SECTION EIGHT
COLD-FORMED STEEL CONSTRUCTION

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The term cold-formed steel construction, as used in this section, refers to structural components that are made of flat-rolled steel. This section deals with fabricated components made from basic forms of steel, such as bars, plates, sheet, and strip.

COLD-FORMED SHAPES

Cold-formed shapes usually imply relatively small, 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 lends itself well to the manufacture of unique shapes for special purposes and makes it possible to use thin material shaped for maximum stiffness.

The use of cold-formed shapes for ornamental and other non-load-carrying purposes is commonplace. Door and window frames, metal-partition work, non-load-bearing studs, facing, and all kinds of ornamental sheet-metal work employ such shapes. The following deals with cold-formed shapes used for structural purposes in the framing of buildings.

There is no standard series of cold-formed structural sections, such as those for hot-rolled shapes, yet although groups of such sections have been designed (''Cold-formed Steel Design Manual,'' American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036). For the most part, however, cold-formed structural shapes are designed to serve a particular purpose. The general approach of the designer is therefore similar to that involved in the design of built-up structural sections.

Cold-formed shapes invariably cost more per pound than hot-rolled sections. They will be found to be more economical under the following circumstances:

1. Where their use permits a substantial reduction in weight compared to hot-rolled sections. This occurs where relatively light loads are to be supported over short spans, or where stiffness rather than strength is the controlling factor in the design.

2. In special cases where a suitable combination of standard hot-rolled shapes would be heavy and uneconomical.

3. Where quantities required are too small to justify the investment necessary to produce a suitable hot-rolled section.

4. In dual-purpose panel work, where both strength and coverage are desired.

8.1 MATERIAL FOR COLD-FORMED STEEL SHAPES

Cold-formed shapes are usually made from hot-rolled sheet or strip steel, which costs less per pound than cold-rolled steel. The latter, which has been cold-rolled to desired thickness, is used for thinner gages or where, for any reason, the surface finish, mechanical properties, or closer tolerances that result from cold-reducing is desired. Manufacture of cold-formed shapes from plates for use in building construction is possible but is done infrequently.

8.1.1 Plate, Sheet, or Strip

The commercial distinction between steel plates, sheet, and strip is principally a matter of thickness and width of material. In some sizes, however, classification depends on whether the material is furnished in flat form or in coils, whether it is carbon or alloy steel, and, particularly for cold-rolled material, on surface finish, type of edge, temper or heat treatment, chemical composition, and method of production. Although the manufacturers' classification of flat-rolled steel products by size is subject to change from time to time, that given in Table 8.1 for carbon steel is representative.

Carbon steel is generally used. High-strength, low-alloy steel, however, may be used where strength or corrosion resistance justify it, and stainless steel may be used for exposed work.

8.1.2 Mechanical Properties

Material to be used for structural purposes generally conforms to one of the standard specifications of ASTM. Table 8.2 lists the ASTM specifications for structural-quality carbon and low-alloy sheet and strip, and their principal mechanical properties.

TABLE 8.1 Classification by Size of Flat-Rolled Carbon Steel

a. Holt-rolled

Width, in	Thickness, in: 0.2300 and thicker	0.2299–0.2031	0.2030–0.1800	0.1799–0.0470
To 3½ incl.	Bar	Bar	Strip	Strip[a]
Over 3½ to 6 incl.	Bar	Bar	Strip	Strip[b]
Over 6 to 8 incl.	Bar	Strip	Strip	Strip
Over 8 to 12 incl.	Plate[c]	Strip	Strip	Strip
Over 12 to 48 incl.	Plate[d]	Sheet	Sheet	Sheet
Over 48	Plate[d]	Plate[d]	Plate[d]	Sheet

b. Cold-rolled

Width, in	Thicknesses, in: 0.2500 and thicker	0.2499–0.0142	0.0141 and thinner
To 12, incl.	Bar	Strip[e,f]	Strip[e]
Over 12 to 23¹⁵⁄₁₆, incl.	Sheet[g]	Sheet[g]	Strip[h]
Over 23¹⁵⁄₁₆	Sheet	Sheet	Black plate[i]

[a] 0.0255-in minimum thickness.
[b] 0.0344-in minimum thickness.
[c] Strip, up to and including 0.5000-in thickness, when ordered in coils.
[d] Sheet, up to and including 0.5000-in thickness, when ordered in coils.
[e] Except that when the width is greater than the thickness, with a maximum width of ½ in and a cross-sectional area not exceeding 0.05 in^2, and the material has rolled or prepared edges, it is classified as flat wire.
[f] Sheet, when slit from wider coils and supplied with cut edge (only) in thicknesses 0.0142 to 0.0821 and widths 2 to 12 in. inclusive, and carbon content 0.25% maximum by ladle analysis.
[g] May be classified as strip when a special edge, a special finish, or single-strand rolling is specified or required.
[h] Also classified as black plate[i], depending on detailed specifications for edge, finish, analysis, and other features.
[i] Black plate is a cold-rolled, uncoated tin-mill product that is supplied in relatively thin gages.

8.1.3 Stainless-Steel Applications

Stainless-steel cold-formed shapes, although not ordinarily used in floor and roof framing, are widely used in exposed components, such as stairs, railings, and balustrades; doors and windows; mullions, fascias; curtain walls and panel work; and other applications in which a maximum degree of corrosion resistance, retention of appearance and luster, and compatibility with other materials are primary considerations. Stainless-steel sheet and strip are available in several types and grades, with different strength levels and different degrees of formability, and in a wide range of finishes.

Information useful in design of stainless-steel cold-formed members can be obtained from the ''Specification for the Design of Cold-Formed Stainless Steel Structural Members,'' American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191-4400. The specification is applicable to material covered by ASTM A666, ''Austenitic Stainless Steel, Sheet, Strip, Plate and Flat

TABLE 8-2 Principal Mechanical Properties of Structural Quality Sheet, Strip, and Plate Steel

ASTM designation	Material	Grade	Minimum yield point, ksi	Minimum tensile strength, ksi: Hot rolled	Minimum tensile strength, ksi: Cold rolled	Minimum elongation, % in 2 in	Bend test, 180°, ratio of inside diameter to thickness
A570	Hot-rolled sheet and strip, carbon steel	30	30	49		●	1
		36	36	53		●	1½
		40	40	55		●	2
		45	45	60		●	2½
		50	50	65		●	3
A606	Hot-rolled and cold-rolled sheet and strip, high-strength, low-alloy steel	Cut lengths	50	70		22	1
		Coils	45	65		22	1
		Annealed or normalized	45	65		22	1
		Cold rolled	45		65	22	1
						HR§ CR§	
A607	Hot-rolled and cold-rolled, high-strength, low-alloy columbium or vanadium steels, sheet and strip, cut lengths or coils	45	45	60	60	23 22	1
		50	50	65	65	20 20	1
		55	55	70	70	18 18	1½
		60	60	75	75	16 16	2
		65	65	80	80	14 15	2½
		70	70	85	85	12 14	3
A611	Cold-rolled sheet, structural carbon-steel sheet, cut lengths or coils	A	25		42	26	0
		B	30		45	24	1
		C	33		48	22	1½
		D	40		52	20	2

TABLE 8-2 Principal Mechanical Properties of Structural Quality Sheet, Strip, and Plate Steel (*Continued*)

ASTM designation	Material	Grade	Minimum yield point, ksi	Minimum tensile strength, ksi		Minimum elongation, % in 2 in	Bend test, 180°, ratio of inside diameter to thickness
				Holt rolled	Cold rolled		
A653	Galvanized sheet steel, zinc-coated by the hot-dip process, structural quality	SQ 33	33	45		20	1½
		37	37	52		18	2
		40	40	55		16	2½
		50 class 1	50	65		12	†
		80	80	82			†
		50 class 2	50	70		12	†
		HSLA				TYP1 TYP2	
		50	50	60		20 22	
		60	60	70		16 18	
		70	70	80		12 14	
		80	80	90		10 12	
A36	Structural steel (plates only)		36	58–80		23	‡
A242	High-strength, low-alloy structural steel (plates ¾ in and under)		50	70		†	‡
A283	Low and intermediate tensile strength carbon steel plates	A	24	45–60		30	
		B	27	50–65		28	
		C	30	55–75		25	
		D	33	60–80		23	
A500	Cold-formed welded and seamless carbon steel structural tubing (round tubing)	A	33	45		25	
		B	42	58		23	
		C	46	62		21	
		D	36	58		23	
	Cold-formed welded and seamless carbon steel structural tubing (shaped tubing)	A	39	45		25	
		B	46	58		23	
		C	50	62		21	
		D	36	58		23	
A529	Structural steel with 42 ksi minimum yield point (½ in maximum thickness) (plates only)	42	42	60–85		22	‡
		50	50	70–100		21	

TABLE 8-2 Principal Mechanical Properties of Structural Quality Sheet, Strip, and Plate Steel (*Continued*)

ASTM designation	Material	Grade	Minimum yield point, ksi	Minimum tensile strength, ksi: Holt rolled	Minimum tensile strength, ksi: Cold rolled	Minimum elongation, % in 2 in	Bend test, 180°, ratio of inside diameter to thickness
A572	High-strength, low-alloy columbium-vanadium steels of structural quality (plates only)	42	42	60		24	‡
		50	50	65		21	‡
		60	60	75		18	‡
		65	65	80		17	‡
A588	High-strength, low-alloy structural steel with 50 ksi minimum yield point to 4 in thick (plates only)	A	50	70		21	‡
		B	50	70		21	‡
		C	50	70		21	‡
		D	50	70		21	‡
		E	50	70		21	‡
		F	50	70		21	‡
		G	50	70		21	‡
		H	50	70		21	‡
		J	50	70		21	‡
A715	High-strength, low-alloy hot-rolled steel with improved formability					HR§ CR§	1
		50	50	60		22 20	1½
		60	60	70		22 18	
		70	70	80		18 16	
		80	80	90		18 16	
A792	Aluminum-zinc alloy coated steel sheet by the hot-dip process, general requirements	33	33	45		20	1½
		37	37	52		18	2
		40	40	65		16	2½
		50A	50			12	—
		50B	50			12	—
		80	80	82		12	

*Varies, see specification. †Not specified or required. ‡S14 bend test. §HR = hot rolled; CR = cold rolled.

Bars for Structural Applications.'' It contains requirements for 201, 202, 301, 302, 304, and 316 types of stainless steels. Further information on these steels as well as steels covered by ASTM A176, A240, and A276 may be obtained from the American Iron and Steel Institute (AISI).

8.1.4 Coatings

Material for cold-formed shapes may be either *black* (uncoated), galvanized, or aluminized. Because of their higher costs, metal-coated steels are used only where exposure conditions warrant paying more for the increased protection afforded against corrosion.

Low-carbon sheets suitable for coating with vitreous enamel are frequently used for facing purposes, but not as a rule to perform load-carrying functions in buildings.

8.1.5 Selection of Grade

The choice of a grade of material, within a given class or specification, usually depends on the severity of the forming operation required to make the required shape, strength desired, weldability requirements, and the economics involved. Grade C of ASTM A611, with a specified minimum yield point of 33 ksi has long been popular for structural use. Some manufacturers, however, use higher-strength grades to good advantage.

8.1.6 Gage Numbers

Thickness of cold-formed shapes was formerly expressed as the manufacturers' standard gage number of the material from which the shapes were formed. *Use of millimeters or decimal parts of an inch, instead of gage numbers,* is now the standard practice. However, for information, the relationships among gage number, weight, and thickness for uncoated and galvanized sheets are given in Table 8.3 for even gages.

8.2 UTILIZATION OF COLD WORK OF FORMING

When strength alone, particularly yield strength, is an all-important consideration in selecting a material or grade for cold-formed shapes (Table 8.2), it is sometimes possible to take advantage of the strength increase that results from cold working of material during the forming operation and thus use a lower-strength, more workable, and possible more economical grade than would otherwise be required. The increase in cold-work strength is ordinarily most noticeable in relatively stocky, compact sections produced in thicker steels. Cold-formed chord sections for open-web steel joists are good examples (Fig. 8.22). Overall average yield strengths of more than 150% of the minimum specified yield strength of the plain material have been obtained in such sections.

The strengthening effect of the forming operation varies across the section but is most pronounced at the bends and corners of a cold-formed section. Accordingly,

TABLE 8.3 Gages, Weights, and Thicknesses of Sheets

Steel manufacturer's standard gage No.	Weight, psf	Equivalent sheet thickness, in*	Galvanized sheet gage No.	Weight, psf	Thickness equivalent, † in
4	9.3750	0.2242			
6	8.1250	0.1943			
8	6.8750	0.1644	8	7.03125	0.1681
10	5.6250	0.1345	10	5.78125	0.1382
12	4.3750	0.1046	12	4.53125	0.1084
14	3.1250	0.0747	14	3.28125	0.0785
16	2.5000	0.0598	16	2.65625	0.0635
18	2.0000	0.0478	18	2.15625	0.0516
20	1.5000	0.0359	20	1.65625	0.0396
22	1.2500	0.0299	22	1.40625	0.0336
24	1.0000	0.0239	24	1.15625	0.0276
26	0.7500	0.0179	26	0.90625	0.0217
28	0.6250	0.0149	28	0.78125	0.0187
30	0.5000	0.0120	30	0.65625	0.0157
32	0.40625	0.0097	32	0.56250	0.0134
34	0.34375	0.0082			
36	0.28125	0.0067			
38	0.25000	0.0060			

*Thickness equivalents of steel are based on 0.023912 in/(lb-ft^2) (reciprocal of 41.820 psf per inch of thickness, although the density of steel is ordinarily taken as 489.6 lb/ft^3, 0.2833 lb/in^3, or 40.80 psf per inch of thickness). The density is adjusted because sheet weights are calculated for specified widths and lengths of sheets, with all shearing tolerances on the over side, and also because sheets are somewhate thicker at the center than at the edges. The adjustment yields a close approximation of the relationship between weight and thickness. (''Steel Products Manual, Carbon Steel Sheets,'' American Iron and Steel Institute.)

†Total thickness, in, including zinc coating. To obtain base metal thickness, deduct 0.0015 in per ounce coating class, or refer to ASTM A653.

for shapes in which bends and corners constitute a high percentage of the whole section, cold working increases the overall strength more than for shapes having a high proportion of thin, wide, flat elements that are not heavily worked in forming. For the latter type of shapes, the strength of the plain, unformed sheet or strip may be the controlling factor in the selection of a grade of material.

Full-section tests constitute a relatively simple, straightforward method of determining as-formed strength. They are particularly applicable to sections that do not contain any elements that may be subject to local buckling. However, each case has to be considered individually in determining the extent to which cold forming will produce an increase in utilizable strength. For further information, refer to the AISI ''Specification for the Design of Cold-Formed Steel Structural Members'' and its ''Commentary,'' 1996, American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036.

8.3 TYPES OF COLD-FORMED SHAPES

Many cold-formed shapes used for structural purposes are similar in their general configurations to hot-rolled structural sections. Channels, angles, and zees 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 sections of this kind may be made with either plain flanges as in Fig. 8.1*a* to *d*, *j*, and *m* or with flanges stiffened by means of lips at outer edges, as in Fig. 8.1*e* to *h*, *k*, and *n*.

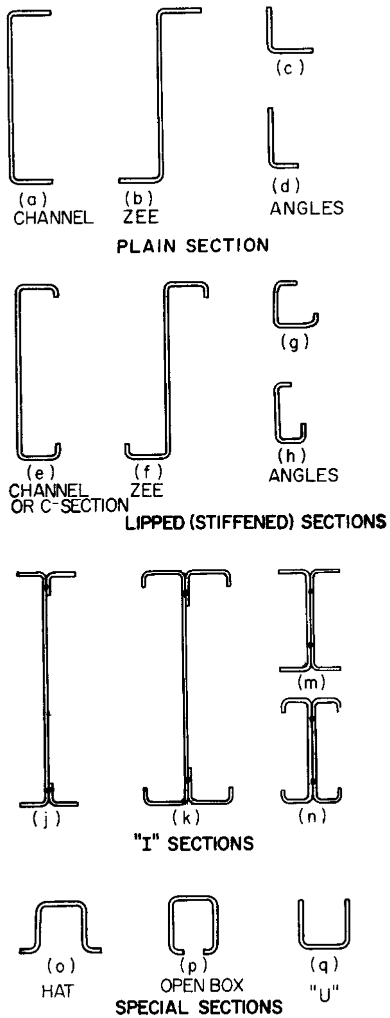


FIGURE 8.1 Typical cold-formed steel structural sections

In addition to these sections, which follow somewhat conventional lines and have their counterparts in hot-rolled structural sections, the flexibility of the forming process makes it relatively easy to obtain inverted U, or hat-shaped, sections and open box sections (Fig. 8.1*o* to *q*). These sections are very stiff in a lateral direction and can be used without lateral support where other more conventional types of sections would fail because of lateral instability.

Other special shapes are illustrated in Fig. 8.2. Some of these are nonstructural in nature; others are used for special-purpose structural members. Figure 8.3 shows a few cold-formed stainless steel sections.

An important characteristic of cold-formed shapes is that the thickness of section is substantially uniform. (A slight reduction in thickness may occur at bends, but that may be ignored for computing weights and section properties.) This means that, for a specified thickness, the amount of flange material in a section, such as a channel, is almost entirely a function of the width of the section, except for shapes where additional flange area is obtained by doubling the material back on itself. Another distinguishing feature of cold-formed sections is that the corners are rounded on both the inside and the outside of the bend, since the shapes are formed by bending flat material.

Sharp corners, such as can be obtained with hot-rolled structural channels, angles, and zees, cannot be obtained in cold-formed shapes by simple bending, although they can be achieved in a coining or upsetting operation. This, however, is not customary in the manufacture of structural cold-formed sections; and in proportioning such sections, the inside radius of bends should never be less, and should preferably be 33 to 100% greater, than specified for the relatively narrow ASTM bend-test specimens. Deck and panel sections, such as are used for floors, roofs, and walls, are as a rule considerably wider, relative to their depth, than are the structural framing members shown in Figs. 8.1 to 8.3.

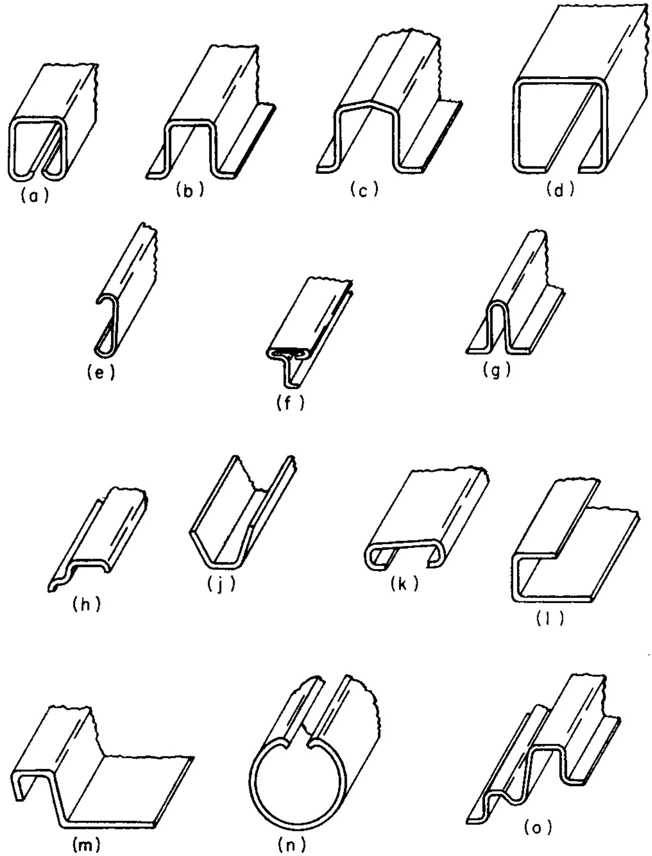


FIGURE 8.2 Miscellaneous cold-formed shapes. (*Bethlehem Steel Corp.*)

DESIGN PRINCIPLES FOR COLD-FORMED STEEL SHAPES

The structural behavior of cold-formed shapes follows the same laws of structural mechanics as does that of conventional structural-steel shapes and plates. Thus, design procedures commonly used in the selection of hot-rolled shapes are generally applicable to cold-formed sections. Although only a portion of a section, in some cases, may be considered structurally effective, computation of the structural properties of the effective option follows conventional procedure.

8.4 SOME BASIC CONCEPTS OF COLD-FORMED STEEL DESIGN

The uniform thickness of most cold-formed sections, and the fact that the widths of the various elements composing such a section are usually large relative to the

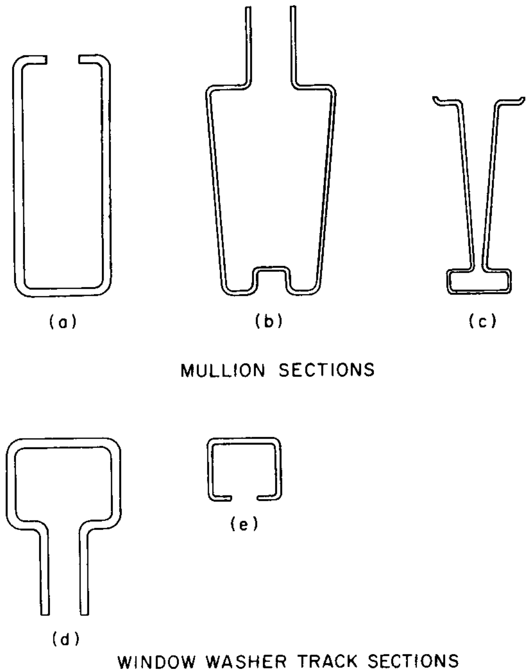


FIGURE 8.3 Cold-formed stainless steel sections. (*The International Nickel Co., Inc.*)

thickness, make it possible to consider, in computing structural properties (moment of inertia, section modulus, etc.) that such properties vary directly as the first power of the thickness. So, in most cases, section properties can be approximated by first assuming that the section is made up of a series of line elements, omitting the thickness dimension. Then, final values can be obtained by multiplying the line-element result by the thickness.

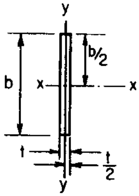
With this method, the final multiplier is always the first power of the thickness, and first-power quantities such as radius of gyration and those locating the centroid of the section do not involve the thickness dimension. The assumption that the area, moment of inertia, and section modulus vary directly as the first power of the thickness is particularly useful in determining the required thickness of a section after the widths of the various elements composing the section have been fixed. This method is sufficiently accurate for most practical purposes. It is advisable, however, particularly when a section is fairly thick compared to the widths of the elements, to check the final result through an exact method of computation.

Properties of thin elements are given in Table 8.4.

Various Failure Modes. One of the distinguishing characteristics of lightweight cold-formed sections is that they are usually composed of elements that are relatively wide and thin. As a result, attention must be given to certain modes of structural behavior ordinarily neglected in dealing with heavier sections, such as hot-rolled structural shapes.

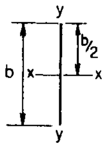
TABLE 8.4 Properties of Area and Line Elements

Area | **Line**

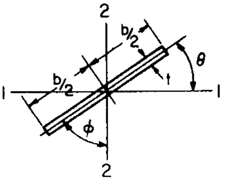


$$A = bt$$
$$I_x = \frac{b^3 t}{12}$$
$$I_y = \frac{bt^3}{12}$$

Rectangle

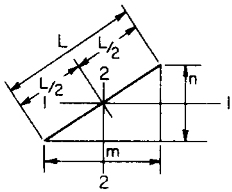


$$A_l = b$$
$$I_{lx} = \frac{b^3}{12}$$
$$I_{ly} = 0$$

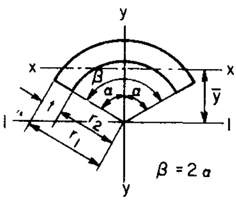


$$I_1 = \frac{bt}{12}(b^2 \sin^2 \theta + t^2 \cos^2 \theta)$$
$$I_2 = \frac{bt}{12}(b^2 \sin^2 \phi + t^2 \cos^2 \phi)$$

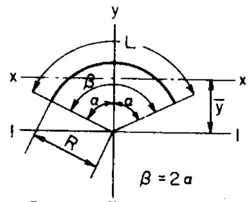
Inclined Rectangle



$$I_1 = \frac{Ln^2}{12}$$
$$I_2 = \frac{Lm^2}{12}$$

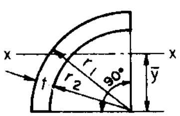


$$A = \beta t\left(\frac{r_1 + r_2}{2}\right)$$
$$\bar{y} = \frac{2}{3A}(r_1^3 - r_2^3) \sin \frac{\beta}{2}$$
$$I_x = ⅛(r_1^4 - r_2^4)(\beta + \sin \beta) - A\bar{y}^2$$
$$I_y = ⅛(r_1^4 - r_2^4)(\beta - \sin \beta)$$

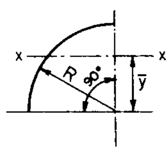


$$L = R\beta$$
$$\bar{y} = 2R \frac{\sin \frac{\beta}{2}}{\beta}$$
$$I_x = R^3\left(\frac{\beta + \sin \beta}{2} - \frac{4 \sin^2 \frac{\beta}{2}}{\beta}\right)$$
$$I_y = ½ R^3(\beta - \sin \beta)$$

Circular Arc



$$A = \frac{\pi}{4} t(r_1 + r_2)$$
$$= 0.7854t(r_1 + r_2)$$
$$\bar{y} = \frac{4}{3\pi} \frac{(r_1^3 - r_2^3)}{t(r_1 + r_2)}$$
$$= 0.424 \frac{(r_1^3 - r_2^3)}{t(r_1 + r_2)}$$
$$I_x = \frac{\pi}{16}(r_1^4 - r_2^4) - A\bar{y}^2$$
$$= 0.1964(r_1^4 - r_2^4) - A\bar{y}^2$$



$$L = \frac{\pi R}{2}$$
$$\bar{y} = \frac{2}{\pi} R = 0.637R$$
$$I_x = 0.1488R^3$$

For small radii:

r_2	A	$\bar{y}$	I_x
$2t$	$3.927t^2$	$1.613t$	$2.549t^4$
$1.5t$	$3.142t^2$	$1.300t$	$1.369t^4$
t	$2.356t^2$	$0.990t$	$0.635t^4$
$0.75t$	$1.963t^2$	$0.838t$	$0.400t^4$
$0.5t$	$1.571t^2$	$0.690t$	$0.235t^4$

90° Circular Corner

When thin, wide elements are in axial compression, as in the case of a beam flange or a part of a column, they tend to buckle elastically at stresses below the yield point of the steel. This local buckling is not to be confused with the general buckling that occurs in the failure of a long column or of a laterally unsupported beam. Rather, local buckling represents failure of a single element of a section, and conceivably may be relatively unrelated to buckling of the entire member. In addition, there are other factors, such as shear lag, which gives rise to nonuniform stress distribution; torsional instability, which may be more pronounced in thin sections than in thicker ones and requires more attention to bracing; and other related structural phenomena customarily ignored in conventional structural design that sometimes must be considered with thin material. Means of taking care of these factors in ordinary structural design are described in the ''Specification for the Design of Cold-Formed Steel Structural Members.''

Design Bases. The allowable stress design method (ASD) is used currently in structural design of cold-formed steel structural members and described in the rest of this section. In addition, the load and resistance factor design method (LRFD) can also be used for design. Both methods are included in the 1996 edition of the AISI ''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 stress 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:

$$R \leq R_n/\Omega$$

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 stress 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:

$$R_u \leq \varphi R_n$$

where $R_u = \Sigma \gamma_i Q_i$ = requires 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 provided in Chapter A of the AISI Specification for the design of different types of cold-formed steel structural members and connections. For design examples, see AISI ''Cold-Formed Steel Design Manual,'' 1996 edition.

The Committee on Specifications of the American Iron and Steel Institute has strived to put all formulas in the "Specification for the Design of Cold-Formed Steel Structural Members" on nondimensional bases so that their use with English or SI units is rigorous and convertible.

(AISI "Cold-Formed Steel Design Manual," American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036.)

8.5 STRUCTURAL BEHAVIOR OF FLAT COMPRESSION ELEMENTS

In buckling of flat, thin 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 any edge fillets, to the thickness t of the element (Fig. 8.4). Local buckling of elements with large w/t may be resisted with stiffeners or bracing.

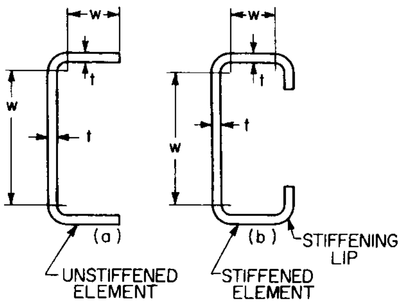


FIGURE 8.4 Compression elements.

Flat compression elements of cold-formed structural members are accordingly classified as stiffened or unstiffened. Stiffened compression elements have both edges of the element parallel to the direction of stress stiffened by a web, flange, or stiffening lip. If the sections in Fig. 8.1a to n are used as compression members, the webs are considered as stiffened compression elements. The wide, lipless flange elements and the lips that stiffen the outer edges, however, are unstiffened elements. Any section can be broken down into a combination of stiffened and unstiffened elements.

Only part of an element may be considered effective under compression in computation of net section properties. The portion that may be treated as effective depends on w/t for the element.

The cold-formed structural cross sections shown in Fig. 8.5 indicate that the effective portions b of the width of a stiffened compression element are considered to be divided into two parts, located next to the two edge stiffeners of that element. (A stiffener may be a web, another stiffened element, or a lip in beams. Lips in these examples are presumed to be fully effective.) In computation of net section properties, only the effective portions of stiffened compression elements are used and the ineffective portions are disregarded. For beams, because flange elements subjected to uniform compression may not be fully effective, reduced section properties, such as moments of inertia and section moduli, must be used. For computation of the effective widths of webs, see Art. 8.7. Effective areas of column cross sections are based on full cross-sectional areas less all ineffective portions for use in the formula for axially loaded columns, Eq. (8.22), in Art. 8.13.

The critical load, P_{cr}, kips, for elastic flexural buckling of a bar of uniform cross section, concentrically end loaded as a column, is given by the Euler formula:

$$P_{cr} = \pi^2 EI/L^2 \tag{8.1}$$

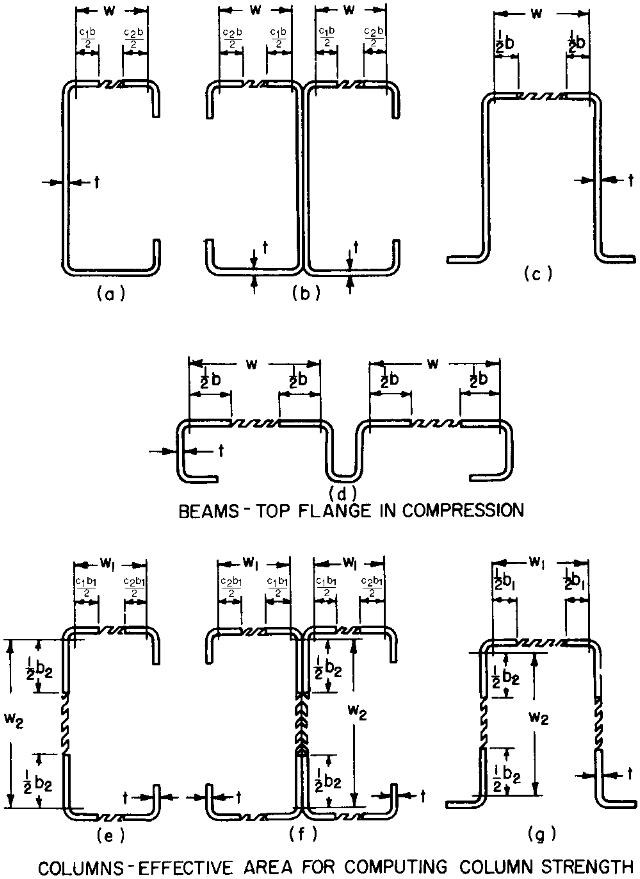


FIGURE 8.5 Effective width of stiffened compression elements with stiffening lips assumed to be fully effective.

where 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

Bryan, in 1891, determined the critical buckling stress, f_{cr}, ksi, for a thin rectangular plate compressed between two opposite edges with the other two edges supported, to be given by

$$f_{cr} = k\pi^2 E(t/w)^2/12(1 - \nu^2) \tag{8.2}$$

where k = a coefficient depending on edge-support restraint
w = width of late, in
t = thickness of plate, in
ν = Poisson's ratio

In 1932, von Karman gave the following formula for determining the effective width-to-thickness ratio b/t at yielding along the simply supported edges of a thin rectangular plate subjected to compression between the other two opposite edges:

$$b/t = 1.9t\sqrt{E/f_y} \tag{8.3}$$

where b = effective width for a plate of width w, in, and f_y = yield strength of plate material, ksi.

After extensive tests of cold-formed steel structural sections, Winter, in 1947, recommended that von Karman's formula be modified to

$$b/t = 1.9t\sqrt{E/f_{\max}}\left(1 - \frac{0.475\sqrt{E/f_{\max}}}{w/t}\right) \tag{8.4}$$

where $f_{\max}$ = maximum stress at simply supported edges, ksi. This formula for determining the effective widths of stiffened, thin, flat elements was first used in the AISI "Light-Gage Steel Design Manual," 1949. Subsequent studies showed that the factor 0.475 was unnecessarily conservative and that 0.415 was more appropriate. It was used in AISI specifications between 1968 and 1980 to evaluate post-buckling strength of thin, flat elements.

Until 1986, all AISI specifications based strength of thin, flat elements stiffened along one edge on *buckling stress.* In contrast, *effective width* was used for thin, flat elements stiffened along both edges. This treatment changed after Pekoz in 1986 presented a 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. Pekoz proposed the following three equations to generalize Eq. (8.4) with a factor of 0.415:

$$\lambda = [1.052(w/t)\sqrt{f/E}]/\sqrt{k} \tag{8.5}$$

where k = 4.00 for stiffened elements
= 0.43 for unstiffened elements
f = stress in the compression elements of the section computed on the basis of the design width, in
w = flat width of the element exclusive of radii, in
t = base thickness of element, in
λ = a slenderness factor

The effective width is computed from

$$b = w \qquad \lambda \le 0.673 \tag{8.6a}$$

$$b = \rho w \qquad \lambda > 0.673 \tag{8.6b}$$

where ρ is a reduction factor to be computed from

$$\rho = \frac{1 - 0.22/\lambda}{\lambda} \tag{8.7}$$

These equations were adopted in the AISI "Specification for the Design of Cold-Formed Steel Structural Members," 1986 and are retained in the 1996 edition of the AISI Specifications. See also Arts. 8.6 to 8.8.

8.6 UNSTIFFENED COLD-FORMED ELEMENTS SUBJECT TO LOCAL BUCKLING

As indicated in Art. 8.5, the effective width of an unstiffened element in compression may be computed from Eqs. (8.5) to (8.7). By definition, unstiffened elements have only one edge in the direction of compression stress supported by a web or stiffened element while the other edge has no auxiliary support (Fig. 8.6*a*). The coefficient k in Eq. (8.5) is 0.43 for such an element. When the flat-width-to-thickness ratio does not exceed $72/\sqrt{f}$, where f = compressive stress, ksi, an unstiffened element is fully effective and $b = w$. Generally, however, Eq. (8.5) becomes

$$\lambda = \frac{1.052(w/t)\sqrt{f/E}}{\sqrt{0.43}} = 0.0093(w/t)\sqrt{f} \tag{8.8}$$

where E = 29,500 ksi for steel. Substitution of λ in Eq. (8.7) yields $b/w = \rho$. Fig. (8.7*a*) shows a nest of curves for the relationship of b/t to w/t for unstiffened elements for w/t between 0 and 60 with f between 15 and 90 ksi.

In *beam deflection* determinations requiring use of the moment of inertia of the cross section, the allowable stress f is used to calculate the effective width of an unstiffened element in a cold-formed steel member loaded as a beam. However, in *beam strength* determinations requiring use of the section modulus of the cross section, $1.67f$ is the stress to be used in Eq. (8.8) to calculate the effective width of the unstiffened element and provide an adequate margin of safety.

In determination of safe loads for a cold-formed steel section used as a column, the effective width for an unstiffened element should be determined for a nominal buckling stress, F_n, to ensure an adequate margin of safety.

8.7 STIFFENED COLD-FORMED ELEMENTS SUBJECT TO LOCAL BUCKLING

As indicted in 8.5, the effective width of a stiffened element in compression may be computed from Eqs. (8.5) to (8.7). By definition, stiffened elements have one edge in the direction of compression stress supported by a web or stiffened element and the other edge also supported by a qualified stiffener (Fig. 8.6*b*). The coefficient k in Eq. (8.5) is 4.00 for such an element. When the flat-width-to-thickness ratio does not exceed $220/\sqrt{f}$, where f = compressive stress, ksi, computed on the basis of the effective section, a stiffened element is fully effective and $b = w$. Generally, however, Eq. (8.5) becomes

$$\lambda = \frac{1.052(w/t)\sqrt{f/E}}{\sqrt{4}} = 0.0031(w/t)\sqrt{f} \tag{8.9}$$

where E = 29,500 ksi for steel. Substitution of λ in Eq. (8.7) yields $b/w = \rho$. Moreover, when $\lambda \leq 0.673$, $b = w$ and when $\lambda > 0.673$, $b = \rho w$. Figure 8.7*b* shows a nest of curves for the relationship of b/t to w/t for stiffened elements for w/t between 0 and 500 with f between 10 and 90 ksi.

In *beam deflection* determinations requiring use of the moment of inertia of the cross section, the allowable stress f is used to calculate the effective width of a

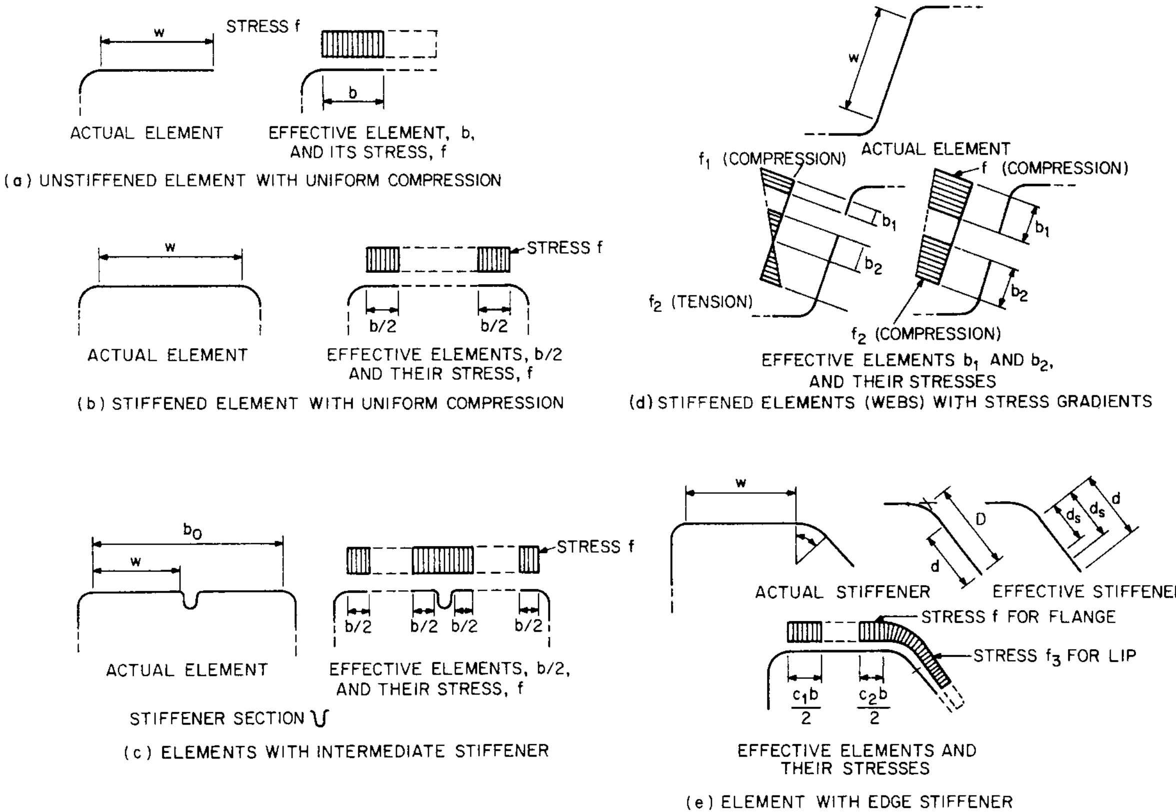


FIGURE 8.6 Schematic diagrams showing effective widths for unstiffened and stiffened elements, intermediate stiffeners, beam webs, and edge stiffeners.

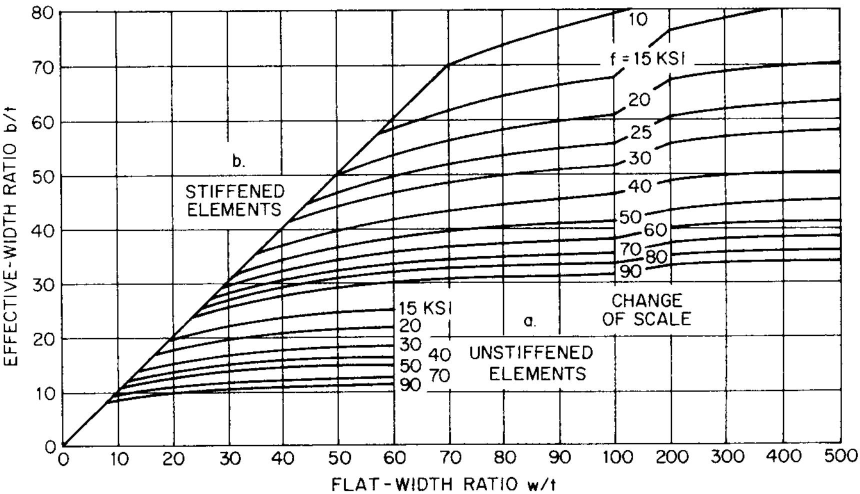


FIGURE 8.7 Curves relate effective-width ratio b/t to flat-width ratio w/t at various stresses f for (a) unstiffened elements and (b) stiffened elements.

stiffened element in a cold-formed steel member loaded as a beam. However, in *beam strength* determinations requiring use of the section modulus of the cross section, $1.67f$ is the stress to be used in Eq. (8.9) to calculate the effective width of the stiffened element and provide a margin of safety.

In determination of the safe loads for a cold-formed steel section used as a column, effective width for a stiffened element must be determined for a nominal buckling stress, F_n, to ensure an adequate margin of safety.

Since effective widths are proportional to $\sqrt{k}$, the effective width of a stiffened element is $\sqrt{4.00/0.43} = 3.05$ times as large as that of an unstiffened element at applicable combinations of f and w/t. Thus, stiffened elements offer greater strength and economy.

Single Intermediate Stiffener. For uniformly compressed stiffened elements with a single intermediate stiffener, as shown in Fig. 8.6c, calculations for required moment of inertia I_a of the stiffener are based on a parameter S.

$$S = 1.28\sqrt{E/f} \tag{8.10}$$

For Case I, $S \geq b_o/t$, where b_o = flat width, in, including the stiffener. $I_a = 0$ and no stiffener is required.

For Case II, $S < b_o/t < 3S$. The required moment of inertia is determined from

$$I_a/t^4 = [50(b_o/t)/S] - 50 \tag{8.11a}$$

For Case III, $b_o/t \geq 3S$. The required moment of inertia is determined from

$$I_a/t^4 = [128(b_o/t)/S] - 285 \tag{8.11b}$$

Webs Subjected to Stress Gradients. Effective widths also are applicable to stiffened elements subject to stress gradients in compression, such as in the webs of beams. Figure 8.6*d* illustrates the application. The effective widths b_1 and b_2 are determined with the use of the following equations:

$$b_1 = b_e/(3 - \psi) \tag{8.12}$$

where $\psi = f_2/f_1$
f_1 = stress, ksi, in compression flange (Fig. 8.6*d*)
f_2 = stress, ksi, in opposite flange (Fig. 8.6*d*)
b_e = effective width b determined from Eqs. (8.5) to (8.7) with f_1 substituted for f and with k calculated from Eq. (8.14)

Stress f_2 may be tensile (negative) or compressive (positive). When both f_1 and f_2 are compressive, $f_1 \geq f_2$.

$$b_2 = \tfrac{1}{2} b_e \qquad \text{for } \psi \leq -0.236 \tag{8.13a}$$

where $b_1 + b_2$ should not exceed the depth of the compression portion of the web calculated for the effective cross section.

$$b_2 = b_e - b_1 \qquad \text{for } \psi > -0.236 \tag{8.13b}$$

$$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \tag{8.14}$$

Uniformly Compressed Elements with Edge Stiffener. While a slanted lip, as depicted in Fig. 8.6*e*, may be used as an edge stiffener for a cold-formed steel section, calculation of stresses for such a section is complex. (See AISI "Specification for the Design of Cold-Formed Steel Structural Members.") Consequently, the following is primarily applicable to 90° lips.

Calculation of the required moment of inertia, I_a, falls into one of three cases:

For Case I, $w/t \leq S/3$. $b = w$, where b is the effective width, and no edge support is needed. S is defined by Eq. (8.10) and is the maximum w/t for full effectiveness of the flat width without auxiliary support.

For Case II, $S/3 < w/t < S$. The required moment of inertia of the lip is determined from

$$I_a/t^4 = 399\{[(w/t)/S] - \sqrt{k_u/4}\}^3 \tag{8.15}$$

where $k_u = 0.43$. When $S/3$ is substituted for w/t in Eq. (8.15), $I_a = 0$ and no support is needed at the edge for which a lip is being considered (see Case I). When $w/t = S$, a stiffening lip would be required to have a depth-thickness ratio d/t of 11.3. The maximum stress in a lip with this value of d/t, however, could be only 40.6 ksi, which corresponds to a maximum allowable stress of 24.3 ksi in bending and 22.6 ksi in compression, with safety factors of 1.67 and 1.80, respectively.

For Case III, $w/t \geq S$. The required moment of inertia of the edge stiffener is determined from

8.8 APPLICATION OF EFFECTIVE WIDTHS

The curves of Fig. 8.7 were plotted from values of Eqs. (8.8) and (8.9). They may be used to determine b/t for different values of w/t and unit stresses f. The effective width b is dependent 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 and curves. A direct solution for the correct value of b/t can be obtained from the formulas, however, when f is known or is held to a specified maximum allowable value for deflection determination (20 ksi for $F_y = 33$ ksi, for example). This is true, though, only when compression controls; for example, for symmetrical channels and Z and I sections used as flexural members bending about their major axis (Fig. 8.1*e*, *f*, *k* and *n*) or for unsymmetrical channels and Z and I sections with neutral axis closer to the tension flange than to the compression flange. If w/t of the compression flange does not exceed about 60, little error will result in assuming that $f = 0.60 \times 33 = 20$ ksi for $F_y = 33$ ksi. This is so even though the neutral axis is above the geometric centerline. For wide, inverted, pan-shaped sections, such as deck and panel sections, a somewhat more accurate determination using successive approximations will prove 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. (8.8) and (8.9) to obtain appropriate effective widths.

Example. As an example of effective-width determination, consider the hat section of Fig. 8.8. The section is to be made of steel with a specified minimum yield strength $F_y = 33$ ksi. It is to be used as a simply supported beam with the top flange in compression, at a basic working stress of 20 ksi. Safe load-carrying capacity is to be computed; so $f = 20 \times 1.67 = 33$ ksi is used to obtain b/t.

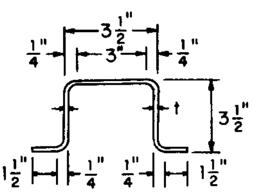


FIGURE 8.8 Hat section.

The top flange is a stiffened compression element with 3-in flat width. If the thickness is 1⁄16 in, then the flat-width-thickness ratio (w/t) is 48 (greater than $w/t = 220/\sqrt{33} = 38$), stiffening is required, and Eq. (8.9) applies. For $w/t = 48$ and $f = 33$ ksi, Eq. (8.9) gives $b/t = 41$. Thus, with $b/w = 41/48$, only 85% of the top-flange flat width can be considered effective. The neutral axis will lie below the horizontal center line, and compression will control. In this case, the assumption that $f = 33$ ksi, made at the start, controls maximum stress, and b/t can be determined directly from Eq. (8.9) without successive approximations. However, for a wide hat section in which the horizontal axis is nearer the compression than the tension flange, stress in the tension flange controls, and successive approximations are required for the determination of unit stress and effective width of the compression flange.

(“Cold-Formed Steel Design Manual,” American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036.)

8.9 MAXIMUM FLAT-WIDTH RATIOS OF COLD-FORMED SHAPES

When the flat-width-thickness ratio (w/t) exceeds about 30 for an unstiffened element and about 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 to take advantage of what is known as post-buckling strength of the section. The effective-width formulas, Eqs. (8.5) to (8.7), are based on this practice. To avoid intolerable deformations, however, w/t, disregarding intermediate stiffeners and based on the actual thickness t 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 with both longitudinal edges connected to a web or flange element, such as in a hat, U, or box-type section	500
Unstiffened compression element	60

8.10 UNIT STRESSES FOR COLD-FORMED STEEL

For sheet and strip of A611, Grade C steel with a specified minimum yield strength $F_y = 33$ ksi, use a basic allowable stress $f = 20$ ksi in tension and bending. For other strengths of steels, f is determined by taking 60% of the specified minimum yield strength F_y. (This procedure implies a safety factor of 1.67.) However, an increase of 33⅓% in allowable stress is customary for combined wind or earthquake forces with other loads. It should be noted that the 1996 AISI specification uses "strength" (moment, force, etc.) rather than unit stress.

8.11 LATERALLY UNSUPPORTED COLD-FORMED BEAMS

If cold-formed steel sections are not laterally supported at frequent intervals, the allowable unit stress 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. (See AISI "Specification for the Design of Cold-Formed Steel Structural Members.")

Because of the torsional flexibility of lightweight channel and Z sections, their use as beams without close lateral support is not recommended. When a compression flange is fully connected to a deck or sheathing material, the flange is considered braced for its full length and bracing of the other flange may not be needed to prevent buckling of the beam. This depends on the collateral material and its connections, dimensions of the member, and the span.

When laterally unsupported beams must be used, or where lateral buckling of a flexural member is likely to occur, consideration should be given to the use of relatively bulky sections that have two webs, such as hat or box sections (Fig. 8.1*o*, *p*, and *q*).

8.12 ALLOWABLE SHEAR STRENGTH IN WEBS

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

For $h/t \leq 0.96\sqrt{k_v E/F_y}$,

$$V_a = 0.4F_y ht \tag{8.17}$$

For $0.96\sqrt{k_v E/F_y} < h/t \leq 1.415\sqrt{k_v E/F_y}$,

$$V_a = 0.38t^2\sqrt{k_v EF_y} \tag{8.18}$$

For $h/t > 1.415\sqrt{k_v E/F_y}$,

$$V_a = 0.54k_v Et^3/h \tag{8.19}$$

where t = web thickness, in
h = depth of the flat portion of the web measured along the plane of the web, in
E = modulus of elasticity of the steel = 29,500 ksi
k_v = shear buckling coefficient = 5.34 for unreinforced webs for which $(h/t)_{max}$ does not exceed 200
F_y = specified yield stress of the steel, ksi

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

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

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

$$(M/M_{axo})^2 + (V/V_a)^2 \leq 1.0 \tag{8.20}$$

where M_{axo} = allowable moment about the centroidal axis, in-kips, when bending alone is present
V_a = allowable shear, kips, when shear alone exists
M = applied bending moment, in-kips
V = actual shear, kips

In addition to above, web crippling should also be checked.

8.13 CONCENTRICALLY LOADED COMPRESSION MEMBERS

The following formulas apply to members in which the resultant of all loads acting on a member is an axial load passing through the centroid of the effective section (calculated at the nominal buckling stress F_n, ksi). The axial load should not exceed P_a, kips, calculated from

$$P_a = P_n/\Omega_c \tag{8.21}$$

$$P_n = A_e F_n \tag{8.22}$$

where 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:
For $\lambda_c \leq 1.5$,

$$F_n = (0.658^{\lambda_c^2})\ F_y \tag{8.23a}$$

For $\lambda_c > 1.5$,

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right] F_y \tag{8.23b}$$

where $\lambda_c = \sqrt{\dfrac{F_y}{F_e}}$
F_y = yield stress of the steel, ksi
F_e = the least of the elastic flexural, torsional and torsional-flexural buckling stress

Figure 8.9 shows the ratio between the column buckling stress F_n and the yield strength F_y.
For elastic flexural behavior,

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \tag{8.24}$$

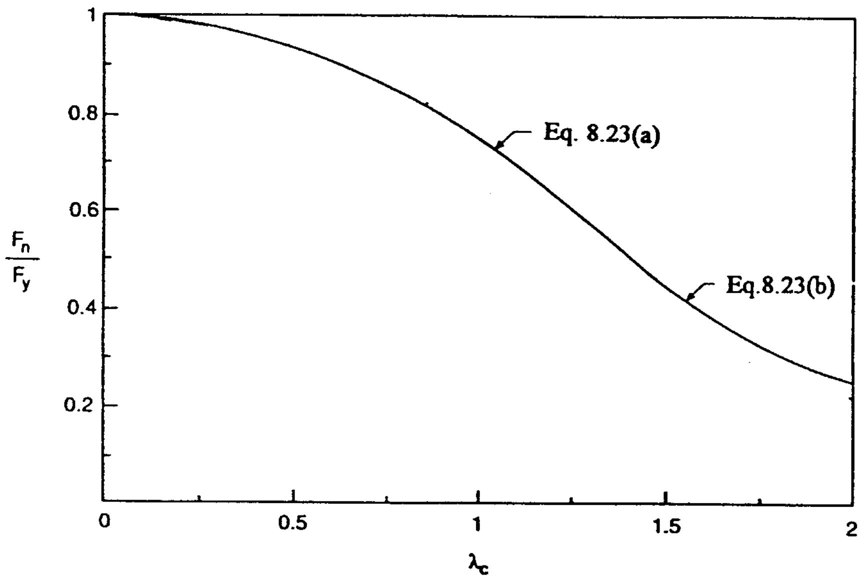


FIGURE 8.9 Ratio of nominal column buckling stress to yield strength.

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 of the steel, ksi

Moreover, 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 and 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 sections that may be subject to torsional or torsional-flexural buckling, refer to AISI ''Specification for the Design of Cold-Formed Steel Structural Members,'' American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036.

8.14 COMBINED AXIAL AND BENDING STRESSES

Combined axial and bending stresses in cold-formed sections can be handled the same way as for structural steel. The interaction criterion to be used is given in the AISI ''Specification for the Design of Cold-Formed Structural Members.''

JOINING OF COLD-FORMED STEEL

Cold-formed members may be assembled into desired shapes or spliced or joined to other members with any of various types of fasteners. For the purpose, welds, bolts, and screws are most frequently used, but other types, such as rivets, studs, and metal stitching, can also be used.

8.15 WELDING OF COLD-FORMED STEEL

Electric currents are generally used in either of two ways to joint cold-formed steel components, with electric-arc welding or resistance welding. The former method is described in Art. 8.16 and the latter in Art. 8.17.

Welding offers important advantages to fabricators and erectors in joining steel 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 where notch effects are minimal; the final appearance 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 damaged by welding. Coatings may adversely affect weld quality. Metal-coated steels, such as galvanized (zinc-coated), aluminized, and terne-coated (lead-tin alloy), however may be successfully welded using procedures tailored for the steel and its coating.

Generally, steel to be welded should be clean and free of contaminants such as oil, grease, paints, and scale. Paint should be applied only after the welding process. (See "Welding Handbook," American Welding Society, 550 NW LeJeune Rd., Miami, FL 33126 and O. W. Blodgett, "Design of Weldments," James F. Lincoln Welding Foundation, Cleveland, OH 44117.)

8.16 ARC WELDING OF COLD-FORMED STEEL

Arc welding may be done in the shop or in the field. The basic sheet-steel weld types are shown in Fig. 8.10. Factors favoring arc welding are portability and versatility of equipment as well as freedom in joint design. Only one side of a joint need be accessible, and overlap of parts is not required if joint fit-up is good.

8.16.1 Helpful Hints for Welding

Distortion may occur with lightweight steel weldments, but it can be minimized by avoiding overwelding. Weld sizes should be matched with service requirements.

Always design welded 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 weld sequence can also be used to control distortion.

Groove welds (made by butting sheet edges together, Fig. 8.10*a*) can be designed for 100% joint efficiency. Calculation of design stress is usually unnecessary if the weld penetrates 100% of the section.

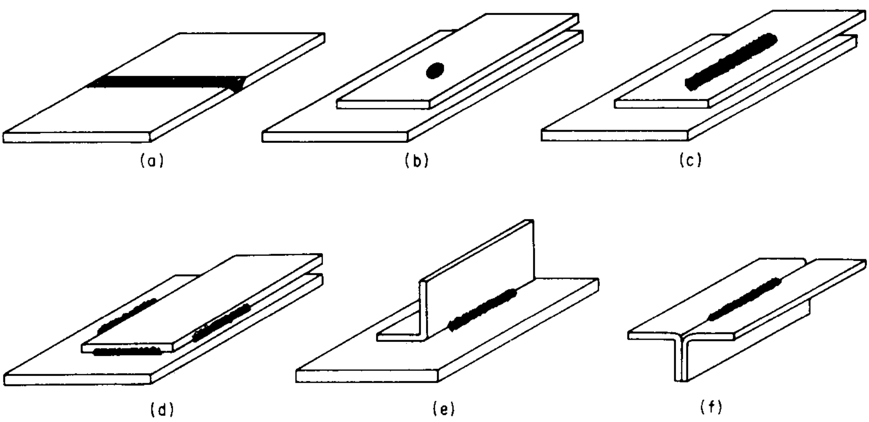


FIGURE 8.10 Types of sheet-steel welds: (*a*) square-groove weld: (*b*) arc spot weld (round puddle weld); (*c*) arc seam weld (oblong puddle weld); (*d*) fillet welds; (*e*) flare bevel-groove weld; (*f*) flare V-groove weld.

Stresses in fillet welds should be considered as shear on the throat for any direction of 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, ¼-in-fillet weld has a leg dimension of ¼ 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 stress design method; the safety factor is 2.50, unless otherwise noted.

8.16.2 Types of Arc Welding

Shielded metal arc welding, also called manual stick electrode, is the most common arc-welding process because of its versatility. The method, however, requires skilled operators. The welds can be made in any position, but 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 applicable to materials 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 spot welding is an adaptation 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 layer 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 is feasible. This makes the process advantageous.

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 also treated in the American Welding Society ‘‘Specification for Welding Sheet Steel in Structures,’’ AWS D1.3.

8.16.3 Groove Welds in Butt Joints

The maximum load for a groove weld in a butt joint, welded from one or both sides, should be determined on the basis of the lower-strength base steel in the connection, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained.

8.16.4 Arc Spot Welds

Arc spot welds (Fig. 8.10*b*), also known as puddle welds, are permitted for welding sheet steel to thicker supporting members in the flat position. Such welds, which result when coalescence proceeds from the surface of one sheet into one or more other sheets of a lapped joint without formation of a hole, should not be made on steel where the thinnest connected part is more than 0.15 in thick, or through a combination of steel sheets having a total thickness exceeding 0.15 in. Arc spot welds are specified by minimum effective diameter of fused area, d_e. Minimum effective allowable diameter is ⅜ in. The nominal shear load P_n, kips, on each arc spot weld between sheet or between sheets and a supporting member should not exceed the smaller of the values given by Eqs. (8.25) to (8.28).

$$P_n = 0.589 d_e^2 F_{xx} \tag{8.25}$$

For $d_a/t \le 0.815\sqrt{E/F_u}$,

$$P_n = 2.20 t d_a F_u \tag{8.26}$$

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

$$P_n = 0.280\left[1 + \frac{5.59\sqrt{E/F_u}}{d_a/t}\right] t d_a F_u \tag{8.27}$$

For $d_a/t \ge 1.397\sqrt{E/F_u}$,

$$P_n = 1.40 t d_a F_u \tag{8.28}$$

where d_a = average diameter, in, of the arc spot weld at midthickness of sheet
= $d - t$ for a single sheet
= $d - 2t$ for multiple sheets (not more than four lapped sheets over a supporting member)
d = visible diameter of outer surface of arc spot weld, in
d_e = effective diameter of fused area, in
= $0.7d - 1.5t \le 0.55d$
t = total combined base steel thickness, in (exclusive of coatings) of sheets involved in shear transfer
F_{xx} = stress-level designation in AWS electrode classification, ksi
F_u = tensile strength of the base steel 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 be at least $e_{\min}$, in, as given by

$$e_{\min} = e\Omega_e \tag{8.29}$$

where e = $P/(F_u t)$
Ω_e = factor of safety for sheet tearing
= 2.0 when $F_u/F_{sy} \ge 1.08$
= 2.22 when $F_u/F_{sy} < 1.08$
P = force transmitted by weld, kips
F_{sy} = yield strength of sheet steel, ksi, as specified
t = thickness of thinnest connected sheet, in

In addition, the distance from the centerline of any weld to the end or boundary of the connected member should be at least $1.5d$. In no case should the clear distance between welds and the end of the member be less than d.

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

$$P_n = 0.785 d_e^2 F_{xx} \tag{8.30a}$$

or either:

For $F_u/E < 0.00187$

$$P_n = [6.59 - 3150(F_u/E)]td_aF_u \leq 1.46td_aF_u \tag{8.30b}$$

For $F_u/E \geq 0.00187$

$$P_n = 0.70td_aF_u \tag{8.30c}$$

The following limitations also apply: $e_{\min} \geq d$, $F_{xx} \geq 60$ ksi, $F_u \leq 82$ ksi, and $t \geq 0.028$ in.

As for arc spot welds (Art. 8.16.4), if measurements indicate that a given weld procedure will consistently give larger diameters d_a or d_e, as applicable, the larger diameter may be used to calculate the maximum allowable load, if that procedure will be used.

8.16.5 Arc Seam Welds

These are basically the same as arc spot welds but are made linearly without slots in the sheets (Fig. 8.10*c*). Arc seam welds apply to the following types of joints:

1. Sheet to a thicker supporting member in the flat position

2. Sheet to sheet in the horizontal or flat position

The shear load P_n, kips, on an arc seam weld should not exceed the values given by either Eq. (8.31) or (8.32).

$$P_n = \left[\frac{\pi d_e^2}{4} + Ld_e\right] 0.75F_{xx} \tag{8.31}$$

$$p_n = 2.5tF_u(0.25L + 0.96d_a) \tag{8.32}$$

where d_a = average width, in, of arc seam weld
= $d - t$ for a single sheet
= $d - 2t$ for a double sheet
d = width, in, of arc seam weld
L = length, in, of weld not including the circular ends (in computations, L should not exceed $3d$)
d_e = effective width, in, of weld at fused surfaces
= $0.7d - 1.5t$

F_u and F_{xx} are defined as for arc spot welds (Art. 8.16.4). Minimum edge distances also are defined as for arc spot welds.

If measurements indicate that a given weld procedure will consistently give a larger effective width d_e or larger average diameter d_a, as applicable, these values may be used to calculate the maximum allowable load on an arc seam weld, if that welding procedure will actually be used.

8.16.6 Fillet Welds

These are made along the edges of sheets in lapped or T joints (Fig. 8.10*d*). The fillet welds may be made in any position and either sheet to sheet or sheet to thicker steel member.

The shear load P_n, kips, on a fillet weld in lapped or T joints should not exceed the value of P_n computed from Eqs. (8.33) to (8.34).

For longitudinal loading along the weld:

$$P_n = (1 - 0.01L/t)tLF_u \qquad L/t < 25 \tag{8.33}$$

$$P_n = 0.75tLF_u \qquad L/t \geq 25 \tag{8.34}$$

where t = smaller thickness of sheets being welded, in
L = length, in, of the fillet weld
F_u = specified tensile strength of base steel, ksi

For loading transverse to the weld:

$$P_n = tLF_u \tag{8.35}$$

For $t > 0.15$ in,

$$P_n = 0.75t_wLF_{xx} \tag{8.36}$$

where F_{xx} = stress-level designation in AWS electrode classification, ksi
t_w = effective throat of weld, in
= 0.707 times the smaller of the weld-leg lengths

8.16.7 Flare Groove Welds

These are made on the outsides of curved edges of bends in cold-formed shapes (Fig. 8.10*e* and *f*). The welds may be made in any position to join:

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 shear load P_n, kips, on a weld is governed by the thickness t, in, of the sheet adjacent to the weld. The load should not exceed the values of P_n given by Eqs. (8.37) to (8.40).

For flare bevel-groove welds subject to transverse loading,

$$P_n = 0.833tLF_u \tag{8.37}$$

where L = length, in, of the weld and F_u = specified tensile strength, ksi, of the base steel.

For flare V-groove welds, subject to longitudinal loading,

$$P_n = 0.75tLF_u \qquad t \leq t_w < 2t \text{ or } h < L \tag{8.38}$$

where t_w = effective throat of the weld, in and h = lip height, in

$$P_n = 1.50tLF_u \qquad t_w \geq 2t \text{ and } h \geq L \tag{8.39}$$

In addition, if $t > 0.15$ in,

$$P_n = 0.75t_wLF_{xx} \tag{8.40}$$

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

8.17 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 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 ⅜ 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.

8.17.1 Spot Welding

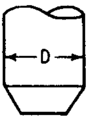
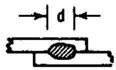
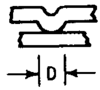
Spot welding is the most common resistance-welding process. The weld is formed at the interface between the pieces being joined and consists of a cast-steel nugget. The nugget has a diameter about equal to that of the electrode face and should penetrate about 60 to 80% of each sheet thickness.

For structural design purposes, spot welds can be treated the same way as bolts, except that no reduction in net section due to holes need be made. Table 8.5 gives the essential information for design purposes for uncoated steel based on ‘‘Recommended Practices for Resistance Welding.’’ American Welding Society, 1966. The maximum allowable loads per weld for design purposes are based on shear strengths of welds observed in tests after application of a safety factor of 2.5 bounds of data. Note that the thickest steel for plain spot welding is ⅛ in. Thicker material can be resistance welded by projection or by pulsation methods if high capacity spot welders for material thicker than ⅛ in are not available.

8.17.2 Projection Welding

This 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 steel using spot welders ordinarily limited to thinner stocks.

TABLE 8.5 Design Data for Spot and Projection Welding of Low-Carbon Sheet Steel

Thickness t of thinnest outside piece, in	Min OD of electrode, *D*. ir. 	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	5/8	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

8.17.3 Pulsation Welding

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

8.17.4 Recommended Practices for Spot Welding

The spot welding of higher-strength steels than those contemplated under Table 8.5 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. High-carbon steels such as ASTM A653, SQ Grade 50 (formerly, A446, 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.

TABLE 8.6 Nominal Shear Strength per Spot for Low-Carbon Sheet Steel

Thickness of thinnest outside sheet, in	Nominal shear strength per spot, kips	Thickness of thinnest outside sheet, in	Nominal shear strength per spot, kips
0.010	0.13	0.080	3.33
0.020	0.48	0.090	4.00
0.030	1.00	0.100	4.99
0.040	1.42	0.110	6.07
0.050	1.65	0.125	7.29
0.060	2.28	0.190	10.16
0.070	2.83	0.250	15.00

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 8.5 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 further information on spot welding of coated steels, see ''Recommended Practices for Resistance Welding of Coated Low-Carbon Steel,'' American Welding Society, 550 N.W. Lejeune Rd., Miami, FL 33126.

The nominal shear strength per spot, is a function of the thickness of the thinnest outside sheet. Table 8.6 lists spot shear strengths for sheets with thicknesses from 0.010 to 0.250 in, as recommended for design by the American Iron and Steel Institute.

8.18 BOLTING OF COLD-FORMED STEEL MEMBERS

Bolting is convenient in cold-formed construction. Bolts, nuts, and washers should generally conform to the requirements of the ASTM specifications listed in Table 8.7. The maximum sizes of bolt holes are given in Table 8.8. Standard holes should be used in bolted connections when possible. If slotted holes are used, the length of the holes should be normal to the direction of the shear load. Washers should be installed atop oversized or slotted holes.

8.18.1 Spacing of Bolts

The distance e, in, measured in the direction of applied 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 $e_{\min}$.

$$e_{\min} = e\Omega_e \tag{8.41}$$

$$e = P/F_u t \tag{8.42}$$

TABLE 8.7 ASTM Bolt, Nut, and Washer Steels

A194	Carbon and alloy steel nuts for high-pressure and high-temperature service
A307	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 bolt diameter less than ½ in)
A449	Quenched and tempered steel bolts and studs (for bolt diameter less 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

TABLE 8.8 Maximum Size of Bolt Holes, in.

Nominal bolt diameter, in	Standard hole diameter d_h, in	Oversized hole diameter d_h, in	Short-slotted hole, in	Long-slotted hole, in
Less than ½	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\frac{1}{2}d)$
½ or larger	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\frac{1}{2}d)$

where Ω_e = safety factor for sheet tearing
= 2.00 when $F_u/F_{sy} \geq 1.08$
= 2.22 when $F_u/F_{sy} < 1.08$
P = force, kips, transmitted by a bolt
t = thickness, in, of thinnest connected part
F_u = tensile strength, ksi, of connected part
F_{sy} = yield strength, ksi, of connected part

In addition, the minimum distance between centers of bolt holes should provide sufficient clearance for bolt heads, nuts, washers, and wrench but be at least 3 times the nominal diameter d, in. The distance from the center of any standard hole to the end or boundary of the connecting member should be at least $1\frac{1}{2}d$.

8.18.2 Bolted Cold-Formed Members in Tension

Calculation of the allowable tension force on the net section of a bolted connection depends on the thickness t, in, of the thinnest connected part. When t exceeds 3⁄16 in, design of the connection is governed by the AISC "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," American Institute of Steel Construction, One East Wacker Drive, Chicago, IL 60601. When t does not exceed 3⁄16 in and washers are provided under the bolt head and nut, the following is applicable:

The tension force on the net section should not exceed P_a, kips, calculated from Eq. (8.43).

$$P_a = P_n/\Omega_t \tag{8.43}$$

$$P_n = A_n F_t \tag{8.44}$$

where Ω_t = safety factor for tension on net section
= 2.22 for single shear
= 2.00 for double shear
A_n = area of net section of thinnest sheet, in^2

The nominal limiting tension stress F_t, kips, is given by

$$F_t = (1 - 0.9r + 3rd/s)F_u \leq F_u \tag{8.45}$$

where s = bolt spacing, in, measured normal to line of stress
= width of sheet for a single bolt in the net section
F_u = tensile strength, ksi, of connected part
d = nominal diameter, in, of bolt
r = ratio of force transmitted by the bolts at the section to the tension force in the member at that section (if $r < 0.2$, it may be taken equal to zero)

When washers are not provided under the bolt head and nut, see AISI specification.

8.18.3 Bearing Stresses and Bolt Tension

The bearing force should not exceed P_a, kips, calculated from Eq. (8.46).

$$P_a = P_n/\Omega_b \tag{8.46}$$

$$P_n = F_p dt \tag{8.47}$$

where Ω_b = safety factor for bearing = 2.22
F_p = nominal bearing stress, ksi, in connected part
d = nominal diameter of bolt, in
t = thickness, in, of thinnest connected part

Table 8.9 lists nominal bearing stresses for bolted connections.

Table 8.10 lists nominal shear and tension stresses for various grades of bolts. The bolt force resulting in shear, tension, or combinations of shear and tension should not exceed the allowable force P_a, kips, calculated from Eq. (8.48).

$$P_a = A_b F/\Omega \tag{8.48}$$

where A_b = gross cross-sectional area of bolt, in^2
F = nominal stress, ksi, F_{nv}, F_{nt}, or F'_{nt} in Tables 8.10 and 8.11

Safety factors given in Tables 8.10 and 8.11 may be used with Eq. (8.48) to compute allowable loads on bolted joints.

Table 8.11 lists nominal tension stresses for bolts subjected to a combination of shear and tension.

TABLE 8.9 Nominal Bearing Stresses for Bolted Connections of Cold-Formed Steel Components[a]

Type of joint	Nominal bearing stress F_p, ksi: With washers under both bolt head and nut[b]	Nominal bearing stress F_p, ksi: Without washers under bolt head and nut or with only one washer[c]
Inside sheet of double-shear connection	$3.33F_u$ ($F_u/F_{sy} \geq 1.08$)[d] $3.00F_u$ ($F_u/F_{sy} < 1.08$)[d]	$3.00F_u$[e]
Sheets in single shear and outside sheets of double-shear connection	$3.00F_u$	$2.22F_u$[e]

[a] For joints with parts ³⁄₁₆ in or more thick, see the "Specification for Structural Steel Buildings," American Institute of Steel Construction.

[b] For joints with parts 0.024 in or more thick.

[c] For joints with parts 0.036 in or more thick.

[d] F_u/F_{sy} is the ratio of the tensile strength of a connected part to its yield strength.

[e] For $F_u/F_{sy} \geq 1.08$

8.18.4 Example—Tension Joints with Two Bolts

Assume that the bolted tension joints of Fig. 8.11 comprise two sheets of ³⁄₁₆-in-thick, A611, Grade C steel. For this steel, $F_{sy} = 33$ ksi and $F_u = 48$ ksi. The sheets in each joint are 4 in wide and are connected by two ⅝-in-diameter, A325 bolts, with washers under both bolt head and nut.

Case 1 of Fig. 8.11 has 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. (8.45), $r = 2(T/2)/T = 1$. Spacing of the bolts $s = 2$ in and $d/s = \frac{5}{8}/2 = 0.312$. The tension stress in the net section, computed from Eq. (8.45), is then

$$F_t = (1 - 0.9 \times 1 + 3 \times 1 \times 0.312)F_u = 1.04\ F_u > F_u$$

Use $F_t = F_u$.

Substitution in Eq. (8.44) with $F_u = 48$ ksi yields the nominal tension load on the net section:

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

The allowable load is

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

This compares with the tensile strength of each sheet for tension member design:

$$P_n = A_n F_{sy} = [4 - (2 \times \tfrac{11}{16})] \times \tfrac{3}{16} \times 33 = 16.24 \text{ kips}$$

The allowable load is

$$P_a = P_n/\Omega = 16.24/1.67 = 9.72 \text{ kips}$$

Use $P_a = 9.72$ kips.

TABLE 8.10 Nominal Tensile and Shear Strength for Bolts

Description of bolts	Tensile strength: Factor of safety Ω	Tensile strength: Nominal stress F_{nt}, ksi	Shear strength: Factor of safety Ω	Shear strength: 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 8.11 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			2.25
When ¼ in $\leq d <$ ½ in	$52 - 4f_v \leq 40.5$		
When $d \geq$ ½ in	$58.5 - 4f_v \leq 45$		

The shear stress, f_v, shall also satisfy Table 8.10.

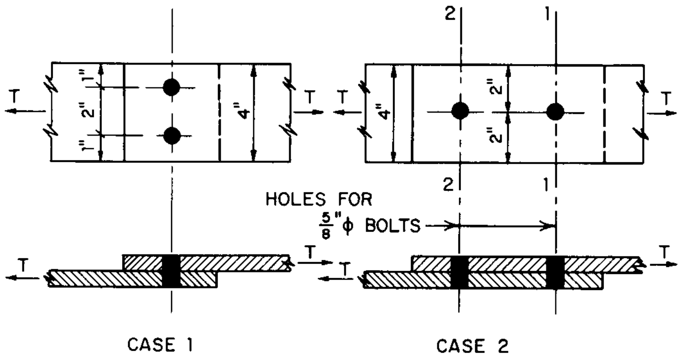


FIGURE 8.11 Bolted connections with two bolts.

Case 2 of Fig. 8.11 has the two bolts, with 4-in spacing, arranged in a single line along the direction of applied force. For the top sheet (Fig. 8.11) at section 1-1 then, $r = (T/2)/T = ½$, and for this sheet at section 2-2, $r = (T/2)/(T/2) = 1$. For the top sheet at both sections, $d/s = ⅝/4 = 0.156$.

From Eq. (8.45), for the top sheet at section 1-1,

$$F_t = (1 - 0.9 \times ½ + 3 \times ½ \times 0.156)F_u = 0.784\ F_u$$

The maximum load for that sheet would then be

$$P_n = [4 - {}^{11}\!/_{16}] \times {}^3\!/_{16} \times 0.784 \times 48 = 23.37 \text{ kips}$$

For section 2-2, top sheet,

$$F_t = (1 - 0.9 \times 1 + 3 \times 1 \times 0.156)F_u = 0.568F_u$$

Maximum load for section 2-2, top sheet, would then be

$$P_n/2 = (4 - 11/16) \times (3/16) \times 0.568 \times 48 = 16.93 \text{ kips}$$

$$P_n = 33.86 \text{ kips}$$

Compare sections 1-1 and 2-2, $P_n = 23.37$ kips. The allowable load is:

$$P_a = P_n/\Omega = 23.37/2.22 = 10.53 \text{ kips}$$

This compares with the tensile strength of each sheet for tension member design:

$$P_n = A_n F_{sy} = [4 - 11/16](3/16) \times 33 = 20.50 \text{ kips}$$

The allowable load is:

$$P_a = P_n/\Omega = 20.50/1.67 = 12.28 \text{ kips}$$

Use $P_a = 10.53$ kips.

The minimum distance between a bolt center and adjacent bolt edge or sheet edge is for Case 1:

$$e = P/F_u t = (9.72/2)/(48 \times 3/16) = 0.54 \text{ in}$$

$$e_{\min} = e\Omega = 0.54 \times 2 = 1.08 \text{ in}$$

For Case 2:

$$e = (10.53/2)/(48 \times 3/16) = 0.59 \text{ in}$$

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

The bearing strength P_n per bolt of the 3/16-in-thick steel sheet is:

$$P_n = F_p\, dt = (3 \times 48) \times 5/8 \times 3/16 = 16.88 \text{ kips}$$

The allowable bearing load for two bolts:

$$P_a = 2P_n/\Omega = 2 \times 16.88/2.22 = 15.21 \text{ kips} > 10.53 \text{ kips} \quad \text{O.K.}$$

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

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

For two bolts, the allowable load is:

$$P_a = 2 \times 6.9 = 13.8 \text{ kips} > 10.53 \text{ kips} \quad \text{O.K.}$$

In summary, the allowable loads for Cases 1 and 2 are 9.72 kips and 10.53 kips, respectively. The shear capacity of bolts should also be checked.

8.19 SELF-TAPPING SCREWS FOR JOINING SHEET STEEL COMPONENTS

Self-tapping screws that are hardened so that their threads form or cut mating threads in one or both of the sheet steel parts being connected are frequently used for making field joints. Such screws provide a rapid and efficient means of making light-duty connections. The screws are especially useful for such purposes as fastening sheet-metal siding, roofing, and decking to structural steel; making attachments at joints, side laps, and closures in siding, roofing, and decking; fastening collateral materials to steel framing; and fastening steel studs to sill plates or channel tracks. The screws may also be used for fastening bridging to steel joists and studs, fastening corrugated decking to steel joists, and similar connections to secondary members.

Since 1996, the AISI specification included design rules for determining nominal loads for shear and tension. The safety factor to be used for computing the allowable load is 3.0.

Several types of tapping screws are shown in Fig. 8.12. Other types are available. There are many different head styles—slotted, recessed, hexagonal, flat, round, etc. Some types, called *Sems,* are supplied with preassembled washers under the heads. Other types are supplied with neoprene washers for making watertight joints in roofing.

All the types of screws shown in Fig. 8.12 require prepunched or predrilled holes. Self-drilling screws, which have a twist drill point that drills the proper size of hole just ahead of threading, are especially suited for field work, because they eliminate separate punching or drilling operations. Another type of self-drilling screw, capable of being used in relatively thin sheets of material in situations where the parts being joined can be firmly clamped together, has a very sharp point that pierces the material until the threads engage.

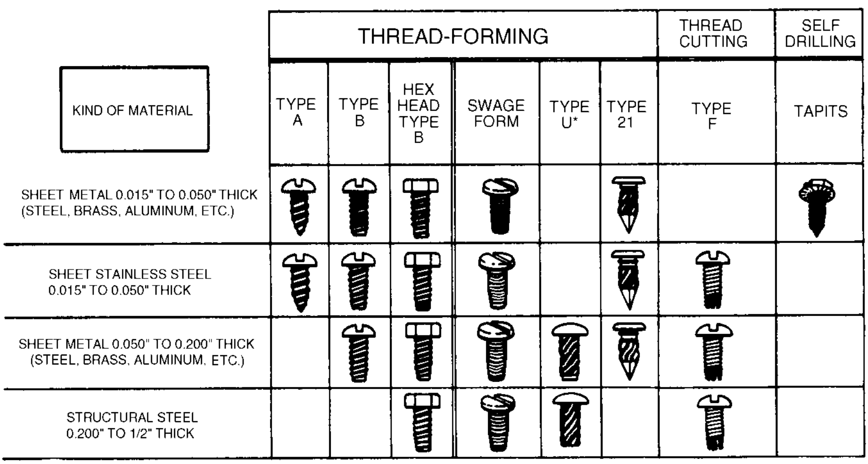


FIGURE 8.12 Tapping screws. NOTE: A blank space does not signify necessarily that the type of screw cannot be used for this purpose; it denotes that the type of self-tapping screw will not generally give the best results in this type of material. (*Parker-Kalon Corp., Emhart Corp., Campbellsville, Ky.*)

TABLE 8.12 Average Diameters of Self-Tapping Screws, in*

Number or size, in	Types AB and B		Type F†	Type U
	Outside	Root	Outside	Outside
No. 4	0.112	0.084	0.110	0.114
No. 6	0.137	0.102	0.136	0.138
No. 8	0.164	0.119	0.161	0.165
No. 10	0.186	0.138	0.187	0.180
No. 12	0.212	0.161	0.213	0.209
No. 14‡ or ¼	0.243	0.189	0.247	0.239‡
5⁄16	0.312	0.240	0.309	0.312
3⁄8§	0.376	0.304	0.371	0.375

* Averages of standard maximum and minimum dimensions adopted under ANSI B18.6.4-1966.

† Type F has threads of machine-screw type approximating the Unified Thread Form (ANSI B1.101960). The figures shown are averages of those for two different thread pitches for each size of screw.

‡ Size No. 14 for Type U.

§ Does not apply to Type AB.

Torsional-strength requirements for self-tapping screws have been standardized under American National Standards Institute B18.6.4, ''Slotted and Recessed Head Tapping Screws and Metallic Drive Screws.'' Safe loads in shear and tension on such screws can vary considerably, depending on type of screw and head, tightening torque, and details of the assembly. When screws are used for structural load-carrying purposes, the user should rely on experience with the particular application, manufacturer's recommendations, or actual tests of the type of assembly involved.

Essential body dimensions of some types of self-tapping screws are given in Table 8.12. Complete details on these and other types, and recommended hole sizes, may be found in ANSI B18.6.4 and in manufacturers' publications.

8.20 SPECIAL FASTENERS FOR COLD-FORMED STEEL

Special fasteners, such as tubular rivets, blind rivets (capable of being driven from one side only), special bolts used for ''blind insertion,'' special studs, lock nuts, and the like, and even metal stitching, which is an outgrowth of the common office stapling device for paper, are used for special applications. When such a fastener is required, refer to manufacturers' catalogs for design information, and base any structural strength attributed to the fastener on the results of carefully made tests or the manufacturer's recommendations.

COLD-FORMED STEEL FLOOR, ROOF, AND WALL CONSTRUCTION

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-

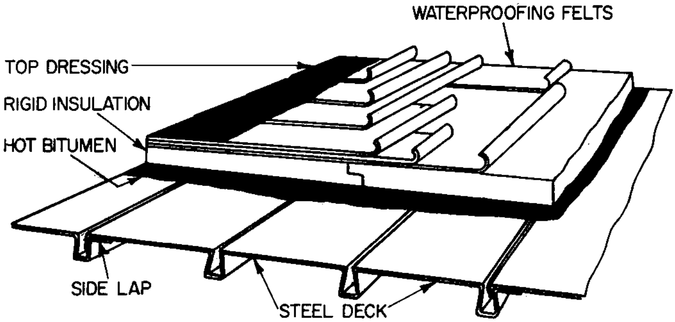


FIGURE 8.13 Roof-deck assembly.

deck assembly is shown in Fig. 8.13. The Steel Deck Institute, P.O. Box 25, Fox River Grove, IL 60021, has developed much useful information on steel roof deck.

8.21 STEEL ROOF DECK

Various types of steel roof deck are available and may be classified in accordance with recommendations of the Steel Deck Institute (SDI). All types consist of long, narrow sections with longitudinal ribs at least 1½ in deep and spaced about 6 in on centers (Fig. 8.14). Other rib dimensions are shown in Fig. 8.14*a* to *c* for some standard styles.

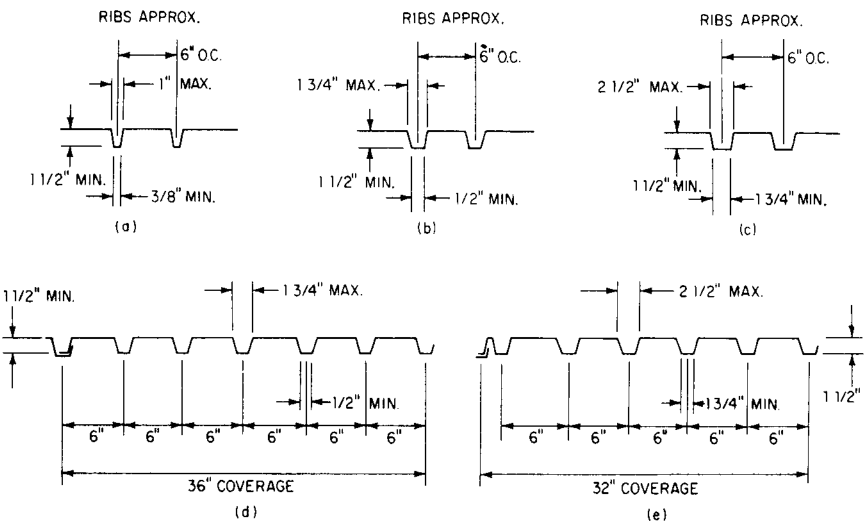


FIGURE 8.14 Typical cold-formed steel roof-deck sections. (*a*) Narrow rib: (*b*) intermediate rib; (*c*) wide rib; (*d*) intermediate rib in 36-in-wide sheets with nested side laps; (*e*) wide rib in 32-in-wide sheets with upstanding seams.

8.21.1 Types of Steel Roof Deck

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. Thickness of steel commonly used is 0.048 or 0.036 in, but most building codes permit 0.030-in-thick steel to be used. Figure 8.14*d* and *e* shows full-width decking 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 steel thicknesses, 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. 8.15, in 24- to 16-ga black and galvanized.

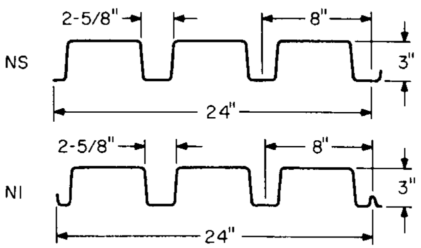


FIGURE 8.15 Cross sections of types NS and NI roof deck for 9- to 15-ft spans.

The weight of the steel roof deck shown in Fig. 8.14 depends 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 (ASTM A611, Grades C, D or E) or galvanized (A653 SQ Grade 33 or higher). Both steels have minimum yield strengths 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. Aluminized steel is another metal-coated steel option.

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

8.21.2 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. 8.14*a* to *c*, and publishes allowable uniform loading tables for narrow-, intermediate-, and wide-rib steel roof deck (Table 8.13, for example). 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 Structural Members" (Art. 8.8). SDI has banned compression flange widths otherwise assumed to be effective. Moreover, SDI "Basic Design Specifications" recommends the following:

Moment and Deflection Coefficients. Where steel roof decks are welded to supports, a moment coefficient of $\frac{1}{10}$ (applied to WL) should be used for three or more spans. Deflection coefficients of 0.0054 and 0.0069 (applied to WL^3/EI) should be used for two span and three span, respectively. All other steel roof-deck installations should be designed as simple spans, with moment and deflection coefficients $\frac{1}{8}$ and $\frac{5}{384}$, respectively. (W = total uniform load, L = span, E = modulus of elasticity, I = moment of inertia.)

TABLE 8.13 Allowable Total (Dead plus Live) Uniform Loads, psf, on Steel Roof Deck*

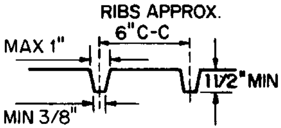
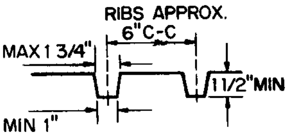
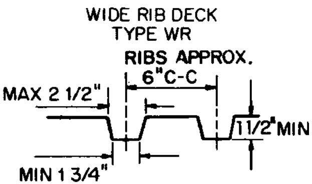
	Deck type	Span condition	Design thickness, in	Span—c to c joists or purlins, ft-in										
				4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0
NARROW RIB DECK TYPE NR	NR 22	SIMPLE	0.0295	73	58	47								
	NR 20	SIMPLE	0.0358	91	72	58	48	40						
	NR18	SIMPLE	0.0474	125	99	80	66	55	47					
	NR 22	2-SPAN	0.0295	80	63	51	42							
	NR 20	2-SPAN	0.0358	97	76	62	51	43						
	NR 18	2-SPAN	0.0474	128	101	82	68	57	48	42				
	NR 22	3 OR MORE	0.0295	100	79	64	53	44						
	NR 20	3 OR MORE	0.0358	121	96	77	64	54	46					
	NR 18	3 OR MORE	0.0474	160	126	102	85	71	61	52	45			
				4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0
INTERMEDIATE RIB DECK TYPE IR	IR 22	SIMPLE	0.0295	84	66	54	44							
	IR 20	SIMPLE	0.0358	104	82	67	55	46						
	IR 18	SIMPLE	0.0474	142	112	91	75	63	54	46	40			
	IR 22	2-SPAN	0.0295	90	71	58	48	40						
	IR 20	2-SPAN	0.0358	110	87	70	58	49	41					
	IR 18	2-SPAN	0.0474	145	114	93	77	64	55	47	41			
	IR 22	3 OR MORE	0.0295	113	89	72	60	50	43					
	IR 20	3 OR MORE	0.0358	137	108	88	72	61	52	45				
	IR 18	3 OR MORE	0.0474	181	143	116	96	81	69	59	52	45	40	

TABLE 8.13 Allowable Total (Dead plus Live) Uniform Loads, psf, on Steel Roof Deck* (Continued)



Deck type	Span condition	Design thickness, in	Span—c to c joists or purlins, ft-in										
			5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0	9-6	10-0
WR 22	SIMPLE	0.0295	90	70	56	46							
WR 20	SIMPLE	0.0358	113	88	70	57	48	40					
WR 18	SIMPLE	0.0474	159	122	96	77	64	54	46	40			
WR 22	2-SPAN	0.0295	96	79	67	57	49	43					
WR 20	2-SPAN	0.0358	123	102	86	73	63	55	48	43			
WR 18	2-SPAN	0.0474	164	136	114	98	84	73	64	57	51	46	41
WR 22	3 OR MORE	0.0295	119	99	83	71	61	53	47	41	36		
WR 20	3 OR MORE	0.0358	153	127	107	91	79	68	58	50	43		
WR 18	3 OR MORE	0.0474	204	169	142	121	105	91	79	67	58	51	

*Load tables were calculated with sectional properties for minimum thicknesses of 0.028, 0.034, and 0.045 in, corresponding respectively to design thickness of 0.0295, 0.0358, and 0.0474 in, exclusive of coating on base metal.

Loads shown in tables are uniformly distributed total (dead plus live) loads, psf. Loads in shaded areas are governed by live-load deflection not in excess of $1/240 \times$ span. The dead load included is 10 psf. All other loads are governed by the allowable flexural stress limit of 20 ksi for a 33-ksi minimum yield point.

Rib-width limitations shown are taken at the theoretical intersection points of flange.

Span length assumes c-to-c spacing of supports. Tabulated loads shall not be increased by assuming clear-span dimensions.

Bending moment formulas used for flexural stress limitation are: for simply supported and two-span decking, $M = wl^2/8$; for decking with three continuous spans or more, $M = wl^2/10$.

¶Deflection formulas for deflection limitation are: For simply supported decking, $\Delta = 5wl^4/384El$; for two- and three-span decking, $\Delta = 0.0054\ wl^4/EI$ and $0.0069\ wl^4/EI$, respectively.

Normal installations covered by these tables do not require midspan fasteners for spans of 5 ft or less.

From "Design Manual for Composite Decks, Form Decks, Roof Decks and Cellular Deck Floor Systems with Electrical Distribution," Steel Deck Institute.

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

Anchorage. Steel roof deck should be anchored to the supporting framework to resist the following 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 uplife forces.

8.21.3 Diaphragm Action of Decks

In addition to their normal function as roof panels under gravity loading, steel roof deck assemblies can be used as shear diaphragms under lateral loads, such as wind and seismic forces. When steel roof deck is used for these purposes, special attention should be paid to connections between panels and attachments of panels to building frames. For design purposes, see SDI "Diaphragm Design Manual."

8.21.4 Details and Accessories of 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 roof-deck manufacturers (Fig. 8.16).

Roof decks are commonly arc welded to structural steel supports with puddle welds at least ¼ in in diameter or with elongated welds of equal perimeter. Electrodes should be selected for amperage adjusted to fuse all layers of steel roof decking to supporting members without creating blowholes around the welds. Welding washers are recommended for thicknesses less than 0.028 in.

Fillet welds at least 1 in long 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 members at intervals not exceeding 18 in. When standard steel roof deck spans 5 ft or more, adjacent sheets should be fastened together at midspan with either welds or screws.

8.21.5 Roof Deck Insulation and Fire Resistance

Although insulation is not ordinarily supplied by the roof-deck manufacturer, it is standard practice to install ¾- or 1-in-thick mineral fiberboard between roof deck and roofing. SDI further recommends that all steel decks be covered with a material of sufficient insulating value to prevent condensation under normal occupancy conditions. Insulation should be adequately attached to the steel deck by means of adhesives or mechanical fasteners. Insulation materials should be protected from the elements at all times during storage and installation.

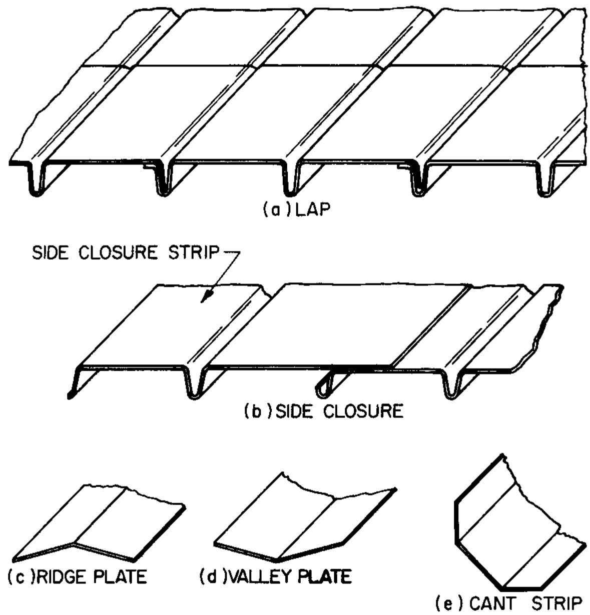


FIGURE 8.16 Roof-deck details.

The UL ''Fire Resistance Directory,'' Underwriter's Laboratories, Inc., 333 Pfingsten Rd., Northbrook, IL 60062, lists fire-resistance ratings for steel roof-deck construction. Some systems with fire ratings up to 2 h are listed in Table 8.14.

*8.22 CELLULAR STEEL FLOOR AND ROOF PANELS**

Several different designs of cellular steel panels and fluted steel panels for floor and roof construction are available. Sections of some of these panels are illustrated in Fig. 8.17.

8.22.1 Cellular-Steel-Floor Raceway System

One form of cellular steel floor assembly with a distribution system for electrical wiring, telephone cables, and data cables is described below and is illustrated in Fig. 8.18. This system is used in many kinds of structures, including massive high-rise buildings for institutional, business, and mercantile occupancies.

The cellular-steel-floor raceway system is basically a profiled steel deck containing wiring raceways and having structural concrete on top. The cellular deck

*Courtesy of R. E. Albrecht, Engineer, H. H. Robertson Company, Ambridge, Pa.

TABLE 8.14 Fire Resistance Ratings for Steel Floor and Roof Assemblies*

Roof construction	Insulation	Underside protection	Authority
		2-h rating†	
Min. 1½-in-deep steel deck on steel joists or steel beams	Min. 1¾-in-thick listed mineral fiberboard	Min. 1¾-in-thick, direct-applied, sprayed vermiculite plaster, UL listed	UL design P711†
Min. 1½-in-deep steel deck on steel joists or steel beams	Min. 1¹⁄₁₆-in-thick listed mineral fiberboard	Min. 1⁹⁄₁₆-in-thick, direct-applied, sprayed fiber protection, UL listed	UL design P818†
Floor construction	**Concrete**	**Underside protection**	**Authority**
		2-h rating‡	
1½-, 2-, or 3-in-deep steel floor units on steel beams	2½-in-thick normal-weight or lightweight concrete	Min. ⅜-in-thick, direct-applied, sprayed vermiculite plaster, UL listed	UL design D739‡
1½-, 2, or 3-in-deep steel floor units on steel beams	2½-in-thick normal-weight or lightweight concrete	Min. ⅜-in-thick, direct-applied, sprayed fiber protection, UL listed	UL design D858†

* Based on "Fire Resistance Directory," 1990, Underwriters Laboratories, Inc., 333 Pfingsten Rd., Northbrook, IL 60062.
† 1½-h and 1-h ratings are also available.
‡ 1-h, 2½-h, 3-h, and 4-h ratings are also available.

consists of closely spaced cellular raceways. These are connected to a main trench header duct with removable cover plate for lay-in wiring. Set on a repetitive module, the cellular raceways are assigned to electrical power, telephone, and data wiring. At prescribed intervals, as close as 2 ft longitudinally and 2 ft transversely over the floor, preset inserts may be provided for access to the wiring and activation workstations. When an insert is activated at a workstation, connections for electrical power, telephone, and data are provided at one outlet. Insert fittings may be flush with the top floor surface or project above it.

This system provides the required fire-resistive barrier between stories of a building. The cellular metal floor units also serve the structural purposes of acting as working platforms and concrete forms during construction and as tensile reinforcement for the concrete floor slab after the building is occupied.

Cellular steel floor raceways have many desirable features including moderately low cost, good flexibility, which contributes to lower life-cycle cost, and minimal

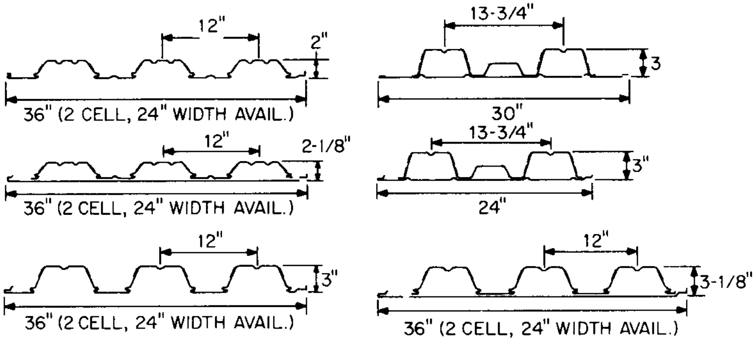


FIGURE 8.17 Composite cellular and fluted steel floor sections. (*Courtesy H. H. Robertson Co., Ambridge, Pa.*)

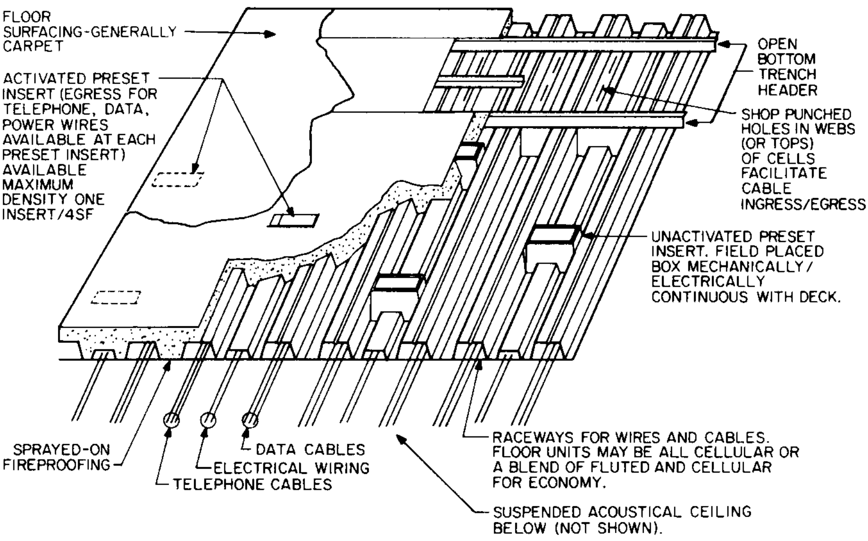


FIGURE 8.18 Composite cellular and fluted steel floor sections. (*Courtesy H. H. Robertson Co., Ambridge, Pa.*)

limitations on placement of outlets. Little or no increase over floor depth required for strictly structural purposes is necessary to accommodate the system.

Wiring may penetrate the floor surface only at outlet fittings. Therefore, if carpet is used, it will have to be cut and a flap peeled back to provide access to the fittings. Use of carpet tiles rather than sheet carpet facilitates access to the preset inserts.

Where service outlets are not required to be as close as 2 ft on centers, a blend of fluted and cellular floor sections may be used. As an example, alternating 3-ft-wide fluted floor deck with 2-ft-wide cellular floor panels 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 available.

8.22.2 Steels Used for Cellular and Fluted Decking

Cellular and fluted floor and roof sections (decking) usually are made of steel 0.030 in or more thick complying with the requirements of ASTM A611, Grades C, D, or E, for uncoated steel or ASTM A653 structural quality, for galvanized steel, with a minimum yield points of 33 ksi. The steel may be either galvanized or painted.

8.22.3 Structural Design of Steel Floor and Roof Panels

Design is usually based on the ‘‘Specification for the Design of Cold-Formed Steel Structural Members,’’ American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036. Structural design of composite floor slabs incorporating sheet-steel floor and roof panels is usually based on ‘‘Standard for the Structural Design of Composite Slabs,’’ ANSI/ASCE 3-91 and ‘‘Standard Practice for Construction and Inspection of Composite Slabs,’’ ANSI/ASCE 9-91, American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191-44001.

Details of design and installation vary with types of panels and manufacturers. In any particular instance, refer to the manufacturer’s recommendations.

8.22.4 Fire Resistance of Cellular and Fluted Steel Decking

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 a considerable number of assemblies are available. (See ‘‘Fire-Resistant Steel-Frame Construction,’’ American Institute of Steel Construction,’’ and ‘‘Fire Resistance Directory,’’ Underwriters Laboratories).

8.23 CORRUGATED SHEETS FOR ROOFING, SIDING, AND DECKING

Although the use of corrugated sheets of thin steel for roofing and siding leaves something to be desired for weathertightness and appearance, they are used for barns and similar buildings for some protection against weather elements. They are cheap, easy to install on a wood frame, and last for many years if galvanized. (Corrugated steel sheets are the oldest type of cold-formed steel structural members. They have been used since 1784, when Henry Cort introduced sheet rolling in England.)

The commonest form of corrugated sheet, the arc-and-tangent type, has the basic cross section shown in Fig. 8.19*a*. Its section properties are readily calculated with factors taken from Fig. 8.19*b* to *f* and substituted in the following formulas.

The area, in^2, of the corrugated sheet may be determined from

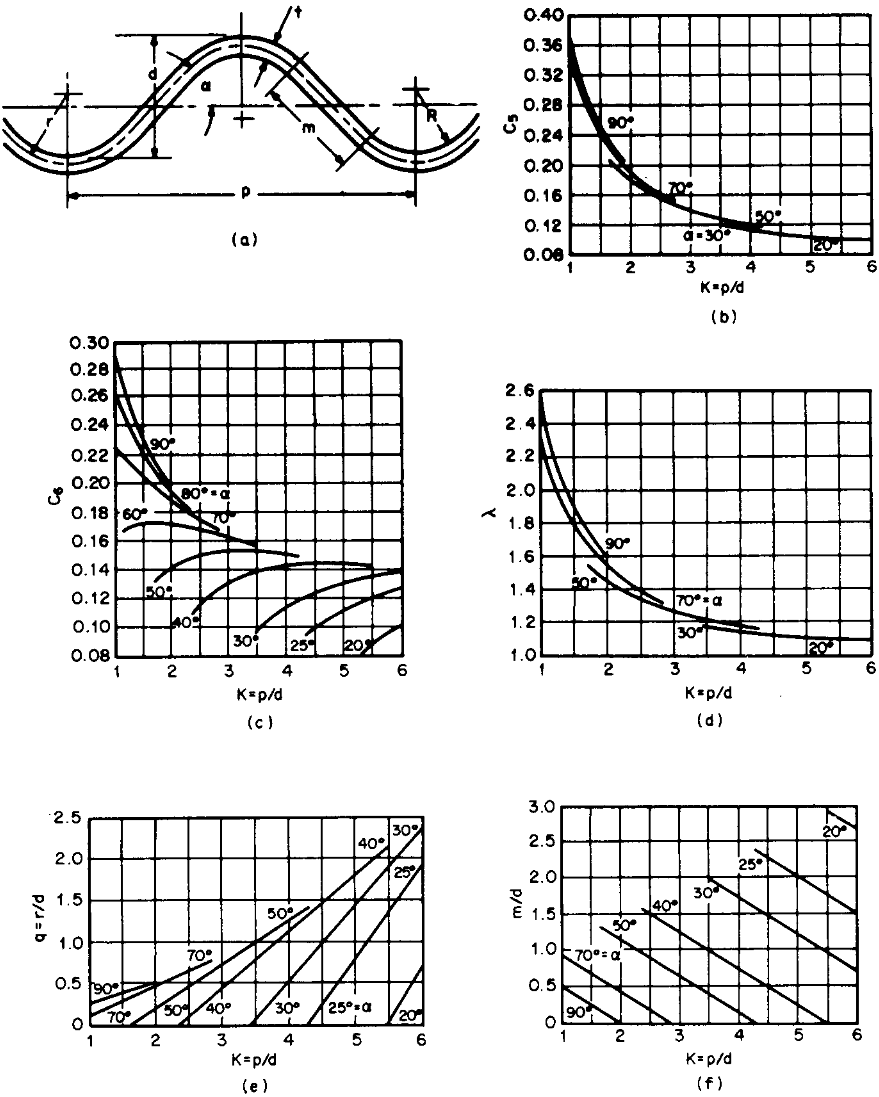


FIGURE 8.19 Factors for determining section properties of the arc-and-tangent type of corrugated steel sheet shown in (*a*).

$$A = \lambda bt \tag{8.49}$$

where b = width of sheet, in

t = sheet thickness, in

$$\lambda = \frac{(2/K + \alpha)\sin\alpha + (1 - 2\alpha/K)\cos\alpha - 1}{1 - \cos\alpha} \qquad \text{(See Fig. 8.19}d\text{)}$$

K = pitch-depth ratio of a corrugation = p/d
p = pitch, in, of corrugation
d = depth, in, of corrugation
α = tangent angle, radians, or angle of web with respect to the neutral axis of the sheet cross section

The moment of inertia, in^4, of the corrugated sheet may be obtained from

$$I = C_5 bt^3 + C_6 bd^2 t \tag{8.50}$$

where $C_5 = \dfrac{q(6\alpha + \sin 2\alpha - 8 \sin \alpha) + 4 \sin \alpha + K \cos \alpha}{12K}$ (See Fig. 8.19*b*)

$$C_6 = \frac{1}{K}\left[q^3\left(6\alpha + \sin 2\alpha - 8 \sin \alpha - \frac{4}{3} \tan^3 \alpha \sin^2 \alpha\right) + q^2(4 \sin \alpha + K \tan^3 \alpha \sin \alpha - 4\alpha) + q\left(\alpha - \frac{1}{4} K^2 \tan^3 \alpha\right) + \frac{K^3 \tan^2 \alpha}{48 \cos \alpha}\right]$$ (See Fig. 8.19*c*)

$$q = \frac{r}{d} = \frac{K \tan \alpha - 2}{4(\sec \alpha - 1)}$$ (See Fig. 8.19*e*)

The section modulus of the corrugated sheet may be computed from

$$S = \frac{2I}{d + t} \tag{8.51}$$

The radius of gyration, in, is given by

$$\rho = \sqrt{\frac{I}{A}} \tag{8.52}$$

and the tangent length-depth ratio is

$$\frac{m}{d} = \frac{\sin \alpha}{1 - \cos \alpha} - \frac{K}{2} \tag{8.53}$$

(See Fig. 8.19*f*.)

Example—Corrugated Sheet Properties. Consider a corrugated sheet with a 6-in pitch, 2-in depth, inside radius R of 1⅛ in, and thickness t of 0.135 in. The mean radius r is then $1.125 + 0.135/2 = 1.192$ in; $q = r/d = 1.192/2 = 0.596$ in, and $K = p/d = 6/2 = 3$. From Fig. 8.19*e*, angle α is found to be nearly 45°. For $p/d = 3$ and $\alpha = 45°$, Fig. 8.19*b*, *c*, *d*, and *f* indicate that $C_5 = 0.14$, $C_6 = 0.145$ $\lambda = 1.24$, and $m/d = 0.93$. Section properties per inch of corrugated width are then computed as follows:

From Eq. (8.49),

$$A = 1.24 \times 1 \times 0.135 = 0.167 \text{ in}^2$$

From Eq. (8.50),

$$I = 0.14 \times 1(0.135)^3 + 0.145 \times 1(2)^2 0.135 = 0.0786 \text{ in}^4$$

From Eq. (8.51),

$$S = \frac{2 \times 0.0786}{2 + 0.135} = 0.0736 \text{ in}^3$$

From Eq. (8.52),

$$\rho = \sqrt{\frac{0.0786}{0.167}} = 0.686 \text{ in}$$

and from Eq. (8.53),

$$m = 0.93 \times 2 = 1.86 \text{ in}$$

I, *S*, and *A* for corrugated sheets with widths *b* are obtained by multiplying the per-inch values by *b*.

Unit Stresses. The allowable unit bending stress F_r, ksi, at extreme fibers of corrugated sections of carbon or low-alloy steel may be taken as $0.6F_y$, if r/t does not exceed $1650/F_y$. For $1650/F_y \leq r/t < 6500/F_y$,

$$F_r = 331t/r + 0.399F_y \tag{8.54}$$

where F_y = specified minimum yield point of the steel, ksi.

Section properties of corrugated sheets with cross sections composed of flat elements may be computed with the linear method given in Art. 8.4, by combining properties of the various elements as given in Table 8.4. (See also "Sectional Properties of Corrugated Sheets Determined by Formula," *Civil Engineering,* February 1954.)

8.24 LIGHTWEIGHT STEEL METRIC SHEETING

Metric sheeting, the cross section of which is shown in Fig. 8.20, has a corrugation-like conformation with locking side edges. It has a laying width of 500 mm or 0.5 m (19⅔ in), and is available in thicknesses of 5, 7, 8, 10, and 12 ga. Sheets are installed vertically in soil with edges of successive units interlocking. For additional corrosion protection, metric sheeting may be ordered galvanized after continuous cold forming in lengths of 4 to 40 ft.

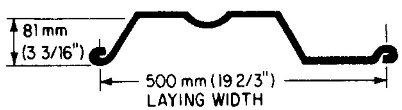


FIGURE 8.20 Steel metric sheeting.

Applications include checkdams, core walls, wingwalls, trench walls, excavations, low retaining walls, ditch checks, jetties and lagoon baffles. The sheeting often can be put into soft ground with the aid of a backhoe, although for harder subgrades, conventional drop, vibratory, or diesel hammers applied to a light driving head make emplacement easier. The tight metal-to-metal interlock at the edges of metric sheeting contains soil and controls water movement. Table 8.15 lists its structural properties.

TABLE 8.15 Physical Properties of Metric Sheeting*

Thickness		Weight		Section properties			
				Section modulus, in^3		Moment of inertia, in^4	
Gage	in	lb/lin ft of pile	lb/ft^2 of wall	Per section	Per ft	Per section	Per ft
5	0.2092	19.1	11.6	5.50	3.36	9.40	5.73
7	0.1793	16.4	10.0	4.71	2.87	7.80	4.76
8	0.1644	15.2	9.3	4.35	2.65	7.36	4.49
10	0.1345	12.5	7.6	3.60	2.20	6.01	3.67
12	0.1046	9.9	6.0	2.80	1.71	4.68	2.85

*Based on ''CONTECH Metric Sheeting,'' 1990, CONTECH Construction Products Inc., Middletown, Ohio.

Metric sheeting should not be confused with steel sheetpiling, which is a heavier hot-rolled steel product used for major construction projects, including breakwaters, bulkheads, cofferdams, and docks. Metric sheeting is nevertheless an economical product suitable for many less-demanding applications for both temporary and permanent uses. An advantage for contractors is that it can be withdrawn and reused on another job.

More information on lightweight steel construction is available from CONTECH Construction Products, 1001 Grove Street, Middletown, OH 45044

8.25 STAINLESS STEEL STRUCTURAL DESIGN

Cold-formed, stainless-steel structural members require different design approaches from those presented in Arts. 8.1 through 8.13 for cold-formed structural members of carbon and low-alloy steels. An exception is the stainless steels of the ferritic type that are largely alloyed with chromium and exhibit a sharp-yielding stress-strain curve. The austenitic types of stainless steel, incorporating substantial amounts of nickel as well as chromium, have stress-strain curves that are rounded, do not show sharp yield points, and exhibit proportional limits that are quite low. Because of excellent corrosion resistance, stainless steels are suitable for exterior wall panels and exterior members of buildings as well as for other applications subject to corrosive environments.

The ''Specification for the Design of Cold-Formed Stainless Steel Structural Members,'' ANSI-ASCE 8-90, American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191-4400, presents treatments paralleling those of Arts. 8.1 through 8.13, except the primary emphasis is on the load resistance factor design (LRFD) method. The allowable strength design (ASD) method, however, is also mentioned. For detailed information on austenitic grades of stainless steel, see ASTM A666, ''Austenitic Stainless Steel, Sheet, Strip, Plate and Flat Bar for Structural Applications.''

(W. W. Yu, ''Cold-Formed Steel Design,'' 3rd ed., John Wiley and Sons, Inc., New York.)

PREENGINEERED STEEL BUILDINGS

Preengineered steel buildings may be selected from catalogs. They are fully designed by a manufacturer, who supplies them with all structural and covering material, and all fasteners.

8.26 CHARACTERISTICS OF PREENGINEERED STEEL BUILDINGS

These structures eliminate the need for engineers and architects to design and detail both the structures and the required accessories and openings, as would be done for conventional buildings with components from many individual suppliers. Available with floor areas of up to 1 million ft^2, preengineered buildings readily meet requirements for single-story structures, especially for industrial plants and commercial buildings (Fig. 8.21).

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

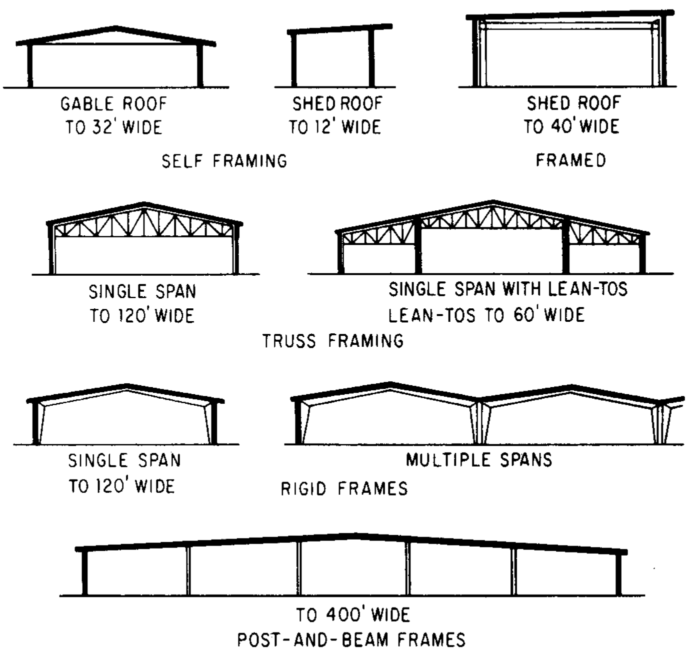


FIGURE 8.21 Principal framing systems for preengineered steel buildings.

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

Preengineered buildings make extensive use of cold-formed steel structural members. These lend themselves to mass production, and their designs can be more accurately fitted to the specific structural requirements. 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 of standard hot-rolled shape, with more weight than required. Also, because this purlin is used on many buildings, the quantity justifies investment in automated equipment for forming and punching. This equipment is nevertheless flexible enough to permit a change of thickness or depth of section to produce similar purlins for other buildings.

The engineers designing a line of preengineered buildings can, because of the repeated use of the design, justify spending additional design time refining and optimizing the design. Most preengineered buildings are designed with the aid of computers. Their programs are specifically tailored to produce systems of such buildings. A rerun of a design to eliminate a few pounds of steel is justified, since the design will be used many times during the life of that building model.

8.27 STRUCTURAL DESIGN OF PREENGINEERED BUILDINGS

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

Standards in ‘‘Metal Building Systems,’’ Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, OH 44115 discuss methods of load application and maximum loading, for use where load requirements are not established by local building codes. Other appropriate design specifications include:

Structural Steel. ‘‘Specification for Structural Steel Buildings,’’ American Institute of Steel Construction, One East Wacker Dr., Chicago, IL 60601.

Cold-Formed Steel. ‘‘Specification for the Design of Cold-Formed Steel Structural Members,’’ American Iron and Steel Institute, 1101 17th St., NW, Washington, DC 20036.

Welding. ‘‘Structural Welding Code,’’ D1.3 and ‘‘Specification for Welding Sheet Steel in Structures,’’ D1.3, American Welding Society, 550 NW LeJeune Rd., Miami, FL 33152.

The Systems Building Association promotes marketing of metal buildings and is located at 28 Lowery Dr., P.O. Box 117, West Milton, OH 45383.

OPEN-WEB STEEL JOISTS

The first steel joist was produced in 1923 and consisted of solid round bars for top and bottom chords and a web formed from a single continuous bent bar, thus simulating a Warren truss. The Steel Joist Institute (SJI) was organized to promote sales of such joists in 1925 and has sponsored further research and development since then.

8.28 DESIGN OF OPEN-WEB STEEL JOISTS

Currently, open-web steel joists are still relatively small, parallel-chord trusses, but *hot-rolled steel shapes* usually make up the components. (For a time, *cold-formed steel shapes* were preferred for chords to utilize higher working stresses available in cold-formed sections of ordinary carbon-steel grades. Unfavorable fabrication costs, however, led to a change to the hot-rolled steel chords.)

Joists are suitable for direct support of floors and roofs of buildings, when designed according to SJI ‘‘Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders,’’ Steel Joist Institute, 3127 10th Ave., North Ext., Myrtle Beach, SC 29577. Moreover, since 1972, the American Institute of Steel

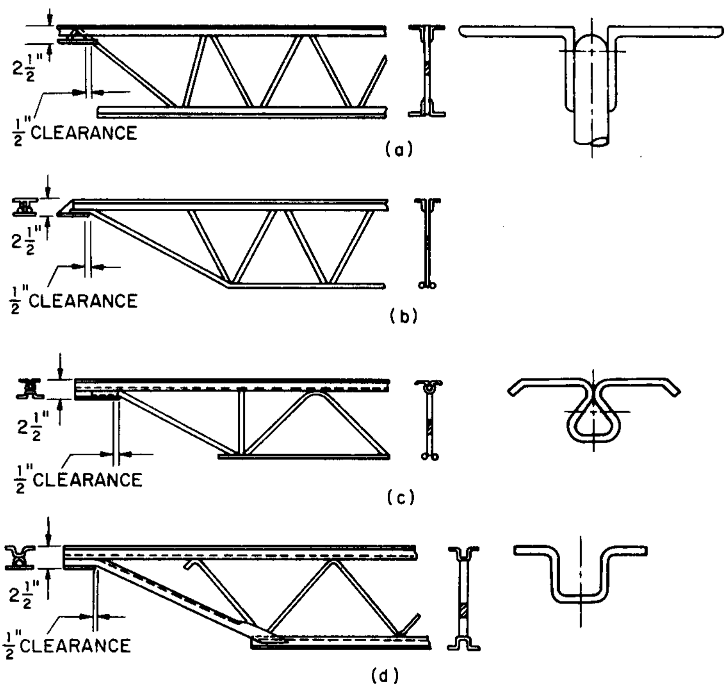


FIGURE 8.22 Some examples of open-web steel joists.

Construction (AISC) has cooperated with SJI in producing an industry standard for steel joist design. However, exact forms of chords and webs, and their methods of manufacture, then as now, have continued to be in the provenance of SJI members. Figure 8.22 shows a number of proprietary steel joists designs.

Joists are designed primarily for use under uniform distributed loading with substantially uniform spacing of joists, as depicted in Fig. 8.23. They can carry concentrated loads, however, especially of loads are applied at joist panel points. Partitions running crosswise to joists usually can be considered as being distributed by the concrete floor slabs, thus avoiding local bending of joist top chords. Even so, joists must always be size-selected to resist the bending moments, shears, and reactions of all loads, uniform or otherwise. So joist loadings given in tables for uniform loading should be used with caution and modified when necessary.

One cardinal rule is that the clear span of a joist should never exceed 24 times its depth. Another rule is that deflections should not exceed $\frac{1}{360}$ of the joist span for floors and roofs to which plaster ceilings are attached or $\frac{1}{240}$ of the span for all other cases.

SJI publishes loading tables for K-series (short span). LH-series (long span), and DLH-series (deep long span) joist girders. The K-series joists are available in depths

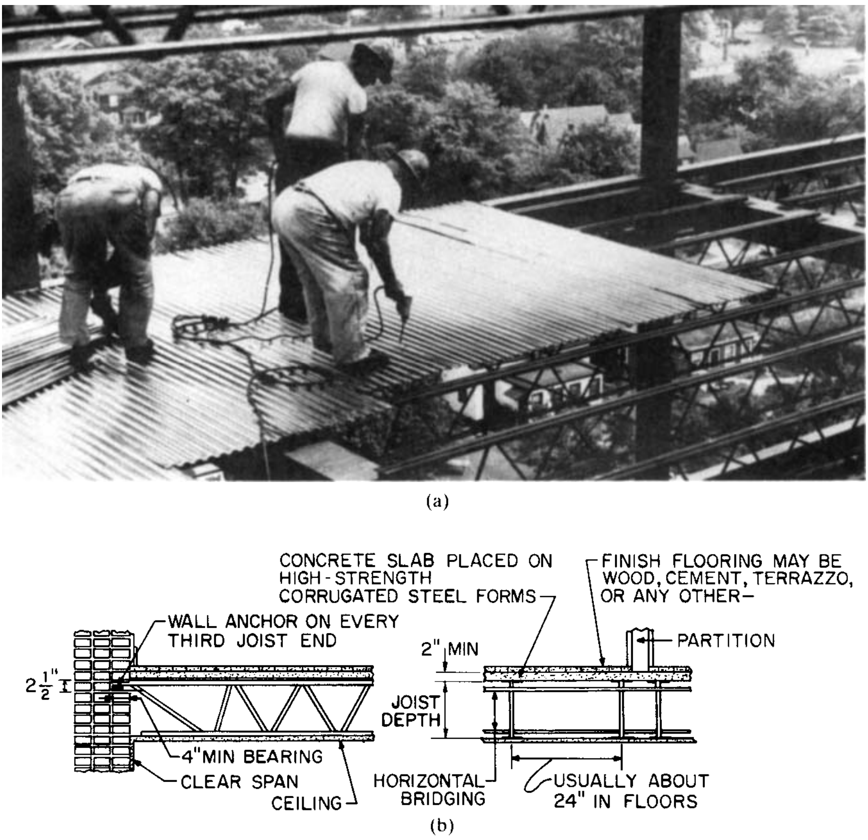


FIGURE 8.23 Some examples of open-web steel-joist floor construction.

of 8 to 30 in and spans of 8 to 60 ft in 13 different chord weights to sustain uniform loads along the span as high as 550 lb/ft. LH-series joists are available in depths from 18 to 48 in and spans of 25 to 96 ft in six different chord weights capable of supporting total loads of 12,000 to 57,600 lb. DLH-series joists are available in depths of 52 to 72 in and spans from 89 to 144 ft in 17 different chord weights with total-load capacities of 26,700 to 80,200 lb. Load capacities in the foregoing were based on a maximum allowable tensile strength of 30 ksi, which calls for high-strength, low-alloy steel having a specified minimum yield strength of 50 ksi or cold-formed steel having the same yield strength.

Fire resistance ratings of 1, 1½, 2, and 3 hours are possible using concrete floors above decks as thin as 2 in and as thick as 3½ 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.

8.29 CONSTRUCTION DETAILS FOR OPEN-WEB STEEL JOISTS

It is essential that bridging be installed between joists as soon as possible after the joists are placed and before application of any construction loads. The most commonly used type of bridging is continuous horizontal bracing composed of steel rods fastened perpendicular to the top and bottom chords of the joists. Diagonal bridging, however, is also permitted. The attachment of the floor or roof is expected to provide additional support of the joists against lateral buckling.

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

Plastered ceilings attached directly to regular open-web steel joists are usually supported at underslung ends by means of *ceiling extensions,* as shown in Fig. 8.24*a*. *Extended ends,* as shown in Fig. 8.24*b*, allow floor and roof treatments beyond outer supporting stringers.

Relatively small openings between joists may usually be framed with angle, channel, and Z-shaped headers supported on adjacent joists. Larger openings should be framed in structural steel. Headers should preferably be located so that they are supported at trimmer-joist panel points.

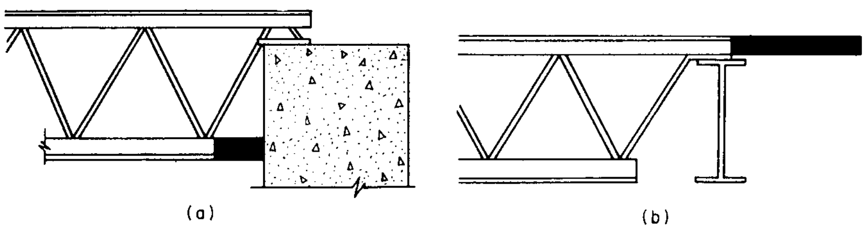


FIGURE 8.24 Open-web steel joist with (*a*) ceiling extension. (*b*) extended end. (*Courtesy of Steel Joist Institute.*)