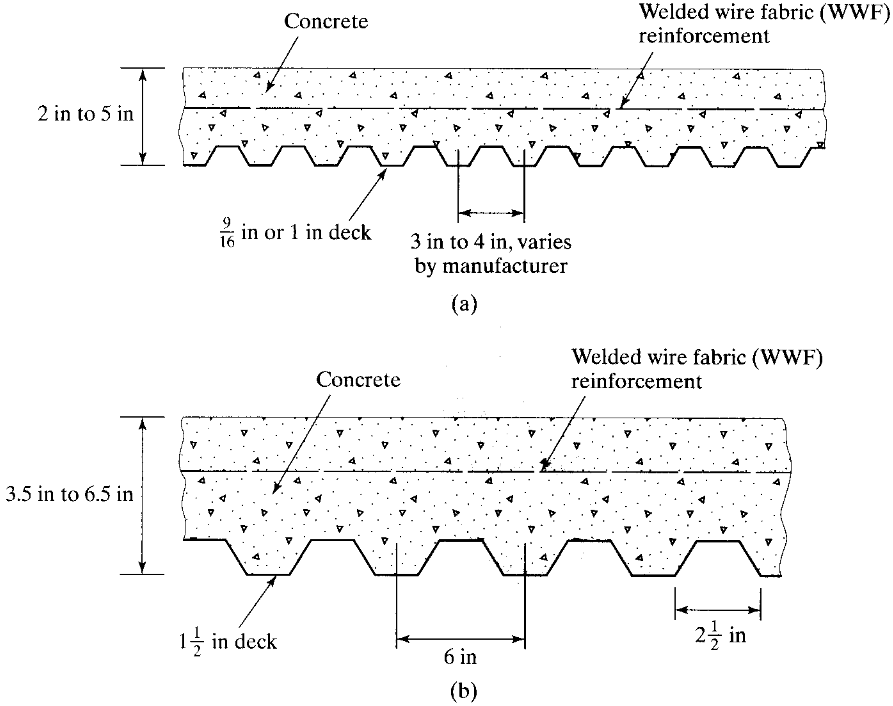


FIGURE 19.10

Steel floor deck systems.

The cells in the decking are convenient for conduits, pipes, and wiring. The steel usually is galvanized, and if exposed underneath, it can be left as it comes from the manufacturer or painted as desired. Should fire resistance be necessary, a suspended ceiling with metal lath and plaster may be used.

19.9 FLAT SLAB FLOORS

Formerly, flat slab floors were limited to reinforced-concrete buildings, but today it is possible to use them in steel-frame buildings. A flat slab is reinforced in two or more directions and transfers its loads to the supporting columns without the use of beams and girders protruding below. The supporting concrete beams and girders are made so wide that they are the same depth as the slab.

Flat slabs are of great value when the panels are approximately square, when more headroom is desired than is provided with the normal beam and girder floors, when heavy loads are anticipated, and when we want to place the windows as near to the tops of the walls as possible. Another advantage is the flat ceiling produced for the floor below. Although the large amounts of reinforcing steel required increased costs, the simple formwork cuts expenses decidedly. The significance of simple formwork will be understood when it is realized that over one-half of the cost of the average poured reinforced-concrete floor slab is in the formwork.

For some reinforced-concrete frame buildings with flat slab floors, it is necessary to flare out the tops of the columns, forming column capitals, and perhaps thicken the slab around the column with the so-called *drop panels*. These items, which are shown in

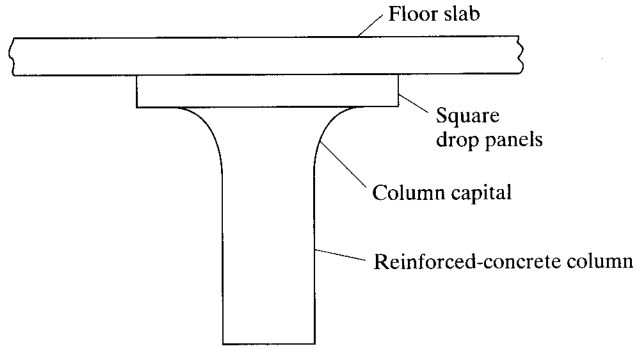


FIGURE 19.11

A flat slab floor for a reinforced-concrete building.

Fig. 19.11, may be necessary to prevent shear or punching failures in the slab around the column.

It is possible in steel-frame buildings to use short steel cantilever beams connected to the steel columns and embedded in the slabs. These beams serve the purposes of the flared columns and drop panels in ordinary flat slab construction. This arrangement often is called a *steel grillage* or *column head.*

The flat slab is not a very satisfactory type of floor system for the usual tall building where lateral forces (wind or earthquake) are appreciable, because protruding beams and girders are desirable to serve as part of the lateral bracing system.

19.10 PRECAST CONCRETE FLOORS

Precast concrete sections are more commonly associated with roofs than they are with floor slabs, but their use for floors is increasing. They are quickly erected and reduce the need for formwork. Lightweight aggregates are often used in the concrete, making the sections light and easy to handle. Some of the aggregates used make the slabs nailable and easily cut and fitted on the job. For floor slabs with their fairly heavy loads, the aggregates should be of a quality that will not greatly reduce the strength of the resulting concrete.

The reader again is referred to *Sweet's Catalog File.* In these catalogs, a great amount of information is available on the various types of precast floor slabs on the market today. Some of the common types available are listed here, and a cross section of each type mentioned is presented in Fig. 19.12. Due to slight variations in the upper

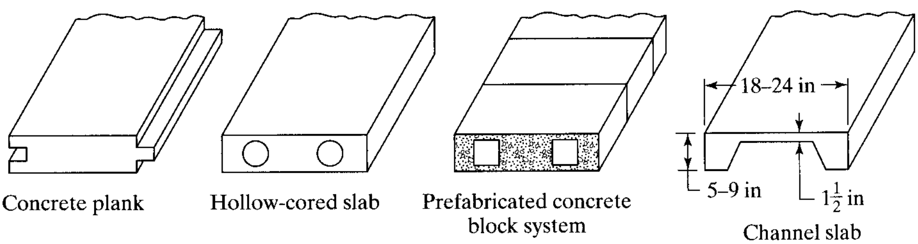
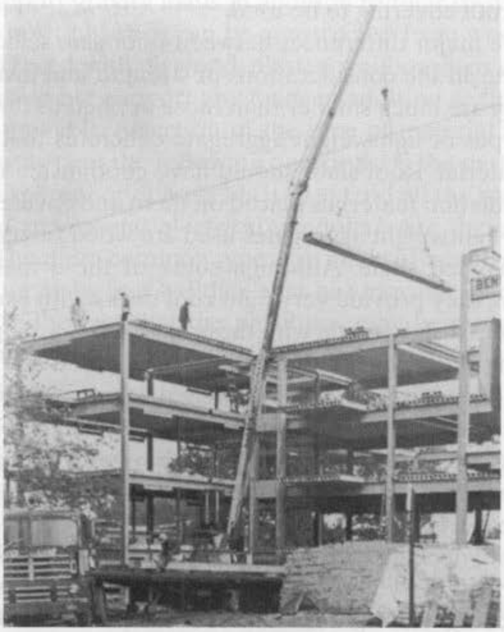


FIGURE 19.12

Precast concrete roof and floor slabs.

101 Hudson St., Jersey City, NJ. (Courtesy of Owen Steel Company, Inc.)



Interns' living quarters, Good Samaritan Hospital, Dayton, OH. (Courtesy of Flexicore Company, Inc.)

surfaces of precast sections, it usually is necessary to use a mortar topping of 1 to 2 in before asphalt tile or other floor coverings can be installed.

1. *Precast concrete planks* are roughly 2 to 3 in thick and 12 to 24 in wide and are placed on joists up to 5 or 6 ft on center. They usually have tongue-and-groove edges.
2. *Hollow-cored slabs* are sections roughly 6 to 8 in deep and 12 to 18 in wide and have hollow, perhaps circular, cores formed in a longitudinal direction, thus reducing their weights by approximately 50 percent (as compared with solid slabs of the same dimensions). These sections, which can be used for spans of roughly 10 to 25 ft, may be prestressed.
3. *Prefabricated concrete block systems* are slabs made by tying together precast concrete blocks with steel rods (may also be prestressed). These seldom-used floors vary from 4 to 8 in deep and can be used for spans of roughly 8 to 32 ft.
4. *Channel slabs* are used for spans of approximately 8 to 24 ft. Rough dimensions of these slabs are given in Fig. 19.12.

19.11 TYPES OF ROOF CONSTRUCTION

The types of roof construction commonly used for steel-frame buildings include concrete slabs on open-web joists, steel roof deck, and various types of precast concrete slabs. For industrial buildings, the roof system in predominant use today involves the use of a cold-rolled steel deck. In addition, the other types mentioned in the preceding sections on floor types are occasionally used, but they usually are unable to compete economically. Among the factors to be considered in selecting the specific types of roof construction are strength, weight, span, insulation, acoustics, appearance below, and type of roof covering to be used.

The major differences between floor-slab selection and roof-slab selection probably occur in the considerations of strength and insulation. The loads applied to roofs generally are much smaller than those applied to floor slabs, thus permitting the use of many types of lightweight aggregate concretes that may be appreciably weaker than floor material. Roof slabs should have good insulation properties, or they will have to have insulation materials placed on them and covered by the roofing. Among the many types of lightweight aggregates used are wood fibers, zonolite, foams, sawdust, gypsum, and expanded shale. Although some of these materials decidedly reduce concrete strengths, they provide very light roof decks with excellent insulating properties.

Precast slabs made with these aggregates are light, are quickly erected, have good insulating properties, and usually can be sawed and nailed. For poured concrete slabs on open-web joists, several lightweight aggregates work very well (zonolite, foams, gypsum, etc.), and the resulting concrete easily can be pumped up to the roofs, thereby facilitating construction. By replacing the aggregates with certain foams, concrete can be made so light it will float in water (for a while). Needless to say, the strength of the resulting concrete is quite low.

Steel decking with similar thin slabs of lightweight and insulating concrete placed on top make very good economical roof systems. A competitive variation consists of steel decking with rigid insulation board placed on top, followed by the regular roofing material. The other concrete-slab types are hard pressed to compete economically with these types, for lightly loaded roofs. Other types of poured concrete decks require much more labor.

19.12 EXTERIOR WALLS AND INTERIOR PARTITIONS

19.12.1 Exterior Walls

The purposes of exterior walls are to provide resistance to atmospheric conditions, including insulation against heat and cold; satisfactory sound-absorption and lightrefraction characteristics; sufficient strength; and acceptable fire ratings. They also should look good and yet be reasonably economical.

For many years, exterior walls were constructed of some type of masonry, glass, or corrugated sheeting. Recently, however, the number of satisfactory materials available for exterior walls has increased tremendously. Precast concrete panels, insulated metal sheeting, and many other prefabricated units are commonly used today. Of increasing popularity are the light prefabricated sandwich panels consisting of three layers. The exterior surface is made from aluminum, stainless steel, ceramics, plastics, and other materials. The center of the panel consists of some type of insulating material, such as fiberglass or fiberboard, while the interior surface is made from metal, plaster, masonry, or some other attractive material.

19.12.2 Interior Partitions

The main purpose of interior partitions is to divide the inside space of a building into rooms. Partitions are selected for appearance, fire rating, weight, and acoustical properties. Partitions may be loadbearing or nonloadbearing.

Bearing partitions support gravity loads in addition to their own weights and are thus permanently fixed in position. They can be constructed from wood or steel studs or from masonry units and faced with plywood, plaster, wallboard, or other material.

Nonbearing partitions do not support any loads in addition to their own weights, and thus may be fixed or movable. Selection of the type of material to be used for a partition is based on the answers to the following questions: Is the partition to be fixed or movable? Is it to be transparent or opaque? Is it to extend all the way to the ceiling? Is it to be used to conceal piping and electrical conduits? Are there fire-rating and acoustical requirements? The more common types are made of metal, masonry, or concrete. For design of the floor slabs in a building that has movable partitions, some allowance should be given to the fact that the partitions may be moved from time to time. Probably the most common practice is to increase the floor-design live load by 15 or 20 psf over the entire floor area.

19.13 FIREPROOFING OF STRUCTURAL STEEL

Although structural steel members are incombustible, their strength is tremendously reduced at temperatures normally reached in fires when the other materials of a building burn. Many disastrous fires have occurred in empty buildings where the only fuel was the buildings themselves. Steel is an excellent heat conductor, and nonfireproofed steel members may transmit enough heat from one burning compartment of a building to ignite materials with which they are in contact in adjoining sections of the building.

The fire resistance of structural steel members can be greatly increased by coating them with fire-protective covers such as concrete, gypsum, mineral fiber sprays, special paints, and other materials. The thickness and kind of fireproofing used depends on the type of structure, the degree of fire hazard, and economics.

In the past, concrete was commonly used for fire protection. Although concrete is not a particularly good insulation material, it is very satisfactory when applied in thicknesses of 1 1/2 to 2 in or more, due to its mass. Furthermore, the water in concrete (16 to 20 percent when fully hydrated) improves its fireproofing qualities appreciably. The boiling off of the water from the concrete requires a great deal of heat. As the water or steam escapes, it greatly reduces the concrete temperature. This is identical to the behavior of steel steam boilers, where the steel temperature is held to maximum values of only a few hundred degrees Fahrenheit due to the escape of the steam. It is true, however, that in very intense fires the boiling off of the water may cause severe cracking and spalling of the concrete.

Although concrete is an everyday construction material, and in mass it is a quite satisfactory fireproofing material, its installation cost is extremely high and its weight is large. As a result, for most steel construction, spray-on fireproofing materials have almost completely replaced concrete.

The spray-on materials usually consist of either mineral fibers or cementious fireproofing materials. The mineral fibers formerly used were made of asbestos. Due to the health hazards associated with this material, its use has been discontinued, and other fibers now are used by manufacturers. The cementious fireproofing materials are composed of gypsum, perlite, vermiculite, and others. Sometimes, when plastered ceilings are required in a building, it is possible to hang the ceilings and light-gage furring channels by wires from the floor systems above and to use the plaster as the fireproofing.

The cost of fireproofing structural steel buildings is high and hurts steel in its economic competition with other materials. As a result, a great deal of research is being conducted by the steel industry on imaginative new fireproofing methods. Among these ideas is the coating of steel members with expansive and insulative paints. When heated to certain temperatures, these paints will char, foam, and expand, forming an insulative shield around the members. These swelling, or extumescent, paints are very expensive.

Other techniques involve the isolation of some steel members outside of the building, where they will not be subject to damaging fire exposure, and the circulation of liquid coolants inside box- or tube-shaped members in the building. It is probable that major advances will be made with these and other fireproofing methods in the near future.[2]

19.14 INTRODUCTION TO HIGH-RISE BUILDINGS

In this introductory text, we do not discuss the design of multistory buildings in detail, but the material of the next few sections gives the student a general idea of the problems involved in the design of such buildings, without presenting elaborate design examples.

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[2]W. A. Rains, "A New Era in Fire Protective Coatings for Steel," *Civil Engineering* (New York: ASCE, September 1976), pp. 80–83.

Office buildings, hotels, apartment houses, and other buildings of many stories are quite common in the United States, and the trend is toward an even larger number of tall buildings in the future. Available land for building in our heavily populated cities is becoming scarcer and scarcer, and costs are becoming higher and higher. Tall buildings require a smaller amount of this expensive land to provide required floor space. Other factors contributing to the increased number of multistory buildings are new and better materials and construction techniques.

On the other hand, several factors that may limit the heights to which buildings will be erected in the future include the following:

1. Certain city building codes prescribe maximum heights for buildings.
2. Foundation conditions may not be satisfactory for supporting buildings of many stories.
3. Floor space may not be rentable above a certain height. Someone will always be available to rent the top floor or two of a 250-story building, but floors numbered 100 through 248 may not be so easy to rent.
4. There are several cost factors that tend to increase with taller buildings. Among these factors are elevators, plumbing, heating and air-conditioning, glazing, exterior walls, and wiring.

Whether a multistory building is used for an office building, a hospital, a school, an apartment house, or something else, the problems of design generally are the same. The construction usually is of the skeleton type in which the loads are transmitted to the foundation by a framework of steel beams and columns. The floor slabs, partitions, and exterior walls all are supported by the frame. This type of framing, which can be erected to tremendous heights, may also be referred to as *beam-and-column* construction. Bearing-wall construction is not often used for buildings of more than a few stories, although it has on occasion been used for buildings up to 20 or 25 stories. The columns of a skeleton frame usually are spaced 20, 25, or 30 ft on centers, with beams and girders framing into them from both directions. (See Fig. 19.2.) On some of the floors, however, it may be necessary to have much larger open areas between columns for dining rooms, ballrooms, and so on. For such cases, very large beams (perhaps built-up I girders) are needed to support column loads for many floors above.

It usually is necessary in multistory buildings to fireproof the members of the frame with concrete, gypsum, or some other material. The exterior walls often are constructed with concrete or masonry units, although an increasing number of modern buildings are erected with large areas of glass in the exterior walls.

For these tall and heavy buildings, the usual spread footings may not be sufficient to support the loads. If the bearing strength of the soil is high, steel grillage footings may be sufficient; for poor soil conditions, pile or pier foundations may be necessary.

For multistory buildings, the beam-to-column systems are superimposed on top of each other, story by story or tier by tier. The column sections can be fabricated for one, two, or more floors, with the two-story lengths probably being the most common. Theoretically, column sizes can be changed at each floor level, but the costs of the splices involved usually would more than cancel any savings in column weights. Columns of three or more stories in height are difficult to erect. The two-story heights work out very well most of the time.

19.15 DISCUSSION OF LATERAL FORCES

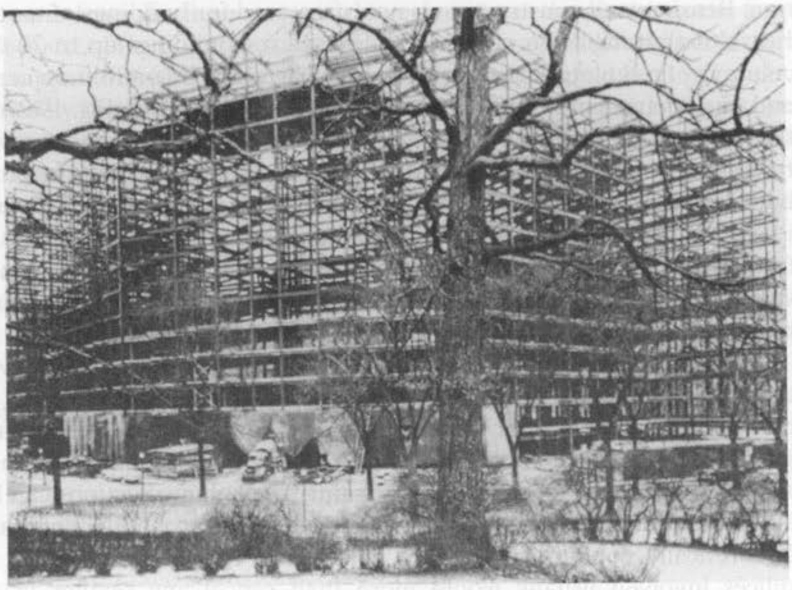
For tall buildings, lateral forces as well as gravity forces must be considered. High wind pressures on the sides of tall buildings produce overturning moments. These moments probably are resisted without difficulty by the axial strengths of the columns, but the horizontal shears produced on each level may be of such magnitude as to require the use of special bracing or moment-resisting connections.

Unless they are fractured, the floors and walls provide a large part of the lateral stiffness of tall buildings. Although the amount of such resistance may be several times that provided by the lateral bracing, it is difficult to estimate and may not be reliable. Today, so many modern buildings have light, movable interior partitions, glass exterior walls, and lightweight floors, that only the steel frame should be assumed to provide the required lateral stiffness.

Not only must a building be sufficiently braced laterally to prevent architectual failure, but it must also be prevented from deflecting so much as to damage its various architectural parts. Another item of importance is the provision of sufficient bracing to give the occupants a feeling of safety. This is commonly referred to as the serviceability limit state. They might not have this feeling of safety in tall buildings that have a great deal of lateral movement in times of high winds. There have actually been tales of occupants of the upper floors of tall buildings complaining of seasickness on very windy days.

The horizontal deflection of a multistory building due to wind or seismic loading is called *drift*. It is represented by Δ in Fig. 19.13. Drift is measured by the *drift index*, Δ/h, where h is the height or distance to the ground.

The usual practice in the design of multistory steel buildings is to provide a structure with sufficient lateral stiffness to keep the drift index between about 0.0015 and 0.0030 radians for the worst storms that occur in a period of approximately 10 years.



Broadview Apartments, Baltimore, MD. (Courtesy of Lincoln Electric Company.)

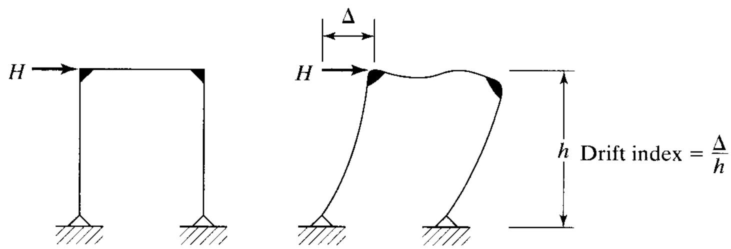


FIGURE 19.13
Lateral drift.

These so-called 10-year storms might have wind in the range of about 90 mph, depending on location and weather records. It is felt that when the index does not exceed these values, the users of the building will be reasonably comfortable.

In addition, multistory buildings should be designed to withstand 50-year storms safely. For such cases, the drift index will be larger than the range mentioned, with the result that the occupants will suffer some discomfort. The 1450-ft twin towers of the World Trade Center in New York City, which were destroyed in 2001, theoretically would deflect or sway about 3 ft in 10-year storms (drift index = 0.0021), while in hurricane winds they theoretically would sway about 7 ft (drift index = 0.0048).

Most buildings can be designed, with little extra expense, to withstand the forces caused during an earthquake of fairly severe intensity. On the other hand, earthquakes during recent years have clearly shown that the average building not designed for earthquake forces can be destroyed by earthquakes which are not particularly severe. The usual practice is to design buildings for additional lateral loads (representing the estimate of the earthquake forces) that are equal to some percentage of the weight of the building and its contents.

The lateral forces require the use of more steel, even though load factors are reduced and safety factors increased for wind and earthquake forces. Additional steel will be used for bracing or moment-resisting connections, as described in the next section.

19.16 TYPES OF LATERAL BRACING

A steel building frame with no lateral bracing is represented in Fig. 19.14(a). Should the beams and columns shown be connected with the standard ("simple beam") connections, the frame would have little resistance to the lateral forces shown. Assuming that the joints act as frictionless pins, the frame would be laterally deflected, as shown in Fig. 19.14(b), eventually collapsing because the structure is unstable.

To resist these lateral deflections, the simplest method, from a theoretical standpoint, is the insertion of full diagonal bracing, as shown in Fig. 19.14(c). From a practical standpoint, however, the student can easily see that in the average building, full diagonal bracing will often be in the way of doors, windows, and other wall openings. Also, many buildings have movable interior partitions, and the presence of interior cross bracing would greatly reduce this flexibility. Usually, diagonal bracing is convenient in solid walls in and around elevator shafts, stairwells, and other walls in which few or no openings are planned.

Another method of providing resistance to lateral forces involves the use of moment-resisting connections, as illustrated in Fig. 19.15. This *moment frame* may be

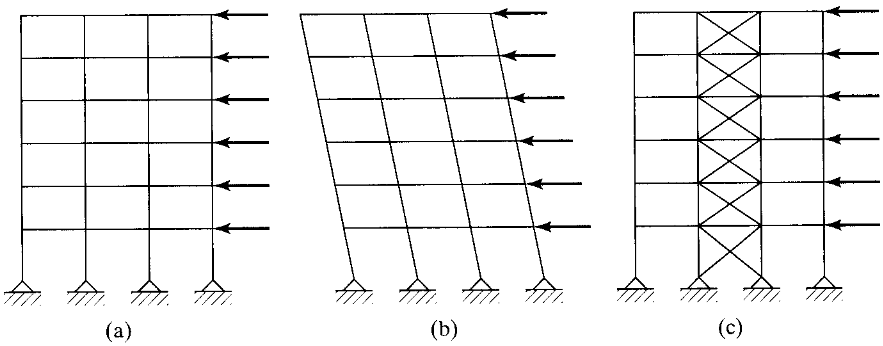


FIGURE 19.14

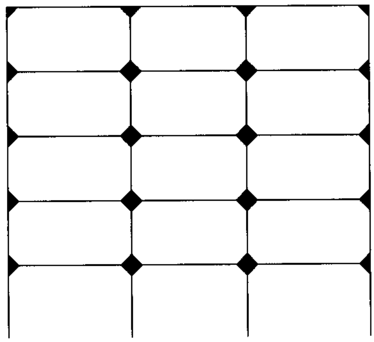


FIGURE 19.15
Moment frame

used economically to provide lateral bracing for lower-rise buildings. As buildings become taller, however, moment frame is not very economical, nor is it very satisfactory in limiting lateral deflections.

Some ways in which we can transmit lateral forces to the ground for buildings in the 20- to 60-story range are shown in Fig. 19.16. The X bracing system of part (a) works very well, except that it might get in the way. Furthermore, as compared with the systems shown in Fig. 19.16(b) and (c), the floor beams have longer spans and will have to be larger.

The K bracing system (b) provides more freedom for the placing of openings than does the full X bracing. K braces are connected at midspan, with the result that the floor beams will have smaller moments. The K bracing system (like the knee bracing system) uses less material than does X bracing. If we need more room than is available with the K bracing system, we can go to the full-story knee bracing system shown in Fig. 19.16(c).[3]

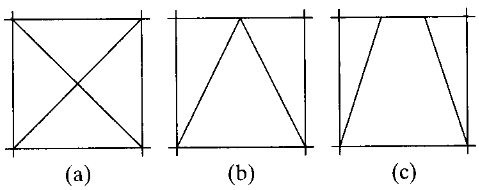


FIGURE 19.16
(a) X brace. (b) K brace.
(c) Full-story knee brace.

[3]E. H. Gaylord, Jr., and C. N. Gaylord, *Structural Engineering Handbook*, 2d ed. (New York: McGraw-Hill, 1979), pp. 19–77 to 19–111.

Construction of the twin Petronas Towers in Kuala Lumpur, Malaysia. (Courtesy of Corbis/Sergio Dolzntes.)

In the usual building, the floor system (beams and slabs) is assumed to be rigid in the horizontal plane, and the lateral loads are assumed to be concentrated at the floor levels. Floor slabs and girders acting together provide considerable resistance to lateral forces. Investigation of steel buildings that have withstood high wind forces has shown that the floor slabs distribute the lateral forces so that all of the columns on a particular floor have essentially equal deflections, as long as twisting of the structure does not occur. When rigid floors are present, they spread the lateral shears to the columns or walls in the building. When lateral forces are particularly large, as in very tall buildings or where seismic forces are being considered, certain specially designed walls may be used to resist large parts of the lateral forces. These walls are called *shear walls*.

It is not necessary to brace every panel in a building. Usually, the bracing can be placed in the outside walls with less interference than in the inside walls, where movable partitions may be desired. Probably, the bracing of the outside panels alone is not enough, and some interior panels may need to be braced. It is assumed that the floors and beams are sufficiently rigid to transfer the lateral forces to the braced panels. Three possible arrangements of braced panels are shown in Fig. 19.17 for lateral forces in one direction. A symmetrical arrangement probably is desirable to prevent uneven lateral deflection, and thus torsion, in the building.

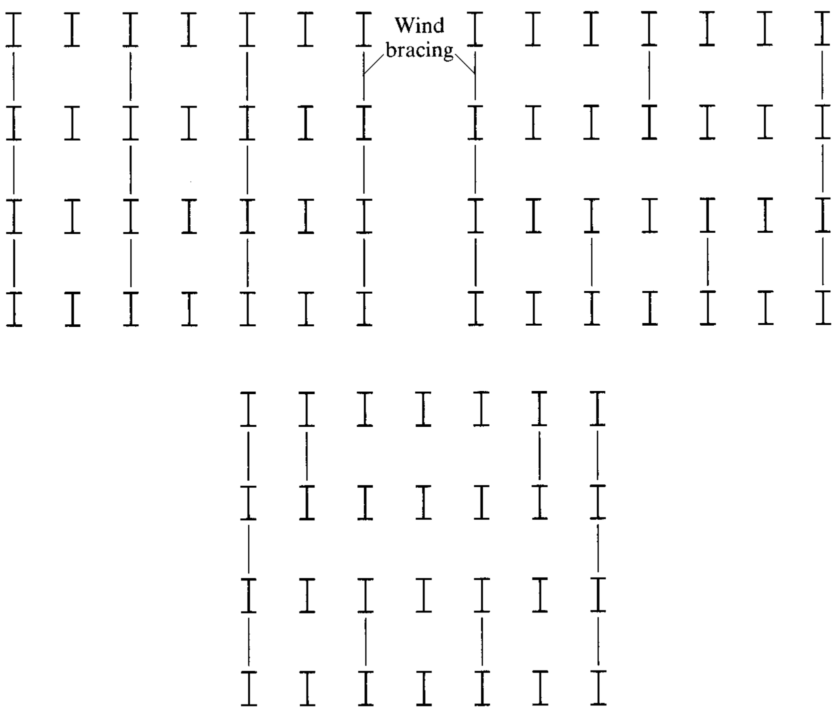


FIGURE 19.17

Possible locations for lateral bracing.

Bracing around elevator shafts and stairwells usually is permissible, while for other locations it often will interfere with windows, doors, movable partitions, glass exterior walls, open spaces, and so on. Should bracing be used around an elevator shaft, as shown in Fig. 19.18(a), and should calculations show that the drift index is too large, it may be possible to use a *hat truss* on the top floor, as shown in part (b). Such a truss will substantially reduce drift. If a hat truss is not feasible because of interference with other items, one or more *belt trusses* may be possible, as shown in Fig. 19.18(c). A belt truss will appreciably reduce drift, although not as much as a hat truss.

The bracing systems described so far are not efficient for buildings with more than approximately 60 stories. These taller buildings have very large lateral wind and, perhaps, seismic forces applied hundreds of feet above the ground. The designer needs to develop a system that will resist these loads without failure and in such a manner that lateral deflections are not so great as to frighten the occupants. The bracing methods used for such buildings usually are based on a tubular frame concept.

With the tubular system, a vertical tubular cantilever frame is created, much like the tube of Fig. 19.19. The tube consists of the building columns and girders in both the longitudinal and transverse directions of the building. The idea is to create a tube that will act like a continuous chimney or stack.

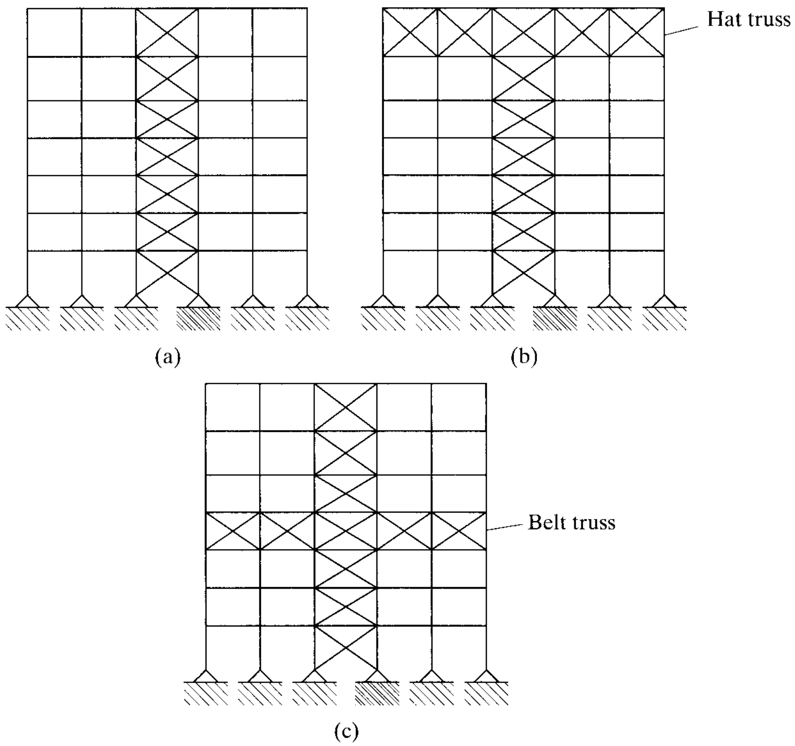


FIGURE 19.18
Bracing systems.

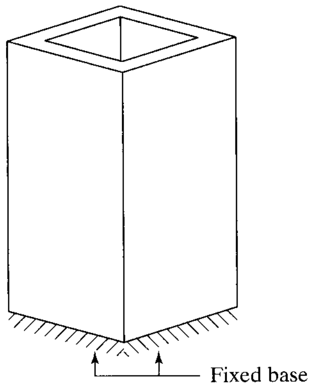


FIGURE 19.19
Solid-wall tube.

To build the tube, the exterior columns are spaced close together—from 3 or 4 ft up to 10 or 12 ft on center. They are connected with spandrel beams at the floor levels, as shown in Fig. 19.20(a).

Further improvements of the tubular system can be made by cross bracing the frame with X bracing over many stories, as illustrated in Fig. 19.20(b). This latter system is very stiff and efficient, and is very helpful in distributing gravity loads rather uniformly over the exterior columns.

Another variation on this system is the tube-within-a-tube system. Interior columns and girders are used to create additional tubes. In tall buildings, it is common to group elevator shafts, utility shafts, and stairs, and these systems can include shear walls and braced bents.[4]

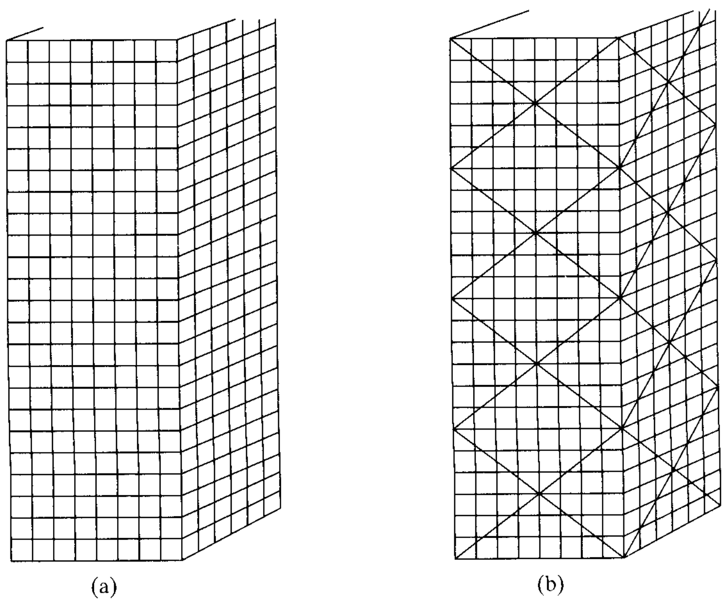


FIGURE 19.20
(a) Tubular frame, (b) Cross-braced tubular frame.

[4]R. N. White, P. Gergely, and R. G. Sexsmith, *Structural Engineering*, vol. 3 (New York: Wiley, 1974), pp. 537–546.

19.17 ANALYSIS OF BUILDINGS WITH DIAGONAL WIND BRACING FOR LATERAL FORCES

Full diagonal cross bracing has been described as being an economical type of wind bracing for tall buildings. Where this type of bracing cannot practically be used due to interference with windows, doors, or movable partitions, moment-resisting brackets often are used. However, should very tall narrow buildings (with height–least-width ratios of 5.0 or greater) be constructed, lateral deflections may become a problem with moment-resisting joints. The joints can satisfactorily resist the moments, but the deflections may be excessive.

Should maximum wind deflections be kept under 0.002 times the building height, there is little chance of injury to the building, according to ASCE Subcommittee 31.[5] The deflection is to be computed neglecting any resistance supplied by floors and walls. To keep lateral deflections within this range, it is necessary to use deep knee bracing, K bracing, or full diagonal cross bracing when the height–least-width ratio is about 5.0; and for greater values of the ratio, full diagonal cross bracing or some other method such as the tubular frame is needed.[6]

The student often may see bracing used in buildings for which he or she would think wind stresses were negligible. Such bracing stiffens up a building appreciably and serves the useful purpose of plumbing the steel frame during erection. Before bracing is installed, the members of a steel building frame may be twisted in many directions. Connecting the diagonal bracing should pull members into their proper positions.

When diagonal cross bracing is used, it is desirable to introduce initial tension into the diagonals. This prestressing will make the building frame tight and reduce its lateral deflection. Furthermore, these light diagonal members can support compressive stress due to their pretensioning. Since the members can resist compression, the horizontal shear to be resisted will be assumed to split equally between the two diagonals. For buildings with several bay widths, equal shear distribution usually is assumed for each bay.

Should the diagonals not be initially tightened, rather stiff sections should be used so that they will be able to resist appreciable compressive forces. The direct axial forces in the girders and columns can be found for each joint from the shear forces assumed in the diagonals. Usually, the girder axial forces so computed are too small to consider, but the values for columns can be quite important. The taller the building becomes, the more critical are the column axial forces caused by lateral forces.

For types of bracing other than cross bracing, similar assumptions can be made for analysis. The student is referred to pages 333–339 of *Theory of Modern Steel Structures*, by L. E. Grinter (New York: Macmillan, 1962), for a discussion of this subject.

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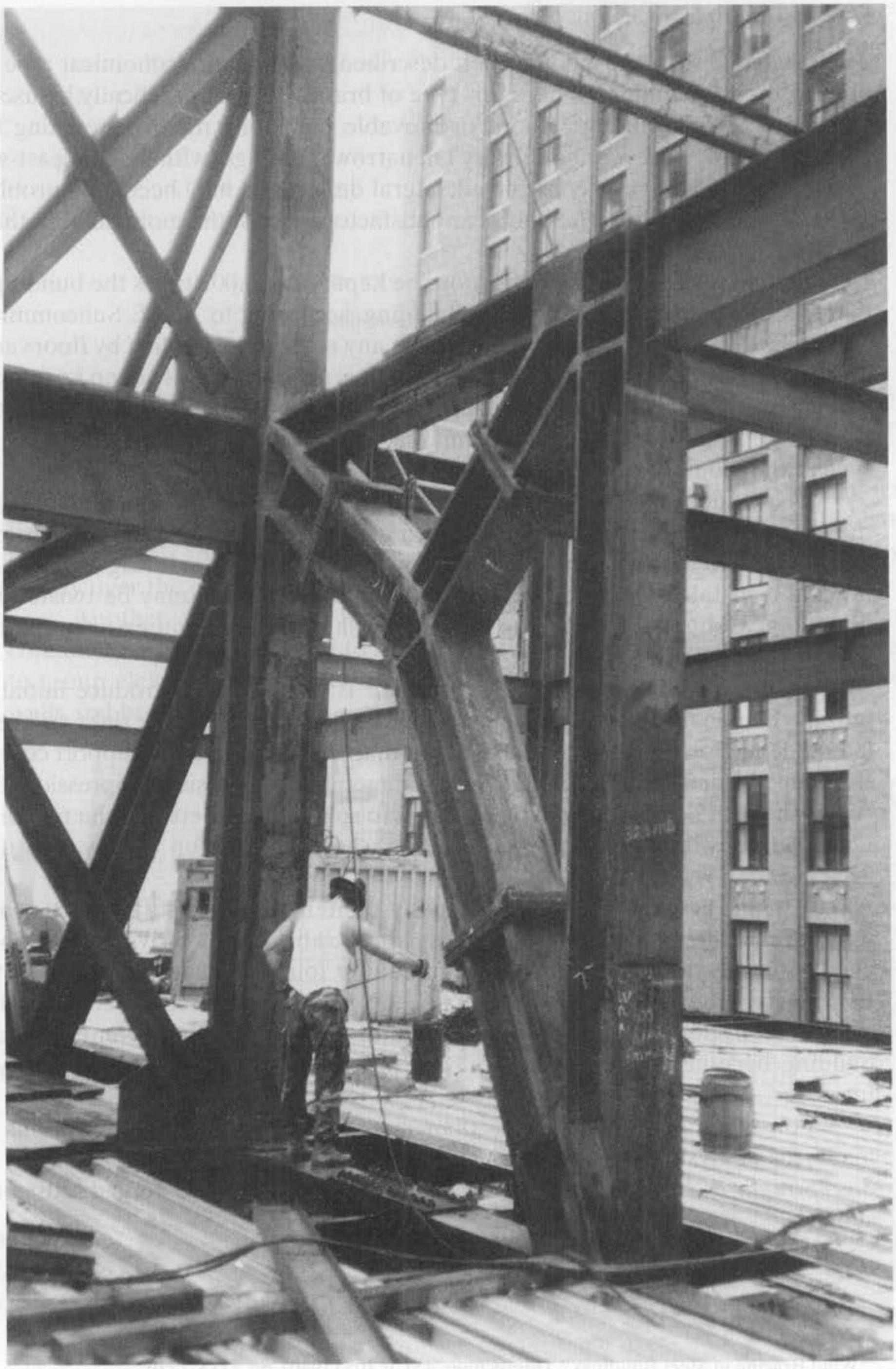
[5]"Wind Bracing in Steel Buildings," *Transactions ASCE* 105 (1940), pp. 1713–1739.

[6]L. E. Grinter, *Theory of Modern Steel Structures* (New York: Macmillan, 1962), p. 326.



Transfer truss, 150 Federal Street, Boston, MA. (Courtesy of Owen Steel Company, Inc.)

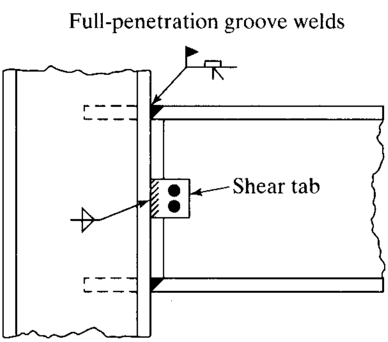
19.18 MOMENT-RESISTING JOINTS

For a large percentage of buildings under eight to ten stories, the beams and girders are connected to each other and to the columns with simple end-framed connections of the types described in Chapter 15. As buildings become taller, it is absolutely necessary to use a definite wind-bracing system or moment-resisting joints. Moment-resisting joints also may be used in lower buildings where it is desired to take advantage of continuity and the consequent smaller beam sizes and depths and shallower floor construction. Moment-resisting brackets also may be necessary in some locations for loads that are applied eccentrically to columns.

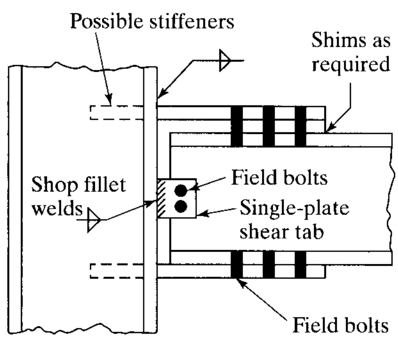
Two types of moment-resisting connections that may be used as wind bracing are shown in Fig. 19.21. The design of connections of these types was also presented in Chapter 14. The average design company through the years probably will develop a file of moment-resisting connections from their previous designs. When they have a wind moment of a certain value, they will refer to their file and select one of their former designs that would provide the required moment resistance.

These are the two most commonly used moment-resisting connections today. Most fabricators select the type shown in 19.21(a) as being the most economical. Single-plate (or shear tab) connectors are shown, but framing angles may be used instead. Column stiffeners (shown as dashed lines in the figures) sometimes are required by AISC Specification J10—or local flange bending, local web yielding, sidesway buckling of the web, and so on. Stiffeners are a nuisance and cost an appreciable amount of money. As a result, we will, when the specification shows they are needed, try to avoid them by increasing column sizes. (The reader is again reminded of the troubles experienced with this type of connection during the Northridge earthquake in California as described in Section 15.11 of this text.)

A very satisfactory variation of these last two connections involves an end plate, as shown in Fig. 12-6 in Part 12 of the AISC Manual. This type of connection may, however, be used only for static load situations.



(a) Field-welded moment connection



(b) Shop-welded, field-bolted moment connection

FIGURE 19.21

Popular moment-resisting connections.

Chase Manhattan Bank Building, New York City. (Courtesy of Bethlehem Steel Corporation.)

19.19 DESIGN OF BUILDINGS FOR GRAVITY LOADS

19.19.1 Simple Framing

If *simple framing* is used, the design of the girders is less complex, because the shears and moments in each girder can be determined by statics. The gravity loads applied to the columns are relatively easy to estimate, but the column moments may be a little more difficult. If the girder reactions on each side of the interior column of Fig. 19.22 are equal, then, theoretically, no moment will be produced in the column at that level. This situation probably is not realistic, however, because it is highly possible for the live load to be applied on one side of the column and not on the other (or at least be unequal in magnitude). The results will be column moments. If the reactions are unequal, the moment produced in the column will equal the difference between the reactions times the distances to the center of gravity of the column.

Exterior columns often may have moments due to spandrel beams opposing the moments caused by the floor loads on the inside of the column. Nevertheless, gravity loads generally will cause the exterior columns to have larger moments than the interior columns.

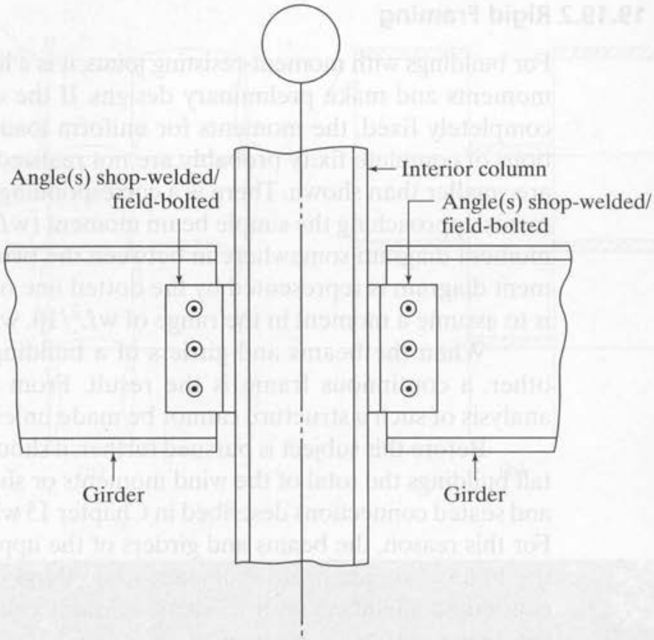


FIGURE 19.22
Simple framing of interior column.

To estimate the moment applied to a column above a certain floor, it is reasonable to assume that the unbalanced moment at that level splits evenly between the columns above and below. In fact, such an assumption is often on the conservative side, as the column below may be larger than the one above.

"Topping out" of Blue Cross–Blue Shield Building in Jacksonville, FL. (Courtesy of Owen Steel Company, Inc.) The Christmas tree is an old north European custom used to ward off evil spirits. It also is used today to show that the steel frame was erected with no lost-time accidents to personnel.

19.19.2 Rigid Framing

For buildings with moment-resisting joints, it is a little more difficult to estimate the girder moments and make preliminary designs. If the ends of each girder are assumed to be completely fixed, the moments for uniform loads are as shown in Fig. 19.23(a). Conditions of complete fixity probably are not realized, with the result that the end moments are smaller than shown. There is a corresponding increase in the positive centerline moments approaching the simple beam moment ($wL^2/8$) shown in Fig. 19.23(b). Probably, a moment diagram somewhere in between the two extremes is more realistic. Such a moment diagram is represented by the dotted line of Fig. 19.23(a). A reasonable procedure is to assume a moment in the range of $wL^2/10$, where L is the clear span.

When the beams and girders of a building frame are rigidly connected to each other, a continuous frame is the result. From a theoretical standpoint, an accurate analysis of such a structure cannot be made unless the entire frame is handled as a unit.

Before this subject is pursued further, it should be realized that in the upper floors of tall buildings the total of the wind moments or shear above will be small, and the framed and seated connections described in Chapter 15 will provide sufficient moment resistance. For this reason, the beams and girders of the upper floors may very well be designed on the basis of simple beam moments, while those of the lower floors may be designed as continuous members with moment-resistant connections due to the larger total of the wind moments or shear above.

From a strictly theoretical viewpoint, there are several live-load conditions that need to be considered to obtain maximum shears and moments at various points in a continuous structure. For the building frame shown in Fig. 19.24, it is desired to place live loads to cause maximum positive moment in span AB. A qualitative influence line for positive moment at the centerline of this span is shown in part (a). This influence line shows that, to obtain maximum positive moment at the centerline of span AB, the live loads should be placed as shown in Fig. 19.24(b).

To obtain maximum negative moment at point B, or maximum positive moment in span BC, other loading situations need to be considered. With the availability of computers, more detailed analyses are being done every day. The student can see, however, that unless the designer uses a computer, he or she probably will not have sufficient time to go through all of these theoretical situations. Furthermore, it is doubtful if the accuracy of our analysis methods would justify all of the work anyway. Nonetheless, it often is feasible to take out two stories of the building at a time as a free body and analyze that part by one of the "exact" methods such as the successive correction method of moment distribution.

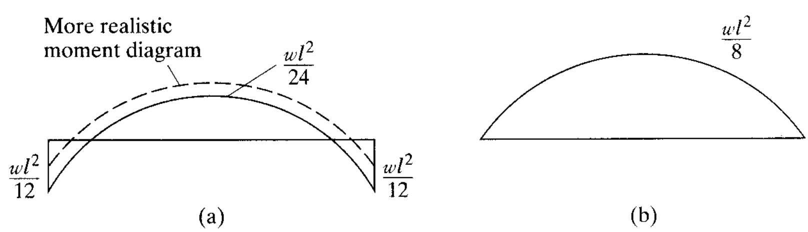


FIGURE 19.23
(a) Fixed-end beam.
(b) Simple beam.

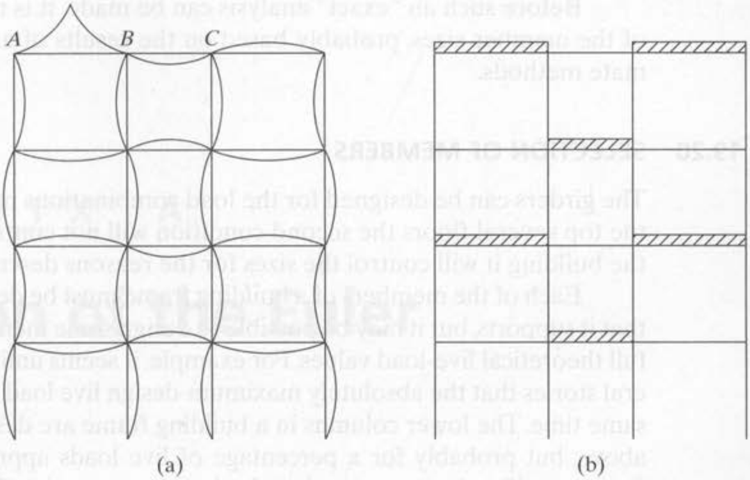
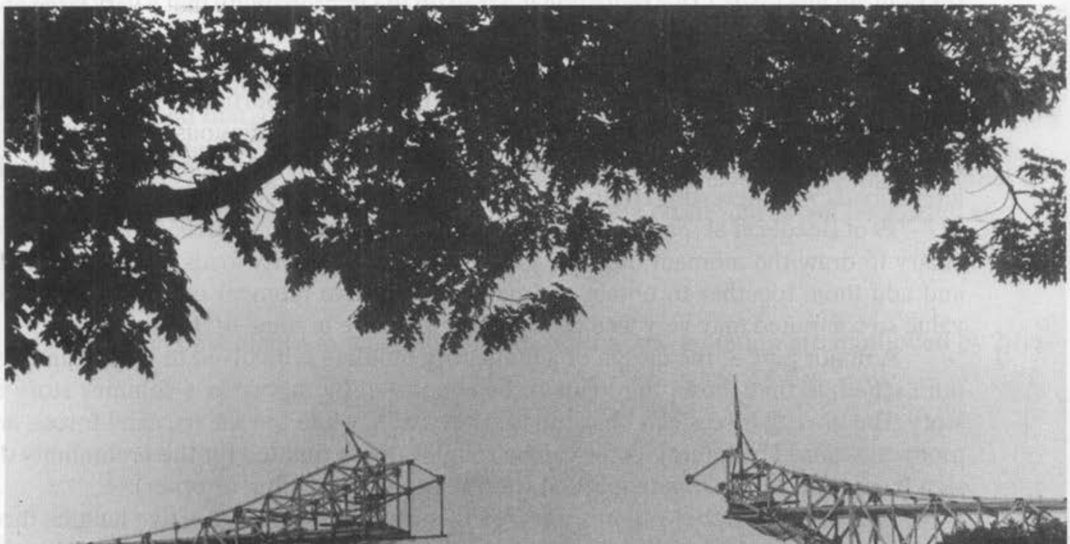


FIGURE 19.24
(a) Influence line for max positive moment at centerline of span AB;
(b) Pattern LL to produce max positive moment in span AB



Construction of bridge spanning the Piscataqua River linking Kittery, Maine, with Portsmouth New Hampshire. (Courtesy of Getty Images/Hilton Archive.)

Before such an "exact" analysis can be made, it is necessary to make an estimate of the member sizes, probably based on the results of analysis by one of the approximate methods.

19.20 SELECTION OF MEMBERS

The girders can be designed for the load combinations mentioned in Section 19.19. For the top several floors the second condition will not control design, but farther down in the building it will control the sizes for the reasons described in the next paragraph.

Each of the members of a building frame must be designed for all of the dead loads that it supports, but it may be possible to design some members for lesser loads than their full theoretical live-load values. For example, it seems unlikely in a building frame of several stories that the absolutely maximum-design live load will occur on every floor at the same time. The lower columns in a building frame are designed for all of the dead loads above, but probably for a percentage of live loads appreciably less than 100 percent. Some specifications require that the beams supporting floor slabs be designed for full dead and live loads, but permit the main girders to be designed under certain conditions for reduced live loads. (This reduction is based on the improbability that a very large area of a floor would be loaded to its full live-load value at any one time.) The ASCE Standard 7 provides some very commonly used reduction expressions.[7]

Should simple framing be used, the girders will be proportioned for simple beam moments plus the moments caused by the lateral loads. For continuous framing, the girders will be proportioned for $wL^2/10$ (for uniform loads) plus the moments caused by the lateral loads. An interesting comparison of designs by the two methods is shown in pages 717–719 of Beedle et al., *Structural Steel Design* (New York: Ronald, 1964). It may be necessary to draw the moment diagram for the two cases (gravity loads and lateral loads) and add them together to obtain the maximum positive moment out in the span. The value so computed may very well control the girder size in some of the members.

A major part of the design of a multistory building is involved in setting up a column schedule that shows the loads to be supported by the various columns story by story. The gravity forces can be estimated very well, while the shears, axial forces, and moments caused by lateral forces can be roughly approximated for the preliminary design from some approximate method (portal, cantilever, factor, or other).

If both axes of the columns are free to sway, the column effective lengths theoretically must be calculated for both axes, as described in Chapter 5. If diagonal bracing is used in one direction, thus preventing sidesway, the K value will be less than 1.0 in that direction. If these frames are braced also in the narrow direction, it is reasonable to use $K = 1.0$ for both axes.

After the sizes of the girders and columns are tentatively selected for a two-story height, an "exact" analysis can be performed and the members redesigned. The two stories taken out for analysis often are referred to as a *tier*. This process can be continued tier by tier down through the building. Many tall buildings have been designed and are performing satisfactorily in which this last step (the two-story "exact" analysis) was omitted.

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[7] *American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures.* ASCE 7-10 (New York: ASCE), Section 4.8.

APPENDIX A

Derivation of the Euler Formula

The Euler formula is derived in this section for a straight, concentrically loaded, homogeneous, long, slender, elastic, and weightless column with rounded ends. It is assumed that this perfect column has been laterally deflected by some means, as shown in Fig. A.1 and that, if the concentric load P were removed, the column would straighten out completely.

The x and y axes are located as shown in the figure. As the bending moment at any point in the column is $-Py$, the equation of the elastic curve can be written as

$$EI\frac{d^2y}{dx^2} = -Py$$

For convenience in integration, both sides of the equation are multiplied by $2dy$:

$$EI2\frac{dy}{dx}d\frac{dy}{dx} = -2Pydy$$

$$EI\left(\frac{dy}{dx}\right)^2 = -Py^2 + C_1$$

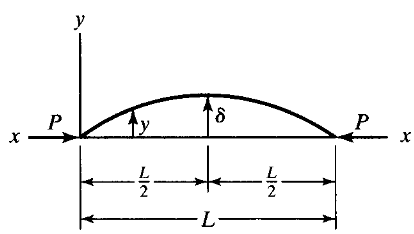

FIGURE A.1

When $y = \delta$, $dy/dx = 0$, and the value of C_1 will equal $P\delta^2$ and

$$EI\left(\frac{dy}{dx}\right)^2 = -Py^2 + P\delta^2$$

The preceding expression is arranged more conveniently as follows:

$$\left(\frac{dy}{dx}\right)^2 = \frac{P}{EI}(\delta^2 - y^2)$$

$$\frac{dy}{dx} = \sqrt{\frac{P}{EI}}\sqrt{\delta^2 - y^2}$$

$$\frac{dy}{\sqrt{\delta^2 - y^2}} = \sqrt{\frac{P}{EI}}dx$$

Integrating this expression, the result is

$$\text{arc sin}\frac{y}{\delta} = \sqrt{\frac{P}{EI}}x + C_2$$

When $x = 0$ and $y = 0$, $C_2 = 0$. The column is bent into the shape of a sine curve expressed by the equation

$$\text{arc sin}\frac{y}{\delta} = \sqrt{\frac{P}{EI}}x$$

When $x = L/2$, $y = \delta$, resulting in

$$\frac{\pi}{2} = \frac{L}{2}\sqrt{\frac{P}{EI}}$$

In this expression, P is the *critical buckling load*, or the maximum load that the column can support before it becomes unstable. Solving for P, we have

$$P = \frac{\pi^2 EI}{L^2}$$

This expression is the Euler formula, but usually it is written in a little different form involving the slenderness ratio. Since $r = \sqrt{I/A}$ and $r^2 = I/A$ and $I = r^2A$, the Euler formula may be written as

$$\frac{P}{A} = \frac{\pi^2 E}{(L/r)^2} = F_e$$

APPENDIX B

Slender Compression Elements

Section B4 of the AISC Specification is concerned with the local buckling of compression elements. In that section, compression elements are classified as non-slender element or slender element sections. As non-slender elements have been discussed previously, this appendix is concerned only with slender elements. A brief summary of the AISC method for determining design stresses for such members is presented in the next few paragraphs.

When b/t ratios exceed the values given in AISC Table B4.1a, those elements will be classified as being slender, and their critical, or F_{cr}, stresses will have to be reduced. The design strength of an axially loaded compression member with slender elements will be reduced by multiplying it by a reduction factor Q.

The value of Q is equal to the product of two reduction factors Q_s and Q_a. Their values are dependent on whether the member consists of stiffened and/or unstiffened elements. Q_a is a reduction factor for slender stiffened compression elements, while Q_s is a reduction factor for slender unstiffened compression elements. Two cases are considered in AISC Specification E7:

1. For members consisting of unstiffened elements only, Q_s is to be determined with the appropriate formulas presented in AISC Section E7.1, and $Q_a = 1.0$.
2. For members consisting of stiffened elements only, $Q_s = 1.0$, and Q_a is to be determined with the formulas presented in AISC Section E7.2.

Equations for computing Q are given in AISC Section E7 for the following types of members: (a) single angles; (b) flanges, angles, and plates projecting from rolled columns or other compression members; (c) flanges, angles, and plates projecting from built-up I-shaped columns or other compression members; and (d) stems of tees. In these equations, the following terms are used:

b = width of unstiffened compression element, as defined in section B4.1, in

t = thickness of unstiffened compression element, in
F_y = specified minimum yield stress, ksi
d = full nominal depth of tee, in

Only the Q_s expressions for case (d), stems of tees, are presented here,

When $\frac{d}{t} \leq 0.75\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \qquad \text{(AISC Equation E7-13)}$$

When $0.75\sqrt{\frac{E}{F_y}} < \frac{d}{t} \leq 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.908 - 1.22\left(\frac{d}{t}\right)\sqrt{\frac{E}{F_y}} \qquad \text{(AISC Equation E7-14)}$$

When $\frac{d}{t} > 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{d}{t}\right)^2} \qquad \text{(AISC Equation E7-15)}$$

In Example B-1, the value of Q_s is computed for a WT section that is used as a compression member. As the member consists only of unstiffened elements, $Q_a = 1.0$. The computed value for Q_s can be checked in the WT shape tables of Part 1 (Table 1-8) of the Manual. In Appendix C, this same WT shape member is further considered as to lateral-torsional buckling, and the Q_s determined here is used there.

Example B-1

A WT10.5 × 31, shown in Fig. B.1, is used as a compression member. Compute Q_s for this member, which is assumed to have an F_y = 50 ksi.

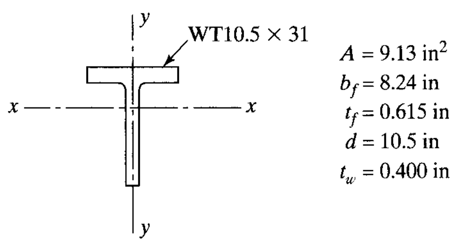

FIGURE B.1

Solution

For tee flange, use AISC Section E7.1a, with

$$\frac{b}{t} = \frac{b_f/2}{t_f} = \frac{8.24/2}{0.615} = 6.70 \leq 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000}{50}} = 13.49$$

$$\therefore Q_s = 1.0 \qquad \text{(AISC Equation E7-4)}$$

For tee stem, use AISC Section E7.1 (d), with

$$\frac{d}{t} = \frac{10.5}{0.400} = 26.25 > 1.03\sqrt{\frac{E}{F_y}} = 1.03\sqrt{\frac{29{,}000}{50}} = 24.81$$

$$\therefore Q_s = \frac{0.69E}{F_y\left(\dfrac{d}{t}\right)^2} = \frac{0.69(29{,}000)}{50(26.25)^2} \qquad \text{(AISC Equation E7-15)}$$

$$Q_s = 0.581 \leftarrow$$

Checks with value in WT shapes tables of AISC (1-8) where $Q_s = 0.581$, for $F_y = 50$ ksi.

If we have slender elements in a compression member, its design compression strength is to be computed as follows:

For $\quad \dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}} \quad \left(\text{or } \dfrac{QF_y}{F_e} \leq 2.25\right)$

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \qquad \text{(AISC Equation E7-2)}$$

For $\quad \dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}} \quad \left(\text{or } \dfrac{QF_y}{F_e} > 2.25\right)$

$$F_{cr} = 0.877F_e \qquad \text{(AISC Equation E7-3)}$$

Where F_e is the elastic buckling stress, calculated using Equations E3-4 and E4-5 for singly symmetric members and $Q = Q_sQ_a$.

A P P E N D I X C

Flexural–Torsional Buckling of Compression Members

Usually, symmetrical members such as W sections are used as columns. Torsion will not occur in such sections if the lines of action of the lateral loads pass through their shear centers. The *shear center* is that point in the cross section of a member through which the resultant of the transverse loads must pass so that no torsion will occur. The calculations necessary to locate shear centers were presented in Chapter 10. The shear centers of the commonly used doubly symmetrical sections occur at their centroids. This is not necessarily the case for other sections, such as channels and angles. Shear center locations for several types of sections are shown in Fig. C.1. Also shown in the figure are the coordinates x_0 and y_0 for the shear center of each section with respect

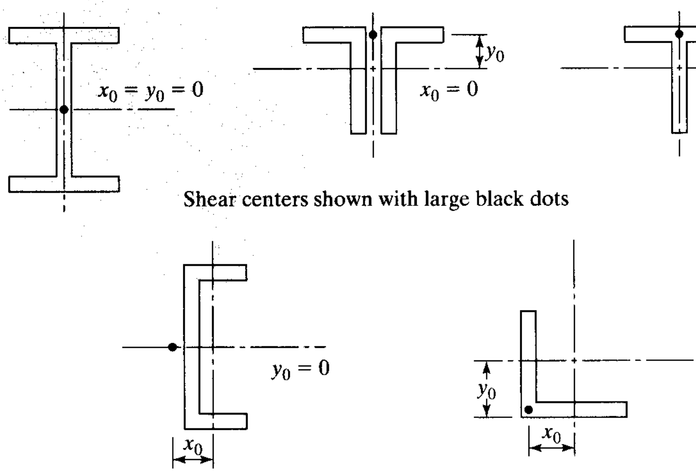

FIGURE C.1

Shear center locations for some common column sections.

to its centroid. These values are needed to solve the flexural-torsional formulas, presented later in this section.

Even though loads pass through shear centers, torsional buckling still may occur. If we load any section through its shear center, no torsion will occur, but we still compute torsional buckling strength for these members—that is, buckling load does not depend on the nature of the axial or transverse loading; rather, it depends on the cross-section properties, column length, and support conditions.

The average designer does not consider the torsional buckling of symmetrical shapes or the flexural-torsional buckling of unsymmetrical shapes. He or she usually thinks that these conditions don't control the critical column loads, or at least that they don't affect them much. If, however, we have unsymmetrical columns or even symmetrical columns made up of thin plates, we will find that torsional buckling or flexural-torsional buckling may significantly reduce column capacities.

In Section E of the AISC Specification, a long list of formulas is presented for computing the flexural-torsional strength of column sections. The values given for column design strengths ($\phi_c P_n$ and P_n/Ω_c values) for double angles, single angles, and tees in Part 4 of the AISC Manual make use of these formulas.

For flexural-torsion, $P_u \leq \phi_c P_n = \phi_c A_g F_{cr}$, with $\phi_c = 0.90$ and F_{cr} to be determined from the formulas to follow from the specification. A list of definitions that are needed for using these formulas also is provided. When the column is defined as a slender-element compression member, the critical stress, F_{cr}, is found from equations in Section E7.

When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}} \quad \left(\text{or } \dfrac{QF_y}{F_e} \leq 2.25\right)$,

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \qquad \text{(AISC Equation E7-2)}$$

and when $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}} \quad \left(\text{or } \dfrac{QF_y}{F_e} > 2.25\right)$,

$$F_{cr} = 0.877F_e, \qquad \text{(AISC Equation E7-3)}$$

When the member meets the width-thickness ratio, λ_r, of AISC Section B4.1, it is classified as a non-slender section. In this case, $Q = 1.0$ and F_{cr} is determined from Equations E3-2 and E3-3 in Section E3.

For either case, F_e, the critical flexural-torsional elastic bucking stress is calculated based on the formulas in Section E4.

For doubly symmetric shapes,

$$F_e = \left[\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right]\frac{1}{I_x + I_y}. \qquad \text{(AISC Equation E4-4)}$$

For singly symmetric shapes where y is the axis of symmetry,

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right]. \qquad \text{(AISC Equation E4-5)}$$

For unsymmetrical sections, F_e is the lowest root of the following cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_0}{\bar{r}_0}\right)^2$$
$$- F_e^2(F_e - F_{ex})\left(\frac{y_0}{\bar{r}_0}\right)^2 = 0 \qquad \text{(AISC Equation E4-6)}$$

The following is a more convenient form of AISC Equation E4-6:

$$HF_e^3 + \left[\frac{1}{\bar{r}_0^2}(y_0F_{ex} + x_0^2F_{ey}) - (F_{ex} + F_{ey} + F_{ez})\right]F_e^2$$
$$+ (F_{ex}F_{ey} + F_{ex}F_{ez} + F_{ey}F_{ez})F_e - F_{ex}F_{ey}F_{ez} = 0$$

Here,

r_o = polar radius of gyration about the shear center (in)
K_z = effective length factor for torsional buckling
G = shear modulus of elasticity of steel = 11,200 ksi
C_w = warping constant (in^6)
J = torsional constant (in^4)

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} \qquad \text{(AISC Equation E4-11)}$$

$$H = 1 - \left(\frac{x_0^2 + y_0^2}{\bar{r}_0^2}\right) \qquad \text{(AISC Equation E4-10)}$$

$$F_{ex} = \frac{\pi^2 E}{(KL/r)_x^2} \qquad \text{(AISC Equation E4-7)}$$

$$F_{ey} = \frac{\pi^2 E}{(KL/r)_y^2} \qquad \text{(AISC Equation E4-8)}$$

$$F_{ez} = \left[\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right]\frac{1}{A_g\bar{r}_0^2} \qquad \text{(AISC Equation E4-9)}$$

The values of C_w, J, $\bar{r}_0$, and H are provided for many sections in the "Flexural-Torsional Properties" tables of Part 1 of the Manual.

In Example C-1, the authors have gone through all of these formulas for a WT shape column. The resulting values for $\phi_c P_n$ is shown to agree with the values given in the Manual.

Example C-1

Determine (a) the flexural buckling strength and (b) the flexural-torsional buckling strength of an 18-ft pinned-end column consisting of A992 Grade 50 steel. The cross-section and other properties of the member are shown in Fig. C.2; $G = 11{,}500$ ksi, and $K = 1.0$.

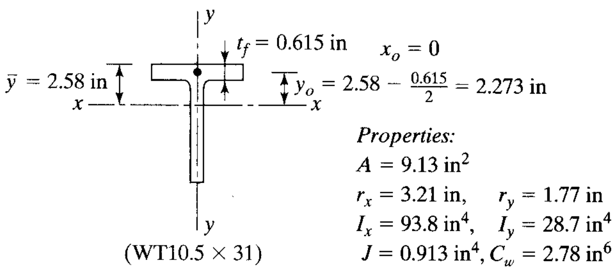

FIGURE C.2

Solution. Using WT1.50 × 31

(a) Flexural buckling about the x-axis (perpendicular to the axis of symmetry)

$$\left(\frac{KL}{r}\right)_x = \frac{(1.0)(12 \times 18)}{3.21} = 67.29$$

From Example B-1, $Q_s = 0.581$

$$Q_a = 1.0$$

$$\therefore Q = Q_s Q_a = 0.581\,(1.0) = 0.581$$

when $\dfrac{KL}{r} = 67.29 \leq 4.71\sqrt{\dfrac{E}{QF_y}} = 4.71\sqrt{\dfrac{29{,}000}{0.581(50)}} = 148.81$

Then

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \qquad \text{(AISC Equation E7-2)}$$

where $F_e = \dfrac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} = \dfrac{\pi^2(29{,}000)}{(67.29)^2} = 63.21$ ksi

Therefore, $$F_{cr} = 0.581\left[0.658^{\frac{0.581(50)}{63.21}}\right]50 = 23.97 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.9\,(23.97)\,(9.13) = 197 \text{ kips}$$

Checks with value in Table 4-7 (Available strength in Axial Compression – WT shapes) where for x-x axis, $\phi_c P_n = 197$ kips.

(b) Flexural-torsional buckling with respect to the y-axis passing through the shear center of the section.

Flexural-torsional properties:

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} \qquad \text{(AISC Equation E4-11)}$$

$$\bar{r}_0^2 = (0)^2 + (2.273)^2 + \frac{93.8 + 28.7}{9.13} = 18.584 \text{ in}^2$$

$$\bar{r}_0 = \sqrt{18.58} = 4.31 \text{ in}$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} = 1 - \frac{(0)^2 + (2.273)^2}{18.58} = 0.722 \text{ in}$$

For tee shaped compression member the critical stress, F_{cr}, is determined from AISC Equation E4-2.

$$F_{cr} = \left[\frac{F_{cry} + F_{crz}}{2H}\right]\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right]$$

where F_{cry} is taken as F_{cr} from Equation E3-2 or E3-3 and $\dfrac{KL}{r} = \left(\dfrac{KL}{r}\right)_y$

Since $$\left(\frac{KL}{r}\right)_y = \frac{1.0(12)(18)}{1.77} = 122.03 < 4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000}{0.581(50)}} = 148.81$$

where $Q = Q_s\,Q_a = 0.581\,(1.0) = 0.581$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_y^2} = \frac{\pi^2(29{,}000)}{(122.03)^2} = 19.22 \text{ ksi} \qquad \text{(AISC Equation E3-4)}$$

Therefore, $F_{cry} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y$ (AISC Equation E3.2)

$$F_{cry} = 0.581\left[0.658^{\frac{0.581(50)}{19.22}}\right]50 = 15.43 \text{ ksi}$$

and where $F_{crz} = \dfrac{GJ}{A_g\bar{r}_0^2}$ (AISC Equation E4-3)

$$F_{crz} = \frac{11{,}200(0.913)}{9.13(18.584)} = 60.27 \text{ ksi}$$

Therefore,

$$F_{cr} = \left[\frac{15.43 + 60.27}{2(0.722)}\right]\left[1 - \sqrt{1 - \frac{4(15.43)(60.27)(0.722)}{(15.43 + 60.27)^2}}\right]$$

$$F_{cr} = 14.21 \text{ ksi.}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.9(14.21)(9.13)$$

$$\phi P_n = 117 \text{ kips} \leftarrow$$

Checks with value in Table 4-7 (Available strength in Axial Compression – WT shapes) where for y-y axis, $\phi_c P_n = 118$ kips.

APPENDIX D

Moment-Resisting Column Base Plates

Column bases frequently are designed to resist bending moments as well as axial loads. An axial load causes compression between a base plate and the supporting footing, while a moment increases the compression on one side and decreases it on the other side. For small moments, the forces may be transferred to the footing through flexure of the base plate. When they are very large, stiffened or booted connections may be used. For a small moment, the entire contact area between the plate and the supporting footing will remain in compression. This will be the case if the resultant load falls within the middle third of the plate length in the direction of bending.

Figures D.1(a) and (b) show base plates suitable for resisting relatively small moments. For these cases, the moments are sufficiently small to permit their transfer to the footings by bending of the base plates. The anchor bolts may or may not have calculable stresses, but they are nevertheless considered necessary for good construction practice. They definitely are needed to hold the columns firmly in place and upright during the initial steel erection process. Temporary guy cables are also necessary during erection. The anchor bolts should be substantial and capable of resisting unforeseen erection forces. Sometimes these small plates are attached to the columns in the shop, and other times they are shipped loose to the job and carefully set to the correct elevations in the field.

Should the eccentricity ($e = M/P$) be sufficiently large that the resultant falls outside the middle third of the plate, there will be an uplift on the other side of the column, putting the anchor bolts on that side in tension.

The moment will be transferred from the column into the footing by means of the anchor bolts, embedded a sufficient distance into the footing to develop the anchor bolt forces. The embedment should be calculated as required by reinforced-concrete design methods.[1] The booted connection shown in Fig. D.1(c) is assumed to be welded

[1] *Building Code Requirements for Reinforced Concrete (ACI 318-05) and Commentary (ACI 318R-05)* (Detroit: American Concrete Institute, 2005), pp. 196–200.

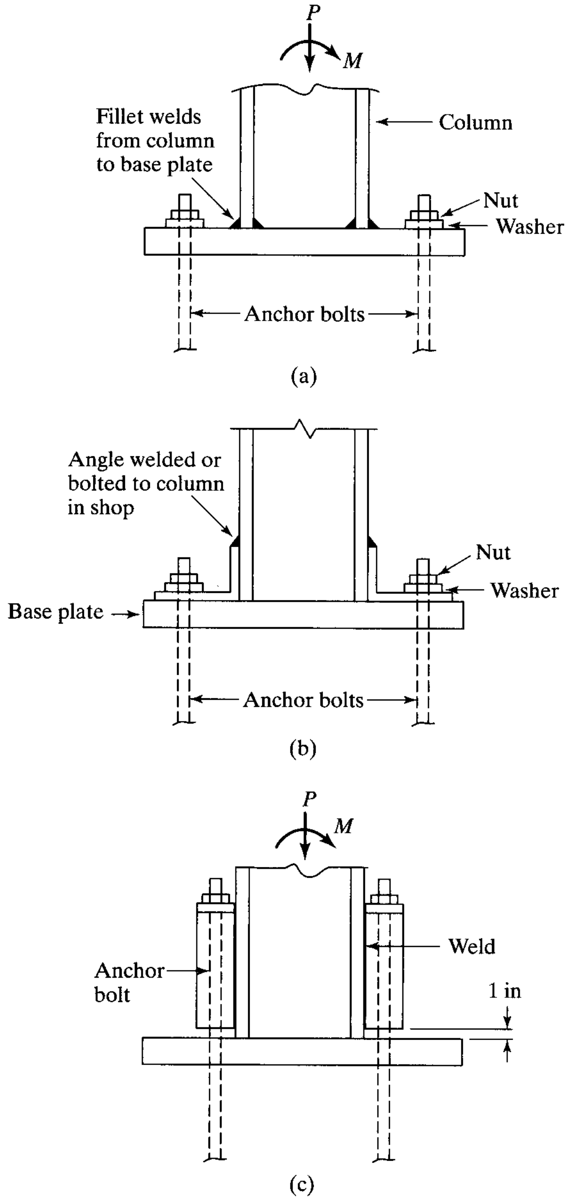

FIGURE D.1

Moment-resisting column base plates.

to the column. The boots generally are made of angles or channels and are not, as a rule, connected directly to the base plate. Rather, the tensile force component induced by the moment is transmitted from the column to the foundation by means of the anchor bolts. When booted connections are used, the base plates normally are shipped loose to the job and carefully set to the correct elevations in the field.

The capacity of these connections to resist rotation is dependent on the lengths of the anchor bolts, which are available to deform elastically. This capacity can be increased somewhat by pretensioning the anchor bolts. (This is similar to the prestress discussion for high-strength bolts presented in Section 13.4.) Actually, prestressing is not very dependable and usually is not done, because of the long-term creep in the concrete.

When a moment-resisting or rigid connection between a column and its footing is used, it is absolutely necessary for the supporting soil or rock beneath the footing to be appreciably noncompressible, or the column base will rotate. If this happens, the rigid connection between the column and the footing is useless. For the purpose of this appendix, the subsoil is assumed to be capable of resisting the moment applied to it without appreciable rotation.

The material presented in this appendix applies to LRFD designs. Should designers wish to use the ASD procedure, they may follow exactly the same steps given herein, but they must use ASD loads and Ω values.

Quite a few methods have been developed through the years for designing moment-resisting base plates. One rather simple procedure used by many designers is presented here. As a first numerical example, a column base plate is designed for an axial load and a relatively small bending moment such that the resultant load falls between the column flanges. Assumptions are made for the width and length of the plate, after which the pressures underneath the plate are calculated and compared with the permissible value. If the pressures are unsatisfactory, the dimensions are changed and the pressures recalculated, and so on, until the values are satisfactory. The moment in the plate is calculated, and the plate thickness is determined. The critical section for bending is assumed to be at the center of the flange on the side where the compression is highest. Various designers will assume that the point of maximum moment is located at some other point, such as at the face of the flange or the center of the anchor bolt.

The moment is calculated for a 1-in-wide strip of the plate and is equated to its resisting moment. The resulting expression is solved for the required thickness of the plate as follows:

$$M_u \leq \phi_b M_n = \frac{\phi_b F_y I}{c} = \frac{\phi_b F_y\left(\frac{1}{12}\right)(1)(t)^3}{t/2}$$

$$t \geq \sqrt{\frac{6M_u}{\phi_b F_y}} \quad \text{with} \quad \phi_b = 0.9$$

From Section J8 of the LRFD Specification,

$$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \qquad \text{(AISC Equation J8-2)}$$

If we assume $\sqrt{\frac{A_2}{A_1}} \geq 2$, then

$$P_p = 1.7 f'_c A_1$$

$$\phi_c P_p = \phi_c 1.7 f'_c A_1 \quad \text{with} \quad \phi_c = 0.65 \quad (\Omega_c = 2.31)$$

Example D-1

Design a moment-resisting base plate to support a W14 × 120 column with an axial load P_u of 620 k and a bending moment M_u of 225 ft-k. Use A36 steel with $F_y = 36$ ksi and a concrete footing with $f'_c = 3.0$ ksi. $\phi_c F_p = (0.65)(1.7)(3.0) = 3.32$ ksi.

Solution. Using a W14 × 120($d = 14.5$ in, $t_w = 0.590$ in, $b_f = 14.70$ in, $t_f = 0.940$ in),

$$e = \frac{(12)(225)}{620} = 4.35 \text{ in}$$

∴ The resultant falls between the column flanges and within the middle third of the plate.

Try a 20 × 28 in plate (after a few trials)

$$f = -\frac{P_u}{A} \pm \frac{P_u ec}{I} = -\frac{620}{(20)(28)} \pm \frac{(620)(4.35)(14)}{(\frac{1}{12})(20)(28)^3}$$

$$= -1.107 \pm 1.032 \begin{cases} -2.139 < \phi_c P_n = 3.32 \text{ ksi} \\ -0.075 \text{ ksi (still compression)} \end{cases} \qquad \text{(OK)}$$

Taking moments to right at center of right flange (see Fig. D.2):

$$M_u = (1.606)(7.22)\left(\frac{7.22}{2}\right) + (2.139 - 1.606)(7.22)\left(\frac{2}{3} \times 7.22\right) = 51.12 \text{ in-k}$$

$$t \geq \sqrt{\frac{6M_u}{\phi_b F_y}} = \sqrt{\frac{(6)(51.12)}{(0.9)(36)}} = 3.08 \text{ in}$$

Checking bending in transverse direction

$$n = \frac{B - 0.8b_f}{2} = \frac{20 - (0.80)(14.7)}{2} = 4.12 \text{ in}$$

$$\text{Average } f_p = \frac{0.075 + 2.139}{2} = 1.107 \text{ ksi}$$

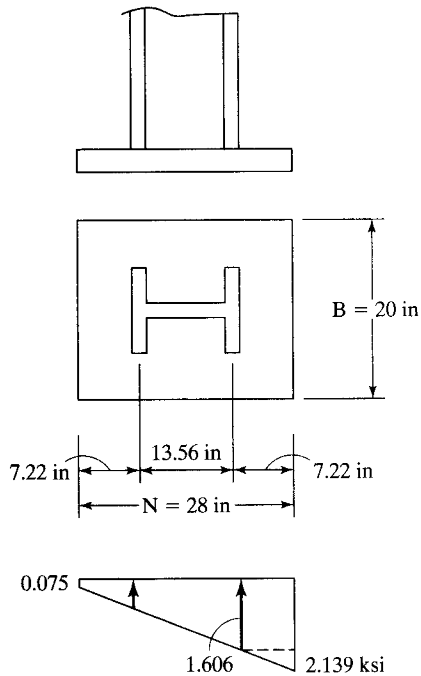

FIGURE D.2

$$M_u = (1.107)(4.12)\left(\frac{4.12}{2}\right) = 9.40 \text{ in-k} < 51.19 \text{ in-k} \qquad \text{(OK)}$$

Use PL3 $\frac{1}{4}$ × 20 × 2 ft 4 in A36

The moment considered in Example D-2 is of such a magnitude that the resultant load falls outside the column flange. As a result, there will be uplift on one side, and the anchor bolt will have to furnish the needed tensile force to provide equilibrium.

In this design, the anchor bolts are assumed to have no significant tension due to tightening. As a result, they are assumed not to affect the force system. As the moment is applied to the column, the pressure shifts toward the flange on the compression side. It is assumed that the resultant of this compression is located at the center of the flange.

Example D-2

Repeat Example D-1 with the same column and design stresses, but with the moment increased from 225 ft-k to 460 ft-k. Refer to Fig. D.3.

Solution. Using a W14 × 120(d = 14.5 in, t_w = 0.590 in, b_f = 14.7 in, t_f = 0.940 in)

$$e = \frac{(12)(460)}{620} = 8.90 \text{ in} > \frac{d}{2} = 7.25 \text{ in}$$

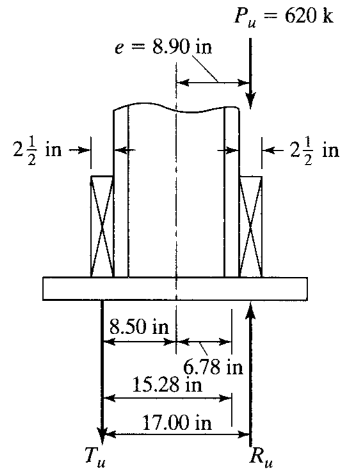

FIGURE D.3

$\therefore$ The resultant falls outside the column flange. Taking moments about the center of the right flange, we have

$$(620)(8.90 - 6.78) - 15.28\,T_u = 0$$

$$T_u = 86.02 \text{ k}$$

$$\text{Anchor bolt } A_{\text{reqd.}} = \frac{T_u}{\phi_t 0.75 F_u} \qquad \text{(AISC Table J3.2)}$$

$$= \frac{86.02}{\phi 0.75 F_u} = \frac{86.02}{(0.75)(0.75)(58)} = 2.64 \text{ in}^2$$

Use two $1\frac{3}{8}$-in-diameter bolts each side. ($A_s = 2.97 \text{ in}^2 > 2.64 \text{ in}^2$)

Approximate plate size, assuming a triangular pressure distribution

$$R_u = P_u + T_u = 620 + 86.02 = 706.02 \text{ k}$$

A of plate reqd. $\geq \dfrac{R_u}{\text{Avg}\phi_c F_p} = \dfrac{R_u}{\phi_c F_p/2}$ with $\phi_c F_p$ given $= 3.32$ ksi in statement of Example D-1.

$$A \text{ reqd.} \geq \frac{706.02}{3.32/2} = 425.31 \text{ in}^2$$

Try a 24-in-long plate.

The load is located $\frac{24}{2}$ in minus the distance from the column c.g. to the column flange c.g. $= \frac{24}{2} - 6.78 = 5.22$ in from edge of the plate. Thus, the pressure triangle will be $3 \times 5.22 = 15.66$ in long, and the required plate length B will equal

$$B = \frac{706.02}{\frac{1}{2} \times 3.32 \times 15.66} = 27.16 \text{ in}$$

Try a 28-in-long plate.

The load R_u is located $\frac{28}{2} - 6.78 = 7.22$ in from the edge of the plate. The triangular pressure length will be $(3)(7.22) = 21.66$ in long, and the required plate width will be

$$B = \frac{706.02}{\left(\frac{1}{2}\right)(3.32)(21.66)} = 19.64 \text{ in}$$

If the plate is made 20 in wide, the pressure zone will have an area $= 20 \times 21.66 = 433.2 \text{ in}^2$, and the maximum pressure will be twice the average pressure, or

$$\frac{706.02}{433.2} \times 2 = 3.26 \text{ ksi} < 3.32 \text{ ksi} \qquad \text{(OK)}$$

Taking moments to right at center of right column flange (see Fig. D.4)

$$M_u = 2.16(7.22)\left(\frac{7.22}{2}\right) + \frac{1}{2}(1.10)(7.22)\left(\frac{2}{3}\right)(7.22) = 75.41 \text{ in-k}$$

$$t = \sqrt{\frac{(6)(75.41)}{(0.9)(36)}} = 3.74 \text{ in}$$

Use PL$3\frac{3}{4} \times 20 \times 2$ ft 4 in A36

Design of weld from column to base PL (see Fig. D.5)

Total length of fillet weld each flange

$$= (2)(14.7) - 0.590 = 28.81 \text{ in}$$

$$C = T = \frac{M_u}{d - t_f} = \frac{(12)(460)}{13.56} = 407.08 \text{ k}$$

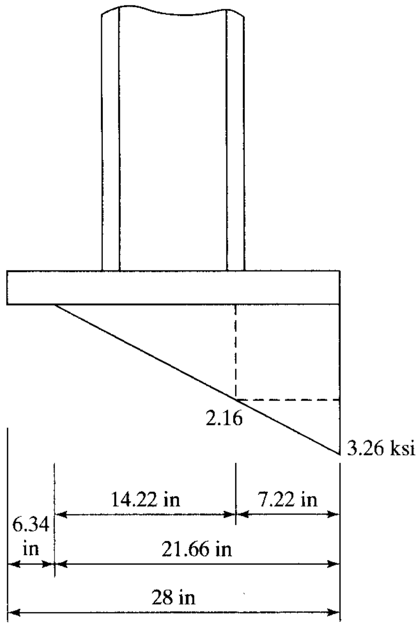

FIGURE D.4

Strength of 1-in-long 1-in fillet weld, using E70 electrodes

$$\phi R_{nw} = \phi(0.60F_{EXX})(0.707)(a) = (0.75)(0.60 \times 70)(0.707)(1.0)$$
$$= 22.3 \text{ k/in}$$

$$\text{Weld size required} = \frac{407.08}{(28.41)(22.3)} = 0.643 \text{ in}$$

Use $\frac{11}{16}$-in fillet welds, E70 electrode, SMAW

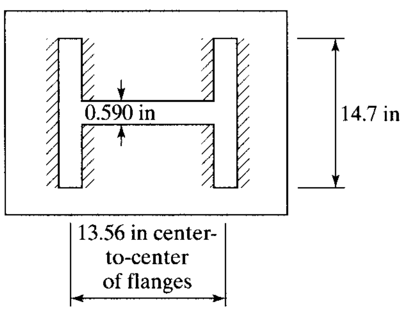

FIGURE D.5
Fillet welds from column to base plate.

Note: Tension flange design strength $= \phi R_n$

$$= \phi F_y A_f = (0.9)(36)(14.7)(0.940) = 448 \text{ k} > T_u \qquad \text{(OK)}$$

In these examples, the variation of stress in the concrete supporting the columns has been assumed to vary in a triangular or straight-line fashion. It is possible to work with an assumed ultimate concrete theory where the concrete in compression under the plate is assumed to fail at a stress of $0.85f'_c$. Examples of such designs are available in several texts.[2] More detailed information concerning moment-resisting bases is available in several places.[3,4]

[2] W. McGuire, *Steel Structures* (Englewood Cliffs, NJ: Prentice-Hall, 1968), pp. 987–1004.
[3] C. G. Salmon, L. Schenker, and B. G. Johnston, "Moment-Rotation Characteristics of Column Anchorages," *Transactions ASCE*, 122 (1957), pp. 132–154.
[4] J. T. DeWolf and E. F. Sarisley, "Column Base Plates with Axial Loads and Moments," *Journal of Structural Division*, ASCE, 106, ST11 (November 1980), pp. 2167–2184.

APPENDIX E

Ponding

It has been claimed that almost 50 percent of the lawsuits faced by building designers are concerned with roofing systems.[1] Ponding, a problem with many flat roofs, is one of the most common subjects of such litigation. If water accumulates more rapidly on a roof than it runs off, ponding results because the increased load causes the roof to deflect into a dish shape that can hold more water, which causes greater deflections, and so on. This process continues until equilibrium is reached, or until collapse occurs. Ponding can be caused by increasing deflections, clogged roof drains, settlement of footings, warped roof slabs, and so on.

The best way to prevent ponding is to use appreciable roof slopes (1/4 in per ft or more), together with good drainage facilities. Supposedly, more than two-thirds of the flat roofs in the United States have slopes less than 1/4 in per ft. The construction of roofs with slopes this large will increase roof building costs by only a few percent compared with perfectly flat roofs. The supporting girders for flat roofs with long spans should definitely be cambered to reduce the possibility of ponding (as well as the sagging that is so disturbing to the people occupying a building).

Appendix 2 of the AISC Specification states that, unless roof surfaces have sufficient slopes to areas of free drainage or sufficient individual drains to prevent water accumulation, the strength and stability of the roof systems during ponding conditions must be investigated. The very detailed work of Marino[2] forms the basis of the ponding provisions of the AISC Specification. Many other useful references also are available.[3–5]

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[1]Gary Van Ryzin, "Roof Design: Avoid Ponding by Sloping to Drain," *Civil Engineering* (New York: ASCE, January 1980), pp. 77–81.
[2]F. J. Marino, "Ponding of Two-Way Roof System," *Engineering Journal*, AISC, vol. 3, no. 3 (3rd Quarter, 1966), pp. 93–100.
[3]L. B. Burgett, "Fast Check for Ponding," *Engineering Journal*, AISC, vol. 10, no. 1 (1st Quarter, 1973), pp. 26–28.
[4]J. Chinn, "Failure of Simply-Supported Flat Roofs by Ponding of Rain," *Engineering Journal*, AISC, no. 2 (2nd Quarter, 1965), pp. 38–41.
[5]J. L. Ruddy, "Ponding of Concrete Deck Floors," *Engineering Journal*, AISC, vol. 23, no. 2 (3rd Quarter, 1986), pp. 107–115.

The amount of water that can be retained on a roof depends on the flexibility of the framing. The specifications state that a roof system can be considered stable and not in need of further investigation if we satisfy the expressions

$$C_p + 0.9C_s \leq 0.25 \qquad \text{(AISC Equation A-2-1)}$$

$$I_d \geq 25(S^4)10^{-6} \qquad \text{(AISC Equation A-2-2)}$$

where

$$C_p = \frac{32L_sL_p^4}{10^7 I_p} \qquad \text{(AISC Equation A-2-3)}$$

$$C_s = \frac{32SL_s^4}{10^7 I_s} \qquad \text{(AISC Equation A-2-4)}$$

L_p = column spacing in direction of girder (primary member length), ft

L_s = column spacing perpendicular to girder direction (secondary member length), ft

S = spacing of secondary members, ft

I_p = moment of inertia of primary members, in^4

I_s = moment of inertia of secondary members, in^4

I_d = moment of inertia of the steel deck (if one is used) supported on the secondary members, in^4 per ft

Should steel roof decks be used, their I_d must at least equal the value given by Equation A-2-2. If the roof decking is the secondary system (i.e., no secondary beams, joists, etc.), it should be handled with Equation A-2-1.

Some other AISC requirements in applying these expressions follow:

1. The moment of inertia I_s must be decreased by 15 percent for trusses and steel joists.
2. Steel decking is considered to be a secondary member supported directly by the primary members.
3. Stresses caused by wind or seismic forces do not have to be considered in the ponding calculations.

Should moments of inertia be needed for open-web joists, they can be computed from the member cross sections or, perhaps more easily, backfigured from the resisting moments and allowable stresses given in the joist tables. (Since $M_R = FI/c$, we can compute $I = M_Rc/F$.)

In effect, these equations reflect *stress indexes*, or percentage stress increases. For instance, here we are considering the percentage increase in stress in the steel members caused by ponding. If the stress in a member increases from $0.60F_y$ to $0.80F_y$, we say that the stress index is given by

$$U = \frac{0.80F_y - 0.60F_y}{0.60F_y} = 0.33$$

The terms C_p and C_s are, respectively, the approximate stiffnesses of the primary and secondary support systems. AISC Equation A-2-1 ($C_p + 0.9C_s \leq 0.25$), which gives us an approximate stress index during ponding, is limited to a maximum value of 0.25. Should we substitute into this equation and obtain a value no greater than 0.25, ponding supposedly will not be a problem. Should the index be larger than 0.25, however, it will be necessary to conduct a further investigation. One method of doing this is presented in Appendix 2 of the AISC Specification and will be described later in this section.

Example E-1 presents the application of AISC Equation A-2-1 to a roofing system.

Example E-1

Check the roof system shown in Fig. E.1 for ponding, using the AISC Specification and A36 steel.

Solution

$$C_p = \frac{32L_sL_p^4}{10^7 I_p} = \frac{(32)(48)(36)^4}{(10^7)(1830)} = 0.141$$

$$C_s = \frac{32SL_s^4}{10^7 I_s} = \frac{(32)(6)(48)^4}{(10^7)(518)} = 0.197$$

$$C_p + 0.9C_s = 0.141 + (0.9)(0.197) = 0.318 > 0.25$$

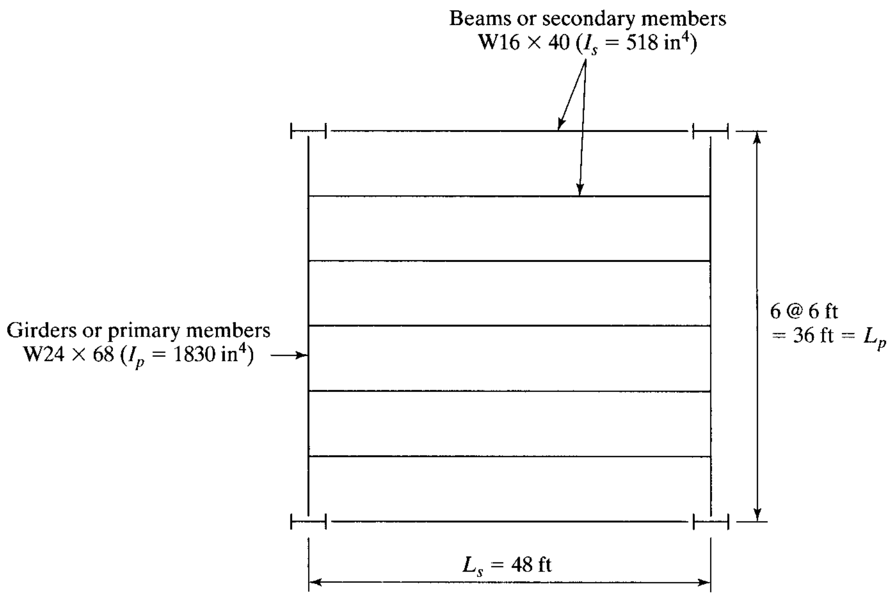

FIGURE E.1

This value indicates insufficient strength and stability, and thus a more precise method of checking should be used.

The curves in Appendix 2 of the AISC Specification provide a design aid for use when we need to compute a more accurate flat-roof framing stiffness than is given by the Specification provision that $C_p + 0.9C_s \leq 0.25$.

The following stress indexes are computed for the primary and secondary members:

For primary members,

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad \text{(AISC Appendix Equation A-2-5)}$$

For secondary members,

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right) \qquad \text{(AISC Appendix Equation A-2-6)}$$

In these expressions, f_0 represents the stress due to $D + R$ (where D is nominal dead load and R is nominal load due to rainwater or ice, exclusive of ponding contribution). These loads should include any snow that is present, although most ponding failures have occurred during torrential summer rains.

We enter Fig. A-2-1 in Appendix 2 of the AISC Specification with our computed U_p and move horizontally to the calculated C_s value of the secondary members. Then we go vertically to the abscissa scale and read the upper limit for the flexibility constant C_p. If this value is more than our calculated C_p value computed for the primary members, the stiffness is sufficient. The same process is followed in Fig. A-2-2, where we enter with our computed U_s and C_p values and pick from the abscissa the flexibility constant C_s, which should be no less than our C_s value. This procedure is illustrated in Example E-2.

Example E-2

Recheck the roof system considered in Example E-1, using the AISC Appendix 2 curves. Assume that, as ponding begins, f_0 is 20 ksi in both girders and beams.

Solution. Checking girders:

$$U_p = \frac{0.8F_y - f_0}{f_0} = \frac{(0.8)(36) - 20}{20} = 0.44$$

For $U_p = 0.44$ and $C_s = 0.197$, we read in AISC Appendix 2, Fig. A-2-1, that $C_p = 0.165$, our calculated C_p of 0.141.

$\therefore$ **Girders are sufficiently stiff.** (OK)

Checking beams:

$$U_s = \frac{0.8F_y - f_o}{f_o} = \frac{(0.8)(36) - 20}{20} = 0.44$$

For $U_s = 0.44$ and $C_p = 0.141$, we read from AISC Appendix 2, Figure A-2-2, that $C_s = 0.150$. This value is $<$ our calculated value of $C_s = 0.197$.

$\therefore$ **Stiffer secondary members are required.**

Glossary

Allowable Strength The nominal strength of a member divided by the safety factor, R_n/Ω.

Amplification Factor A multiplier used to increase the computed moment or deflection in a member to account for the eccentricity of the load.

Annealing A process in which steel is heated to an intermediate temperature range, held at that temperature for several hours, and then allowed to slowly cool off to room temperature. The resulting steel has less hardness and brittleness, but more ductility.

ASD (Allowable Strength Design) Method of sizing structural members such that the allowable strength equals or is greater than the required strength of the member using service loads.

Aspect Ratio The ratio of the lengths of the sides of a rectangular panel to each other.

Available Strength The design strength or allowable strength depending on the design method (ASD or LRFD) used.

Bar Joist See *Open Web-Joist*.

Bays The areas between columns in a building.

Beam A member that supports loads transverse to its longitudinal axis.

Beam–Column A column that is subjected to axial compression loads as well as bending moments.

Bearing–Type Connection Bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

Bearing Wall Construction Building construction where all the loads are transferred to the walls and thence down to the foundations.

Block Shear A fracture type shear where fracture may occur on either the tension plane or the shear plane, followed by yielding on the other plane (see Fig. 3.16).

Braced Frame A frame that has resistance to lateral loads supplied by some type of auxiliary bracing.

Brittle Fracture Abrupt fracture with little or no prior ductile deformation.

Buckling Load The load at which a straight compression member assumes a deflected position.

Built-Up Member A member made up of two or more steel elements bolted or welded together to form a single member.

Camber The construction of a member bent or arched in one direction so that its deflection will not be so noticeable when the service loads bend it in the opposite direction.

Cast Iron An iron with a very high carbon content (2% or more)

Charpy V-Notch Test A test used for measuring the fracture toughness of steel by fracturing it with a pendulum swung from a certain height.

Cladding The exterior covering of the structural parts of a building.

Cold-Formed Light-Gage Steel Shapes Shapes made by cold bending thin sheets of carbon or low-alloy steels into desired cross sections.

Column A structural member whose primary function is to support compressive loads.

Compact Section A section that has a sufficiently stocky profile so that it is capable of developing a fully plastic stress distribution before buckling.

Composite Beam A steel beam made composite with a concrete slab by shear transfer between the two (see Fig. 16.1).

Composite Column A column constructed with rolled or built-up steel shapes, encased in concrete or with concrete placed inside steel pipes or other hollow steel sections (see Fig. 17.1).

Connection The joining of structural members and joints used to transmit forces between two or more members.

Coping The cutting back of the flanges of a beam to facilitate its connection to another beam (see Fig. 10.6).

Cover Plate A plate welded or bolted to the flange of a member to increase cross-sectional area, moment of inertia or section modulus.

Dead Loads Loads of constant magnitude that remain in one position. Examples are weights of walls, floors, roofs, fixtures, structural frames, and so on.

Design Strength The resistance factor times the nominal strength, ΦR_n.

Diagonal Bracing Inclined structural member typically carrying only axial force in a braced frame.

Direct Analysis Method A design method for stability that includes the effects of residual stresses and initial out-of-straightness of frames by reducing member stiffness and applying notional loads in a second-order analysis.

Drift Lateral deflection of a building.

Drift Index The ratio of lateral deflection of a building to its height.

Ductility The property of a material by which it can withstand extensive deformation without failure under high tensile stress.

Effective Length The distance between points of zero moment in a column; that is, the distance between its inflection points.

Effective Length Factor *K* A factor that, when multiplied by the length of a column, will provide its effective length.

Elastic Design A method of design that is based on certain allowable stresses.

Elastic Limit The largest stress that a material can withstand without being permanently deformed.

Elasticity The ability of a material to return to its original shape after it has been loaded and then unloaded.

Elastic Strain Strain that occurs in a member under load before its yield stress is reached.

Endurance Limit The maximum fatigue-type stress in a material for which the material seems to have an infinite life.

Euler Load The compression load at which a long and slender member will buckle elastically.

Eyebar A pin-connected tension member whose ends are enlarged with respect to the rest of the member so as to make the strength of the ends approximately equal to the strength of the rest of the member.

Factored Load A nominal load multiplied by a load factor.

Fasteners A term representing bolts, welds, rivets, or other connecting devices.

Fatigue A fracture situation caused by changing stresses.

Faying Surface The contact or shear area of members being connected.

Fillet Weld A weld placed in the corner formed by two overlapping parts in contact with each other (see Fig. 14.2).

First-Order Analysis Analysis of a structure in which equilibrium equations are written based on an assumed nondeformed structure.

Flexural Buckling A buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-Torsional Buckling A buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Floor Beams The larger beams in many bridge floors that are perpendicular to the roadway of the bridge and that are used to transfer the floor loads from the stringers to the supporting girders or trusses.

Fracture Toughness The ability of a material to absorb energy in large amounts. For instance, steel members can be subjected to large deformations during erection and fabrication, without failure, thus allowing them to be bent, hammered, and sheared, and to have holes punched in them.

Gage Transverse spacing of bolts measured perpendicular to the long direction of the member (see Fig. 12.4).

Girder A rather loosely used term that usually indicates a large beam and, perhaps, one into which smaller beams are framed.

Girts Horizontal members running along the sides of industrial buildings, used primarily to resist bending due to wind. They often are used to support corrugated siding.

Government Anchors Bent steel bars used when ends of steel beam are enclosed by concrete or masonry walls. The bars pass through the beam webs parallel to the walls and are enclosed in the walls. They keep beams from moving longitudinally with respect to the walls.

Gravity Load A load, such as dead load or live load, acting in a downward direction.

Groove Welds Welds made in grooves between members that are being joined. They may extend for the full thickness of the parts (complete-penetration groove welds), or they may extend for only a part of the member thickness (partial-penetration groove welds) (see Fig. 14.2).

Hybrid Member A structural steel member made from parts that have different yield stresses.

Impact Loads The difference between the magnitude of live loads actually caused and the magnitude of those loads had they been applied as dead loads.

Inelastic Action The deformation (of a member) that does not disappear when the loads are removed.

Influence Line A diagram whose ordinates show the magnitude and character of some function of a structure (shear, moment, etc.) as a unit load moves across the structure.

Instability A situation occurring in a member where increased deformation of that member causes a reduction in its load-carrying ability.

Intermediate Columns Columns that fail both by yielding and by buckling. Their behavior is said to be inelastic. Most columns fall in this range, where some of the fibers reach the yield stress and some do not.

Ironworker A person performing steel erection (a name carried over from the days when iron structural members were used).

Joists The closely spaced beams supporting the floors and roofs of buildings.

Jumbo Sections Very heavy steel W sections (and structural tees cut from those sections). Serious cracking problems sometimes occur in these sections when welding or thermal cutting is involved.

Killed Steel Steel that has been deoxidized to prevent gas bubbles and to reduce its nitrogen content.

Lamellar Tearing A separation in the layers of a highly restrained welded joint, caused by "through-the-thickness" strains produced by shrinking of the weld metal.

Lateral Load A load, such as wind load or earthquake load, acting in a lateral or horizontal direction.

Leaning Column A column designed to carry only gravity loads, having connections that do not provide lateral load resistance.

Limit State A condition at which a structure or some point of the structure ceases to perform its intended function either as to strength or as to serviceability.

Lintels Beams over openings in masonry walls, such as windows and doors.

Live Loads Loads that change position and magnitude. They move or are moved. Examples are trucks, people, wind, rain, earthquakes, temperature changes, and so on.

Load Factor A number almost always larger than 1.0, used to increase the estimated loads a structure has to support, to account for the uncertainties involved in estimating loads.

Local Buckling The buckling of a part of a larger member that precipitates failure of the whole member.

Long Columns Columns that buckle elastically and whose buckling loads can be predicted accurately with the Euler formula if the axial buckling stress is below the proportional limit.

LRFD (Load and Resistance Factor Design) A method of sizing structural members such that the design strength equals or is greater than the required strength of the member using factored loads.

Malleability The property of some metals by which they may be hammered, pounded, or rolled into various shapes—particularly, thin sheets.

Mild Steel A ductile low-carbon steel.

Milled Surfaces Those surfaces that have been accurately sawed or finished to a smooth or true plane.

Mill Scale An iron oxide that forms on the surface of steel when the steel is reheated for rolling.

Modulus of Elasticity, or **Young's Modulus** The ratio of stress to strain in a member under load. It is a measure of the stiffness of the material.

Moment Connection A connection that transmits bending moment between connected structural members.

Moment Frame A frame that has resistance to lateral loads supplied by the shear and flexure of the members and their connections.

Net Area Gross cross-sectional area of a member minus any holes, notches, or other indentations.

Nominal Loads The magnitudes of loads specified by a particular code.

Nominal Strength The theoretical ultimate strength of a member or connection.

Noncompact Section A section that cannot be stressed to a fully plastic situation before buckling occurs. The yield stress can be reached in some, but not all of the compression elements before buckling occurs.

Notional Load Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Open-Web Joist A small parallel chord truss whose members are often made from bars (hence the common name *bar joist*) or small angles or other rolled shapes. These joists are very commonly used to support floor and roof slabs (see Fig. 19.5).

P-Delta Effect Changes in column moments and deflections due to lateral deflections.

Partially Composite Section A section whose flexural strength is governed by the strength of its shear connectors.

Pitch The longitudinal spacing of bolts measured parallel to the long direction of a member (see Fig. 12.4).

Plane Frame A frame that, for purposes of analysis and design, is assumed to lie in a single (or two-dimensional) plane.

Plastic Design A method of design that is based on a consideration of failure conditions.

Plastic Modulus The statical moment of the tension and compression areas of a section taken about the plastic neutral axis.

Plastic Moment The yield stress of a section times its plastic modulus. The nominal moment that the section can theoretically resist if it is braced laterally.

Plastic Strain The strain that occurs in a member after its yield stress is reached with no increase in stress.

Plate Girder A built-up steel beam (see Fig. 18.3).

Poisson's Ratio The ratio of lateral strain to axial or longitudinal strain in a member under load.

Ponding A situation on a flat roof where water accumulates faster than it runs off.

Post-Buckling Strength The load a member or frame can support after buckling occurs.

Proportional Limit Largest strain for which Hooke's law applies, or the highest point on the straight-line portion of the stress–strain diagram.

Purlins Roof beams that span between trusses (see Fig. 4.4).

Quenching Rapid cooling of steel with water or oil.

Required Strength The forces, stresses, and deformations produced in a structural member determined from a structural analysis using factor or service loads.

Residual Stresses The stresses that exist in an unloaded member after it's manufactured.

Resistance Factor ϕ A number, almost always less than 1.0, multiplied by the ultimate or nominal strength of a member or connection to take into account the uncertainties in material strengths, dimensions, and workmanship. Also called *overcapacity factor.*

Rigid Frame A structure whose connectors keep substantially the same angles between members before and after loading.

Safety Factor A number, typically greater than 1.0, divided into the nominal strength to take into account the uncertainties of the load and the manner and consequences of failure.

Sag Rods Steel rods that are used to provide lateral support for roof purlins. They also may be used for the same purpose for girts on the sides of buildings (see Fig. 4.5).

St. Venant Torsion The part of the torsion in a member that produces only shear stresses in the member.

Scuppers Large holes or tubes in walls or parapets that enable water above a certain depth to quickly drain from roofs.

Second-Order Analysis Analysis of a structure for which equilibrium equations are written that include the effect of the deformations of the structure.

Section Modulus The ratio of the moment of inertia, taken about a particular axis of a section, to the distance to the extreme fiber of the section, measured perpendicular to the axis in question.

Seismic Of or having to do with an earth-quake.

Serviceability The ability of a structure to maintain its appearance, comfort, durability, and function under normal loading conditions.

Serviceability Limit State A limiting condition affecting the ability of a structure to maintain its appearance, maintainability, durability or comfort of its occupants or function of machinery, under normal usage.

Service Loads The loads that are assumed to be applied to a structure when it is in service (also called *working loads*).

Shape Factor The ratio of the plastic moment of a section to its yield moment.

Shear Center The point in the cross section of a beam through which the resultant of the transverse loads must pass so that no torsion will occur.

Shear Lag A nonuniformity of stress in the parts of rolled or built-up sections occurring when a tensile load is not applied uniformly.

Shear Wall A wall in a structure that is specially designed to resist shears caused by lateral forces such as wind or earthquake in the plane of the wall.

Shims Thin strips of steel that are used to adjust the fitting of connections. Finger shims are installed after the bolts already are in place.

Short Columns Columns whose failure stress will equal the yield stress, and for which no buckling will occur. For a column to fall into this class, it would have to be so short as to have no practical application.

Sidesway The lateral movement of a structure caused by unsymmetrical loads or by an unsymmetrical arrangement of building members.

Simple Connection A connection that transmits negligible bending moment between connected members.

Skeleton Construction Building construction in which the loads are transferred for each floor by beams to the columns and thence to the foundations.

Slenderness Ratio The ratio of the effective length of a column to its radius of gyration, both pertaining to the same axis of bending.

Slender Section A member that will buckle locally while the stress still is in the elastic range.

Slip-Critical Joint A bolted joint that is designed to have resistance to slipping.

Space Frame A three-dimensional structural frame.

Spandrel Beams Beams that support the exterior walls of buildings and perhaps part of the floor and hallway loads (see Fig. 19.3).

Steel An alloy consisting almost entirely of iron (usually, over 98 percent). It also contains small quantities of carbon, silicon, manganese, sulfur, phosphorus, and other materials.

Stiffened Element A projecting piece of steel whose two edges parallel to the direction of a compression force are supported (see Fig. 5.6).

Stiffener A plate or an angle usually connected to the web of a beam or girder to prevent failure of the web (see Figs. 18.10 and 18.11).

Story Drift The difference in horizontal deflection at the top and bottom of a particular story.

Strain-Hardening Range beyond plastic strain in which additional stress is necessary to produce additional strain.

Strength Limit State A limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stringers The beams in bridge floors that run parallel to the roadway.

Tangent Modulus The ratio of stress to strain for a material that has been stressed into the inelastic range.

Tension Field Action The behavior of a plate girder panel which, after the girder initially buckles, acts much like a truss. Diagonal strips of the web act similarly to the diagonals of a parallel chord truss. The stiffeners keep the flanges from coming together, and the flanges keep the stiffeners from coming together (see Fig. 18.8).

Toughness The ability of a material to absorb energy in large amounts. As an illustration, steel members can be subjected to large deformations during fabrication and erection without fracture, thus allowing them to be bent, hammered, and sheared, and to have holes punched in them without visible damage.

Unbraced Frame A frame whose resistance to lateral forces is provided by its members and their connections.

Unbraced Length The distance in a member between points that are braced.

Unstiffened Element A projecting piece of steel having one free edge parallel to the direction of a compression force, with the other edge in that direction unsupported (see Fig. 5.6).

Upset Rods Rods whose ends are made larger than the regular bodies of the rods. Threads are cut into the upset ends, but the area at the root of the thread in each rod is larger than that of the regular part of the bar (see Fig. 4.3).

Warping Torsion The part of the resistance of a member to torsion that is provided by the warping resistance of the member cross section.

Weathering Steel A high-strength low-alloy steel whose surface, when exposed to the atmosphere (not a marine one), oxidizes and forms a tightly adherent film that prevents further oxidation and thus eliminates the need for painting.

Web Buckling The buckling of the web of a member (see Fig. 10.9).

Web Crippling The failure of the web of a member near a concentrated force (see Fig. 10.9).

Working Loads See *Service Loads.*

Wrought Iron An iron with a very low carbon content ($\leq 0.15\%$)

Yield Moment The moment that will just produce the yield stress in the outermost fiber of a section.

Yield Stress The stress at which there is a decided increase in the elongation or strain in a member without a corresponding increase in stress.

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