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Cold-Formed-Steel Design and Construction

The introduction of sheet rolling mills in England in 1784 by Henry Cort led to the first cold-formed-steel structural application, light-gage corrugated steel sheets for building sheathing. Continuous hot-rolling mills, developed in America in 1923 by John Tytus, led to the present fabricating industry based on coiled strip steel. This is now available in widths up to 90 in and in coil weights up to 40 tons, hot- or cold-rolled.

Formable, weldable, flat-rolled steel is available in a variety of strengths and in black, galvanized, or aluminum-coated. Thus, fabricators can choose from an assortment of raw materials for producing cold-formed-steel products. (In cold forming, bending operations are done at room temperature.) Large quantities of cold-formed sections are most economically produced on multistand roll-forming machines from slit coils of strip steel. Small quantities can still be produced to advantage in presses and bending brakes from sheared blanks of sheet and strip steel. Innumerable cold-formed-steel products are now made for building, drainage, road, and construction uses. Design and application of such lightweight-steel products are the principal concern of this section.

10.1 How Cold-Formed Shapes are Made

Cold-formed shapes are relatively thin sections made by bending sheet or strip steel in roll-forming machines, press brakes, or bending brakes. Because of the relative ease and simplicity of the bending operation and the comparatively low cost of forming rolls and dies, the cold-forming process also lends itself well to the manufacture of special shapes for specific architectural purposes and for maximum section stiffness.

Door and window frames, partitions, wall studs, floor joists, sheathing, and moldings are made by cold forming. There are no standard series of cold-formed structural sections, like those for hot-rolled structural shapes, although some dimensional requirements are specified in the American Iron and Steel Institute (AISI) Standards for cold-formed steel framing.

Cold-formed shapes cost a little more per pound than hot-rolled sections. They are nevertheless more economical under light loading.

10.2 Steel for Cold-Formed Shapes

Cold-formed shapes are made from sheet or strip steel, usually from 0.020 to 0.125 in thick. In thicknesses available (usually 0.060 to ½ in), hot-rolled steel usually costs less to use. Cold-rolled steel is used in the thinner gages or where the surface finish, mechanical properties, or more uniform thickness resulting from cold reducing are desired. (The commercial distinction between steel plates, sheets, and strip is principally a matter of thickness and width of material.)

Cold-formed shapes may be either black (uncoated) or galvanized. Despite its higher cost, galvanized material is preferable where exposure conditions warrant paying for increased corrosion protection. Uncoated material to be used for structural purposes generally conforms to one of the standard ASTM Specifications for structural-quality sheet and strip (A1008, A1011 and others). ASTM A653 covers structural-quality galvanized sheets. Steel with a hot-dipped aluminized coating (A792 and A875) is also available.

The choice of grade of material usually depends on the severity of the forming operation required to make the desired shape. Low-carbon steel has wide usage. Most shapes used for structural purposes in buildings are made from material with yield points in the range of 33 to 50 ksi under ASTM Specifications A1008 and A1011. Steel conforming generally to ASTM A606, "High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled Steel Sheet and Strip with Improved Corrosion Resistance," A1008, "Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability," or A1011, "Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability," is often used to achieve lighter weight by designing at yield points from 45 to 70 ksi, although higher yield points are also being used.

Sheet and strip for cold-formed shapes are usually ordered and furnished in decimal or millimetre thicknesses. (The former practice of specifying thickness based on weight and gage number is no longer appropriate.)

For the use of steel plates for cold-formed shapes, see the AISI Specification.

10.3 Types of Cold-Formed Shapes

Some cold-formed shapes used for structural purposes are similar in general configuration to hot-rolled structural shapes. Channels (C-sections), angles, and Z's can be roll-formed in a single operation from one piece of material. I sections are usually made by welding two channels back to back, or by welding two angles to a channel. All such sections may be made with either plain flanges, as in Fig. 10.1*a* to *d*, *j*, and *m*, or with flanges stiffened by lips at outer edges, as in Fig. 10.1*e* to *h*, *k*, and *n*.

In addition to these sections, the flexibility of the forming process makes it relatively easy to obtain hat-shaped sections, open box sections, or inverted-U sections (Fig. 10.1*o*, *p*, and *q*). These sections are very stiff in a lateral direction.

The thickness of cold-formed shapes can be assumed to be uniform throughout in computing weights and section properties. The fact that cold-formed sections have corners rounded on both the inside and outside of the bend has only a slight effect on the section properties, and so computations may be based on sharp corners without serious error.

Cracking at 90° bends can be reduced by use of inside bend radii not smaller than values recommended for specific grades of the steels mentioned in Art. 10.2. For instance, A1008, SS Grade 33 steel, for which a minimum yield point of 33 ksi is specified, should be bent around a die with a radius equal to at least 1½ times the steel thickness. See ASTM Specification grade for appropriate bend radius that can safely be used in making right angle bends.

10.4 Design Principles for Cold-Formed Sections

In 1939, the American Iron and Steel Institute (AISI) started sponsoring studies, which still continue, under the direction of structural specialists associated with the AISI Committees of Sheet and Strip

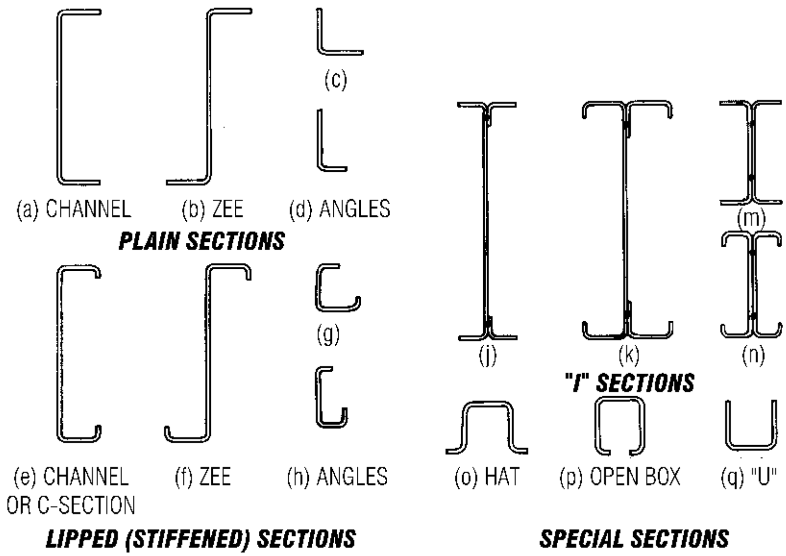


Fig. 10.1 Typical cold-formed-steel structural sections.

Steel Producers, that have yielded the AISI Specification for the Design of Cold-Formed Steel Structural Members. (American Iron and Steel Institute, 1140 Connecticut Ave., N.W., Washington, DC 20036.) The specification, which has been revised and amended repeatedly since its initial publication in 1946, has been adopted by the major building codes of the United States.

Structural behavior of cold-formed shapes conforms to classic principles of structural mechanics, as does the structural behavior of hot-rolled shapes and sections of built-up plates. However, local buckling of thin, wide elements, especially in cold-formed sections, must be prevented with special design procedures. Shear lag in wide elements remote from webs that causes nonuniform stress distribution and torsional instability that causes twisting in columns and beam of open sections also need special design treatment.

Uniform thickness of cold-formed sections and the relative remoteness from the neutral axis of their thin, wide flange elements make possible the assumption that, in computation of section properties, section components may be treated as line elements. (See "Section 3 of Part I of the AISI Cold-Formed Steel Design Manual," 2002.)

(Wei-Wen Yu, "Cold-Formed Steel Design," John Wiley & Sons, Inc., New York.)

Design Basis ▪ The Allowable Strength Design Method (ASD) is used currently in structural design of cold-formed steel structural members and described in the rest of this section using US customary units. In addition, the Load and Resistance Factor Design Method (LRFD) can also be used for design. Both methods are included in the 2001 edition of the AISI "North American Specification for the Design of Cold-Formed Steel Structural Members." However, these two methods cannot be mixed in designing the various cold-formed steel components of a structure.

In the allowable strength design method, the required strengths (bending moments, shear forces, axial loads, etc.) in structural members are computed by structural analysis for the working or service loads using the load combinations given in the AISI Specification. These required strengths are not to exceed the allowable design strengths as follows:

$$R \leq \frac{R_n}{\Omega}$$

where R = required strength

R_n = nominal strength specified in the AISI Specification

Ω = safety factor specified in the AISI Specification

R_n/Ω = allowable design strength

Unlike the allowable strength design method, the LRFD method uses multiple load factors and resistance factors to provide a refinement in the design that can account for different degrees of the uncertainties and variabilities of analysis, design, loading, material properties and fabrication. In this method, the required strengths are not to exceed the design strengths as follows:

$$R_u \leq \phi R_n$$

where $R_u = \Sigma \gamma_i Q_i$ = required strength

R_n = nominal strength specified in the AISI Specification

ϕ = resistance factor specified in the AISI Specification

γ_i = load factors

Q_i = load effects

ϕR_n = design strength

The load factors and load combinations are also specified in the AISI North American Specification for the design of different type of cold-formed steel structural members and connections. For design examples, see AISI "Cold-Formed Steel Design Manual," 2002 edition.

The ASD and LRFD methods discussed above are used in the United States and Mexico. The AISI North American Specification also includes the Limit States Design Method (LSD) for use in Canada. The methodology for the LSD method is the same as the LRFD method, except that the load factors, load combinations, and some resistance factors are different. The North American Specification includes Appendixes A, B, and C, which are applicable in the United States, Canada, and Mexico, respectively.

10.5 Structural Behavior of Flat Compression Elements

For buckling of flat compression elements in beams and columns, the flat-width ratio w/t is an important factor. It is the ratio of width w of a single flat element, exclusive of edge fillets, to the thickness t of the element (Fig. 10.2).

Flat compression elements of cold-formed structural members are classified as stiffened and unstiffened. Stiffened compression elements have both edges parallel to the direction of stress stiffened by a web, flange, or stiffening lip. Unstiffened compression elements have only one edge parallel to the direction of stress stiffened. If the sections in Fig. 10.1*a* to *n* are used as compression members, the webs are considered stiffened compression elements. But the wide, lipless flange elements and the lips that stiffen the outer edges of the flanges are unstiffened elements. Any section composed of a number of plane elements can be broken down into a combination of stiffened and unstiffened elements.

The cold-formed structural cross sections shown in Fig. 10.3 illustrate how effective portions of stiffened compression elements are considered to be divided into two parts located next to the two edge stiffeners of that element. In beams, a stiffener may be a web, another stiffened element, or a lip.

In computing net section properties, only the effective portions of elements are considered and the ineffective portions are disregarded. For beams, flange elements subjected to uniform compression may not be fully effective. Accordingly, section properties, such as moments of inertia and section moduli, should be reduced from those for a fully effective section. (Effective widths of webs can be determined using Section B2.3 of the AISI North American Specification.) Effective areas of column cross sections needed for determination of column loads from Eq. (10.21) of Art. 10.12 are based on full cross-sectional areas less all ineffective portions.

Elastic Buckling ▪ Euler, in 1744, determined the critical load for an elastic prismatic bar end-

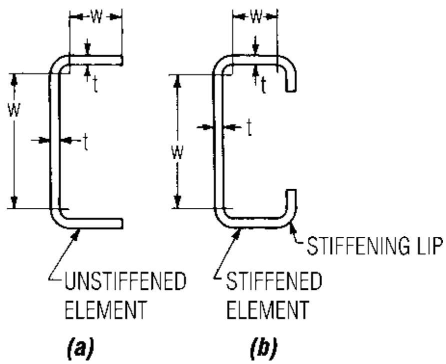


Fig. 10.2 Compression elements.

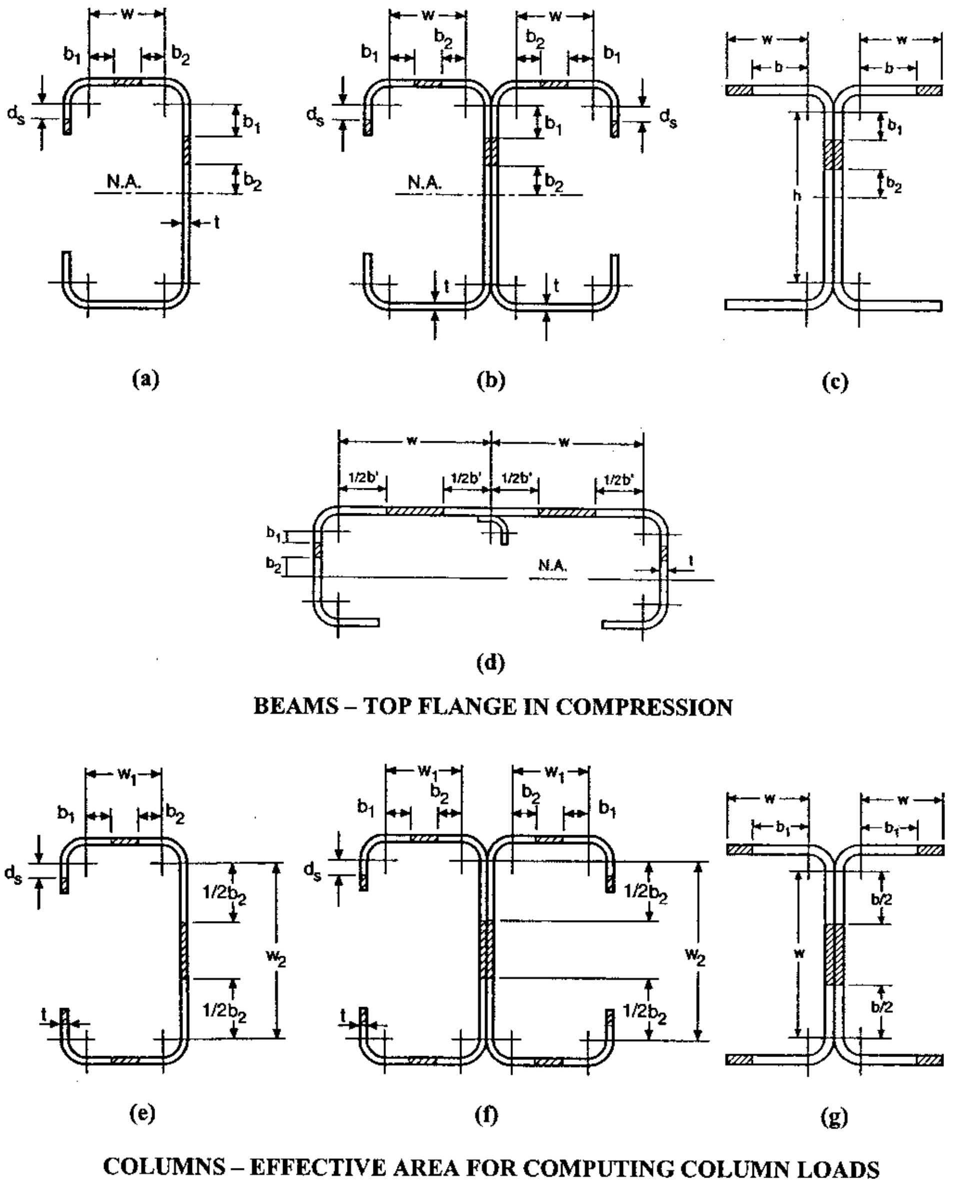


Fig. 10.3 Effective width of compression elements.

loaded as a column from

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{10.1}$$

where P_{cr} = critical load at which bar buckles, kips

E = modulus of elasticity, 29,500 ksi for steel

I = moment of inertia of bar cross section, in^4

L = column length of bar, in

This equation is the basis for designing long columns of prismatic cross section subject to elastic buckling. It might be regarded as the precursor of formulas used in the design of thin rectangular plates in compression.

Bryan, in 1891, proposed for design of a thin rectangular plate compressed between two opposite edges with the other two edges supported:

$$f_{cr} = \frac{k\pi^2 E(t/w)^2}{12(1-\nu^2)} \tag{10.2}$$

where f_{cr} = critical local buckling stress, ksi

k = a coefficient depending on edge-support restraint

w = width of plate, in

ν = Poisson's ratio

t = thickness, in

Until the 1986 edition, all AISI Specifications based strength of thin, flat elements stiffened along one edge on *buckling stress* rather than *effective width* as used for thin, flat elements stiffened along both edges. Although efforts were made by researchers to unify element design using a single concept, unification did not actually occur until Pekoz, in 1986, presented his unified approach using effective width as the basis of design for both stiffened and unstiffened elements and even for web elements subjected to stress gradients. Consequently, the AISI Specification uses the following equations to determine the effective width of uniformly compressed stiffened and unstiffened elements based on a slenderness factor λ:

$$\lambda = \sqrt{\frac{f}{f_{cr}}} = \frac{1.052(w/t)\sqrt{f/E}}{\sqrt{k}} \tag{10.3}$$

where k = 4.00 for stiffened elements

= 0.43 for unstiffened elements

f = unit stress in the compression element of the section, computed on the basis of the design width, ksi

f_{cr} = Eq. (10.2)

w = flat width of the element exclusive of radii, in

t = base thickness of element, in

The effective width is given by

$$b = w \qquad \lambda \leq 0.673 \tag{10.4}$$

$$b = \rho w \qquad \lambda > 0.673 \tag{10.5}$$

The reduction factor ρ is given by

$$\rho = \frac{(1 - 0.22/\lambda)}{\lambda} \tag{10.6}$$

10.6 Unstiffened Elements Subject to Local Buckling

By definition, unstiffened cold-formed elements have only one edge in the compression-stress direction supported by a web or stiffened element, while the other edge has no auxiliary support (Fig. 10.1a). The coefficient k in Eq. (10.3) is 0.43 for such an element. When the ratio of flat width to thickness does not exceed $72/\sqrt{f}$, an unstiffened element with unit stress f is fully effective; that is, the effective width b equals flat width w. Generally, however, Eq. (10.3) becomes

$$\lambda = \frac{1.052}{\sqrt{0.43}}\frac{w}{t}\sqrt{\frac{f}{E}} = 0.0093\frac{w}{t}\sqrt{f} \tag{10.7}$$

where E = 29,500 ksi for steel

f = unit compressive stress, ksi, computed on the basis of effective widths, Eq. (10.3)

When λ is substituted in Eq. (10.6), the b/w ratio ρ results. The lower portion of Fig. 10.5 shows curves for determining the effective-width ratio b/t for unstiffened elements for w/t between 0 and 60, with f between 15 and 90 ksi.

In beam-deflection determinations requiring the use of the moment of inertia of the cross section, f is the allowable stress used to calculate the effective width of an unstiffened element in a cold-formed-steel beam. However, in beam-strength determinations requiring use of the section modulus of the cross section, f is the unit compression stress to be used in Eq. (10.7) to calculate the effective width of the unstiffened element and provide an adequate margin of safety. In determining safe column loads, effective width for the unstiffened element must be determined for a nominal column buckling stress to ensure adequate margin of safety for such elements.

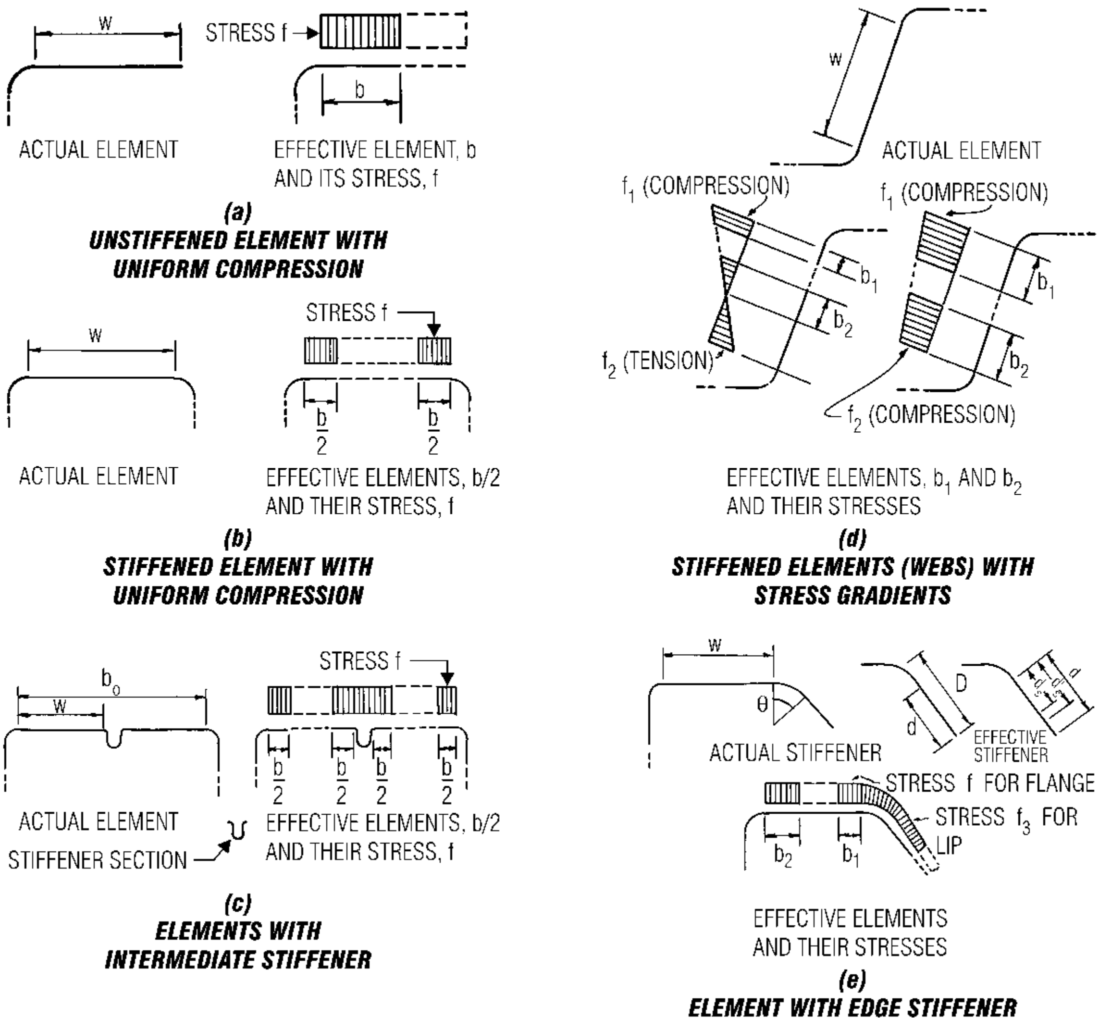


Fig. 10.4 Schematic diagrams showing effective widths for unstiffened and stiffened elements, intermediate stiffeners, beam webs, and edge stiffeners.

("Cold-Formed Steel Design Manual," American Iron and Steel Institute, Washington, D.C.)

10.7 Stiffened Elements Subject to Local Buckling

By definition, stiffened cold-formed elements have one edge in the compression-stress direction supported by a web or stiffened element and the other edge is also supported by a qualified stiffener (Fig. 10.4*b*). The coefficient k in Eq. (10.3) is 4.00 for such an element. When the ratio of flat width to thickness does not exceed $220/\sqrt{f}$, the stiffened element is fully effective, in which $f =$ unit stress, ksi, in the compression element of the structural section computed on the basis of effective widths, Eq. (10.3) becomes

$$\lambda = \frac{1.052}{\sqrt{4}}\frac{w}{t}\sqrt{\frac{f}{E}} = 0.0031\frac{w}{t}\sqrt{f} \qquad (10.8)$$

where $E = 29{,}500$ ksi for steel.

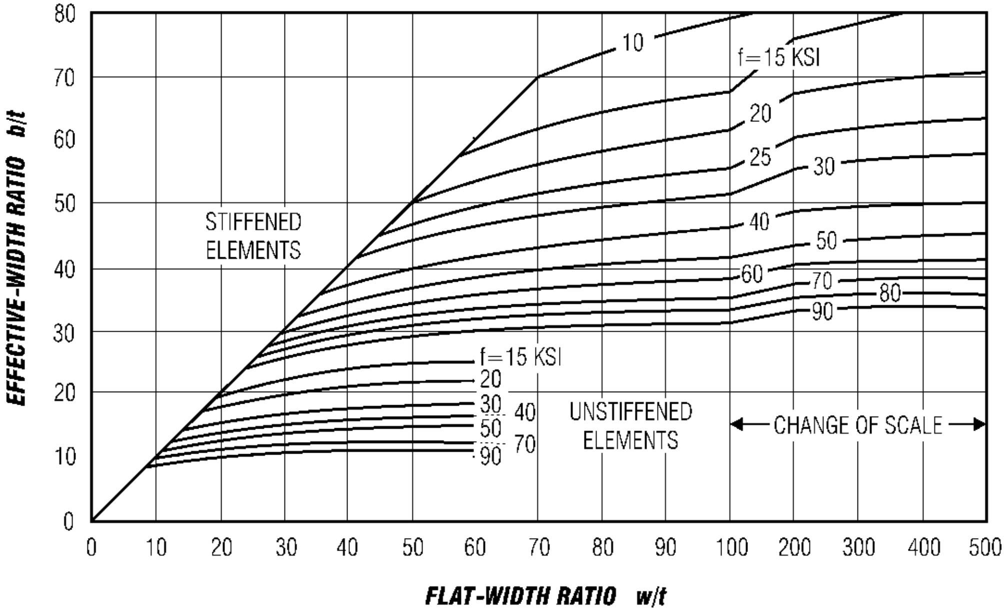


Fig. 10.5 Curves relate the effective-width ratio b/t to the flat-width ratio w/t for various stresses f for unstiffened and stiffened elements.

If λ is substituted in Eq. (10.6), the b/w ratio ρ results. Moreover, when $\lambda \leq 0.673$, $b = w$, and when $\lambda > 0.673$, $b = \rho w$. The upper portion of Fig. 10.5 shows curves for determining the effective-width ratio b/t for stiffened elements w/t between 0 and 500 with f between 10 and 90 ksi.

In beam-deflection determinations requiring the use of the moment of inertia of the cross section, f is the allowable stress used to calculate the effective width of a stiffened element in a cold-formed-steel member loaded as a beam. However, in beam-strength determinations requiring the use of the section modulus of the cross section, f is the unit compression stress to be used in Eq. (10.8) to calculate the width of a stiffened element in a cold-formed-steel beam. In determination of safe column loads, effective width for a stiffened element should be determined for a nominal column buckling stress to ensure an adequate margin of safety for such elements.

Note that the slenderness factor is $\sqrt{4.00/0.43} = 3.05$ times as great for unstiffened elements as for stiffened elements at applicable combinations of stress f and width-thickness ratio w/t. This emphasizes the greater effective width and economy of stiffened elements.

Single Intermediate Stiffener ▪ For uniformly compressed stiffened elements with a single intermediate stiffener, as shown in Fig. 10.4*c*, the required moment of inertia I_a, in^4, is determined by a parameter $S = 1.28\sqrt{E/f}$:

For $b_o/t \leq S$, $I_a = 0$ and no intermediate stiffener is needed, $b = w$.

For $b_o/t > S$, the effective width of the compression flange can be determined by the following local buckling coefficient k:

$$k = 3(R_I)^n + 1 \qquad (10.9a)$$

where

$$n = \left[0.583 - \frac{b_o/t}{12S}\right] \geq \frac{1}{3} \qquad (10.9b)$$

$$R_I = I_s/I_a \leq 1 \qquad (10.9c)$$

For $S < b_o/t < 3S$:

$$I_a = t^4\left[50\frac{b_o/t}{S} - 50\right] \qquad (10.10a)$$

For $b_o/t \geq 3S$:

$$I_a = t^4\left[128\frac{b_o/t}{S} - 285\right] \qquad (10.10\text{b})$$

In the above equations,

b_o = flat width including the stiffener, in

I_s = moment of inertia of full section of stiffener about its own centroidal axis parallel to the element to be stiffened, in^4

Webs Subjected to Stress Gradients ▪ Pekoz's unified approach using effective widths (Art. 10.5) also applies to stiffened elements subjected to stress gradients in compression, such as in webs of beams (Fig. 10.4*d*). The effective widths b_1 and b_2 are determined from the following, with $\psi = |f_2/f_1|$, where f_1 and f_2 are stresses shown in Fig. 10.4*d* calculated on the basis of the effective section. Stress f_1 is assumed to be in compression (positive) and f_2 can be either tension (negative) or compression. In case f_1 and f_2 are both in compression, f_1 is the larger of the two stresses.

$$b_1 = \frac{b_e}{3+\psi} \qquad (10.11)$$

where b_e = effective width b determined from Eqs. (10.3) to (10.6), with f_1 substituted for f and with k calculated from

$$k = 4 + 2(1+\psi)^3 + 2(1+\psi) \qquad (10.12)$$

The value of b_2 is calculated as follows:

For $h_o/b_o \leq 4$:

$$b_2 = b_e/2, \text{ when } \psi > 0.236$$

$$b_2 = b_e - b_1, \text{ when } \psi \leq 0.236$$

For $h_o/b_o > 4$:

$$b_2 = b_e/(1+\psi) - b_1$$

where b_o = out-to-out width of the compression flange, in

h_o = out-to-out depth of web, in

In addition, $b_1 + b_2$ should not exceed the compression portion of the web calculated on the basis of effective section.

Uniformly Compressed Elements with an Edge Stiffener ▪ It is important to understand the capabilities of edge stiffeners (depicted in Fig. 10.4*e* for a slanted lip). However, due to the complexity of this subject, the following presentation is confined primarily to simple lip stiffeners.

Two ranges of w/t values are considered relative to a parameter $0.328\,S$. The limit value of w/t for full effectiveness of the flat width without auxiliary support is

$$0.328\,S = (0.328)(1.28)\sqrt{\frac{E}{f}} = 0.420\sqrt{\frac{E}{f}} \qquad (10.13)$$

where E = modulus of the elasticity, ksi

f = unit compressive stress computed on the basis of effective widths, ksi

1. For the first case, where $w/t \leq 0.328S$, $b = w$, and no edge support is needed.

2. For the second case, where $w/t > 0.328S$, edge support is needed with the required moment of inertia I_a, in^4, determined from

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3$$

$$\leq t^4\left[115\frac{w/t}{S} + 5\right] \qquad (10.14)$$

For a slanted lip, as shown in Fig. 10.4*e*, the moment of inertia of full stiffener I_s, in^4, is

$$I_s = \frac{d^3t}{12}\sin^2\theta \qquad (10.15)$$

where d = flat width of lip, in

θ = angle between normals to stiffened element and its lip (90° for a right-angle lip) (Fig. 10.4*e*)

The effective width, b, of the compression flange can be determined from Eqs. (10.3) to (10.6) with k calculated from the following equations for single lip edge stiffener having $(140° \geq \theta \geq 40°)$:

For $D/w \leq 0.25$, $k = 3.57(R_I)^n + 0.43 \leq 4$ (10.16a)

For $0.25 < D/w \leq 0.8$,

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \qquad (10.16\text{b})$$

$$\text{where } n = \left[0.582 - \frac{w/t}{4S}\right] \geq \frac{1}{3} \qquad (10.16\text{c})$$

$$R_I = I_s/I_a \leq 1 \qquad (10.16\text{d})$$

The values of b_1 and b_2, as shown in Fig. 10.4*e*, can be computed as follows:

$$b_1 = \frac{b}{2}(R_I)$$

$$b_2 = b - b_1$$

The effective width b depends on the actual stress f, which, in turn, is determined by reduced section properties that are a function of effective width. Employment of successive approximations consequently may be necessary in using these equations. This can be avoided and the correct values of b/t obtained directly from the formulas when f is known or is held to a specified maximum value. This is true, though, only when the neutral axis of the section is closer to the tension flange than to the compression flange, so that compression controls. The latter condition holds for symmetrical channels, Z's, and I sections used as flexural members about their major axis, such as Fig. 10.1*e*, *f*, *k*, and *n*. For wide, inverted, pan-shaped sections, such as deck and panel sections, a somewhat more accurate determination, using successive approximations, is necessary.

For computation of moment of inertia for deflection or stiffness calculations, properties of the full unreduced section can be used without significant error when w/t of the compression elements does not exceed 60. For greater accuracy, use Eqs. (10.7) and (10.8) to obtain effective widths.

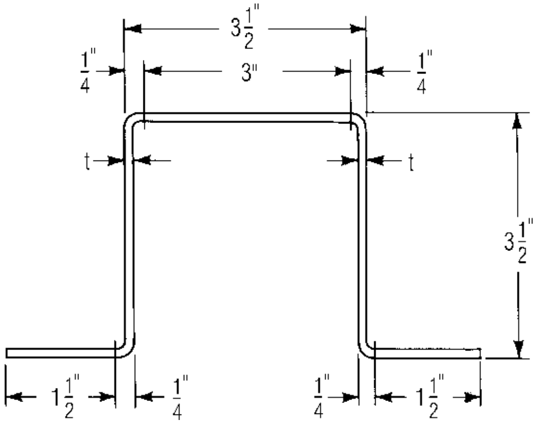


Fig. 10.6 Hat section.

Example ▪ As an example of effective-width determination, consider the hat section in Fig. 10.6. The section is to be made of steel with a specified minimum yield point of $F_y = 33$ ksi. It is to be used as a simply supported beam with the top flange in compression. Safe load-carrying capacity is to be computed. Because the compression and tension flanges have the same width, $f = 33$ ksi is used to compute b/t.

The top flange is a stiffened compression element 3 in wide. If the thickness is $\frac{1}{16}$ in, then the flat-width ratio is 48 ($>220/\sqrt{f}$) and Eq. (10.8) applies. For this value of w/t and $f = 33$ ksi, Eq. (10.8) or Fig. 10.5 gives b/t as 41. Thus, only 85% of the top-flange flat width can be considered effective in this case. The neutral axis of the section will lie below the horizontal center line, and compression will control. In this case, the assumption that $f = F_y = 33$ ksi, made at the start, controls maximum stress, and b/t can be determined directly from Eq. (10.8), without successive approximations.

For a wide hat section in which the horizontal centroidal axis is nearer the compression than the tension flange, the stress in the tension flange controls. So determination of unit stress and effective width of the compression flange requires successive approximations.

("Cold-Formed Steel Design Manual," American Iron and Steel Institute, Washington, D.C., 2002 Edition.)

10.8 Maximum Flat-Width Ratios for Cold-Formed Elements

When the flat-width ratio exceeds about 30 for an unstiffened element and 250 for a stiffened element, noticeable buckling of the element may develop at relatively low stresses. Present practice is to permit buckles to develop in the sheet and take advantage of what is known as the postbuckling strength of the section. The effective-width formulas [Eqs. (10.3), (10.6), (10.7), and (10.8)] are based on this practice of permitting some incipient buckling to occur at the allowable stress. To avoid intolerable deformations, however, overall flat-width ratios, disregarding intermediate stiffeners and based on the actual thickness of the element, should not exceed the following:

Stiffened compression element having one longitudinal edge connected to a web or flange, the other to a simple lip	60
Stiffened compression element having one longitudinal edge connected to a web or flange, the other stiffened by any other kind of stiffener	90
Stiffened compression element with both longitudinal edges connected to a web or flange element, such as in a hat, U, or box type of section	500
Unstiffened compression element	60

10.9 Beam Design Considerations

For the design of beams, considerations should be given to (a) bending strength and deflection, (b) web strength for shear, combined bending and shear, web crippling, and combined bending and web crippling, (c) bracing requirements, (d) shear lag, and (e) flange curling.

Based on the AISI ASD method, the required bending moment computed from working loads shall not exceed the allowable design moment determined by dividing the nominal bending moment by a factor of safety. For *laterally supported beams*, the nominal bending moment is based on the nominal section strength calculated on the basis of either (a) initiation of yielding in the effective section or (b) the inelastic reserve capacity in accordance with the AISI Specification. The factor of safety for bending is taken as 1.67.

10.10 Laterally Unsupported Cold-Formed Beams

In the relatively infrequent cases in which cold-formed sections used as beams are not laterally supported at frequent intervals, the strength must be reduced to avoid failure from lateral instability. The amount of reduction depends on the shape and proportions of the section and the spacing of lateral supports. This is not a difficult obstacle. (For details, see the AISI "North American Specification for the Design of Cold-Formed Steel Structural Members," 2001.)

Because of the torsional flexibility of cold-formed channel and Z sections, their use as beams without lateral support is not recommended. When one flange is connected to a deck or sheathing material, the nominal flexural strength of the member can be determined in accordance with the AISI specification.

When laterally unsupported beams must be used, or where lateral buckling of a flexural member is likely to be a problem, consideration should be given to the use of relatively bulky sections that have two webs, such as hat or box sections (Fig. 10.1*o* and *p*).

10.11 Allowable Shear Strength and Web Crippling Strength in Webs

The shear force at any section should not exceed the allowable shear V_a, kips, calculated as follows:

1. For $h/t \leq 1.510\sqrt{Ek_v/F_y}$,

$$V_a = 0.375t^2\sqrt{K_vF_yE} \leq 0.375F_yht \qquad (10.17a)$$

2. For $h/t > 1.510\sqrt{Ek_v/F_y}$,

$$V_a = 0.565\frac{Ek_vt^3}{h} \qquad (10.17b)$$

where t = web thickness, in

h = depth of the flat portion of the web measured along the plane of the web, in

k_v = shear buckling coefficient = 5.34 for unreinforced webs for which $(h/t)_{\max}$ does not exceed 200

F_y = design yield stress, ksi

E = modulus of elasticity = 29,500 ksi

For design of reinforced webs, especially when h/t exceeds 200, see AISI "North American Specification for the Design of Cold-Formed Steel Structural Members," 2001.

For a web consisting of two or more sheets, each sheet should be considered a separate element carrying its share of the shear force.

For beams with unreinforced webs, the moment M, and the shear V, should satisfy the following interaction equation:

$$\left(\frac{M}{M_{axo}}\right)^2 + \left(\frac{V}{V_a}\right)^2 \leq 1.0 \qquad (10.18)$$

where M_{axo} = allowable moment about the centroidal axis, in-kips

V_a = allowable shear force when shear alone exists, kips

M = applied bending moment, in-kips

V = actual shear load, kips

For beams with reinforced webs, the interaction equation for combined bending and shear is given in the AISI North American Specification.

In addition to the design for shear strength of beam webs, consideration should also be given to the web crippling strength and combined bending and web crippling strength as necessary. The web crippling strength depends on several parameters including h/t, N/t, R/t, F_y, t, and the angle between the plane of the web and the plane of the bearing surface. In the above ratios, N is the actual bearing length and R is the inside bend radius. Other symbols were defined previously.

The 2001 edition of the AISI North American Specification includes the following equation for determining the nominal web crippling strength of webs without holes:

$$P_n = Ct^2 F_y \sin\theta \left(1 - C_R\sqrt{\frac{R}{t}}\right) \times \left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (10.19)$$

In the above equation, coefficients C, C_h, C_N, and C_R together with factors of safety are given in the Specification for built-up sections, single web channel and C-sections, single web Z-sections, single hat sections, and multi-web deck sections under different support and loading conditions. For beam webs with holes, the web crippling strength should be multiplied by the reduction factor, R_c. In addition, the AISI Specification provides interaction equations for combined bending and web crippling strength.

10.12 Concentrically Loaded Compression Members

The following applies to members in which the resultant of all loads acting on the member is an axial load passing though the centroid of the effective section calculated for the nominal buckling stress F_n, ksi. The axial load should not exceed P_a calculated as follows:

$$P_a = \frac{P_n}{\Omega_c} \qquad (10.20)$$

$$P_n = A_e F_n \qquad (10.21)$$

where P_a = allowable compression load, kips

P_n = ultimate compression load, kips

Ω_c = factor of safety for axial compression = 1.80

A_e = effective area at stress F_n, in^2

The magnitude of F_n is determined as follows, ksi:

$$\text{For } \lambda_c \leq 1.5, \quad F_n = (0.658^{\lambda_c^2})F_y \qquad (10.22)$$

$$\text{For } \lambda_c > 1.5, \quad F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad (10.23)$$

where $\lambda_c = \sqrt{F_y/F_e}$

F_y = yield stress of the steel, ksi

F_e = the least of the elastic flexural, torsional and torsional-flexural buckling stress

Figure 10. 7 shows the ratio between the column buckling stress F_n and the yield strength F_y.

For the elastic flexural mode,

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (10.24)$$

where K = effective-length factor

L = unbraced length of member, in

r = radius of gyration of full, unreduced cross section, in

E = modulus of elasticity, ksi

Moreover, non-compact angle sections should be designed for the applied axial load P acting simultaneously with a moment equal to $PL/1000$ applied about the minor principal axis causing compression in the tips of the angle legs.

The slenderness ratio KL/r of all compression members preferably should not exceed 200 except that, during construction only, KL/r preferably should not exceed 300.

For treatment of open cross sections which may be subject to torsional-flexural buckling, refer to

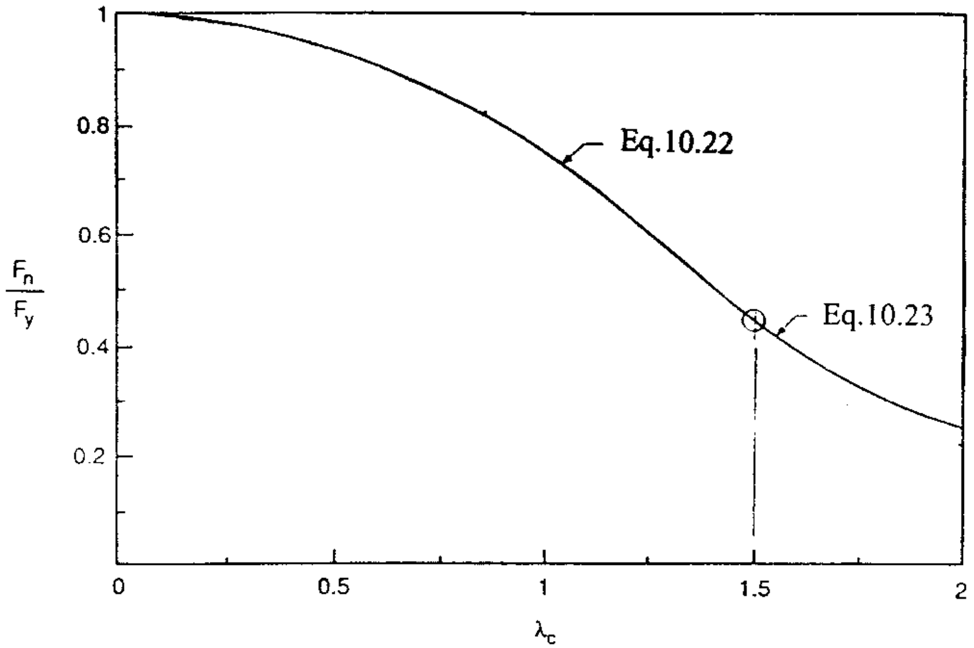


Fig. 10.7 Ratio of nominal column buckling stress to yield strength.

AISI "North American Specification for the Design of Cold-Formed Steel Structural Members," 2001.

10.13 Combined Axial and Bending Stresses

Combined axial and bending stresses in cold-formed sections can be handled in a similar way as for structural steel. The interaction criterion to be used is given in the AISI "North American Specification for the Design of Cold-Formed-Steel Structural Members," 2001.

10.14 Welding of Cold-Formed Steel

Welding offers important advantages to fabricators and erectors in joining metal structural components. Welded joints make possible continuous structures, with economy and speed in fabrication; 100% joint efficiencies are possible.

Conversion to welding of joints initially designed for mechanical fasteners is poor practice. Joints should be specifically designed for welding, to take full advantage of possible savings. Important considerations include the following: The overall assembly should be weldable, welds should be located so that notch effects are minimized, the final appearance of the structure should not suffer from unsightly welds, and welding should not be expected to correct poor fit-up.

Steels bearing protective coatings require special consideration. Surfaces precoated with paint or plastic are usually damaged by welding. And coatings may adversely affect weld quality. Metallically coated steels, such as galvanized (zinc-coated), aluminized, and terne-coated (lead-tin alloy), are now successfully welded using procedures tailored for the steel and its coating.

Generally, steel to be welded should be clean and free of oil, grease, paints, scale, and so on. Paint should be applied only after the welding operation.

("Welding Handbook," American Welding Society, 550 N.W. LeJeune Rd., Miami, FL 33135 www.aws.org; O. W. Blodgett, "Design of Weldments," James F. Lincoln Arc Welding Foundation, Cleveland, OH 44117 www.weldinginnovation.com.)

10.15 Arc Welding of Cold-Formed Steel

Arc welding may be done in the shop and in the field. The basic sheet-steel weld types are shown in Fig. 10.8. Factors favoring arc welding are portability and versatility of equipment and freedom in joint design. (See also Art. 10.14.) Only one side of a joint need be accessible, and overlap of parts is not required if joint fit-up is good.

Distortion is a problem with lightweight steel weldments, but it can be minimized by avoiding overwelding. Weld sizes should be matched to service requirements.

Always design joints to minimize shrinking, warping, and twisting. Jigs and fixtures for holding lightweight work during welding should be used to control distortion. Directions and amounts of distortion can be predicted and sometimes counteracted by preangling the parts. Discrete selection of welding sequence can also be used to control distortion.

Groove welds (made by butting the sheet edges together) can be designed for 100% joint efficiency. Calculations of design stress is usually unnecessary if the weld penetrates 100% of the section.

Stresses in fillet welds should be considered as shear on the throat for any direction of the applied stress. The dimension of the throat is calculated as 0.707 times the length of the shorter leg of the weld. For example, a 12-in-long, $\frac{1}{4}$-in fillet weld has a leg dimension of $\frac{1}{4}$ in, a throat of 0.177 in, and an equivalent area of 2.12 in^2. For all grades of steel, fillet and plug welds should be proportioned according to the AISI specification. For the allowable strength design method, the factors of safety for various weld types are given in the AISI North American Specification.

Shielded-metal-arc welding, also called manual stick electrode, is the most common arc welding process because of its versatility, but it calls for skilled operators. The welds can be made in any position. Vertical and overhead welding should be avoided when possible.

Gas-metal-arc welding uses special equipment to feed a continuous spool of bare or flux-cored wire into the arc. A shielding gas such as argon or carbon dioxide is used to protect the arc zone from the contaminating effects of the atmosphere. The process is relatively fast, and close control can be maintained over the deposit. The process is not

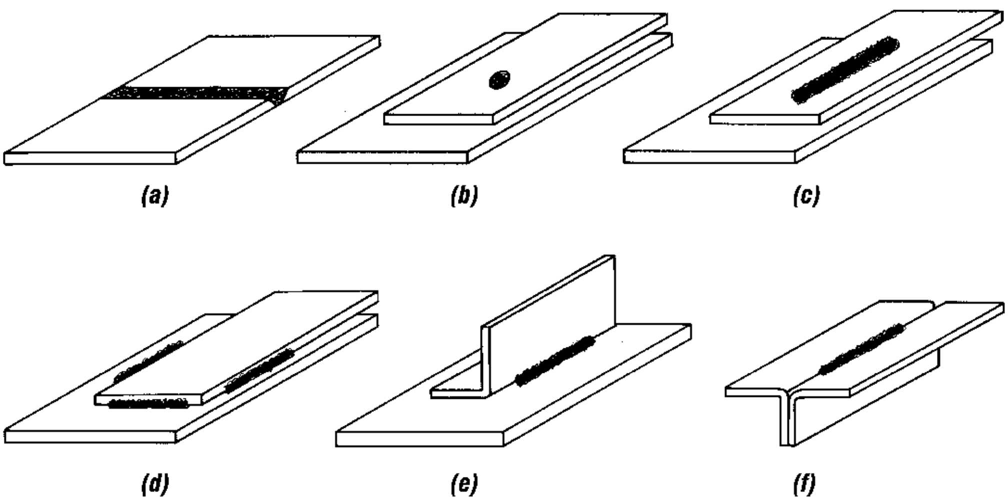


Fig. 10.8 Types of sheet-steel welds: (*a*) Square-groove weld; (*b*) arc spot weld (round puddle weld); (*c*) arc seam weld (oblong puddle weld); (*d*) fillet welds; (*e*) flare-bevel-groove weld; (*f*) flare-V-groove weld.

applicable to materials below $\frac{1}{32}$ in thick but is extensively used for thicker steels.

Gas-tungsten-arc welding operates by maintaining an arc between a nonconsumable tungsten electrode and the work. Filler metal may or may not be added. Close control over the weld can be maintained. This process is not widely used for high-production fabrication, except in specialized applications, because of higher cost.

One form of **arc spot welding** is an adaption of gas-metal-arc welding wherein a special welding torch and automatic timer are employed. The welding torch is positioned on the work and a weld is deposited by burning through the top component of the lap joint. The filler wire provides sufficient metal to fill the hole, thereby fusing together the two parts. Access to only one side of the joint is necessary. Field welding by unskilled operators often makes this process desirable.

Another form of arc spot welding utilizes gas-tungsten arc welding. The heat of the arc melts a spot through one of the sheets and partly through the second. When the arc is cut off, the pieces fuse. No filler metal is added. Design of arc-welded joints of sheet steel is fully treated in the American Welding Society "Structural Welding Code-Sheet Steel," AWS D1.3, www.aws.org. Allowable maximum-load capacities of arc-welded joints of sheet steel, including cold-formed members 0.180 in or less thick, are determined in the following ways.

Groove Welds in Butt Joints ▪ The maximum load for a groove weld in a butt joint, welded from one or both sides, is determined by the base steel with the lower strength in the connection, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained.

Arc Spot Welds ▪ These are permitted for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) may not be made on steel where the thinnest connected part is over 0.15 in thick, nor through a combination of steel sheets having a total thickness of over 0.15 in. Arc spot welds should be specified by minimum effective diameter of fused area d_e. Minimum effective allowable diameter is $\frac{3}{8}$ in. The nominal shear load P_n, on each arc spot weld between two or more sheets and a supporting member should not exceed the smaller of the values calculated from Eq. (10.25) or, as appropriate, Eqs. (10.26), (10.27), (10.28).

$$P_n = 0.589 d_e^2 F_{xx} \tag{10.25}$$

For $d_a/t \leq 0.815\sqrt{E/F_u}$:

$$P_n = 2.20 t d_a F_u \tag{10.26}$$

For $0.815\sqrt{E/F_u} < d_a/t < 1.397\sqrt{E/F_u}$:

$$P_n = 0.280\left[1 + 5.59\frac{t}{d_a}\sqrt{\frac{E}{F_u}}\right] t d_a F_u \tag{10.27}$$

For $d_a/t \geq 1.397\sqrt{E/F_u}$:

$$P_n = 1.40 t d_a F_u \tag{10.28}$$

where t = sum of thicknesses, in (exclusive of coatings), of all the sheets involved in shear transfer through the spot weld

d_a = average diameter, in, of spot weld at middepth of the shear transfer zone

= $d - t$ for a single sheet or multiple sheets (not more than four lapped sheets over a supporting member)

d = visible diameter, in, of outer surface of spot weld

d_e = effective diameter, in, of fused area

= $0.7d - 1.5t$ but not more than $0.55d$

F_{xx} = stress-level designation, ksi, in AWS electrode classification

F_u = tensile strength of base metal as specified, ksi

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed should not be less than the value of $e_{\min}$ as given by

$$e_{\min} = e\Omega_e \tag{10.29}$$

where $e = P/(F_u t)$

Ω_e = factor of safety for sheet tearing

= 2.20 when $F_u/F_{sy} \geq 1.08$

= 2.55 when $F_u/F_{sy} < 1.08$

F_u = tensile strength of base metal as specified, ksi

P = force transmitted by weld, kips

t = total combined base steel thickness, in

In addition, the distance from the centerline of any weld to the end or boundary of the connected member may not be less than $1.5d$. In no case may the clear distance between welds and the end of the member be less than d.

The nominal tension load P_n on each arc spot weld between sheet and supporting member should be computed as the smaller of either:

$$P_n = 0.785 d_e^2 F_{xx} \tag{10.30a}$$

or

$$P_n = 0.8\left(\frac{F_u}{F_y}\right)^2 t d_a F_u \tag{10.30b}$$

and the following limitations also apply:

$$t d_a F_u \leq 3;\ e_{\min} \geq d;\ F_{xx} \geq 60\ \text{ksi};$$
$$F_u \leq 82\ \text{ksi};\ F_{xx} > F_u$$

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter d_e, or larger average diameter d_a, as applicable, this larger diameter may be used, if the welding procedure required for making those welds is followed.

Arc Seam Welds ▪ These apply to the following joints:

1. Sheet to thicker supporting member in the flat position
2. Sheet to sheet in the horizontal or flat position

The nominal shear load P_n on each arc seam weld should not exceed the values calculated from either Eq. (10.31) or (10.32).

$$P_n = \left[\frac{\pi d_e^2}{4} + L d_e\right] 0.75 F_{xx} \tag{10.31}$$

$$P_n = 2.5 t F_u (0.25L + 0.96 d_a) \tag{10.32}$$

where d = width of arc seam weld, in

L = length of seam weld not including the circular ends, in (For computation purposes, L should not exceed $3d$)

d_a = average width of arc seam weld, in

$= d - t$ for a single sheet or double sheets

d_e = effective width of arc seam weld at fused surfaces, in

$= 0.7d - 1.5t$

F_u and F_{xx} are strengths as previously defined for arc spot welds. Also, minimum edge distance is the same as that defined for arc spot welds. If it can be shown by measurement that a given weld procedure will consistently give a larger effective width d_e or larger average width, d_a, as applicable, this value may be used, if the welding procedure required for making the welds that were measured is followed.

Fillet Welds ▪ These may be used for welding of joints in any position, either sheet to sheet or sheet to thicker steel member. The nominal shear load P_n, kips, on a fillet weld in lap or T joints should not exceed the following:

For Longitudinal Loading

For $L/t < 25$:

$$P_n = \left(1 - \frac{0.01L}{t}\right) t L F_u \tag{10.33}$$

For $L/t \geq 25$:

$$P_n = 0.75 t L F_u \tag{10.34}$$

For Transverse Loading

$$P_n = t L F_u \tag{10.35}$$

where t = least thickness of sheets being fillet welded, in

L = length of fillet weld, in

In addition, for $t > 0.10$ in, the nominal load for a fillet weld in lap and T joints should not exceed

$$P_n = 0.75 t_w L F_{xx} \tag{10.36}$$

where t_w = effective throat, in, = lesser of $0.707w_1$ or $0.707w_2$; w_1 and w_2 are the width of the weld legs; and F_u and F_{xx} are strengths as previously defined.

Flare-Groove Welds ▪ These may be used for welding of joints in any position, either:

1. Sheet to sheet for flare-V-groove welds
2. Sheet to sheet for flare-bevel-groove welds
3. Sheet to thicker steel member for flare-bevel-groove welds

The nominal shear load, P_n, kips, on a weld is governed by the thickness, t, in, of the sheet steel adjacent to the weld.

For flare-bevel-groove welds, the transverse load should not exceed

$$P_n = 0.833tLF_u \tag{10.37}$$

For flare-V-groove welds, when the effective throat t_w is equal to or greater than the least thickness t of the sheets being joined but less than $2t$, or if the lip height is less than the weld length L, in, the longitudinal loading should not exceed

$$P_n = 0.75tLF_u \tag{10.38}$$

If t_w is equal to or greater than $2t$ and the lip height is equal to or greater than L,

$$P_n = 1.50tLF_u \tag{10.39}$$

In addition, if $t > 0.10$ in

$$P_n = 0.75t_wLF_{xx} \tag{10.40}$$

10.16 Resistance Welding of Cold-Formed Steel

Resistance welding comprises a group of welding processes wherein coalescence is produced by the heat obtained from resistance of the work to flow of electric current in a circuit of which the work is a part and by the application of pressure. Because of the size of the equipment required, resistance welding is essentially a shop process. Speed and low cost are factors favoring its selection.

Almost all resistance-welding processes require a lap-type joint. The amount of contacting overlap varies from $\frac{3}{8}$ to 1 in, depending on sheet thickness. Access to both sides of the joint is normally required. Adequate clearance for electrodes and welder arms must be provided.

Spot welding is the most common resistance-welding process. The work is held under pressure between two electrodes through which an electric current passes. A weld is formed at the interface between the pieces being joined and consists of a cast-steel nugget. The nugget has a diameter about equal to that of the electrode face and should penetrate about 60 to 80% of each sheet thickness.

For structural design purposes, spot welding can be treated the same way as rivets, except that no reduction in net section due to holes need be made. Table 10.1 gives the essential information for uncoated material based on "Recommended Practices for Resistance Welding," American Welding Society. Note that the thickest material for plain spot welding is $\frac{1}{8}$ in. Thicker material can be resistance-welded by projection or by pulsation methods if high-capacity spot welders for material thicker than $\frac{1}{8}$ in are not available.

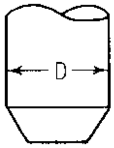
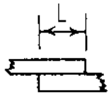
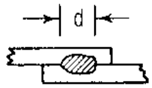
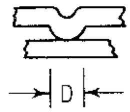
Projection welding is a form of spot welding in which the effects of current and pressure are intensified by concentrating them in small areas of projections embossed in the sheet to be welded. Thus, satisfactory resistance welds can be made on thicker material using spot welders ordinarily limited to thinner stocks.

Pulsation welding, or **multiple-impulse welding**, is the making of spot welds with more than one impulse of current, a maneuver that makes some spot welders useful for thicker materials. The trade-offs influencing choice between projection welding and impulse welding involve the work being produced, volume of output, and equipment available.

The spot welding of higher-strength steels than those contemplated under Table 10.1 may require special welding conditions to develop the higher shear strengths of which the higher-strength steels are capable. All steels used for spot welding should be free of scale; therefore, either hot-rolled and pickled or cold-rolled steels are usually specified. Steels containing more than 0.15% carbon are not as readily spot welded as lower-carbon steels, unless special techniques are used to ensure ductile welds. However, high-carbon steels such as ASTM A653, SS Grade 50 (formerly, Grade D), which can have a carbon content as high as 0.40% by heat analysis, are not recommended for resistance welding. Designers should resort to other means of joining such steels.

Maintenance of sufficient overlaps in detailing spot-welded joints is important to ensure consistent weld strengths and minimum distortions at joints. Minimum weld spacings specified in Table 10.1 should be observed, or shunting to previously made adjacent welds may reduce the electric current to a level below that needed for welds being made. Also, the joint design should provide sufficient clearance between electrodes and work to prevent short-circuiting of current needed to make satisfactory spot welds. For design purposes, the AISI North American Specification provides design equations and a factor of safety on the basis of "Recommended Practices for Resistance Welding of Coated Low-Carbon Steel," American Welding Society, 550 N.W. LeJeune Rd., Miami, FL 33135, www.aws.org.

Table 10.1 Test Data for Spot and Projection Welding

Thickness t of Thinnest Piece, in	Min OD of Electrode D, in	Min Contacting Overlap, in	Min Weld Spacing c to c, in	Approx Dia of Fused Zone, in	Min Shear Strength per Weld, lb	Dia of Projection D, in
						
			Spot Welding			
0.021	3/8	7/16	3/8	0.13	320	
0.031	3/8	7/16	1/2	0.16	570	
0.040	1/2	1/2	3/4	0.19	920	
0.050	1/2	9/16	7/8	0.22	1,350	
0.062	1/2	5/8	1	0.25	1,850	
0.078	5/8	11/16	1 1/4	0.29	2,700	
0.094	5/8	3/4	1 1/2	0.31	3,450	
0.109	5/8	13/16	1 5/8	0.32	4,150	
0.125	7/8	7/8	1 3/4	0.33	5,000	
			Projection Welding			
0.125		11/16	9/16	0.338	4,800	0.281
0.140		3/4	3/8	7/16	6,000	0.312
0.156		13/16	11/16	1/2	7,500	0.343
0.171		7/8	3/4	9/16	8,500	0.375
0.187		15/16	13/16	9/16	10,000	0.406

10.17 Bolting of Cold-Formed-Steel Members

Bolting is convenient in cold-formed-steel construction. Bolts, nuts, and washers should generally conform to the requirements of the ASTM specifications listed in Table 10.2.

Maximum sizes permitted for bolt holes are given in Table 10.3. Holes for bolts may be standard or oversized round or slotted. Standard holes should be used in bolted connections when possible. The length of slotted holes should be normal to the direction of shear load. Washers should be installed over oversized or slotted holes.

Hole Locations ▪ The distance e, measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed, should not be less than the value of e_{min}

Table 10.2 ASTM Bolt, Nut, and Washer Steels

A194	Carbon and Alloy Steel Nuts for High-Pressure and High-Temperature Service
A307	(Type A) Carbon Steel Bolts and Studs
A325	High Strength Bolts for Structural Steel Joints
A354	(Grade BD) Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than ½ in)
A449	Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than ½ in)
A490	Heat-Treated Steel Structural Bolts
A563	Carbon and Alloy Steel Nuts
F436	Hardened Steel Washers
F844	Washers, Steel, Plain (Flat), Unhardened for General Use
F959	Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

determined by Eq. (10.41),

$$e_{\min} = e\Omega_e \tag{10.41}$$

where $e = \dfrac{P}{F_u t}$ (10.42)

Ω_e = factor of safety for sheet tearing

= 2.00 when $F_u/F_{sy} \geq 1.08$

= 2.22 when $F_u/F_{sy} < 1.08$

P = force transmitted by bolt, kips

t = thickness of thinnest connected part, in

F_u = tensile strength of connected part, ksi

F_{sy} = yield strength of connected part, ksi

In addition, the minimum distance between centers of bolt holes should provide sufficient clearance for bolt heads, nuts, washers, and the wrench but not less than three times the nominal bolt diameter d. The distance from the center of any standard hole to the end or boundary of the connecting member should not be less than $1\frac{1}{2}\,d$.

Allowable Tension ▪ The tension force on the net sectional area A_n of a bolted connection should not exceed P_a calculated from Eq. (10.43).

$$P_a = \frac{P_n}{\Omega_t} \tag{10.43}$$

where $P_n = A_n F_t$ (10.44)

F_t = nominal limit for tension stress on net section, ksi

F_t and Ω_t are determined as follows:

1. When $t \geq \frac{3}{16}$ in, as required by the AISC Specification.
2. When $t < \frac{3}{16}$ in, the tensile capacity of a bolted member should be determined from Section C2 of the AISI North American Specification. For fracture in the effective net section of flat sheet connections having washers provided under the bolt head and the nut, the tensile stress F_t can be computed as follows:
   a. For a single bolt or a single row of bolts perpendicular to the force,

$$F_t = \left(0.1 + \frac{3d}{s}\right)F_u \leq F_u \tag{10.45a}$$

   b. For multiple bolts in the line parallel to the force,

$$F_t = F_u \tag{10.45b}$$

where Ω_t = factor of safety for tension on the net section

Table 10.3 Maximum Size of Bolt Holes, in

Nominal Bolt Dia, d, in	Standard Hole Dia, d, in	Oversized Hole Dia, d, in	Short-Slotted Hole Dimensions, in	Long-Slotted Hole Dimensions, in
$<\frac{1}{2}$	$d + \frac{1}{32}$	$d + \frac{1}{16}$	$(d + \frac{1}{32}) \times (d + \frac{1}{4})$	$(d + \frac{1}{32}) \times (2\frac{1}{2}\,d)$
$\geq\frac{1}{2}$	$d + \frac{1}{16}$	$d + \frac{1}{8}$	$(d + \frac{1}{16}) \times (d + \frac{1}{4})$	$(d + \frac{1}{16}) \times (2\frac{1}{2}\,d)$

= 2.22 for single shear and 2.00 for double shear

d = nominal bolt diameter, in

s = sheet width divided by number of bolt holes in cross section, in

F_u = tensile strength of the connected part, ksi

When washers are not provided under the bolt head and nut, see AISI Specification. The Specification also provides the design information for flat sheet connections having staggered hole patterns and structural members such as angles and channels.

Table 10.4b Modification Factor, m_f, for Type of Bearing Connection

Type of Bearing Connection	m_f
Single Shear and Outside Sheets of Double Shear Connection with Washers under Both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection without Washers under Both Bolt Head and Nut, Or with only One Washer	0.75
Inside Sheet of Double Shear Connection with or without Washers	1.33

Allowable Bearing ▪ The bearing force should not exceed P_a calculated from Eq. (10.46).

$$P_a = \frac{P_n}{\Omega_b} \tag{10.46}$$

where $P_n = m_f C d t F_u$, kips (10.47)

Ω_b = factor of safety for bearing = 2.50

C = bearing factor determined from Table 10.4a

d = nominal bolt diameter, in

t = uncoated sheet thickness, in

F_u = tensile strength of sheet, ksi

m_f = modification factor determined from Table 10.4b

Allowable Bolt Sresses ▪ Table 10.5 lists nominal shear and tension for various grades of bolts. The bolt force resulting in shear, tension, or combination of shear and tension should not exceed allowable bolt force P_a calculated from Eq. (10.48).

$$P_a = \frac{A_b F}{\Omega} \tag{10.48}$$

where A_b = gross cross-sectional area of bolt, in^2

F = nominal unit stress given by F_{nv}, F_{nt} or F'_{nt} in Tables 10.5 and 10.6

Factors of safety given in Tables 10.5 and 10.6 should be used to compute allowable loads on bolted joints.

Table 10.6 lists nominal tension stresses for bolts subject to the combination of shear and tension.

Table 10.4a Bearing Factor, C

Thickness of Connected Part, t, in	Ratio of Fastener Diameter to Member Thickness, d/t	C
$0.024 \le t < 0.1875$	$d/t < 10$	3.0
	$10 \le d/t \le 22$	$4 - 0.1(d/t)$
	$d/t > 22$	1.8

Example—Tension Joints with Two Bolts ▪ Assume that the bolted tension joints of Fig. 10.9 comprise two sheets of $\frac{3}{16}$-in-thick, A1008 SS Grade 33 steel. For this steel, $F_y = 33$ ksi and $F_u = 48$ ksi. The sheets in each joint are 4 in wide and are connected to two $\frac{5}{8}$-in-diameter, A325 bolts, with washers under both bolt head and nut. Determine the allowable load based on the ASD method.

A. *Based on Tensile Strength of Steel Sheets*

Case 1 shows the two bolts arranged in a single transverse row. A force $T/2$ is applied to each bolt and the total force T has to be carried by the net section of each sheet through the bolts. So, in Eq. (10.45a), spacing of the bolts $s = 2$ in and $d/s = (\frac{5}{8})/2 = 0.312$. The tension stress in the net section,

Table 10.5 Nominal Tensile and Shear Strength for Bolts

Description of Bolts	Tensile Strength		Shear Strength	
	Factor of Safety Ω	Nominal Stress F_{nt}, ksi	Factor of Safety Ω	Nominal Stress F_{nv}, ksi
A307 Bolts, Grade A, ¼ in $\leq d <$ ½ in.	2.25	40.5	2.4	24.0
A307 Bolts, Grade A, $d \geq$ ½ in	2.25	45.0		27.0
A325 bolt, when threads are not excluded from shear planes	2.0	90.0		54.0
A325 bolts, when threads are excluded from shear planes		90.0		72.0
A354 Grade BD Bolts ¼ in $\leq d <$ ½ in, when threads are not excluded from shear planes		101.0		59.0
A354 Grade BD Bolts ¼ in $\leq d <$ ½ in, when threads are excluded from shear planes		101.0		90.0
A449 Bolts, ¼ in $\leq d <$ ½ in, when threads are not excluded from shear planes		81.0		47.0
A449 Bolts, ¼ in $\leq d <$ ½ in, when threads are excluded from shear planes		81.0		72.0
A490 Bolts, when threads are not excluded from shear planes		112.5		67.5
A490 Bolts, when threads are excluded from shear planes		112.5		90.0

Table 10.6 Nominal Tension Stress, F'_{nt} (ksi), for Bolts Subject to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Factor of Safety Ω
A325 Bolts	$110-3.6f_v \leq 90$	$110-2.8f_v \leq 90$	2.0
A354 Grade BD Bolts	$122-3.6f_v \leq 101$	$122-2.8f_v \leq 101$	
A449 Bolts	$100-3.6f_v \leq 81$	$100-2.8f_v \leq 81$	
A490 Bolts	$136-3.6f_v \leq 112.5$	$136-2.8f_v \leq 112.5$	
A307 Bolts, Grade A			
When ¼ in $\leq d <$ ½ in	$52 - 4f_v \leq 40.5$		2.25
When $d \geq$ ½ in	$58.5 - 4f_v \leq 45$		

The shear stress, f_v, shall also satisfy Table 10.5.

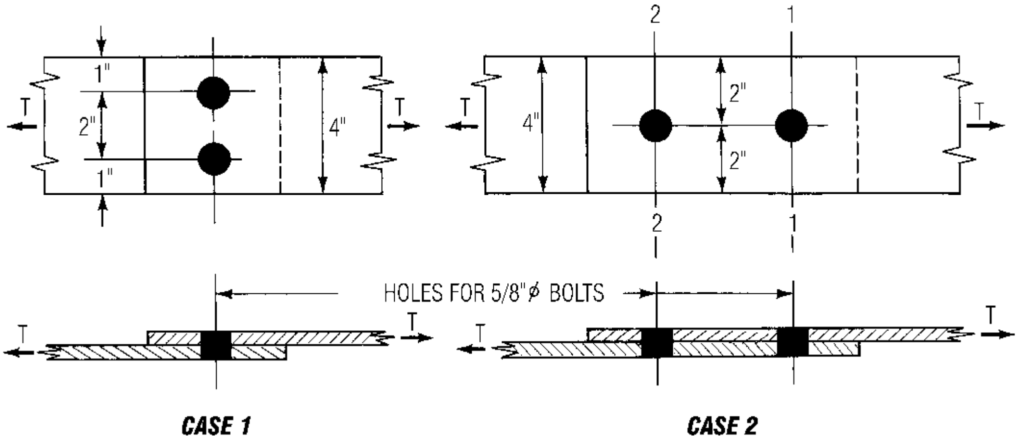


Fig. 10.9 Bolted connections with two bolts.

computed from Eq. (10.45a), is then

$$F_t = (0.1 + 3 \times 0.312)F_u = 1.04F_u > F_u$$

Use $F_t = F_u$.

Substitution in Eq. (10.44) with $F_u = 48\,\text{ksi}$ yields the nominal tension load on the net section:

$$P_n = [4 - (2 \times 11/16)] \times 3/16 \times 48 = 23.63\,\text{kips}$$

The allowable load is

$$P_a = \frac{P_n}{\Omega} = \frac{23.63}{2.22} = 10.64\,\text{kips}$$

This compares with the tensile strength of each sheet for tension member design according to Section C2 of Appendix A of the 2001 edition of the AISI North American Specification:

For yielding:

$$T_n = A_g F_y = (4 \times 3/16)(33) = 24.75\,\text{kips}$$

$$T_a = \frac{T_n}{\Omega} = \frac{24.75}{1.67} = 14.82\,\text{kips}$$

For fracture away from connection:

$$T_n = A_n F_u = [4 - (2 \times 11/16)] \times 3/16 \times 48 = 23.63\,\text{kips}$$

$$T_a = \frac{T_n}{\Omega} = \frac{23.63}{2.00} = 11.82\,\text{kips} < 14.82\,\text{kips}$$

Use $T_a = 11.82\,\text{kips}$. Since $T_a > P_a$, use $P_a = 10.64\,\text{kips}$ for Case 1.

Case 2 shows the two bolts, with 4-in spacing, arranged in a single line along the direction of applied force. From Eq. (10.45b),

$$F_t = F_u \text{ and } P_n = A_n F_t$$

$$P_a = 10.64\,\text{kips (same as Case 1)}$$

Compare with the tensile strength for tension member design:

For yielding (same as Case 1):

$$T_a = 14.82\,\text{kips}$$

For fracture away from connection:

$$T_n = A_n F_u = (4 - 11/16) \times 3/16 \times 48 = 29.81\,\text{kips}$$

$$T_a = 29.81/2.00 = 14.91\,\text{kips} > 14.82\,\text{kips}$$

Use $T_a = 14.82\,\text{kips}$. Since $T_a > P_a$, use $P_a = 10.64\,\text{kips}$ for Case 2.

B. *Check for Bearing Capacity*

From Eq. (10.47), the bearing strength P_n per bolt of the $\frac{3}{16}$-in-thick steel sheet is:

$$P_n = m_f C d t F_u$$

Since $d/t = (5/8)/(3/16) = 3.33 < 10$, $C = 3.0$. For single shear connection with washers under both bolt head and nut, $m_f = 1.00$. Therefore,

$$P_n = 1 \times 3 \times 5/8 \times 3/16 \times 48 = 16.88\,\text{kips}$$

The allowable bearing load for two bolts:

$$P_a = 2\frac{P_n}{\Omega} = 2 \times \frac{16.88}{2.50} = 13.50\,\text{kips}$$

$$> 10.64\,\text{kips O.K.}$$

C. *Check for Shear Strength of Bolts*

Using the A325 bolts with threads not excluded from the shear plane, the allowable shearing strength of each bolt is:

$$P_s = A_b \frac{F_{nv}}{\Omega} = (5/8)^2 \times 0.7854 \times \frac{54}{2.4} = 6.9\,\text{kips}$$

For two bolts, the allowable load is:

$$P_a = 2 \times 6.9 = 13.8\,\text{kips} > 10.64\,\text{kips O.K.}$$

D. *Bolt Spacing and Edge Distance*

From the above calculations, the allowable load for Cases 1 and 2 is 10.64 kips. The minimum distance between a bolt center and adjacent bolt edge or sheet edge in the direction of applied force for Cases 1 and 2 is:

$$e = \frac{P}{F_u t} = \frac{10.64/2}{(48)(3/16)} = 0.59\,\text{in}$$

$$e_{\min} = e\Omega = 0.59 \times 2 = 1.18\,\text{in}$$

The bolt spacing and edge distance should also be checked for other AISI dimensional requirements.

In addition to the above calculations, block shear rupture should also be considered according to the AISI North American Specification.

10.18 Tapping Screws for Joining Light-Gage Members

Tapping screws are often used for making field joints in lightweight construction, especially in connections that do not carry any calculated gravity load. Such screws are of several types (Fig. 10.10). Tapping screws used for fastening sheet-metal siding and roofing are generally preassembled with Neoprene washers for effective control of leaks, squeaks, cracks, or crazing, depending on the surface of the material. For best results, when Type A sheet-metal screws are specified, screws should be fully threaded to the head to assure maximum hold in sheet metal.

Tapping screws are made of steel so hardened that their threads form or cut mating threads in one or both relatively soft materials being joined. Slotted, hexagon, and plain heads are provided for installing them. The thread-forming types all

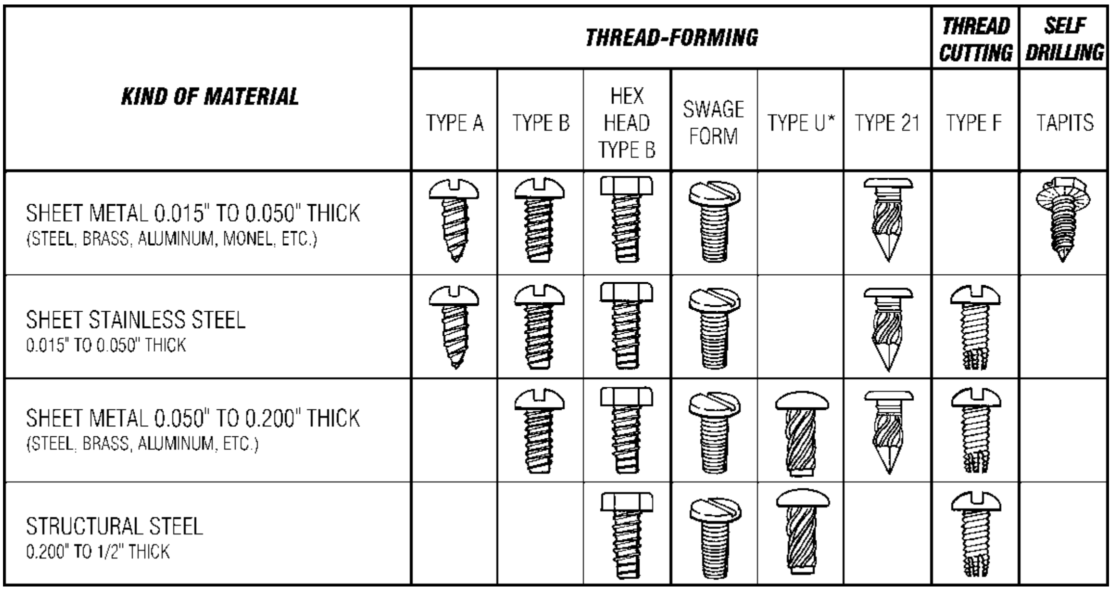


Fig. 10.10 Tapping screws. Note: A blank space does not necessarily signify that the type of screw cannot be used for this purpose; it denotes that the type of self-tapping screw will not generally give the best results in the material. (*Parker-Kalon Corp., Emhart Corp.*)

require predrilled holes appropriate in diameter to the hardness and thickness of the material being joined. Types A and B are screwed, whereas types U and 21 are driven. Predrilled holes are required for thread-cutting Type F, but no hole is required for self-drilling TAPIT type.

Tapping screws may be used for light-duty connections, such as fastening bridging to sheet-metal joists and studs. Since 1996 the AISI Specification included design rules for determining nominal load for shear and tension. The factors of safety to be used for computing the allowable load is 3.0.

Steel Roof and Floor Deck

Steel roof deck consists of ribbed sheets with nesting or upstanding-seam joints designed for the support of roof loads between purlins or frames. A typical roof-deck assembly is shown in Fig. 10.11. The Steel Deck Institute, P.O. Box 25, Fox River Grove, IL 6002, www.sdi.org, has developed much useful information on steel roof-deck.

10.19 Types of Steel Roof Deck

As a result of the Steel Deck Institute's efforts to improve standardization, steel roof deck is now classified. All types consist of long, narrow sections with longitudinal ribs at least $1\frac{1}{2}$ in deep spaced about 6 in on centers. Other rib dimensions are shown in Fig. 10.12*a* to *c* for some standard styles. Such steel roof deck is commonly available in 24- and 30-in covering widths, but sometimes in 18- and 36-in widths, depending on the manufacturer. Figure 10.12*d* and *e* shows full-width executions in cross section. Usual spans, which may be simple, two-span continuous, or three-span continuous, range from 4 to 10 ft. The SDI "Design Manual for Composite Decks, Form Decks, Roof Decks and Cellular Deck Floor Systems with Electrical Distribution" gives allowable total uniform loading (dead and live), lb/ft^2, for various gages, spans, and rib widths.

Some manufacturers make special long-span roof-deck sections, such as the 3-in-deep Type N roof deck shown in Fig. 10.13.

The weight of the steel roof deck shown in Fig. 10.12 varies, depending on rib dimensions and edge details. For structural design purposes, weights of 2.8, 2.1, and 1.7 lb/ft^2 can be used for the usual design thicknesses of 0.048, 0.036, and 0.030 in, respectively, for black steel in all rib widths, as commonly supplied.

Steel roof deck is usually made of structural-quality sheet or strip, either black or galvanized, ASTM A611, Grade C, D or E or A653 Structural Quality with a minimum yield strength of 33 ksi. Black steel is given a shop coat of priming paint by the roof-deck manufacturer. Galvanized steel may or may not be painted; if painted, it should first be bonderized to ensure paint adherence.

The thicknesses of steel commonly used are 0.048 and 0.036 in, although most building codes also permit 0.030-in-thick steel to be used.

SDI Design Manual includes "Recommendations for Site Storage and Erection," and also provides standard details for accessories. See also SDI "Manual of Construction with Steel Deck."

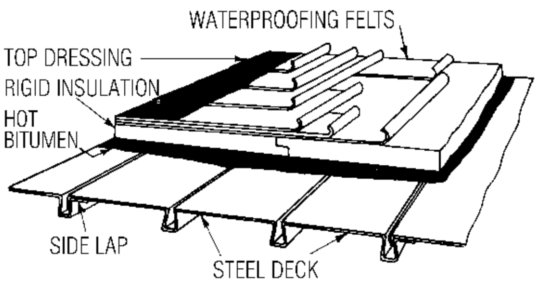


Fig. 10.11 Roof-deck assembly.

10.20 Load-Carrying Capacity of Steel Roof Deck

The Steel Deck Institute has adopted a set of basic design specifications, with limits on rib dimensions, as shown in Fig. 10.12*a* to *c*, to foster standardization of steel roof deck. This also has made possible publication by SDI of allowable uniform loading tables. These tables are based on section moduli and moments of inertia computed with effective-width procedures stipulated in the AISI "Specification for the Design of Cold-Formed Steel

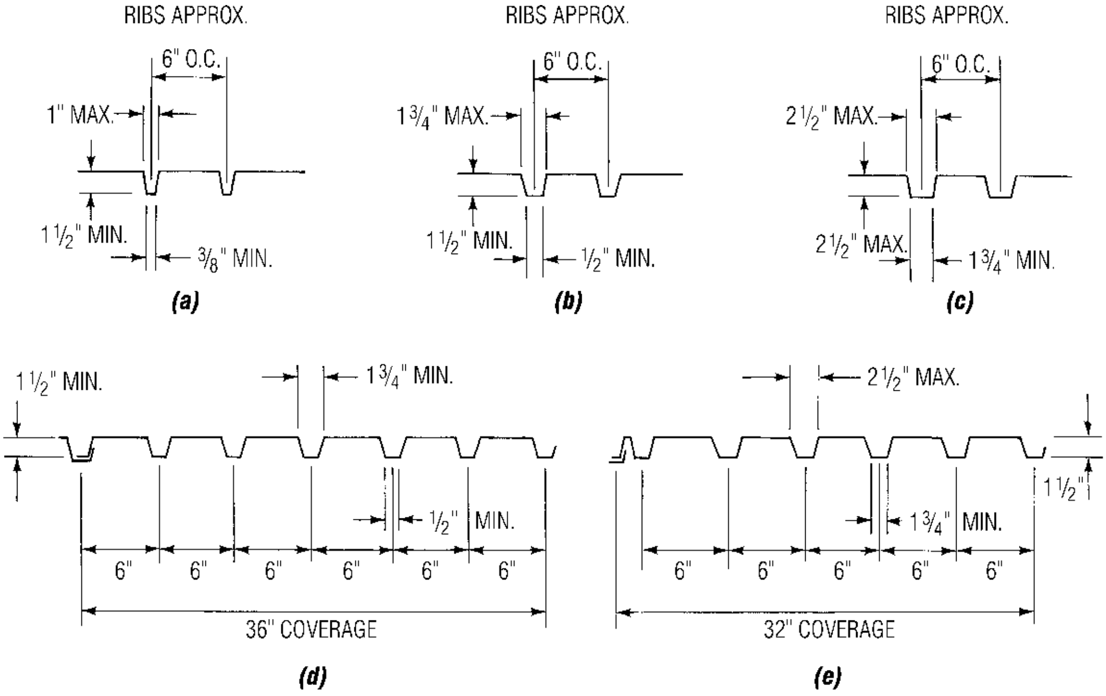


Fig. 10.12 Typical cold-formed-steel roof-deck sections: (*a*) Narrow-rib; (*b*) intermediate rib; (*c*) wide rib; (*d*) intermediate rib in 36-in-wide sheets with nested side laps; (*e*) wide rib in 32-in-wide sheets with upstanding seams.

Structural Members" (Art. 10.4). SDI has banned compression flange widths otherwise assumed to be effective. SDI "Basic Design Specifications" contain the following provisions:

Moment and Deflection Coefficients ▪ Where steel roof decks are welded to the supports, a moment coefficient of $\frac{1}{10}$ (applied to *WL*) shall be used for three or more spans. Deflection coefficients of 0.0054 and 0.0069 (applied to WL^3/EI) shall be used for two span and three span, respectively. All other steel roof-deck installations shall be designed as simple spans, for which moment and deflection coefficients are $\frac{1}{8}$ and $\frac{5}{384}$, respectively.

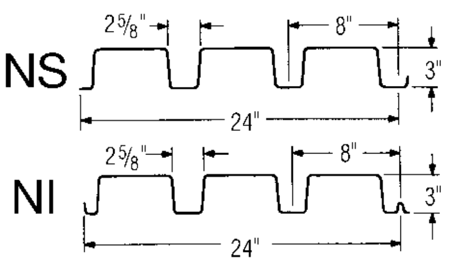


Fig. 10.13 Roof-deck cross sections types NS and NI of 9- to 15-ft spans.

Maximum Deflections ▪ The deflection under live load shall not exceed $\frac{1}{240}$ of the clear span, center to center of supports. (Suspended ceiling, lighting fixtures, ducts, or other utilities shall not be supported by the roof deck.)

Anchorage ▪ Steel roof deck shall be anchored to the supporting framework to resist the following gross uplifts:

45 lb/ft^2 for eave overhang

30 lb/ft^2 for all other roof areas

The dead load of the roof-deck construction may be deducted from the above uplift forces.

Diaphragm Action ▪ Steel deck when properly attached to a structural frame becomes a diaphragm capable of resisting in-plane shear forces. A major SDI steel-deck diaphragm testing program at West Virginia University has led to tentative shear-design recommendations given in two publications that can be ordered from SDI. For design purposes, see SDI Diaphragm Design Manual.

10.21 Details and Accessories for Steel Roof Deck

In addition to the use of nesting or upstanding seams, most roof-deck sections are designed so that ends can be lapped shingle fashion.

Special ridge, valley, eave, and cant strips are provided by the roof-deck manufacturers.

Connections ▪ Roof decks are commonly arc welded to structural steel with puddle welds at least $\frac{1}{2}$ in in diameter or with elongated welds of equal perimeter. Electrodes should be selected and amperage adjusted to fuse all layers of roof deck to steel supporting members without creating blow-holes around the welds. Welding washers are recommended for thicknesses less than 0.030 in.

One-inch-long fillet welds should be used to connect lapped edges of roof deck.

Tapping screws are an alternative means of attaching steel roof deck to structural support members, which should be at least $\frac{1}{16}$ in thick. All edge ribs and a sufficient number of interior ribs should be connected to supporting frame members at intervals not exceeding 18 in. When standard steel roof-deck spans are 5 ft or more, adjacent sheets should be fastened together at midspan with either welds or screws. Details to be used depend on job circumstances and manufacturer's recommendations.

Insulation ▪ Although insulation is not ordinarily supplied by the roof-deck manufacturer, it is standard practice to install $\frac{3}{4}$- or 1-in-thick mineral fiberboard between roof deck and roofing. The Steel Deck Institute further recommends: All steel decks shall be covered with a material of sufficient insulating value as to prevent condensation under normal occupancy conditions. Insulation shall be adequately attached to the steel deck by means of adhesives or mechanical fasteners. Insulation materials shall be protected from the elements at all times during storage and installation.

Fire Resistance ▪ The "Fire Resistance Directory," Underwriters' Laboratories Inc., 333 Pfingsten Rd., Northbrook, IL 60062, lists fire-resistance ratings for steel roof-deck construction. SDI Design Manual provides the UL Designs for 2-hour rating with directly-applied protection, 2-hour rating with metal lath and plaster ceiling, and 1-hour rating with suspended acoustical ceiling.

10.22 Composite Floor Deck

Research on the structural behavior of cold-formed-steel decks filled with concrete has demonstrated that composite action between these materials can be achieved in floors. Floor deck from one supplier is available in the thicknesses from 0.030 to 0.060 in and rib depths of $1\frac{1}{2}$, 2, and 3 in, with embossed surfaces for improved bonding with the concrete in-fill. Figure 10.14 shows three cross sections of composite floor deck.

10.23 Cellular Steel Floor and Roof Panels*

Several designs of cellular steel panels and fluted steel panels for floor and roof construction are shown in Fig. 10.15. One form of cellular steel floor for distribution of electrical wiring, telephone cables, and data cables is described in the following and illustrated in Fig. 10.16. This system is used in many kinds of structures, including massive high-rise buildings for institutional, business, and mercantile occupancies. It consists of profiled steel deck containing multiple wiring cells with structural concrete on top. The closely spaced, parallel, cellular raceways are connected

*Written by R. E. Albrecht.

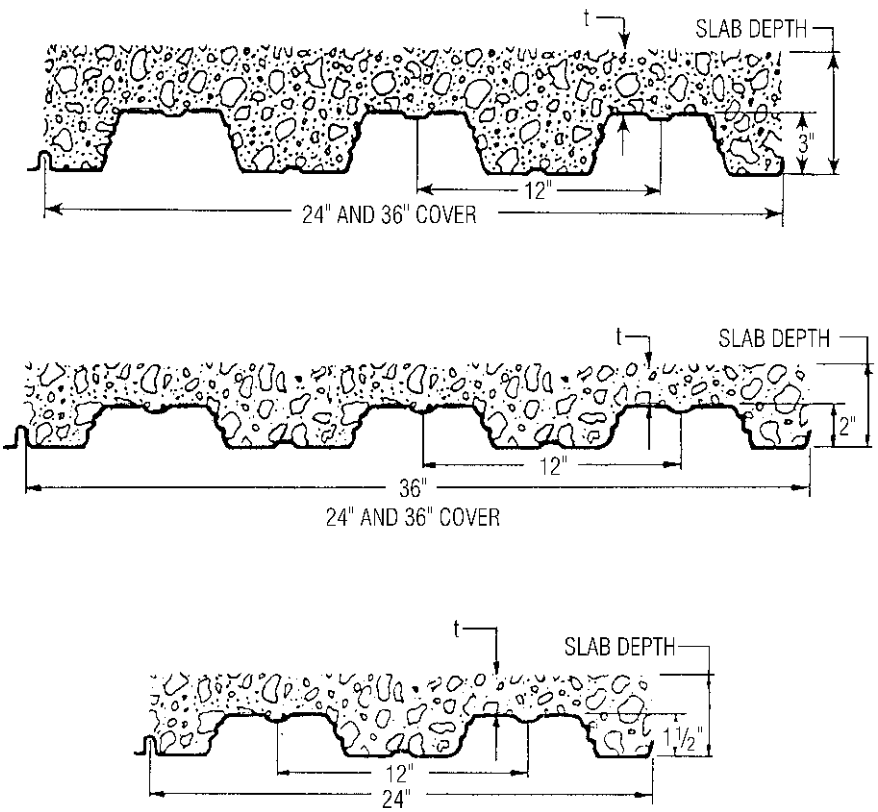


Fig. 10.14 Types of composite floor deck (LOK-FLOR, by United Steel Deck, Inc.).

to a header duct usually placed perpendicular to the cells. The header duct is equipped with a removable cover plate for lay-in wiring. On a repetitive module, the cellular raceways are assigned to electrical power, telephone, and data wiring. Preset inserts for activation of work-stations may be installed at prescribed intervals, as close as 2 ft longitudinally and transversely.

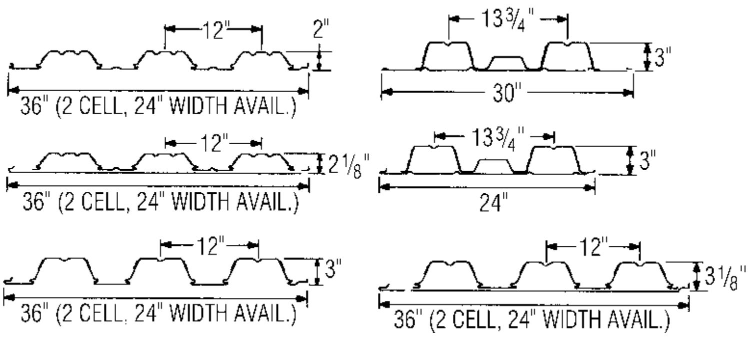


Fig. 10.15 Composite cellular and fluted steel floor sections. (*H. H. Robertson Co.*)

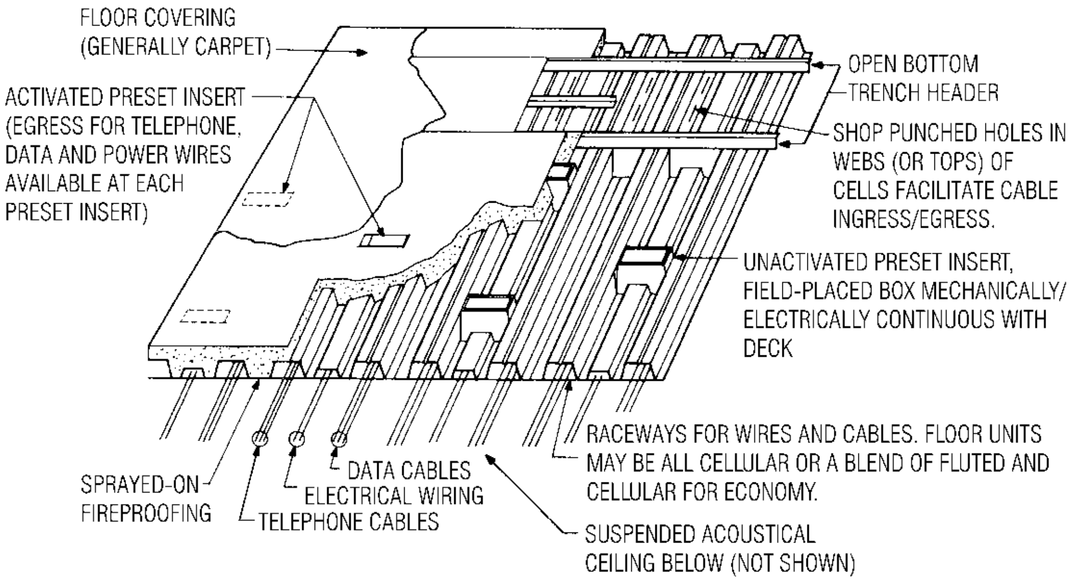


Fig. 10.16 Cellular steel floor raceway system. (*H. H. Robertson Co.*)

When an insert is activated at a workstation, connections for electrical power, telephone, and data are provided at one outlet.

Features ▪ During construction, the cellular steel floor decking serves as a working platform and as concrete forms. Afterward, the steel deck serves as the tensile reinforcement for the composite floor slab. The system also provides the required fire-resistive barrier between stories of the building.

Cellular steel floor raceway systems have many desirable features, including moderate first cost, flexibility in accommodating owners' needs (which lowers life-cycle costs), and minimal limitations on placement of outlets, which may be installed as close as 2 ft on centers in longitudinal and transverse directions. Physically, the wiring must penetrate the floor surface at outlet fittings. Therefore, the carpet (or other floor covering) has to be cut and a flap peeled back to expose each outlet. Use of carpet tiles rather than sheet carpet facilitates activation of preset inserts.

Where service outlets are not required to be as close as 2 ft, a blend of cellular and fluted floor sections may be used. For example, alternating 3-ft-wide fluted floor sections with 2-ft-wide cellular floor sections results in a module for service outlets of 5 ft in the transverse direction and as close as 2 ft in the longitudinal direction. Other modules and spacings are also available.

Flexibility in meeting owners' requirements can be achieved with little or no change in required floor depth to accommodate the system. Service fittings may be flush with the floor or may project above the floor surface, depending on the owners' desires.

Specifications ▪ Cellular steel floor and roof sections (decking) usually are made of steel 0.030 in or more thick complying with requirements of ASTM A1008, SS Grade 33, for uncoated steel or ASTM A653, SS Grade 33, for galvanized steel, both having specified minimum yield strengths of 33 ksi. Steel for decking may be galvanized or painted.

Structural design of cold-formed-steel floor and roof panels is usually based on the American Iron and Steel Institute "Specification for the Design of Cold-Formed Steel Structural Members." Structural design of composite slabs incorporating cold-formed-steel floor and roof panels is usually based on the American Society of Civil Engineers "Standard for the Structural Design of Composite Slabs" and "Standard Practice for Construction and Inspection of Composite Slabs" (www.asce.org).

Details of design and installation vary with types of panels and manufacturers. For a specific installation, follow the manufacturer's recommendations.

Fire Resistance ▪ Any desired degree of fire protection for cellular and fluted steel floor and roof assemblies can be obtained with concrete toppings and plaster ceilings or direct-application compounds (sprayed-on fireproofing). Fire-resistance ratings for many assemblies are available (Table 10.7). ("Fire-Resistant Steel-Frame Construction," American Institute of Steel Construction www.aisc.org; "Fire Resistance Directory," 1990, Underwriters' Laboratories, www.ul.com.)

Open-Web Steel Joists

As defined by the Steel Joist Institute, 3127 10th Avenue, North Ext., Myrtle Beach, SC 29577 (www.steeljoist.org), open-web steel joists are load-carrying members suitable for the direct support of floors and roof decks in buildings when these members are designed in accordance with SJI specifications and standard load tables.

As usually employed in floor construction, open-web steel joists support on top a slab of concrete, 2 to 2½ in thick, placed on permanent forms (Fig. 10.17). In addition to light weight, one advantage claimed for open-web steel-joist construction is that the open-web system provides space for electrical work, ducts, and piping.

10.24 Joist Fabrication

Standardization under the specifications of the Steel Joist Institute consists of definition of product; specification of materials, design stresses, manufacturing features, accessories, and installation procedures; and handling and erection techniques. Most manufacturers have made uniform certain details, such as end depths, which are desirably standardized for interchangeability. Exact forms of the members, configuration of web systems, and methods of manufacture are matters for the individual manufacturers of these joists. A number of proprietary designs have been developed.

Open-web steel joists are different in one important respect from fabricated structural-steel framing members commonly used in building construction: The joists usually are manufactured by production line methods with special equipment designed to produce a uniform product. Components generally are joined by either resistance or electric-arc welding. Various joist designs are shown in Fig. 10.18.

K-series open-web joists are manufactured in standard depths from 8 to 30 in in 2-in increments and in different weights. The K series is designed with higher allowable stresses, for either high-strength, hot-rolled steel or cold-worked sections that utilize an increase in base-material yield point. Thus, such steel having a specified minimum yield point of 50 ksi can be designed at a basic allowable stress of 30 ksi. The K series is intended for spans from 8 to 60 ft.

LH-series, longspan joists have been standardized with depths from 18 to 48 in for clear spans from 25 to 96 ft. DLH-series, deep, longspan joists have been standardized with depths from 52 to 72 in for clear spans from 89 to 144 ft. Basic allowable design stress is taken at 0.6 times the specified minimum yield point for the LH and DLH series, values from 36 to 50 ksi being feasible.

Joist girders have been standardized with depths from 20 to 72 in for clear spans from 20 to 60 ft. Basic allowable design stress is taken at 0.6 times the specified minimum yield point for joist girders, values from 36 to 50 ksi being contemplated.

The safe load capacities of each series are listed in SJI "Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders," 1994.

10.25 Design of Open-Web Joist Floors

Open-web joists are designed primarily for use under uniformly distributed loading and at substantially uniform spacing. But they can safely carry concentrated loads if proper consideration is given to the effect of such loads. Good practice requires that heavy concentrated loads be applied at joist panel points. The weight of a partition running crosswise to the joists usually is considered satisfactorily distributed by the floor slab and assumed not to cause local bending in the top chords of the joists. Even so, joists must be selected to resist the bending moments, shears, and end reactions due to such loads.

The method of selecting joist sizes for any floor depends on whether or not the effect of any cross

Table 10.7 Fire-Resistance Ratings for Steel Floor and Roof Assemblies (2-h Ratings)*

Roof Construction	Insulation	Underside Protection	Authority
Min $1\frac{1}{2}$-in-deep steel deck on steel joists or steel beams	Min $1\frac{3}{4}$-in-thick, listed mineral fiberboard	Min $1\frac{3}{4}$-in-thick, direct-applied, sprayed vermiculite plaster, UL Listed	UL Design P711†
Min $1\frac{1}{2}$-in-deep steel deck on steel joists or steel beams	Min $1\frac{1}{16}$-in-thick, listed mineral fiberboard	Min $1\frac{9}{16}$-in-thick, direct-applied, sprayed fiber protection, UL Listed	UL Design P818†
Floor Construction	**Concrete**	**Underside Protection**	**Authority**
$1\frac{1}{2}$-, 2-, or 3-in-deep, steel floor units on steel beams	$2\frac{1}{2}$-in-thick, normal-weight or light-weight concrete	Min. $\frac{3}{8}$-in-thick, direct-applied, sprayed vermiculite plaster, UL Listed	UL Design P739†
$1\frac{1}{2}$-, 2-, or 3-in-deep, steel floor units on steel beams	$2\frac{1}{2}$-in-thick, normal-weight or light-weight concrete	Min. $\frac{3}{8}$-in-thick, direct-applied, sprayed fiber protection, UL Listed	UL Design P858†

*For roof construction, $1\frac{1}{2}$-h and 1-h ratings are also available. For floor construction, $2\frac{1}{2}$-h, 3-h, and 4-h ratings are also available.

†"Fire Resistance Index," Underwriters' Laboratories, Inc., 1990.

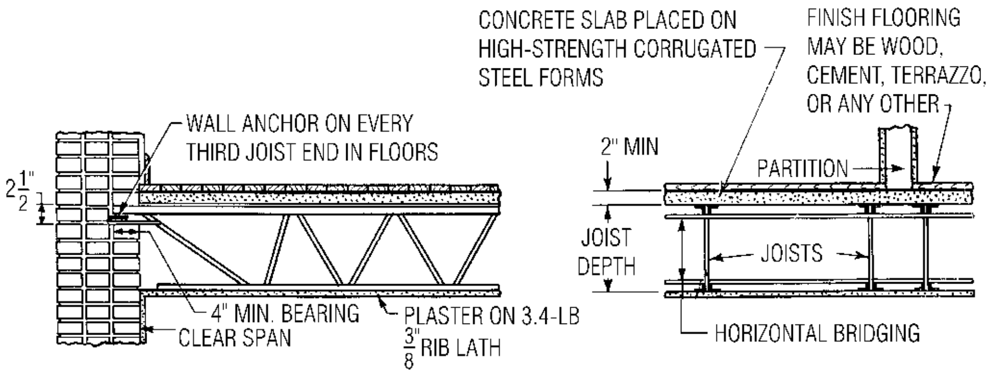


Fig. 10.17 Open-web steel joist construction.

partitions or other concentrated loads must be considered. Under uniform loading only, joist sizes and spacings are most conveniently selected from a table of safe loads. Where concentrated or nonuniform loads exist, calculate bending moments, end reactions, and shears, and select joists accordingly.

The chord section and web details are different for different joist designs made by different manufacturers. Information relating to the size and properties of the members may be obtained from manufacturers' catalogs.

Open-web steel-joist specifications require that the clear span not exceed 24 times the depth of the joist.

10.26 Construction Details for Open-Web Steel Joists

It is essential that bridging be installed between joists as soon as possible after the joists have been placed and before application of construction loads. The most commonly used type of bridging is a continuous horizontal bracing composed of rods fastened to the top and bottom chords of the joists. Diagonal bridging, however, also is permitted. The attachment of the floor or roof deck must provide lateral support for design loads.

It is important that masonry anchors be used on wall-bearing joists. Where the joists rest on steel beams, the joists should be welded, bolted, or clipped to the beams.

Fire resistance ratings of 1, $1\frac{1}{2}$, 2 and 3 hours are possible using concrete floors above decks as thin as 2 in and as thick as $3\frac{1}{2}$ in with various types of ceiling protection systems. The Steel Joist Institute identifies such ceiling protection systems as exposed grids, concealed grids, gypsum board, cementitious or sprayed fiber.

When the usual cast-in-place concrete floor slab is used, it is customary to install reinforcing bars in two perpendicular directions or welded-wire fabric. Stirrups are not usually necessary. Forms for the concrete slab usually consist of corrugated steel sheets, expanded-metal rib lath, or welded-wire fabric. Corrugated sheets can be fastened with self-tapping screws or welded to the joists, with a bent washer to reinforce the weld and anchor the slab.

Pre-Engineered Steel Buildings and Housing

10.27 Characteristics of Pre-Engineered Steel Buildings

These structures may be selected from a catalog fully designed and supplied with all structural and covering material, with all components and fasteners. Such buildings eliminate the need for engineers and architects to design and detail both the structure and the required accessories and openings, as would be done for conventional buildings with components from many individual suppliers. Available with floor area of up to 1 million ft^2, pre-engineered buildings readily meet requirements for single-story structures, especially for industrial plants and commercial buildings (Fig. 10.19).

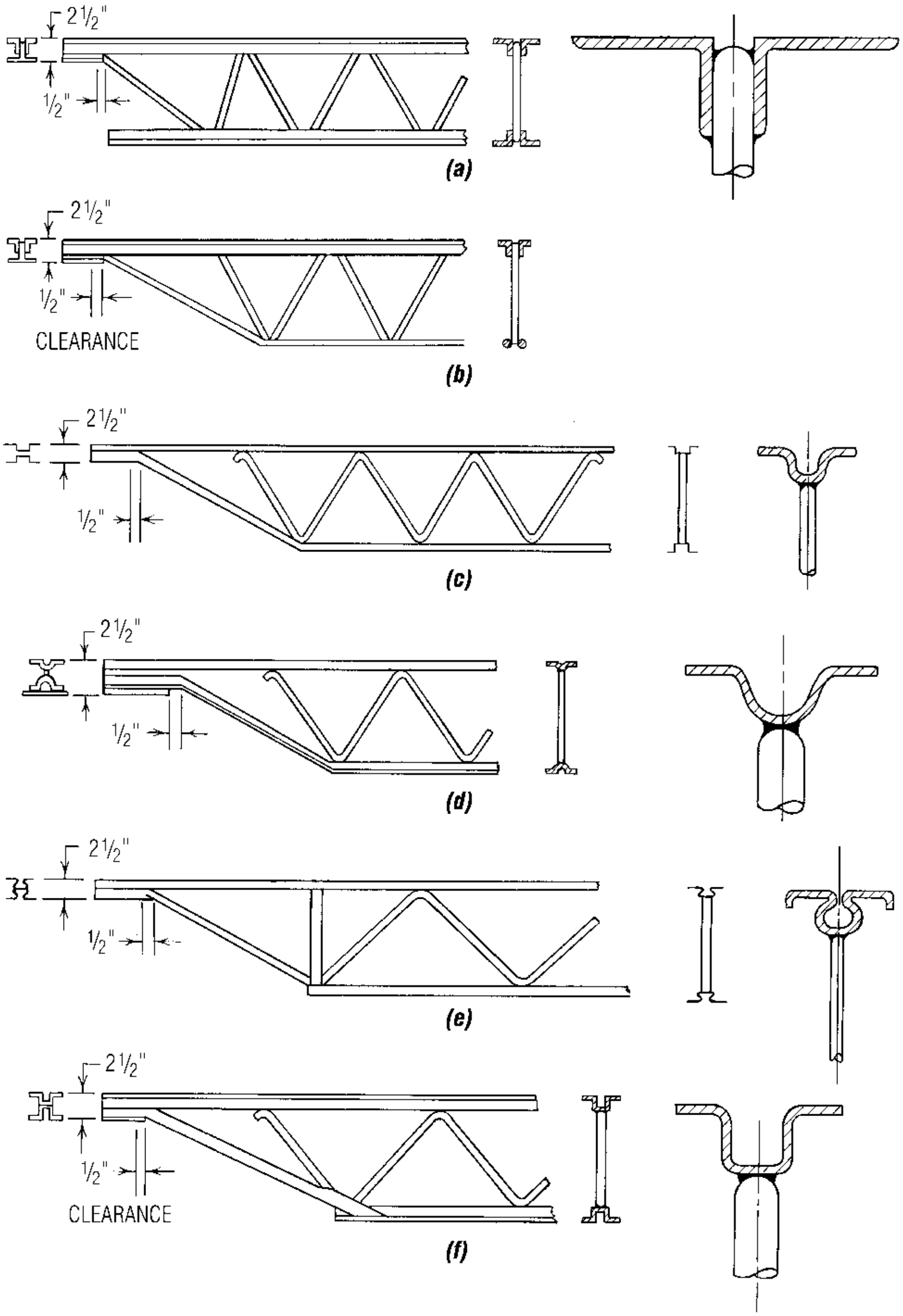


Fig. 10.18 Open-web steel joists.

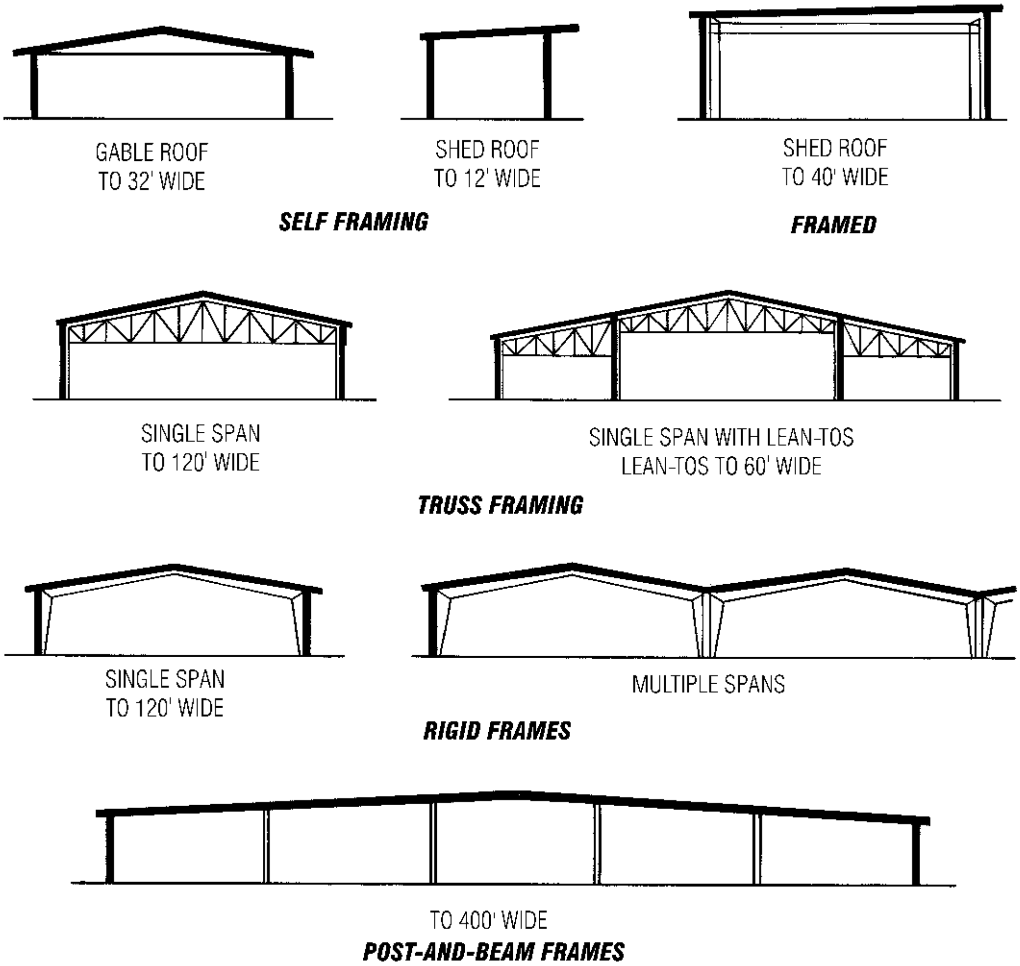


Fig. 10.19 Principal framing systems for pre-engineered buildings.

Pre-engineered buildings may be provided with custom architectural accents. Also, standard insulating techniques may be used and thermal accessories incorporated to provide energy efficiency. Exterior wall panels are available with durable factory-applied colors.

Many pre-engineered metal building suppliers are also able to modify structurally their standard designs, within certain limits, while retaining the efficiencies of predesign and automated volume fabrication. Examples of such modification include the addition of cranes; mezzanines; heating, ventilating, and air-conditioning equipment; sprinklers; lighting; and ceiling loads with special building dimensions.

Pre-engineered buildings make extensive use of cold-formed structural members. These parts lend themselves to mass production, and their design can be more accurately fitted to the specific structural requirement. For instance, a roof purlin can be designed with the depth, moment of inertia, section modulus, and thickness required to carry the load, as opposed to picking the next-higher-size standard hot-rolled shape, with more weight than required. Also, because this purlin is used on thousands of buildings, the quantity justifies investment in automated equipment for forming and punching. This equipment is flexible enough to permit a change of thickness or depth of section to produce similar purlins for other loadings.

The engineers designing a line of pre-engineered buildings can, because of the repeated use of the design, justify spending additional design time refining and optimizing the design. Most pre-engineered buildings are designed with the aid of electronic computers. Their programs are specifically tailored for the product. A rerun of a problem to eliminate a few pounds of steel is justified since the design will be reused many times during the life of that model.

10.28 Structural Design of Pre-Engineered Buildings

The buildings are designed for loading criteria in such a way that any building may be specified to meet the geographical requirements of any location. Combinations of dead load, snow load, live load, and wind load conform with requirements of several model building codes.

The Metal Building Dealers Association, 1406 Third National Building, Dayton, OH 45402, and the Metal Building Manufacturers Association, 1300 Summer Ave., Cleveland, OH 44115 (www.mbma.com), have established design standards (see MBMA, "Metal Building Systems Manual" and "Metal Roofing System Design Manual"). These standards discuss methods of load application and maximum loadings, for use where load requirements are not established by local building codes. Other standard design specifications include:

Structural Steel—"Specification for Structural Steel Buildings," American Institute of Steel Construction (www.asic.org).

Cold-Formed Steel—"Specification for the Design of Cold-Formed Steel Structural Members," American Iron and Steel Institute (www.steel.org).

Welding—"Structural Welding Code—Steel," D1.1, and "Structural Welding Code—Sheet Steel," D1.3, American Welding Society (www.aws.org).

Cold-formed steel structural members have been used for residential construction for many years. To satisfy the needs of design and construction information, the AISI Committee on Framing Standards has developed several "Standards for Cold-Formed Steel Framing," including General Provisions, Truss Design, Header Design, Prescriptive Method for One and Two-Family Dwellings, Wall Stud Design and Lateral Resistance Design. (American Iron and Steel Institute, 1140 Connecticut Ave., N.W., Washington, DC 20036.)

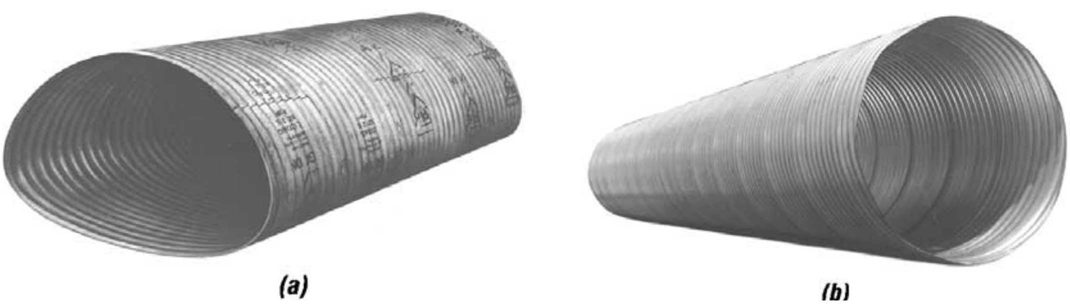
Structural Design of Corrugated Steel Pipe

10.29 Corrugated Steel Pipe

Corrugated steel pipe was first developed and used for culvert drainage in 1896. It is now produced in full-round diameters from 6 in in diameter and 0.064 in thick to 144 in in diameter and 0.168 in thick. Heights of cover up to 100 ft are permissible with highway or railway loadings.

Riveted corrugated pipe (Fig. 10.20a shows pipe-arch shape) is produced by riveting together circular corrugated sheets to form a tube. The corrugations are annular.

Helically corrugated pipe (Fig. 10.20b) is manufactured by spirally forming a continuously corrugated strip into a tube with a locked or welded seam



(a) (b)

Fig. 10.20 Corrugated steel structures. (*a*) Riveted pipe arch. (*b*) Helical pipe.

joining abutting edges. This pipe is stronger in ring compression because of the elimination of the longitudinal riveted joints. Also, the seam is more watertight than the lap joints of riveted pipe.

Besides being supplied in full-round shapes, both types of pipe are also available in pipe-arch shape. This configuration, with a low and wide waterway area in the invert, is beneficial for headroom conditions. It provides adequate flow capacity without raising the grade.

Corrugated steel pipe and pipe arch are produced with a variety of coatings to resist corrosion and erosion.

The zinc coating provided on these structures is adequate protection under normal drainage conditions with no particular corrosion hazard. Additional coatings or pavings may be specified for placing over the galvanizing.

Asbestos-bonded steel has a coating in which a layer of asbestos fiber is embedded in molten zinc and then saturated with bituminous material. This provides protection for extreme corrosion conditions. Asbestos-bonded steel is available in riveted pipe only. Helical corrugated structures may be protected with a hot-dip coating of bituminous material for severe soil or effluent conditions.

For erosive hazards, a paved invert of bituminous material can be applied to give additional protection to the bottom of the pipe. And for improved flow, these drainage conduits may also be specified with a full interior paving of bituminous material.

Normally, pipe-arch structures are supplied in a choice of span-and-rise combinations that have a periphery equal to that available with full-round corrugated pipe.

10.30 Structural Plate Pipe

To extend the diameter or span-and-rise dimensions of corrugated steel structures beyond that (120 in) available with factory-fabricated drainage conduits, structural plate pipe and other shapes may be used. These are made of heavier gages of steel and are composed of curved and corrugated steel plates bolted together at the installation site. Their shapes include full-round, elliptical, pipe-arch, arch, and horseshoe or underpass shapes. Applications include storm drainage, stream enclosures, vehicular and pedestrian underpasses, and small bridges.

Such structures are field-assembled with curved and corrugated steel plates that may be 10 or 12 ft long (Fig. 10.21). The wall section of the standard structures has 2-in-deep corrugations, 6 in c to c. Thickness ranges from 0.109 to 0.380 in. Each plate is punched for field bolting and special high-strength bolts are supplied with each structure. The number of bolts used can be varied to meet the ring-compression stress.

Circular pipes are available in diameters ranging from 5 to 26 ft, with structures of other configurations available in a similar approximate size range. Special end plates can be supplied to fit a skew or bevel, or a combination of both.

Plates of all structures are hot-dip galvanized. They are normally shipped in bundles for handling convenience. Instructions for assembly are also provided.

10.31 Design of Culverts

Formerly, design of corrugated steel structures was based on observations of how such pipes performed structurally under service conditions. From these observations, data were tabulated and gage tables established. As larger pipes were built and installed and experience was gained, these gage tables were revised and enlarged.

Following is the design procedure for corrugated steel structures recommended in the "Handbook of Steel Drainage and Highway Construction Products" (American Iron and Steel Institute, 1140 Connecticut Ave., N.W., Washington, D.C. 20036).

1. Backfill Density ▪ Select a percent compaction of pipe backfill for design. The value chosen should reflect the importance and size of the structure and the quality that can reasonably be expected. The recommended value for routine use is 85%. This value will usually apply to ordinary installations for which most specifications will call for compaction to 90%. But for more important structures in higher-fill situations, consideration must be given to selecting higher-quality backfill and requiring this quality for construction.

2. Design Pressure ▪ When the height of cover is equal to or greater than the span or diameter of the structure, enter the load-factor chart (Fig. 10.22) to determine the percentage of the total load acting on the steel. For routine use, the

Fig. 10.21 Structural-plate pipe is shown being bolted together at right. Completely assembled structural-plate pipe arch is shown at left.

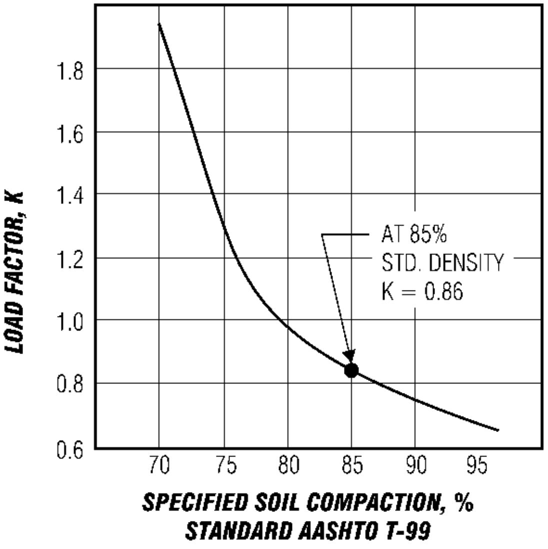


Fig. 10.22 Load factors for corrugated steel pipe are plotted as a function of specified compaction of backfill.

85% soil compaction will provide a load factor $K = 0.86$. The total load is multiplied by K to obtain the design pressure P_n acting on the steel. If the height of cover is less than one pipe diameter, the total load TL is assumed to act on the pipe, and $TL = P_n$; that is,

$$P_n = DL + LL + I \quad H < S \tag{10.49}$$

When the height of cover is equal to or greater than one pipe diameter,

$$P_n = K(DL + LL + I) \quad H \geq S \tag{10.50}$$

where P_n = design pressure, kips/ft^2

K = load factor

DL = dead load, kips/ft^2

LL = live load, kips/ft^2

I = impact, kips/ft^2

H = height of cover, ft

S = span or pipe diameter, ft

3. Ring Compression ▪ The compressive thrust C, kips/ft, on the conduit wall equals the radial pressure P_n kips/ft^2, acting on the wall multiplied by the wall radius R, ft; or $C = P_nR$. This thrust, called ring compression, is the force carried by the steel. The ring compression is an axial load acting tangentially to the conduit wall (Fig. 10.23). For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius. Then

$$C = P_n \frac{S}{2} \tag{10.51}$$

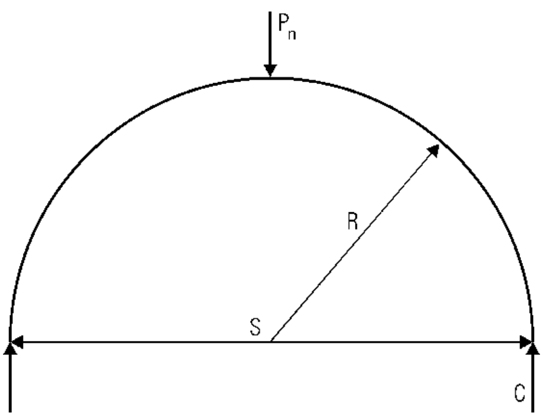


Fig. 10.23 Radical pressure P_n, on the wall of a curved conduit is resisted by compressive thrust, C.

4. Allowable Wall Stress ▪ The ultimate compression in the pipe wall is expressed by Eqs. (10.52) and (10.53). The ultimate wall stress is equal to the specified minimum yield point of the steel and applies to the zone of wall crushing or yielding. Equation (10.52) applies to the interaction zone of yielding and ring buckling; Eq. (10.53) applies to the ring-buckling zone.

When the ratio D/r of pipe diameter—or span D, in, to radius of gyration r, in, of the pipe cross section—does not exceed 294, the ultimate wall stress may be taken as equal to the steel yield strength:

$$F_b = F_y = 33 \text{ ksi}$$

When D/r exceeds 294 but not 500, the ultimate wall stress, ksi, is given by

$$F_b = 40 - 0.000081\left(\frac{D}{r}\right)^2 \tag{10.52}$$

When D/r is more than 500

$$F_b = \frac{4.93 \times 10^6}{(D/r)^2} \tag{10.53}$$

A safety factor of 2 is applied to the ultimate wall stress to obtain the design stress F_c, ksi,

$$F_c = \frac{F_b}{2} \tag{10.54}$$

5. Wall Thickness ▪ Required wall area A, in^2/ft of width, is computed from the calculated compression C in the pipe wall and the allowable stress F_c.

$$A = \frac{C}{F_c} \tag{10.55}$$

From Table 10.8, select the wall thickness that provides the required area with the same corrugation used for selection of the allowable stress.

6. Check Handling Stiffness ▪ Minimum pipe stiffness requirements for practical handling

Table 10.8 Moments of Inertia, Cross-Sectional Areas, and Radii of Gyration for Corrugated Steel Sheets and Plates for Underground Conduits*

Corrugation pitch × depth, in	Specified Thickness Including Galvanized Coating, in												
	0.040	0.052	0.064	0.079	0.109	0.138	0.168	0.188	0.218	0.249	0.280	0.310	0.380
	Base-Metal Thickness, in												
	0.0359	0.0478	0.0598	0.0747	0.1046	0.1345	0.1644	0.1838	0.2145	0.2451	0.2758	0.3125	0.3750
	Moment of Inertia I, in^4/ft of Width												
$1\frac{1}{2} \times \frac{1}{4}$	0.0030	0.0041	0.0053	0.0068	0.0103	0.0145	0.0196						
$2 \times \frac{1}{2}$	0.0137	0.0184	0.0233	0.0295	0.0425	0.0566	0.0719						
$2\frac{2}{3} \times \frac{1}{2}$	0.0135	0.0180	0.0227	0.0287	0.0411	0.0544	0.0687						
3×1	0.0618	0.0827	0.1039	0.1306	0.1855	0.2421	0.3010						
5×1			0.1062	0.1331	0.1878	0.2438	0.3011						
6×2					0.725	0.938	1.154	1.296	1.523	1.754	1.990	2.280	2.784
	Cross-Sectional Wall Area A, in^2/ft of Width												
$1\frac{1}{2} \times \frac{1}{4}$	0.456	0.608	0.761	0.950	1.331	1.712	2.093						
$2 \times \frac{1}{2}$	0.489	0.652	0.815	1.019	1.428	1.838	2.249						
$2\frac{2}{3} \times \frac{1}{2}$	0.465	0.619	0.775	0.968	1.356	1.744	2.133						
3×1	0.534	0.711	0.890	1.113	1.560	2.008	2.458						
5×1			0.794	0.992	1.390	1.788	2.186						
6×2					1.556	2.003	2.449	2.739	3.199	3.658	4.119	4.671	5.613
	Radius of Gyration r, in												
$1\frac{1}{2} \times \frac{1}{4}$	0.0816	0.0824	0.0832	0.0846	0.0879	0.0919	0.0967						
$2 \times \frac{1}{2}$	0.1676	0.1682	0.1690	0.1700	0.1725	0.1754	0.1788						
$2\frac{2}{3} \times \frac{1}{2}$	0.1702	0.1707	0.1712	0.1721	0.1741	0.1766	0.1795						
3×1	0.3403	0.3410	0.3417	0.3427	0.3448	0.3472	0.3499						
5×1			0.3657	0.3663	0.3677	0.3693	0.3711						
6×2					0.682	0.684	0.686	0.688	0.690	0.692	0.695	0.698	0.704

*Corrugation dimensions are nominal, subject to manufacturing tolerances. Section properties were calculated from base-metal thicknesses, that is, with galvanized-coating thickness excluded.

and installation, without undue care or bracing, have been established through experience. The resulting flexibility factor FF limits the size of each combination of corrugation pitch and metal thickness.

$$FF = \frac{D^2}{EI} \tag{10.56}$$

where E = modulus of elasticity, ksi, of steel = 30,000 ksi

I = moment of inertia of wall, in^4/in

The following maximum values of FF are recommended for ordinary installations:

$FF = 0.0433$ for factory-made pipe with riveted, welded, or helical seams

$FF = 0.0200$ for field-assembled pipe with bolted seams

Higher values can be used with special care or where experience indicates. Trench condition, as in sewer design, can be one such case; use of aluminum pipe is another. For example, the flexibility factor permitted for aluminum pipe in some national specifications is more than twice that recommended here for steel because aluminum has only one-third the stiffness of steel, the modulus for aluminum being about 10,000 ksi vs. 30,000 ksi for steel. Where a high degree of flexibility is acceptable for aluminum, it will be equally acceptable for steel.

7. Check Longitudinal Seams ▪ Most pipe seams develop the full yield strength of the pipe wall. However, there are exceptions in standard pipe manufacture and these are identified here. Shown in Table 10.9 are those standard riveted and bolted seams which do not develop a strength equivalent to $F_y = 33$ ksi. To maintain a consistent factor of safety of 2.0 for these pipes, it is necessary to reduce the maximum ring compression to one half the indicated seam strength. Nonstandard, or new longitudinal seam details should be checked for this same possible condition.

Other Types of Lightweight-Steel Construction

10.32 Lightweight-Steel Bridge Decking

This trapezoidal-corrugated plank, welded to steel (Fig. 10.24) or lagged to wood stringers, gives a strong, secure base for a smooth bituminous traffic surface. It may be used for replacement of old wood decks and for new construction.

10.33 Beam-Type Guardrail

The beam-type guardrail in Fig. 10.25 has the flexibility necessary to absorb impact as well as the beam strength to prevent pocketing of a car against a post. Standard post spacing is 12½ ft. The rail is anchored with one bolt to each post, and there are eight bolts in the rail splice to assure continuous-

Table 10.9 Ultimate Longitudinal Seam Strengths (kips/ft)

Thickness, in				2⅔ × ½ in Rivet Seams		
Corrugated Steel Pipe	Structural Plate	6 × 2 in 4 Bolts Per Ft	3 × 1 in	5/16 in Single Rivet	⅜ in Single Rivet	⅜ in Double Rivet
0.064			28.7[1]	16.7		
0.079			35.7[1]	18.2		
0.109	0.111	42.0			23.4	
0.138	0.140	62.0	63.7[2]		24.5	49.0
0.168			70.7[2]		25.6	51.3

Standard seams not shown develop full yield strength of pipe wall.

[1]Double ⅜-in. rivets.

[2]Double 7/16-in. rivets.

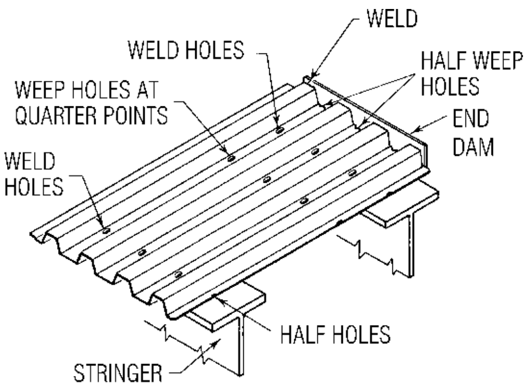


Fig. 10.24 Lightweight-steel bridge plank.

beam strength. Available lengths are $12\frac{1}{2}$ and 25 ft. Standard steel thickness is 0.109 in; heavy-duty is 0.138 in thick. The guardrail is furnished galvanized or as prime-painted steel. (See also Art. 16.17.)

10.34 Bin-Type Retaining Wall

A bin-type retaining wall (Fig. 10.26) is a series of closed-face bins, which when backfilled transform the soil mass into an economical retaining wall. The flexibility of steel allows for adjustments due to uneven ground settlement. There are standard designs for these walls with vertical or battered face, heights to 30 ft, and various conditions of surcharge.

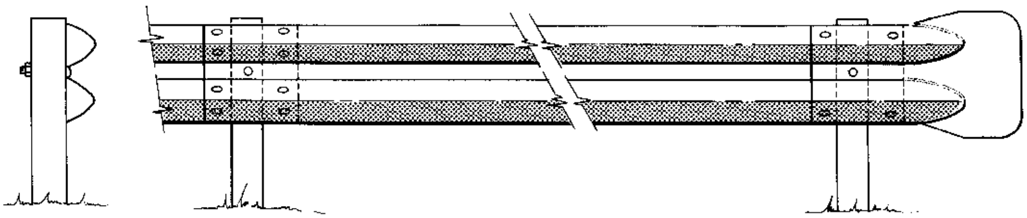


Fig. 10.25 Beam-type guardrail of steel.

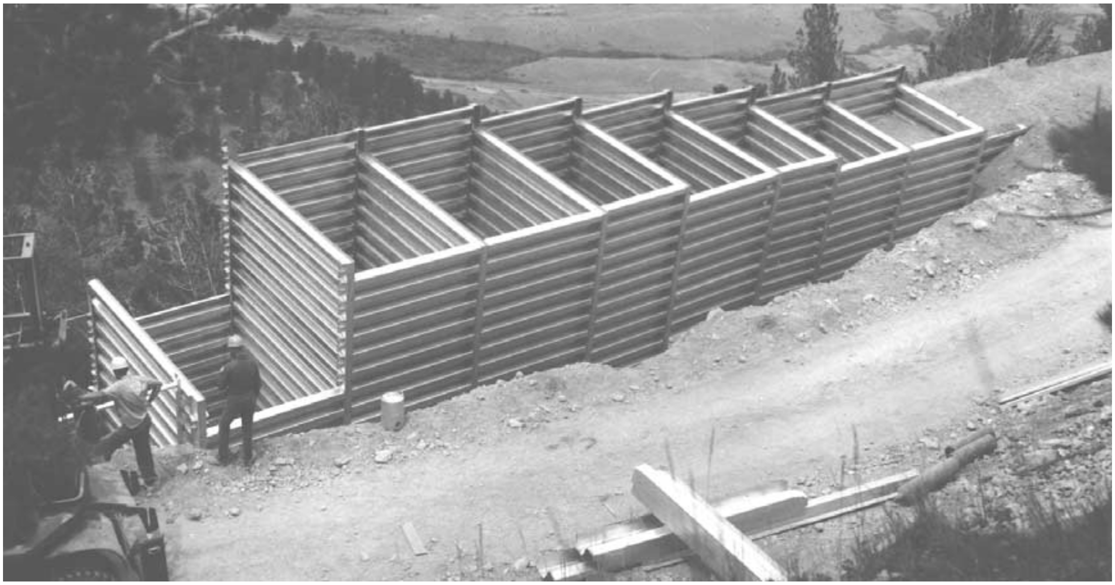


Fig. 10.26 Bin-type retaining wall of cold-formed steel.

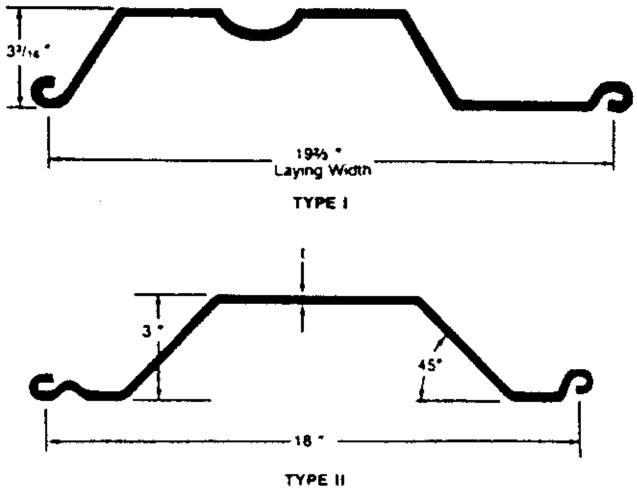


Fig. 10.27 Lightweight steel sheeting

Table 10.10(a) Physical Properties of Type I Lightweight Steel Sheeting

Gage	Uncoated Thickness in	Weight		Section Properties			
		lbs/ft of pile	lbs/ft^2 of wall	Section Modulus, in^3		Moment of Inertia, in^4	
				per section	per ft	per section	per ft
5	0.209	19.1	11.6	5.50	3.36	9.40	5.73
7	0.179	16.4	10.0	4.71	2.87	8.00	4.88
8	0.164	15.2	9.3	4.35	2.65	7.36	4.49
10	0.134	12.5	7.6	3.60	2.20	6.01	3.67
12	0.105	9.9	6.0	2.80	1.71	4.68	2.85

Based on AISI "Handbook of Steel Drainage & Highway Construction Products," 1994.

Table 10.10(b) Physical Properties of Type II Lightweight Steel Sheeting

Gage	Uncoated Thickness, in.	Weight		Section Modulus, in^3/ft	Moment of Inertia in^4/ft
		lbs/ft	lbs/ft^2		
7	0.179	13.86	9.24	2.37	3.56
8	0.164	12.70	8.47	2.13	3.20
10	0.134	10.37	6.91	1.84	2.74

Based on AISI "Handbook of Steel Drainage & Highway Construction Products," 1994.

10.35 Lightweight-Steel Sheeting

Corrugated sheeting has beam strength to support earth pressure on walls of trenches and excavations, and column strength for driving. The sheeting presents a small end cross section for easy driving (Fig. 10.27). Physical properties of the sheeting shown in Fig. 10.27 are listed in Table 10.10.