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Cold-Formed Steel Structures

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7.1 Introduction

Cold-formed steel members as shown in Figure 7.1 are widely used in building construction, bridge construction, storage racks, highway products, drainage facilities, grain bins, transmission towers, car bodies, railway coaches, and various types of equipment. These sections are cold-formed from carbon or low alloy steel sheet, strip, plate, or flat bar in cold-rolling machines or by press brake or bending brake operations. The thicknesses of such members usually range from 0.0149 in. (0.378 mm) to about 1/4 in. (6.35 mm) even though steel plates and bars as thick as 1 in. (25.4 mm) can be cold-formed into structural shapes.

FIGURE 7.1: Various shapes of cold-formed steel sections. (From Yu, W.W. 1991. *Cold-Formed Steel Design,* John Wiley & Sons, New York. With permission.)

The use of cold-formed steel members in building construction began in the 1850s in both the U.S. and Great Britain. However, such steel members were not widely used in buildings in the U.S. until the 1940s. At the present time, cold-formed steel members are widely used as construction materials worldwide.

Compared with other materials such as timber and concrete, cold-formed steel members can offer the following advantages: (1) lightness, (2) high strength and stiffness, (3) ease of prefabrication and mass production, (4) fast and easy erection and installation, and (5) economy in transportation and handling, just to name a few.

From the structural design point of view, cold-formed steel members can be classified into two major types: (1) individual structural framing members (Figure 7.2) and (2) panels and decks (Figure 7.3).

In view of the fact that the major function of the individual framing members is to carry load, structural strength and stiffness are the main considerations in design. The sections shown in Figure 7.2 can be used as primary framing members in buildings up to four or five stories in height. In tall multistory buildings, the main framing is typically of heavy hot-rolled shapes and the secondary elements such as wall studs, joists, decks, or panels may be of cold-formed steel members. In this case, the heavy hot-rolled steel shapes and the cold-formed steel sections supplement each other.

The cold-formed steel sections shown in Figure 7.3 are generally used for roof decks, floor decks, wall panels, and siding material in buildings. Steel decks not only provide structural strength to carry loads, but they also provide a surface on which flooring, roofing, or concrete fill can be applied as shown in Figure 7.4. They can also provide space for electrical conduits. The cells of cellular panels

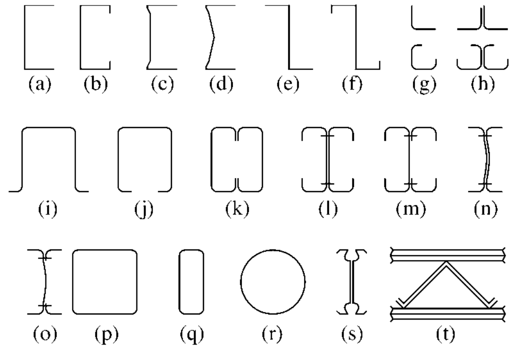


FIGURE 7.2: Cold-formed steel sections used for structural framing. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

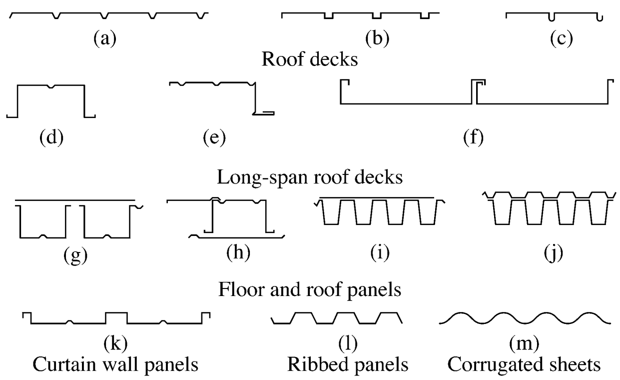


FIGURE 7.3: Decks, panels, and corrugated sheets. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

can also be used as ducts for heating and air conditioning. For composite slabs, steel decks are used not only as formwork during construction, but also as reinforcement of the composite system after the concrete hardens. In addition, load-carrying panels and decks not only withstand loads normal to their surface, but they can also act as shear diaphragms to resist forces in their own planes if they are adequately interconnected to each other and to supporting members.

During recent years, cold-formed steel sections have been widely used in residential construction and pre-engineered metal buildings for industrial, commercial, and agricultural applications. Metal building systems are also used for community facilities such as recreation buildings, schools, and churches. For additional information on cold-formed steel structures, see Yu [49], Rhodes [36], and Hancock [28].

7.2 Design Standards

Design standards and recommendations are now available in Australia [39], Austria [31], Canada [19], Czechoslovakia [21], Finland [26], France [20], Germany [23], India [30], Japan [14], The Netherlands [27], New Zealand [40], The People's Republic of China [34], The Republic of South Africa [38], Sweden [44], Romania [37], U.K. [17], U.S. [7], USSR [41], and elsewhere. Since 1975, the European Convention for Constructional Steelwork [24] has prepared several documents for the design and

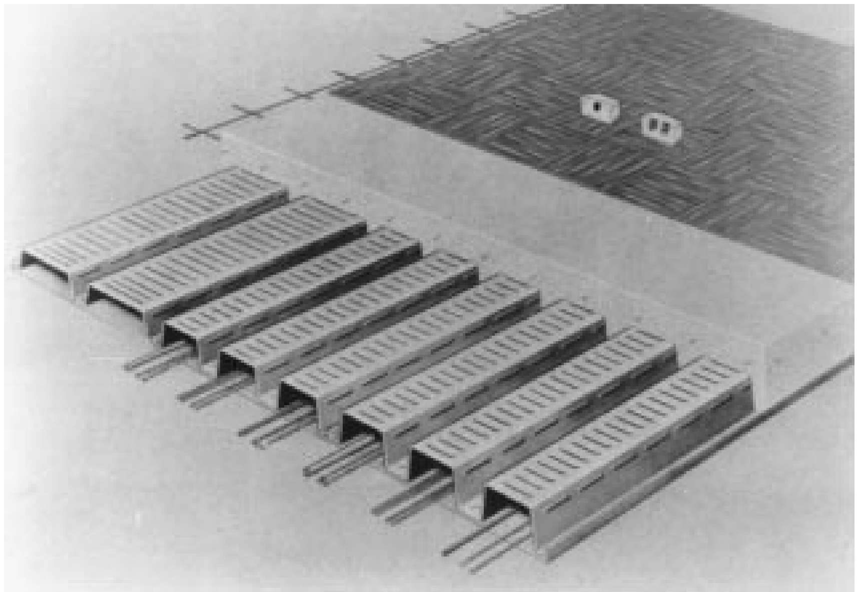


FIGURE 7.4: Cellular floor decks. (From Yu, W.W. 1991. *Cold-Formed Steel Design,* John Wiley & Sons, New York. With permission.)

testing of cold-formed sheet steel used in buildings. In 1989, Eurocode 3 provided design information for cold-formed steel members.

This chapter presents discussions on the design of cold-formed steel structural members for use in buildings. It is mainly based on the current AISI combined specification [7] for allowable stress design (ASD) and load and resistance factor design (LRFD). It should be noted that in addition to the AISI specification, in the U.S., many trade associations and professional organizations have issued special design requirements for using cold-formed steel members as floor and roof decks [42], roof trusses [6], open web steel joists [43], transmission poles [10], storage racks [35], shear diaphragms [7, 32], composite slabs [11], metal buildings [33], light framing systems [15], guardrails, structural supports for highway signs, luminaries, and traffic signals [4], automotive structural components [5], and others. For the design of cold-formed stainless steel structural members, see ASCE Standard 8-90 [12].

7.3 Design Bases

For cold-formed steel design, two design approaches are being used. They are: (1) ASD and (2) LRFD. Both methods are briefly discussed in this section.

7.3.1 Allowable Stress Design (ASD)

In the ASD approach, the required strengths (moments, axial forces, and shear forces) in structural members are computed by accepted methods of structural analysis for the specified nominal or working loads for all applicable load combinations listed below [7].

1. D
2. $D + L + (L_r \text{ or } S \text{ or } R_r)$

3. $D + (W \text{ or } E)$
4. $D + L + (L_r \text{ or } S \text{ or } R_r) + (W \text{ or } E)$

where

D = dead load
E = earthquake load
L = live load due to intended use and occupancy
L_r = roof live load
R_r = rain load, except for ponding
S = snow load
W = wind load

In addition, due consideration should also be given to the loads due to (1) fluids with well-defined pressure and maximum heights, (2) weight and lateral pressure of soil and water in soil, (3) ponding, and (4) contraction or expansion resulting from temperature, shrinkage, moisture changes, creep in component materials, movement due to different settlement, or combinations thereof.

The required strengths should not exceed the allowable design strengths permitted by the applicable design standard. The allowable design strength is determined by dividing the nominal strength by a safety factor as follows:

$$R_a = R_n / \Omega \tag{7.1}$$

where

R_a = allowable design strength
R_n = nominal strength
Ω = safety factor

For the design of cold-formed steel structural members using the AISI ASD method [7], the safety factors are given in Table 7.1.

When wind or earthquake loads act in combination with dead and/or live loads, it has been a general practice to permit the allowable design strength to be increased by a factor of one-third because the action of wind or earthquake on a structure is highly localized and of very short duration. This can also be accomplished by permitting a 25% reduction in the combined load effects without the increase of the allowable design strength.

7.3.2 Limit States Design or Load and Resistance Factor Design (LRFD)

Two types of limit states are considered in the LRFD method. They are: (1) the limit state of strength required to resist the extreme loads during the life of the structure and (2) the limit state of serviceability for a structure to perform its intended function.

For the limit state of strength, the general format of the LRFD method is expressed by the following equation:

$$\Sigma \gamma_i Q_i \leq \phi R_n \tag{7.2}$$

where

$\Sigma \gamma_i Q_i$ = required strength
ϕR_n = design strength
γ_i = load factors
Q_i = load effects
ϕ = resistance factor
R_n = nominal strength

The load factors and load combinations are specified in various standards. According to the AISI Specification [7], the following load factors and load combinations are used for cold-formed steel design:

TABLE 7.1 Safety Factors, Ω, and Resistance Factors, ϕ, used in the AISI Specification [7]

Type of strength	ASD safety factor, Ω	LRFD resistance factor, ϕ
(a) Stiffeners		
Transverse stiffeners	2.00	0.85
Shear stiffeners[a]	1.67	0.90
(b) Tension members (see also bolted connections)	1.67	0.95
(c) Flexural members		
Bending strength		
For sections with stiffened or partially stiffened compression flanges	1.67	0.95
For sections with unstiffened compression flanges	1.67	0.90
Laterally unbraced beams	1.67	0.90
Beams having one flange through-fastened to deck or sheathing (C- or Z-sections)	1.67	0.90
Beams having one flange fastened to a standing seam roof system	1.67	0.90
Web design		
Shear strength[a]	1.67	0.90
Web crippling		
For single unreinforced webs	1.85	0.75
For I-sections	2.00	0.80
For two nested Z-sections	1.80	0.85
(d) Concentrically loaded compression members	1.80	0.85
(e) Combined axial load and bending		
For tension	1.67	0.95
For compression	1.80	0.85
For bending	1.67	0.90-0.95
(f) Cylindrical tubular members		
Bending strength	1.67	0.95
Axial compression	1.80	0.85
(g) Wall studs and wall assemblies		
Wall studs in compression	1.80	0.85
Wall studs in bending	1.67	0.90-0.95
(h) Diaphragm construction	2.00-3.00	0.50-0.65
(i) Welded connections		
Groove welds		
Tension or compression	2.50	0.90
Shear (welds)	2.50	0.80
Shear (base metal)	2.50	0.90
Arc spot welds		
Welds	2.50	0.60
Connected part	2.50	0.50-0.60
Minimum edge distance	2.00-2.22	0.60-0.70
Tension	2.50	0.60
Arc seam welds		
Welds	2.50	0.60
Connected part	2.50	0.60
Fillet welds		
Longitudinal loading (connected part)	2.50	0.55-0.60
Transverse loading (connected part)	2.50	0.60
Welds	2.50	0.60
Flare groove welds		
Transverse loading (connected part)	2.50	0.55
Longitudinal loading (connected part)	2.50	0.55
Welds	2.50	0.60
Resistance Welds	2.50	0.65
(j) Bolted connections		
Minimum spacing and edge distance	2.00-2.22	0.60-0.70
Tension strength on net section		
With washers		
Double shear connection	2.00	0.65
Single shear connection	2.22	0.55
Without washers	2.22	0.65
Bearing strength	2.22	0.55-0.70
Shear strength of bolts	2.40	0.65
Tensile strength of bolts	2.00-2.25	0.75
(k) Screw connections	3.00	0.50
(l) Shear rupture	2.00	0.75
(m) Connections to other materials (Bearing)	2.50	0.60

[a] When $h/t \leq 0.96\sqrt{Ek_v/F_y}$, $\Omega = 1.50$, $\phi = 1.0$

1. $1.4D + L$
2. $1.2D + 1.6L + 0.5(L_r$ or S or $R_r)$
3. $1.2D + 1.6(L_r$ or S or $R_r) + (0.5L$ or $0.8W)$
4. $1.2D + 1.3W + 0.5L + 0.5(L_r$ or S or $R_r)$
5. $1.2D + 1.5E + 0.5L + 0.2S$
6. $0.9D - (1.3W$ or $1.5E)$

All symbols were defined previously.
Exceptions:

1. The load factor for E in combinations (5) and (6) should be equal to 1.0 when the seismic load model specified by the applicable code or specification is limit state based.
2. The load factor for L in combinations (3), (4), and (5) should be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.
3. For wind load on individual purlins, girts, wall panels, and roof decks, multiply the load factor for W by 0.9.
4. The load factor for L_r in combination (3) should be equal to 1.4 in lieu of 1.6 when the roof live load is due to the presence of workmen and materials during repair operations.

In addition, the following LRFD criteria apply to roof and floor composite construction using cold-formed steel:

$$1.2D_s + 1.6C_w + 1.4C$$

where

D_s = weight of steel deck
C_w = weight of wet concrete during construction
C = construction load, including equipment, workmen, and formwork, but excluding the weight of the wet concrete.

Table 7.1 lists the ϕ factors, which are used for the AISI LRFD method for the design of cold-formed steel members and connections [7]. It should be noted that different load factors and resistance factors may be used in different standards. These factors are selected for the specific nominal strength equations adopted by the given standard or specification.

7.4 Materials and Mechanical Properties

In the AISI Specification [7], 14 different steels are presently listed for the design of cold-formed steel members. Table 7.2 lists steel designations, ASTM designations, yield points, tensile strengths, and elongations for these steels.

From a structural standpoint, the most important properties of steel are as follows:

1. Yield point or yield strength, F_y
2. Tensile strength, F_u
3. Stress-strain relationship
4. Modulus of elasticity, tangent modulus, and shear modulus
5. Ductility
6. Weldability
7. Fatigue strength

TABLE 7.2 Mechanical Properties of Steels Referred to in the AISI 1996 Specification

Steel designation	ASTM designation	Yield point, F_y (ksi)	Tensile strength, F_u (ksi)	Elongation (%) In 2-in. gage length	Elongation (%) In 8-in. gage length
Structural steel	A36	36	58-80	23	—
High-strength low-alloy structural steel	A242 (3/4 in. and under)	50	70	—	18
	(3/4 in. to 1-1/2 in.)	46	67	21	18
Low and intermediate tensile strength carbon plates, shapes and bars	A283 Gr. A	24	45-60	30	27
	B	27	50-65	28	25
	C	30	55-75	25	22
	D	33	60-80	23	20
Cold-formed welded and seamless carbon steel structural tubing in rounds and shapes	A500 Round tubing				
	A	33	45	25	—
	B	42	58	23	—
	C	46	62	21	—
	D	36	58	23	—
	Shaped tubing				
	A	39	45	25	—
	B	46	58	23	—
	C	50	62	21	—
	D	36	58	23	—
Structural steel with 42 ksi minimum yield point	A529 Gr. 42	42	60-85	—	19
	50	50	70-100	—	18
Hot-rolled carbon steel sheets and strips of structural quality	A570 Gr. 30	30	49	21-25	—
	33	33	52	18-23	—
	36	36	53	17-22	—
	40	40	55	15-21	—
	45	45	60	13-19	—
	50	50	65	11-17	—
High-strength low-alloy columbium-vanadium steels of structural quality	A572 Gr. 42	42	60	24	20
	50	50	65	21	18
	60	60	75	18	16
	65	65	80	17	15
High-strength low-alloy structural steel with 50 ksi minimum yield point	A588	50	70	21	18
Hot-rolled and cold-rolled high-strength low-alloy steel sheet and strip with improved corrosion resistance	A606 Hot-rolled as rolled coils; annealed, or normalized; and cold-rolled	45	65	22	—
	Hot-rolled as rolled cut lengths	50	70	22	—
Hot-rolled and cold-rolled high-strength low-alloy columbium and/or vanadium steel sheet and strip	A607 Gr. 45	45	60 (55)	Hot-rolled 23-25	
				Cold-rolled 22	
	50	50	65 (60)	Hot-rolled 20-22	—
				Cold-rolled 20	—
	55	55	70 (65)	Hot-rolled 18-20	—
				Cold-rolled 18	—
	60	60	75 (70)	Hot-rolled 16-18	—
				Cold-rolled 16	—
	65	65	80 (75)	Hot-rolled 14-16	—
				Cold-rolled 15	—
	70	70	85 (80)	Hot-rolled 12-14	—
				Cold-rolled 14	—
Cold-rolled carbon structural steel sheet	A611 Gr. A	25	42	26	—
	B	30	45	24	—
	C	33	48	22	—
	D	40	52	20	—
	E	80	82	—	—

TABLE 7.2 Mechanical Properties of Steels Referred to in the AISI 1996 Specification *(continued)*

Steel designation	ASTM designation	Yield point, F_y (ksi)	Tensile strength, F_u (ksi)	Elongation (%) In 2-in. gage length	Elongation (%) In 8-in. gage length
Zinc-coated steel sheets of structural quality	A653 SQ Gr. 33	33	45	20	—
	37	37	52	18	—
	40	40	55	16	—
	50 (class 1)	50	65	12	—
	50 (class 3)	50	70	12	—
	80	80	82	—	—
	HSLA Gr. 50	50	60	20	—
	60	60	70	16	—
	70	70	80	12(14)	—
	80	80	90	10(12)	—
Hot-rolled high-strength low-alloy steel sheets and strip with improved formability	A715 Gr. 50	50	60	22-24	—
	60	60	70	20-22	—
	70	70	80	18	—
	80	80	90	14	—
Aluminum-zinc alloy-coated by the hot-dip process general requirements	A792 Gr. 33	33	45	20	—
	37	37	52	18	—
	40	40	55	16	—
	50	50	65	12	—
	80	80	82	—	—

Notes:
1. The tabulated values are based on ASTM Standards.
2. 1 in. = 25.4 mm; 1 ksi = 6.9 MPa.
3. A653 Structural Quality Grade 80, Grade E of A611, and Structural Quality Grade 80 of A792 are allowed in the AISI Specification under special conditions. For these grades, $F_y = 80$ ksi, $F_u = 82$ ksi, elongations are unspecified. See AISI Specification for reduction of yield point and tensile strength.
4. For A653 steel, HSLA Grades 70 and 80, the elongation in 2-in. gage length given in the parenthesis is for Type II. The other value is for Type I.
5. For A607 steel, the tensile strength given in the parenthesis is for Class 2. The other value is for Class 1.

In addition, formability, durability, and toughness are also important properties for cold-formed steel.

7.4.1 Yield Point, Tensile Strength, and Stress-Strain Relationship

As listed in Table 7.2, the yield points or yield strengths of all 14 different steels range from 24 to 80 ksi (166 to 552 MPa). The tensile strengths of the same steels range from 42 to 100 ksi (290 to 690 MPa). The ratios of the tensile strength-to-yield point vary from 1.12 to 2.22. As far as the stress-strain relationship is concerned, the stress-strain curve can either be the sharp-yielding type (Figure 7.5a) or the gradual-yielding type (Figure 7.5b).

7.4.2 Strength Increase from Cold Work of Forming

The mechanical properties (yield point, tensile strength, and ductility) of cold-formed steel sections, particularly at the corners, are sometimes substantially different from those of the flat steel sheet, strip, plate, or bar before forming. This is because the cold-forming operation increases the yield point and tensile strength and at the same time decreases the ductility. The effects of cold-work on the mechanical properties of corners usually depend on several parameters. The ratios of tensile strength-to-yield point, F_u/F_y, and inside bend radius-to-thickness, R/t, are considered to be the most important factors to affect the change in mechanical properties of cold-formed steel sections. Design equations are given in the AISI Specification [7] for computing the tensile yield strength of corners and the average full-section tensile yield strength for design purposes.

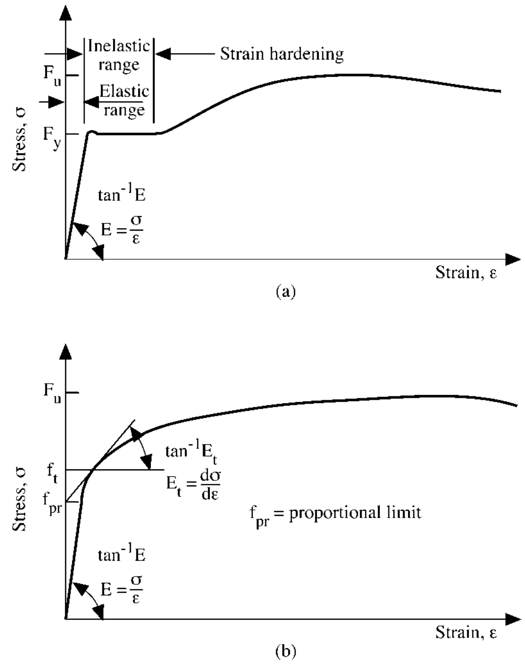


FIGURE 7.5: Stress-strain curves of steel sheet or strip. (a) Sharp-yielding. (b) Gradual-yielding. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

7.4.3 Modulus of Elasticity, Tangent Modulus, and Shear Modulus

The strength of cold-formed steel members that are governed by buckling depends not only on the yield point but also on the modulus of elasticity, E, and the tangent modulus, E_t. A value of $E =$ 29,500 ksi (203 GPa) is used in the AISI Specification for the design of cold-formed steel structural members. This E value is slightly larger than the value of 29,000 ksi (200 GPa), which is being used in the AISC Specification for the design of hot-rolled shapes. The tangent modulus is defined by the slope of the stress-strain curve at any given stress level as shown in Figure 7.5b. For sharp-yielding steels, $E_t = E$ up to the yield, but with gradual-yielding steels, $E_t = E$ only up to the proportional limit, f_{pr} (Figure 7.5b). Once the stress exceeds the proportional limit, the tangent modulus E_t becomes progressively smaller than the initial modulus of elasticity. For cold-formed steel design, the shear modulus is taken as $G = 11{,}300$ ksi (77.9 GPa) according to the AISI Specification.

7.4.4 Ductility

According to the AISI Specification, the ratio of F_u/F_y for the steels used for structural framing members should not be less than 1.08, and the total elongation should not be less than 10% for a 2-in. (50.8 mm) gage length. If these requirements cannot be met, an exception can be made for purlins and girts for which the following limitations should be satisfied when such a material is used: (1) local elongation in a 1/2-in. (12.7 mm) gage length across the fracture should not be less than 20% and (2) uniform elongation outside the fracture should not be less than 3%. It should be noted that the required ductility for cold-formed steel structural members depends mainly on the type of application and the suitability of the material. The same amount of ductility that is considered necessary for individual framing members may not be needed for roof panels, siding, and similar applications. For this reason, even though Structural Grade 80 of ASTM A653 steel, Grade E of A611 steel, and Grade 80 of A792 steel do not meet the AISI requirements of the F_u/F_y ratio and the elongation, these steels can be used for roofing, siding, and similar applications provided that (1) the yield strength, F_y, used for design is taken as 75% of the specified minimum yield point or 60 ksi (414 MPa), whichever is less, and (2) the tensile strength, F_u, used for design is taken as 75% of the specified minimum tensile stress or 62 ksi (427 MPa), whichever is less.

7.5 Element Strength

For cold-formed steel members, the width-to-thickness ratios of individual elements are usually large. These thin elements may buckle locally at a stress level lower than the yield point of steel when they are subject to compression in flexural bending and axial compression as shown in Figure 7.6. Therefore, for the design of such thin-walled sections, local buckling and postbuckling strength of thin elements have often been the major design considerations. In addition, shear buckling and web crippling should also be considered in the design of beams.

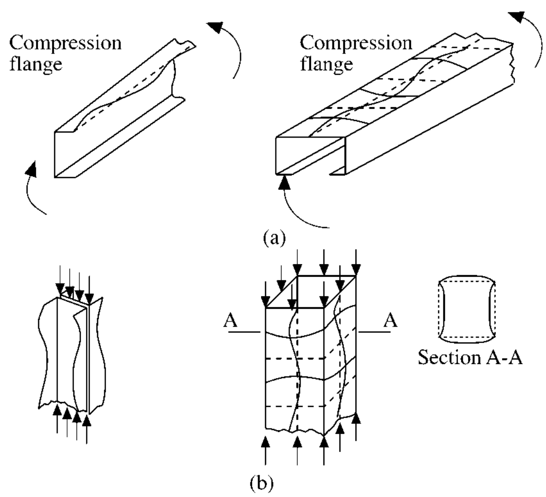


FIGURE 7.6: Local buckling of compression elements. (a) Beams. (b) Columns. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

7.5.1 Maximum Flat-Width-to-Thickness Ratios

In cold-formed steel design, the maximum flat-width-to-thickness ratio, w/t, for flanges is limited to the following values in the AISI Specification:

1. Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by

   Simple lip 60

   Any other kind of stiffener 90

2. Stiffened compression element with both longitudinal edges connected to other stiffened element 500

3. Unstiffened compression element and elements with an inadequate edge stiffener . . . 60

For the design of beams, the maximum depth-to-thickness ratio, h/t, for webs are:

1. For unreinforced webs: $(h/t)_{\max} = 200$
2. For webs that are provided with transverse stiffeners:

   Using bearing stiffeners only: $(h/t)_{\max} = 260$

   Using bearing stiffeners and intermediate stiffeners: $(h/t)_{\max} = 300$

7.5.2 Stiffened Elements under Uniform Compression

The strength of a stiffened compression element such as the compression flange of a hat section is governed by yielding if its w/t ratio is relatively small. It may be governed by local buckling as shown in Figure 7.7 at a stress level less than the yield point if its w/t ratio is relatively large.

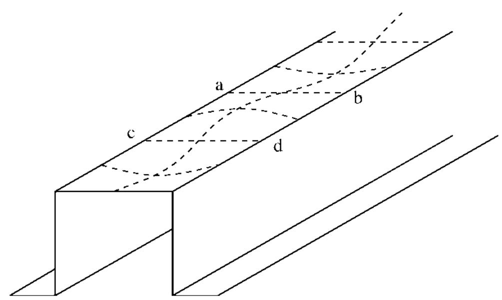


FIGURE 7.7: Local buckling of stiffened compression flange of hat-shaped beam.

The elastic local buckling stress, f_{cr}, of simply supported square plates and long plates can be determined as follows:

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(w/t)^2} \tag{7.3}$$

where

k = local buckling coefficient
E = modulus of elasticity of steel = 29.5×10^3 ksi (203 GPa)
w = width of the plate
t = thickness of the plate
μ = Poisson's ratio

It is well known that stiffened compression elements will not collapse when the local buckling stress is reached. An additional load can be carried by the element after buckling by means of a redistribution of stress. This phenomenon is known as postbuckling strength and is most pronounced for elements with large w/t ratios.

The mechanism of the postbuckling action can be easily visualized from a square plate model as shown in Figure 7.8 [48]. It represents the portion *abcd* of the compression flange of the hat section illustrated in Figure 7.7. As soon as the plate starts to buckle, the horizontal bars in the grid of the model will act as tie rods to counteract the increasing deflection of the longitudinal struts.

In the plate, the stress distribution is uniform prior to its buckling. After buckling, a portion of the prebuckling load of the center strip transfers to the edge portion of the plate. As a result, a

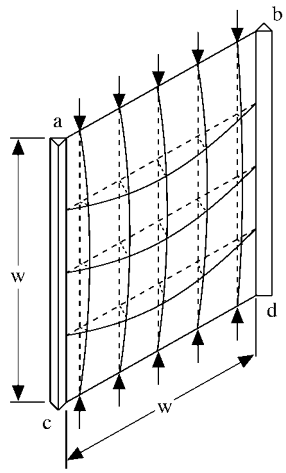


FIGURE 7.8: Postbuckling strength model. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

nonuniform stress distribution is developed, as shown in Figure 7.9. The redistribution of stress continues until the stress at the edge reaches the yield point of steel and then the plate begins to fail.

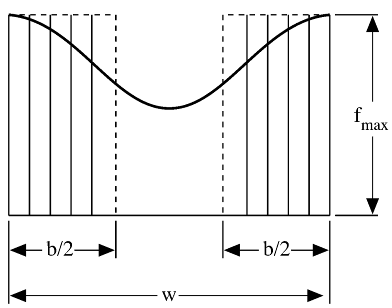


FIGURE 7.9: Stress distribution in stiffened compression elements.

For cold-formed steel members, a concept of "effective width" has been used for practical design. In this approach, instead of considering the nonuniform distribution of stress over the entire width of the plate, w, it is assumed that the total load is carried by a fictitious effective width, b, subjected to a uniformly distributed stress equal to the edge stress, $f_{\max}$, as shown in Figure 7.9. The width, b, is selected so that the area under the curve of the actual nonuniform stress distribution is equal to the sum of the two parts of the equivalent rectangular shaded area with a total width, b, and an intensity of stress equal to the edge stress, $f_{\max}$. Based on the research findings of von Karman, Sechler, and Donnell [45], and Winter [47], the following equations have been developed in the AISI Specification for computing the effective design width, b, for stiffened elements under uniform compression [7]:

(a) Strength Determination

$$1.\ \text{When}\quad \lambda \leq 0.673, b = w \tag{7.4}$$

$$2.\ \text{When}\quad \lambda > 0.673, b = \rho w \tag{7.5}$$

where

b = effective design width of uniformly compressed element for strength determination (Figure 7.10)

w = flat width of compression element

ρ = reduction factor determined from Equation 7.6:

$$\rho = (1 - 0.22/\lambda)/\lambda \leq 1 \tag{7.6}$$

where λ = plate slenderness factor determined from Equation 7.7:

$$\lambda = (1.052/\sqrt{k})(w/t)(\sqrt{f/E}) \tag{7.7}$$

where

k = plate buckling coefficient = 4.0 for stiffened elements supported by a web on each longitudinal edge as shown in Figure 7.10

t = thickness of compression element

E = modulus of elasticity

f = maximum compressive edge stress in the element without considering the safety factor

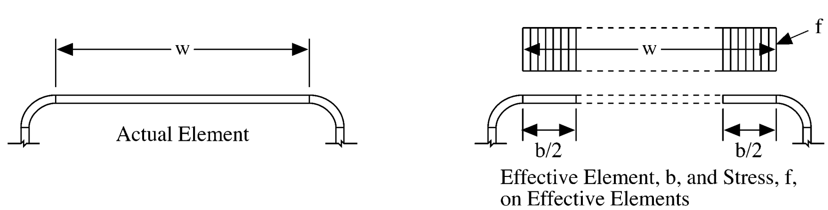


FIGURE 7.10: Effective design width of stiffened compression elements.

(b) Deflection Determination

For deflection determination, Equations 7.4 through 7.7 can also be used for computing the effective design width of compression elements, except that the compressive stress should be computed on the basis of the effective section at the load for which deflection is calculated.

The relationship between ρ and λ according to Equation 7.6 is shown in Figure 7.11.

EXAMPLE 7.1:

Calculate the effective width of the compression flange of the box section (Figure 7.12) to be used as a beam bending about the x-axis. Use $F_y = 33$ ksi. Assume that the beam webs are fully effective and that the bending moment is based on initiation of yielding.

Solution Because the compression flange of the given section is a uniformly compressed stiffened element, which is supported by a web on each longitudinal edge, the effective width of the flange for strength determination can be computed by using Equations 7.4 through 7.7 with $k = 4.0$.

Assume that the bending strength of the section is based on Initiation of Yielding, $\bar{y} \geq 2.50$ in. Therefore, the slenderness factor λ for $f = F_y$ can be computed from Equation 7.7, i.e.,

$$\begin{aligned} k &= 4.0 \\ w &= 6.50 - 2(R + t) = 6.192 \text{ in.} \\ w/t &= 103.2 \end{aligned}$$

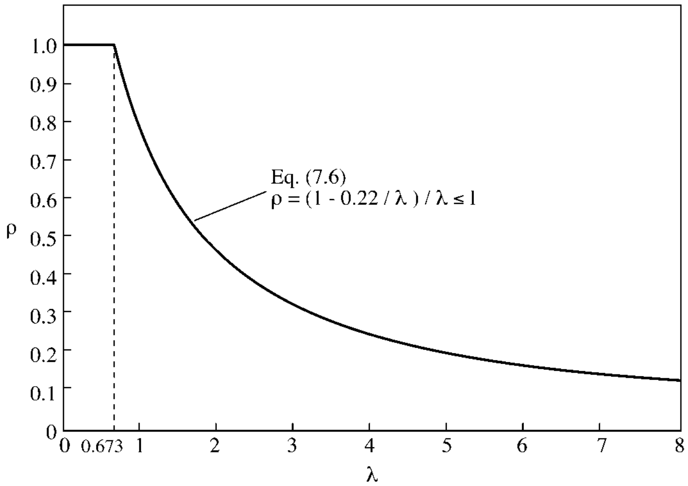


FIGURE 7.11: Reduction factor, ρ, vs. slenderness factor, λ. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

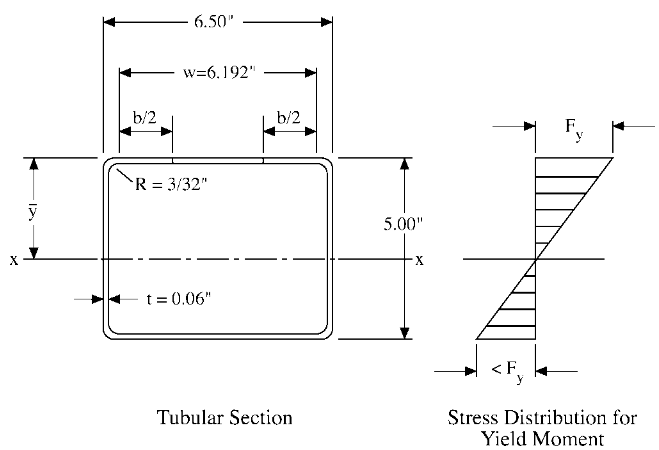


FIGURE 7.12: Example 7.1. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

$$
\begin{aligned}
f &= 33 \text{ ksi} \\
\lambda &= (1.052/\sqrt{k})(w/t)\sqrt{f/E} \\
&= (1.052/\sqrt{4.0})(103.2)\sqrt{33/29{,}500} = 1.816
\end{aligned}
$$

Since $\lambda > 0.673$, use Equations 7.5 and 7.6 to compute the effective width, b, as follows:

$$
\begin{aligned}
b &= \rho w = [(1-0.22/\lambda)/\lambda]w \\
&= [(1-0.22/1.816)/1.816](6.192) = 3.00 \text{ in.}
\end{aligned}
$$

7.5.3 Stiffened Elements with Stress Gradient

When a flexural member is subject to bending moment, the beam web is under the stress gradient condition (Figure 7.13), in which the compression portion of the web may buckle due to the compressive stress caused by bending. The effective width of the beam web can be determined from the following AISI provisions:

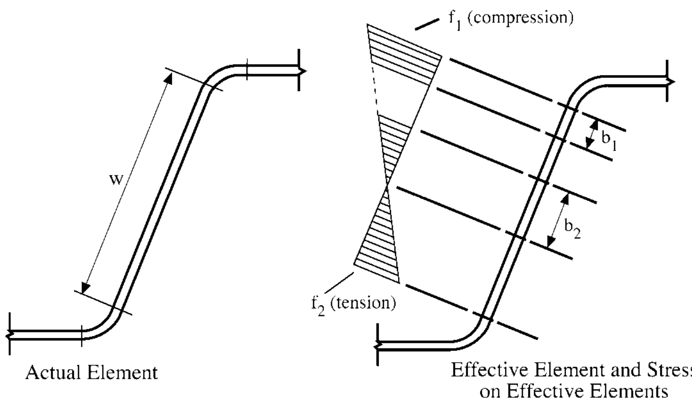


FIGURE 7.13: Stiffened elements with stress gradient.

(a) Strength Determination

The effective widths, b_1 and b_2, as shown in Figure 7.13, should be determined from the following equations:

$$b_1 = b_e/(3-\psi) \tag{7.8}$$

For $\psi \leq -0.236$

$$b_2 = b_e/2 \tag{7.9}$$

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

For $\psi > -0.236$

$$b_2 = b_e - b_1 \tag{7.10}$$

where b_e = effective width b determined by Equation 7.4 or Equation 7.5 with f_1 substituted for f and with k determined as follows:

$$k = 4 + 2(1-\psi)^3 + 2(1-\psi) \tag{7.11}$$

$$\psi = f_2/f_1 \tag{7.12}$$

f_1, f_2 = stresses shown in Figure 7.13 calculated on the basis of effective section. f_1 is compression (+) and f_2 can be either tension (−) or compression. In case f_1 and f_2 are both compression, $f_1 \geq f_2$

(b) Deflection Determination

The effective widths used in computing deflections should be determined as above, except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1} and f_{d2} are the computed stresses f_1 and f_2 as shown in Figure 7.13 based on the effective section at the load for which deflection is determined.

7.5.4 Unstiffened Elements under Uniform Compression

The effective width of unstiffened elements under uniform compression as shown in Figure 7.14 can also be computed by using Equations 7.4 through 7.7, except that the value of k should be taken as 0.43 and the flat width w is measured as shown in Figure 7.14.

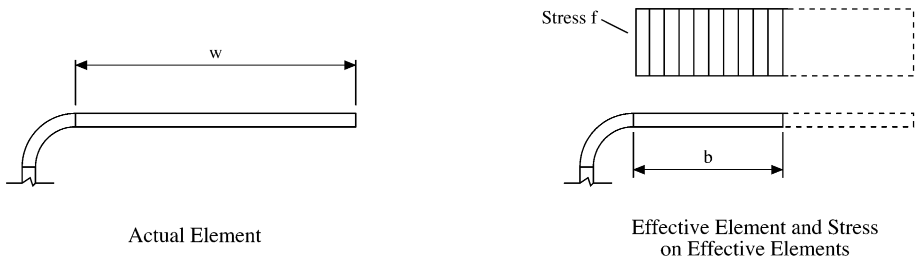


FIGURE 7.14: Effective design width of unstiffened compression elements.

7.5.5 Uniformly Compressed Elements with an Edge Stiffener

The following equations can be used to determine the effective width of the uniformly compressed elements with an edge stiffener as shown in Figure 7.15.

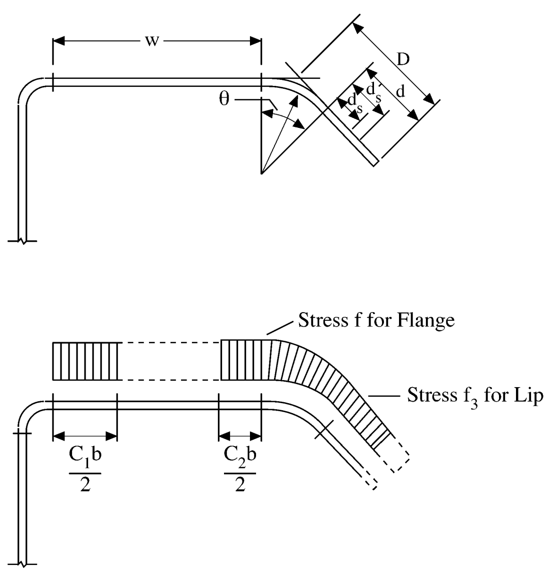


FIGURE 7.15: Compression elements with an edge stiffener.

Case I: For $w/t \leq S/3$

$$I_a = 0 \quad \text{(no edge stiffener needed)}$$

$$
\begin{aligned}
b &= w \\
d_s &= d_s' \quad \text{for simple lip stiffener} \\
A_s &= A_s' \quad \text{for other stiffener shapes}
\end{aligned}
\tag{7.13}
$$

Case II: For $S/3 < w/t < S$

$$
\begin{aligned}
I_a/t^4 &= 399\{[(w/t)/S] - \sqrt{k_u/4}\}^3 \\
n &= 1/2 \\
C_2 &= I_s/I_a \leq 1 \\
C_1 &= 2 - C_2
\end{aligned}
\tag{7.14}
$$

b should be calculated according to Equations 7.4 through 7.7, where

$$
\begin{aligned}
k &= C_2^n(k_a - k_u) + k_u \\
k_u &= 0.43
\end{aligned}
$$

For simple lip stiffener with $140° \geq \theta \geq 40°$ and $D/w \leq 0.8$ where θ is as shown in Figure 7.15:

$$
\begin{aligned}
k_a &= 5.25 - 5(D/w) \leq 4.0 \\
d_s &= C_2 d_s'
\end{aligned}
$$

For a stiffener shape other than simple lip:

$$
\begin{aligned}
k_a &= 4.0 \\
A_s &= C_2 A_s'
\end{aligned}
$$

Case III: For $w/t \geq S$

$$
I_a/t^4 = [115(w/t)/S] + 5 \tag{7.15}
$$

C_1, C_2, b, k, d_s, and A_s are calculated per Case II with $n = 1/3$

where

S	=	$1.28\sqrt{E/f}$
k	=	buckling coefficient
d, w, D	=	dimensions shown in Figure 7.15
d_s	=	reduced effective width of the stiffener
d_s'	=	effective width of the stiffener calculated as unstiffened element under uniform compression
C_1, C_2	=	coefficients shown in Figure 7.15
A_s	=	reduced area of the stiffener
I_a	=	adequate moment of inertia of the stiffener, so that each component element will behave as a stiffened element
I_s, A_s'	=	moment of inertia of the full section of the stiffener about its own centroidal axis parallel to the element to be stiffened, and the effective area of the stiffener, respectively

For the stiffener shown in Figure 7.15,

$$
\begin{aligned}
I_s &= (d^3 t \sin^2\theta)/12 \\
A_s' &= d_s' t
\end{aligned}
$$

7.5.6 Uniformly Compressed Elements with Intermediate Stiffeners

The effective width of uniformly compressed elements with intermediate stiffeners can also be determined from the AISI Specification, which includes separate design rules for compression elements with only one intermediate stiffener and compression elements with more than one intermediate stiffener.

Uniformly Compressed Elements with One Intermediate Stiffener

The following equation can be used to determine the effective width of the uniformly compressed elements with one intermediate stiffener as shown in Figure 7.16.

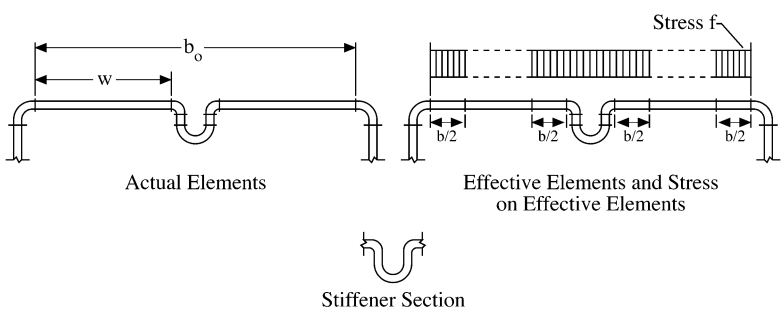


FIGURE 7.16: Compression elements with one intermediate stiffener.

Case I: For $b_0/t \leq S$

$$\begin{aligned} I_a &= 0 \quad \text{(no intermediate stiffener needed)} \\ b &= w \\ A_s &= A_s' \end{aligned}$$

Case II: For $S < b_0/t < 3S$

$$I_a/t^4 = [50(b_0/t)/S] - 50$$

b and A_s are calculated according to Equations 7.4 through 7.7, where

$$\begin{aligned} k &= 3(I_s/I_a)^{1/2} + 1 \leq 4 \\ A_s &= A_s'(I_s/I_a) \leq A_s' \end{aligned}$$

Case III: For $b_0/t \geq 3S$

$$I_a/t^4 = [128(b_0/t)/S] - 285$$

b and A_s are calculated according to Equations 7.4 through 7.7, where

$$\begin{aligned} k &= 3(I_s/I_a)^{1/3} + 1 \leq 4 \\ A_s &= A_s'(I_s/I_a) \leq A_s' \end{aligned}$$

In the above equations, all symbols were defined previously.

Uniformly Compressed Elements with More Than One Intermediate Stiffener

For the determination of the effective width of sub-elements, the stiffeners of a stiffened element with more than one stiffener should be disregarded unless each intermediate stiffener has the minimum I_s as follows:

$$I_{\min}/t^4 = 3.66\sqrt{(w/t)^2 - (0.136E)/F_y} \geq 18.4$$

where

w/t = width-thickness ratio of the larger stiffened sub-element

I_s = moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened

For additional requirements, see the AISI Specification.

7.6 Member Design

This chapter deals with the design of the following cold-formed steel structural members: (a) tension members, (b) flexural members, (c) concentrically loaded compression members, (d) combined axial load and bending, and (e) cylindrical tubular members. The nominal strength equations with safety factors (Ω) and resistance factors (ϕ) are provided in the Specification [7] for the given limit states.

7.6.1 Sectional Properties

The sectional properties of a member such as area, moment of inertia, section modulus, and radius of gyration are calculated by using the conventional methods of structural design. These properties are based on either full cross-section dimensions, effective widths, or net section, as applicable.

For the design of tension members, the nominal tensile strength is presently based on the net section. However, for flexural members and axially loaded compression members, the full dimensions are used when calculating the critical moment or load, while the effective dimensions, evaluated at the stress corresponding to the critical moment or load, are used to calculate the nominal strength.

7.6.2 Linear Method for Computing Sectional Properties

Because the thickness of cold-formed steel members is usually uniform, the computation of sectional properties can be simplified by using a “linear” or “midline” method. In this method, the material of each element is considered to be concentrated along the centerline or midline of the steel sheet and the area elements are replaced by straight or curved “line elements”. The thickness dimension, t, is introduced after the linear computations have been completed. Thus, the total area is $A = Lt$, and the moment of inertia of the section is $I = I't$, where L is the total length of all line elements and I' is the moment of inertia of the centerline of the steel sheet. The moments of inertia of straight line elements and circular line elements are shown in Figure 7.17.

7.6.3 Tension Members

The nominal tensile strength of axially loaded cold-formed steel tension members is determined by the following equation:

$$T_n = A_n F_y \tag{7.16}$$

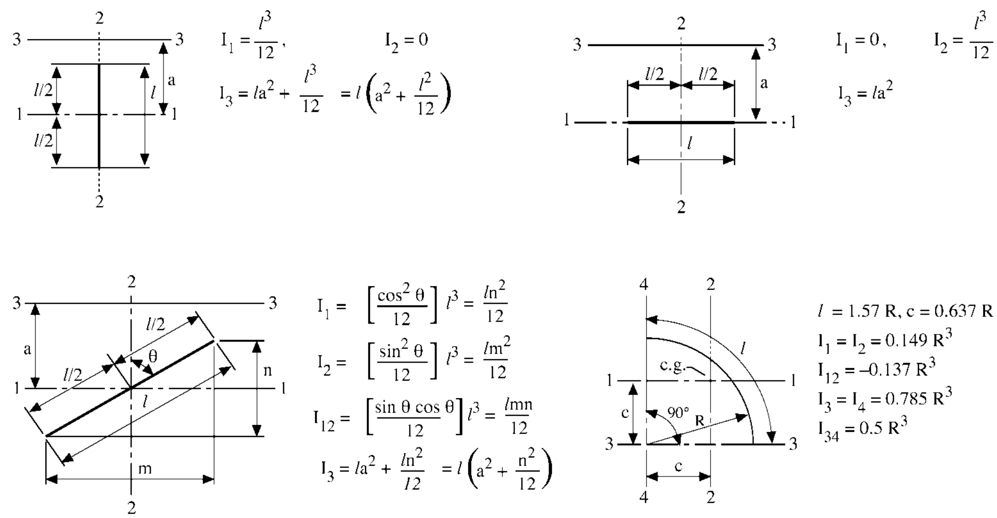


FIGURE 7.17: Properties of line elements.

where

T_n = nominal tensile strength
A_n = net area of the cross-section
F_y = design yield stress

When tension members use bolted connections or circular holes, the nominal tensile strength is also limited by the tensile capacity of connected parts treated separately by the AISI Specification [7] under the section title of Bolted Connections.

7.6.4 Flexural Members

For the design of flexural members, consideration should be given to several design features: (a) bending strength and deflection, (b) shear strength of webs and combined bending and shear, (c) web crippling strength and combined bending and web crippling, and (d) bracing requirements. For some cases, special consideration should also be given to shear lag and flange curling due to the use of thin materials.

Bending Strength

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, they are designed according to the nominal section strength. Otherwise, if they are laterally unbraced, then the bending strength may be governed by the lateral buckling strength. For channels or Z-sections with tension flange attached to deck or sheathing and with compression flange laterally unbraced, and for such members having one flange fastened to a standing seam roof system, the nominal bending strength should be reduced according to the AISI Specification.

Nominal Section Strength

Two design procedures are now used in the AISI Specification for determining the nominal bending strength. They are: (I) *Initiation of Yielding* and (II) *Inelastic Reserve Capacity*.

According to Procedure I on the basis of initiation of yielding, the nominal moment, M_n, of the cross-section is the effective yield moment, M_y, determined for the effective areas of flanges and the beam web. The effective width of the compression flange and the effective depth of the web can be computed from the design equations given in Section 7.5. The yield moment of a cold-formed steel flexural member is defined as the moment at which an outer fiber (tension, compression, or both) first attains the yield point of the steel. Figure 7.18 shows three types of stress distribution for yield

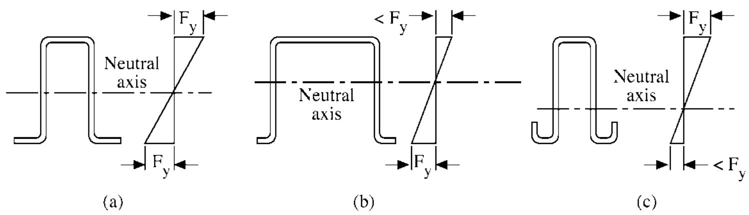


FIGURE 7.18: Stress distribution for yield moment (based on initiation of yielding).

moment based on different locations of the neutral axis. Accordingly, the nominal section strength for initiation of yielding can be computed as follows:

$$M_n = M_y = S_e F_y \tag{7.17}$$

where

S_e = elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y

F_y = design yield stress

For cold-formed steel design, S_e is usually computed by using one of the following two cases:

1. If the neutral axis is closer to the tension than to the compression flange (Case c), the maximum stress occurs in the compression flange, and therefore the plate slenderness ratio λ (Equation 7.7) and the effective width of the compression flange are determined by the w/t ratio and $f = F_y$. This procedure is also applicable to those beams for which the neutral axis is located at the mid-depth of the section (Case a).
2. If the neutral axis is closer to the compression than to the tension flange (Case b), the maximum stress of F_y occurs in the tension flange. The stress in the compression flange depends on the location of the neutral axis, which is determined by the effective area of the section. The latter cannot be determined unless the compressive stress is known. The closed-form solution of this type of design is possible but would be a very tedious and complex procedure. It is, therefore, customary to determine the sectional properties of the section by successive approximation.

See Examples 7.2 and 7.3 for the calculation of nominal bending strengths.

EXAMPLE 7.2:

Use the ASD and LRFD methods to check the adequacy of the I-section with unstiffened flanges as shown in Figure 7.19. The nominal moment is based on the initiation of yielding using $F_y = 50$ ksi. Assume that lateral bracing is adequately provided. The dead load moment $M_D = 30$ in.-kips and the live load moment $M_L = 150$ in.-kips.

Solution

(A) ASD Method

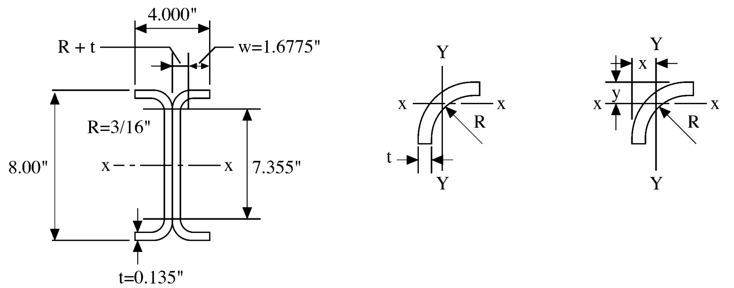


FIGURE 7.19: Example 7.2. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

1. Location of Neutral Axis. For $R = 3/16$ in. and $t = 0.135$ in., the sectional properties of the corner element are as follows:

$$\begin{aligned} I_x &= I_y = 0.0003889 \quad \text{in.}^4 \\ A &= 0.05407 \quad \text{in.}^2 \\ x &= y = 0.1564 \quad \text{in.} \end{aligned}$$

   For the unstiffened compression flange,

$$w = 1.6775 \ \text{ in.}, \ w/t = 12.426$$

   Using $k = 0.43$ and $f = F_y = 50$ ksi,

$$\begin{aligned} \lambda &= (1.052/\sqrt{k})(w/t)\sqrt{f/E} = 0.821 > 0.673 \\ b &= [(1 - 0.22/0.821)/0.821](1.6775) = 1.496 \quad \text{in.} \end{aligned}$$

   Assuming the web is fully effective, the neutral axis is located at $y_{cg} = 4.063$ in. as shown in Figure 7.20. Since $y_{cg} > d/2$, initial yield occurs in the compression flange. Therefore, $f = F_y$.

2. Check the web for full effectiveness as follows (Figure 7.20):

$$\begin{aligned} f_1 &= 46.03 \quad \text{ksi} \ \ \text{(compression)} \\ f_2 &= -44.48 \quad \text{ksi} \ \ \text{(tension)} \\ \psi &= f_2/f_1 = -0.966. \end{aligned}$$

   Using Equation 7.11,

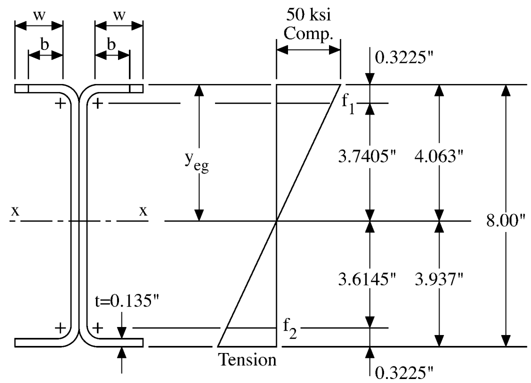


FIGURE 7.20: Stress distribution in webs. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

$$
\begin{aligned}
k &= 4 + 2(1-\psi)^3 + 2(1-\psi) = 23.13 \\
h &= 7.355 \quad \text{in.} \\
h/t &= 54.48 \\
\lambda &= (1.052/\sqrt{k})(54.48)\sqrt{46.03/29{,}500} \\
&= 0.471 < 0.673 \\
b_e &= h = 7.355 \quad \text{in.} \\
b_1 &= b_e/(3-\psi) = 1.855 \quad \text{in.} \\
b_2 &= b_e/2 = 3.6775 \quad \text{in.}
\end{aligned}
$$

Since $b_1 + b_2 = 5.5325$ in. $>$ 3.7405 in., the web is fully effective.

3. The moment of inertia I_x is

$$
\begin{aligned}
I_x &= \Sigma(Ay^2) + 2I_{\text{web}} - (\Sigma A)(y_{cg})^2 \\
&= 25.382 \quad \text{in.}^4
\end{aligned}
$$

The section modulus for the top fiber is

$$S_e = I_x/y_{cg} = 6.247 \quad \text{in.}^3$$

4. Based on initiation of yielding, the nominal moment for section strength is

$$M_n = S_e F_y = 312.35 \quad \text{in.-kips}$$

5. The allowable moment or design moment is

$$M_a = M_n/\Omega = 312.35/1.67 = 187.04 \quad \text{in.-kips}$$

Based on the given data, the required moment is

$$M = M_D + M_L = 30 + 150 = 180 \quad \text{in.-kips}$$

Since $M < M_a$, the I-section is adequate for the ASD method.

(B) LRFD Method

1. Based on the nominal moment M_n computed above, the design moment is

$$\phi_b M_n = 0.90(312.35) = 281.12 \quad \text{in.-kips}$$

2. The required moment for combined dead and live moments is

$$\begin{aligned} M_u &= 1.2M_D + 1.6M_L \\ &= (1.2 \times 30) + (1.6 \times 150) \\ &= 276.00 \quad \text{in.-kips} \end{aligned}$$

Since $\phi_b M_n > M_u$, the I-section is adequate for bending strength according to the LRFD approach.

EXAMPLE 7.3:

Determine the nominal moment about the x-axis for the hat section with stiffened compression flange as shown in Figure 7.21. Assume that the yield point of steel is 50 ksi. Use the linear method. The nominal moment is determined by initiation of yielding.

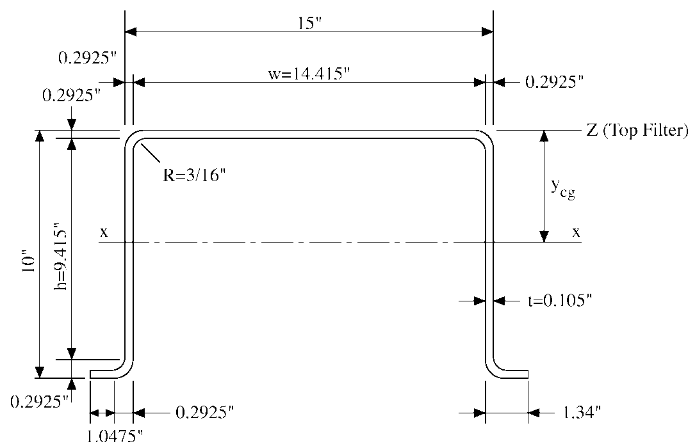


FIGURE 7.21: Example 7.3. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

Solution

1. Calculation of Sectional Properties. In order to use the linear method, midline dimensions are shown in Figure 7.22.

   A. Corner element (Figures 7.17 and 7.22)

$$R' = R + t/2 = 0.240 \quad \text{in.}$$

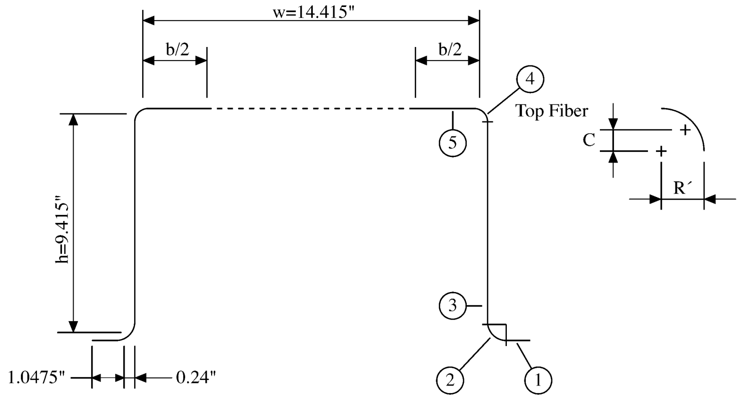


FIGURE 7.22: Line elements. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

Arc length

$$
\begin{aligned}
L &= 1.57R' = 0.3768 \quad \text{in.} \\
c &= 0.637R' = 0.1529 \quad \text{in.}
\end{aligned}
$$

B. Location of neutral axis

a. First approximation. For the compression flange,

$$
\begin{aligned}
w &= 15 - 2(R + t) = 14.415 \quad \text{in.} \\
w/t &= 137.29
\end{aligned}
$$

Using Equations 7.4 through 7.7 and assuming $f = F_y = 50$ ksi,

$$
\begin{aligned}
\lambda &= \frac{1.052}{\sqrt{4}}(137.29)\sqrt{\frac{50}{29500}} = 2.973 > 0.673 \\
\rho &= \left(1 - \frac{0.22}{2.973}\right)/2.973 = 0.311 \\
b &= \rho w = 0.311(14.415) = 4.483 \quad \text{in.}
\end{aligned}
$$

By using the effective width of the compression flange and assuming the web is fully effective, the neutral axis can be located as follows:

Element	Effective length L (in.)	Distance from top fiber y (in.)	Ly (in.^{2})
1	$2 \times 1.0475 =$ 2.0950	9.9475	20.8400
2	$2 \times 0.3768 =$ 0.7536	9.8604	7.4308
3	$2 \times 9.4150 =$ 18.8300	5.0000	94.1500
4	$2 \times 0.3768 =$ 0.7536	0.1396	0.1052
5	4.4830	0.0525	0.2354
Total	26.9152		122.7614
	$y_{cg} = \frac{\Sigma(Ly)}{\Sigma L} = \frac{122.7614}{26.9152} = 4.561$ in.		

Because the distance y_{cg} is less than the half-depth of 5.0 in., the neutral axis is closer to the compression flange and, therefore, the maximum stress occurs in the tension flange. The maximum compressive stress can be computed as follows:

$$f = 50\left(\frac{4.561}{10 - 4.561}\right) = 41.93 \quad \text{ksi}$$

Since the above computed stress is less than the assumed value, another trial is required.

b. Second approximation. Assuming that

$$\begin{aligned} f &= 40.70 \quad \text{ksi} \\ \lambda &= 2.682 > 0.673 \\ b &= 4.934 \quad \text{in.} \end{aligned}$$

Element	Effective length L (in.)	Distance from top fiber y (in.)	Ly (in.^{2})	Ly^2 (in.^{3})
1	2.0950	9.9475	20.8400	207.3059
2	0.7536	9.8604	7.4308	73.2707
3	18.8300	5.0000	94.1500	470.7500
4	0.7536	0.1396	0.1052	0.0147
5	4.9340	0.0525	0.2590	0.0136
Total	27.3662		122.7850	751.3549
	$y_{cg} = \frac{122.7850}{27.3662} = 4.487$ in.			

$$f = \left(\frac{4.487}{10 - 4.487}\right) = 40.69 \quad \text{ksi}$$

Since the above computed stress is close to the assumed value, it is O.K.

C. Check the effectiveness of the web. Use the AISI Specification to check the effectiveness of the web element. From Figure 7.23,

$$\begin{aligned} f_1 &= 50(4.1945/5.513) = 38.04 \quad \text{ksi (compression)} \\ f_2 &= -50(5.2205/5.513) = -47.35 \quad \text{ksi (tension)} \\ \psi &= f_2/f_1 = -1.245. \quad \text{Using Equation 7.11,} \\ k &= 4 + 2(1-\psi)^3 + 2(1-\psi) \\ &= 4 + 2(2.245)^3 + 2(2.245) = 31.12 \end{aligned}$$

$$
\begin{aligned}
h/t &= 9.415/0.105 = 89.67 < 200 \quad \text{O.K.} \\
\lambda &= \frac{1.052}{\sqrt{31.12}}(89.67)\sqrt{\frac{38.04}{29{,}500}} = 0.607 < 0.673 \\
b_e &= h = 9.415 \quad \text{in.} \\
b_1 &= b_e/(3-\psi) = 2.218 \quad \text{in.}
\end{aligned}
$$

Since $\psi < -0.236$,

$$
\begin{aligned}
b_2 &= b_e/2 = 4.7075 \quad \text{in.} \\
b_1 + b_2 &= 6.9255 \quad \text{in.}
\end{aligned}
$$

Because the computed value of $(b_1 + b_2)$ is greater than the compression portion of the web (4.1945 in.), the web element is fully effective.

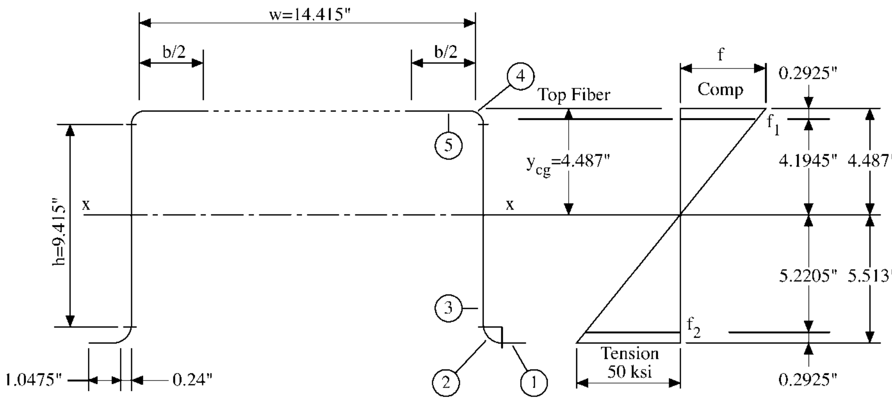


FIGURE 7.23: Effective lengths and stress distribution using fully effective webs. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

D. Moment of inertia and section modulus. The moment of inertia based on line elements is

$$
\begin{aligned}
2I_3' &= 2\left(\frac{1}{12}\right)(9.415)^3 = 139.0944 \\
\Sigma(Ly^2) &= 751.3549 \\
I_z' &= 2I_3' + \Sigma(Ly^2) = 890.4493 \quad \text{in.}^3 \\
(\Sigma L)(y_{cg})^2 &= 27.3662(4.487)^2 = 550.9683 \quad \text{in.}^3 \\
I_x' &= I_z' - (\Sigma L)(y_{cg})^2 = 339.4810 \quad \text{in.}^3
\end{aligned}
$$

The actual moment of inertia is

$$I_x = I'_x t = (339.4810)(0.105) = 35.646 \quad \text{in.}^4$$

The section modulus relative to the extreme tension fiber is

$$S_x = 35.646/5.513 = 6.466 \quad \text{in.}^3$$

2. Nominal Moments. The nominal moment for section strength is

$$M_n = S_e F_y = S_x F_y = (6.466)(50) = 323.30 \quad \text{in.-kips}$$

Once the nominal moment is computed, the design moments for the ASD and LRFD methods can be determined as illustrated in Example 7.2.

According to Procedure II of the AISI Specification, the nominal moment, M_n, is the maximum bending capacity of the beam by considering the inelastic reserve strength through partial plastification of the cross-section as shown in Figure 7.24. The inelastic stress distribution in the cross-section

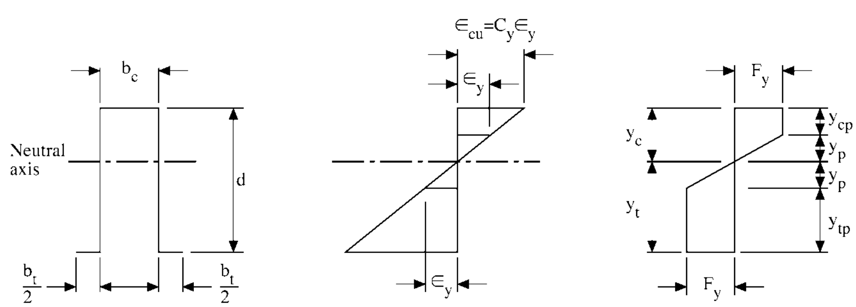


FIGURE 7.24: Stress distribution for maximum moment (inelastic reserve strength). (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

depends on the maximum strain in the compression flange, which is limited by the Specification for the given width-to-thickness ratio of the compression flange. On the basis of the maximum compression strain allowed in the Specification, the neutral axis can be located by Equation 7.18 and the nominal moment, M_n, can be determined by using Equation 7.19:

$$\int \sigma dA = 0 \tag{7.18}$$

$$\int \sigma y dA = M \tag{7.19}$$

where σ is the stress in the cross-section. For additional information, see Yu [49].

Lateral Buckling Strength

The nominal lateral buckling strength of unbraced segments of singly-, doubly-, and point- symmetric sections subjected to lateral buckling, M_n, can be determined as follows:

$$M_n = S_c \frac{M_c}{S_f} \tag{7.20}$$

where

S_f = elastic section modulus of the full unreduced section for the extreme compression fiber

S_c = elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber

M_c = critical moment for singly-, doubly-, and point-symmetric sections calculated as follows:

1. For $M_e \geq 2.78 M_y$:

$$M_c = M_y \tag{7.21}$$

2. For $2.78 M_y > M_e > 0.56 M_y$:

$$M_c = \frac{10}{9} M_y \left(1 - \frac{10 M_y}{36 M_e}\right) \tag{7.22}$$

3. For $M_e \leq 0.56 M_y$:

$$M_c = M_e \tag{7.23}$$

where

M_y = moment causing initial yield at the extreme compression fiber of the full section

= $S_f F_y$

M_e = elastic critical moment calculated according to (a) or (b) below:

(a) For singly-, doubly-, and point-symmetric sections:

M_e = $C_b r_0 A \sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis. For singly-symmetric sections, x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate. For point-symmetric sections, use 0.5 M_e. Alternatively, M_e can be calculated using the equation for doubly-symmetric I-sections or point-symmetric sections given in (b)

M_e = $C_s A \sigma_{ex} [j + C_s \sqrt{j^2 + r_0^2(\sigma_t/\sigma_{ex})}]/C_{TF}$ for bending about the centroidal axis perpendicular to the symmetry axis for singly-symmetric sections only

C_s = $+1$ for moment causing compression on the shear center side of the centroid

C_s = -1 for moment causing tension on the shear center side of the centroid

σ_{ex} = $\pi^2 E/(K_x L_x/r_x)^2$

σ_{ey} = $\pi^2 E/(K_y L_y/r_y)^2$

σ_t = $[GJ + \pi^2 E C_w/(K_t L_t)^2]/A r_0^2$

A = full cross-sectional area

$$C_b = 12.5 M_{\max}/(2.5 M_{\max} + 3M_A + 4M_B + 3M_C) \tag{7.24}$$

In Equation 7.24,

$M_{\max}$ = absolute value of maximum moment in the unbraced segment

M_A = absolute value of moment at quarter point of unbraced segment

M_B = absolute value of moment at centerline of unbraced segment

M_C = absolute value of moment at three-quarter point of unbraced segment

C_b is permitted to be conservatively taken as unity for all cases. For cantilevers or overhangs where the free end is unbraced, C_b shall be taken as unity. For members subject to combined axial load and bending moment, C_b shall be taken as unity.

E = modulus of elasticity

C_{TF} = $0.6 - 0.4(M_1/M_2)$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length in the plane of bending, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined compressive axial load and bending moment, C_{TF} shall be taken as unity.

r_0 = Polar radius of gyration of the cross-section about the shear center

$= \sqrt{r_x^2 + r_y^2 + x_0^2}$

r_x, r_y = radii of gyration of the cross-section about the centroidal principal axes

G = shear modulus

K_x, K_y, K_t = effective length factors for bending about the x- and y-axes, and for twisting

L_x, L_y, L_t = unbraced length of compression member for bending about the x- and y-axes, and for twisting

x_0 = distance from the shear center to the centroid along the principal x-axis, taken as negative

J = St. Venant torsion constant of the cross-section

C_w = torsional warping constant of the cross-section

j = $[\int_A x^3 dA + \int_A xy^2 dA]/(2I_y) - x_0$

(b) For I- or Z-sections bent about the centroidal axis perpendicular to the web (x-axis):
In lieu of (a), the following equations may be used to evaluate M_e:

$$M_e = \pi^2 E C_b d I_{yc}/L^2 \quad \text{for doubly-symmetric I-sections} \tag{7.25}$$

$$M_e = \pi^2 E C_b d I_{yc}/(2L^2) \quad \text{for point-symmetric Z-sections} \tag{7.26}$$

In Equations 7.25 and 7.26,

d = depth of section

E = modulus of elasticity

I_{yc} = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, using the full unreduced section

L = unbraced length of the member

EXAMPLE 7.4:

Determine the nominal moment for lateral buckling strength for the I-beam used in Example 7.2. Assume that the beam is braced laterally at both ends and midspan. Use $F_y = 50$ ksi.

Solution

1. Calculation of Sectional Properties

Based on the dimensions given in Example 7.2 (Figures 7.19 and 7.20), the moment of inertia, I_x, and the section modulus, S_f, of the full section can be computed as shown in the following table.

Element	Area A (in.^{2})	Distance from mid-depth y (in.)	Ay^2 (in.^{4})
Flanges	4(1.6775)(0.135) = 0.9059	3.9325	14.0093
Corners	4(0.05407) = 0.2163	3.8436	3.1955
Webs	2(7.355)(0.135) = 1.9859	0	0
Total	3.1081		17.2048
	$2\,I_{web} = 2(1/12)(0.135)(7.355)^3 =$		8.9522
		$I_x =$	26.1570 in.^{4}
	$S_f = I_x/(8/2) = 6.54$ in.^{3}		

The value of I_{yc} can be computed as shown below.

Element	Area A (in.^{2})	Distance from y axis, x (in.)	Ax^2 (in.^{4})
Flanges	$4(1.6775)(0.135) = 0.9059$	1.1613	1.2217
Corners	$4(0.05407) = 0.2163$	0.1564	0.0053
Webs	$2(7.355)(0.135) = 1.9859$	0.0675	0.0090
Total	3.1081		1.2360
	$I_{\text{flanges}} = 4(1/12)0.135(1.6775)^3 =$		0.2124
		$I_y =$	1.4484 in.^{4}
	$I_{yc} = I_y/2 = 0.724$ in.^{4}		

Considering the lateral supports at both ends and midspan, and the moment diagram shown in Figure 7.25, the value of C_b for the segment AB or BC is 1.30 according to Equation 7.24. Using

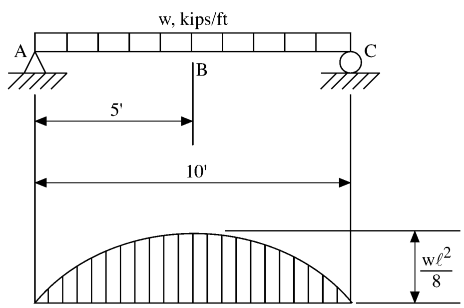


FIGURE 7.25: Example 7.4. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

Equation 7.25,

$$\begin{aligned}
M_e &= \pi^2 E C_b \frac{d I_{yc}}{L^2} \\
&= \pi^2 (29{,}500)(1.30) \frac{(8)(0.724)}{(5 \times 12)^2} = 608.96 \quad \text{in.-kips} \\
M_y &= S_f F_y = (6.54)(50) = 327.0 \quad \text{in.-kips} \\
0.56 M_y &= 183.12 \quad \text{in.-kips} \\
2.78 M_y &= 909.06 \quad \text{in.-kips}
\end{aligned}$$

Since 2.78 $M_y > M_e > 0.56 M_y$, from Equation 7.22,

$$\begin{aligned}
M_c &= \frac{10}{9} M_y \left(1 - \frac{10 M_y}{36 M_e}\right) \\
&= \frac{10}{9}(327.0)\left[1 - \frac{10(327.0)}{36(608.96)}\right] \\
&= 309.14 \quad \text{in.-kips}
\end{aligned}$$

Based on Equation 7.20, the nominal moment for lateral buckling strength is

$$M_n = S_c \frac{M_c}{S_f}$$

in which S_c is the elastic section modulus of the effective section calculated at a compressive stress of $f = M_c/S_f = 309.14/6.54 = 47.27$ ksi. By using the same procedure illustrated in Example 7.2, $S_c = 6.295$ in.^{3}. Therefore, the nominal moment for lateral buckling strength is

$$M_n = (6.295)\left(\frac{309.14}{6.54}\right) = 297.6 \quad \text{in.-kips}$$

For channels or Z-sections having the tension flange through-fastened to deck or sheathing with the compression flange laterally unbraced and loaded in a plane parallel to the web, the nominal flexural strength is determined by $M_n = RS_eF_y$, where R is a reduction factor [7]. A similar approach is used for beams having one flange fastened to a standing seam roof system.

Unusually Wide Beam Flanges and Short Span Beams

When beam flanges are unusually wide, special consideration should be given to the possible effects of shear lag and flange curling. Shear lag depends on the type of loading and the span-to-width ratio and is independent of the thickness. Flange curling is independent of span length but depends on the thickness and width of the flange, the depth of the section, and the bending stresses in both tension and compression flanges.

In order to consider the shear lag effects, the effective widths of both tension and compression flanges should be used according to the AISI Specification.

When a straight beam with unusually wide and thin flanges is subject to bending, the portion of the flange most remote from the web tends to deflect toward the neutral axis due to the effect of longitudinal curvature of the beam and the applied bending stresses in both flanges. For the purpose of controlling the excessive flange curling, the AISI Specification provides an equation to limit the flange width.

Shear Strength

The shear strength of beam webs is governed by either yielding or buckling of the web element, depending on the depth-to-thickness ratio, h/t, and the mechanical properties of steel. For beam webs having small h/t ratios, the nominal shear strength is governed by shear yielding. When the h/t ratio is large, the nominal shear strength is controlled by elastic shear buckling. For beam webs having moderate h/t ratios, the shear strength is based on inelastic shear buckling.

For the design of beam webs, the AISI Specification provides the following equations for determining the nominal shear strength:

For $h/t \leq 0.96\sqrt{Ek_v/F_y}$:

$$V_n = 0.60F_yht \tag{7.27}$$

For $0.96\sqrt{Ek_v/F_y} < h/t \leq 1.415\sqrt{Ek_v/F_y}$:

$$V_n = 0.64t^2\sqrt{k_vF_yE} \tag{7.28}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$:

$$V_n = \pi^2Ek_vt^3/[12(1-\mu^2)h] = 0.905Ek_vt^3/h \tag{7.29}$$

where

V_n = nominal shear strength of beam
h = depth of the flat portion of the web measured along the plane of the web
t = web thickness
k_v = shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the AISI requirements
   when $a/h \leq 1.0$:
$$k_v = 4.00 + \frac{5.34}{(a/h)^2}$$
   when $a/h > 1.0$:
$$k_v = 5.34 + \frac{4.00}{(a/h)^2}$$

where

a = the shear panel length for unreinforced web element
  = the clear distance between transverse stiffeners for reinforced web elements

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

Combined Bending and Shear

For continuous beams and cantilever beams, high bending stresses often combine with high shear stresses at the supports. Such beam webs must be safeguarded against buckling due to the combination of bending and shear stresses. Based on the AISI Specification, the moment and shear should satisfy the interaction equations listed in Table 7.3.

TABLE 7.3 Interaction Equations Used for Combined Bending and Shear

	ASD	LRFD
Beams with unreinforced webs	$\left(\frac{M}{M_{axo}}\right)^2 + \left(\frac{V}{V_a}\right)^2 \leq 1.0$ (7.30)	$\left(\frac{M_u}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \leq 1.0$ (7.32)
Beams with transverse web stiffeners	$M \leq M_a$ and $V \leq V_a$ $0.6\left(\frac{M}{M_{axo}}\right) + \left(\frac{V}{V_a}\right) \leq 1.3$ (7.31)	$M_u \leq \phi_b M_n$ and $V_u \leq \phi_v V_n$ $0.6\left(\frac{M_u}{\phi_b M_{nxo}}\right) + \left(\frac{V_u}{\phi_v V_n}\right) \leq 1.3$ (7.33)

M = bending moment
M_a = allowable moment when bending alone exists
M_{axo} = allowable moment about the centroidal x-axis determined in accordance with the specification excluding the consideration of lateral buckling
V = unfactored shear force
V_a = allowable shear force when shear alone exists
ϕ_b = resistance factor for bending
ϕ_v = resistance factor for shear
M_n = nominal flexural strength when bending alone exists
M_{nxo} = nominal flexural strength about the centroidal x-axis determined in accordance with the specification excluding the consideration of lateral buckling
M_u = required flexural strength
V_n = nominal shear strength when shear alone exists
V_u = required shear strength

Web Crippling

For cold-formed steel beams, transverse stiffeners are not frequently used for beam webs. The webs may cripple due to the high local intensity of the load or reaction as shown in Figure 7.26. Because the theoretical analysis of web crippling is rather complex due to the involvement of many factors, the present AISI design equations are based on extensive experimental investigations under four loading conditions: (1) end one-flange (EOF) loading, (2) interior one-flange (IOF) loading,

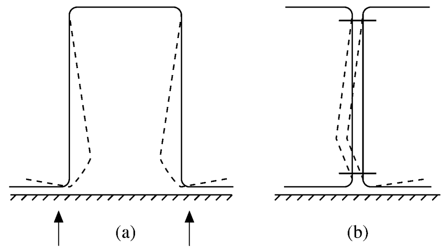


FIGURE 7.26: Web crippling of cold-formed steel beams.

(3) end two-flange (ETF) loading, and (4) interior two-flange (ITF) loading [29, 46, 50]. The loading conditions used for the tests are illustrated in Figure 7.27.

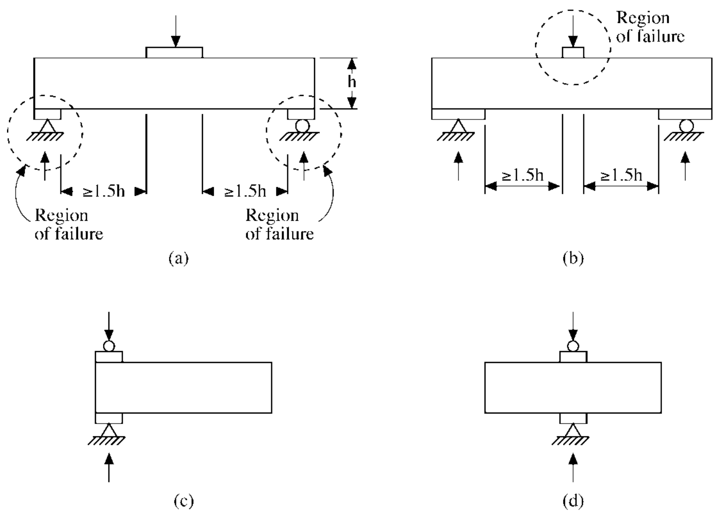


FIGURE 7.27: Loading conditions for web crippling tests. (a) EOF loading. (b) IOF loading. (c) ETF loading. (d) ITF loading. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

The nominal web crippling strength for a given loading condition can be determined from the AISI equations [7] on the basis of the thickness of web element, design yield stress, the bend radius-to-thickness ratio, the depth-to-thickness ratio, the bearing length-to-thickness ratio, and the angle between the plane of the web and the plane of the bearing surface. Tables 7.4a and Table 7.4b list the equations for determining the nominal web crippling strengths of one- and two-flange loading conditions, respectively.

Combined Bending and Web Crippling

For combined bending and web crippling, the design of beam webs should be based on the interaction equations provided in the AISI Specification [7]. These equations are presented in Table 7.5.

Table 7.4a Nominal Web Crippling Strength for One-Flange Loading, per Web, P_n

	Shapes having single webs	Shapes having single webs	I-Sections or similar sections
	Stiffened or partially stiffened flanges	Unstiffened flanges	Stiffened, partially stiffened, and unstiffened flanges
End reaction opposing loads spaced $> 1.5h$	$t^2kC_3C_4C_\theta[331 - 0.61(h/t)] \times[1 + 0.01(N/t)]C_9$ [a]	$t^2kC_3C_4C_\theta[217 - 0.28(h/t)] \times[1 + 0.01(N/t)]C_9$ [a,b]	$t^2F_yC_6(10.0 + 1.25\sqrt{N/t})$
Interior reactions opposing loads spaced $> 1.5h$	$t^2kC_1C_2C_\theta[538 - 0.74(h/t)] \times[1 + 0.007(N/t)]C_9$ [c]	$t^2kC_1C_2C_\theta[538 - 0.74(h/t)] \times[1 + 0.007(N/t)]C_9$ [c]	$t^2F_yC_5(0.88 + 0.12m) \times(15.0 + 3.25\sqrt{N/t})$

[a] When $F_y \geq 66.5$ ksi (459 MPa), the value of KC_3 shall be taken as 1.34.

[b] When $N/t > 60$, the factor $[1 + 0.01(N/t)]$ may be increased to $[0.71 + 0.015(N/t)]$

[c] When $N/t > 60$, the factor $[1 + 0.007(N/t)]$ may be increased to $[0.75 + 0.011(N/t)]$

C_1 = $(1.22 - 0.22k)$
C_2 = $(1.06 - 0.06R/t) \leq 1.0$
C_3 = $(1.33 - 0.33k)$
C_4 = $(1.15 - 0.15R/t) \leq 1.0$ but not less than 0.50
C_5 = $(1.49 - 0.53k) \geq 0.6$
C_6 = $1 + (h/t)/750$ when $h/t \leq 150$
= 1.20, when $h/t > 150$
C_9 = 1.0 for U.S. customary units, kips and in.
= 6.9 for SI units, N and mm
C_θ = $0.7 + 0.3(\theta/90)^2$
F_y = design yield stress of the web
h = depth of the flat portion of the web measured along the plane of the web
k = $894F_y/E$
m = $t/0.075$, when t is in in.
m = $t/1.91$, when t is in mm
t = web thickness
N = actual length of bearing
R = inside bend radius
θ = angle between the plane of the web and the plane of the bearing surface $\geq 45°$, but not more than 90°

Table 7.4b Nominal Web Crippling Strength for Two-Flange Loading, per Web, P_n

	Shapes having single webs	Shapes having single webs	I-Sections or similar sections
	Stiffened or partially stiffened flanges	Unstiffened flanges	Stiffened, partially stiffened, and unstiffened flanges
End reaction opposing loads spaced $\leq 1.5h$	$t^2kC_3C_4C_\theta[244 - 0.57(h/t)] \times[1 + 0.01(N/t)]C_9$ [a]	$t^2kC_3C_4C_\theta[244 - 0.57(h/t)] \times[1 + 0.01(N/t)]C_9$ [a]	$t^2F_yC_8(0.64 + 0.31m) \times(10.0 + 1.25\sqrt{N/t})$
Interior reaction opposing loads spaced $\leq 1.5h$	$t^2kC_1C_2C_\theta[771 - 2.26(h/t)] \times[1 + 0.0013(N/t)]C_9$	$t^2kC_1C_2C_\theta[771 - 2.26(h/t)] \times[1 + 0.0013(N/t)]C_9$	$t^2F_yC_7(0.82 + 0.15m) \times(15.0 + 3.25\sqrt{N/t})$

[a] When $F_y \geq 66.5$ ksi (459 MPa), the value of KC_3 shall be taken as 1.34.

C_7 = $1/k$, when $h/t \leq 66.5$
= $[1.10 - (h/t)/665]/k$, when $h/t > 66.5$
C_8 = $[0.98 - (h/t)/865]/k$
$C_1, C_2, C_3, C_4, C_9, C_\theta, F_y, h, k, m, t, N, R$, and θ are defined in Table 7.4a.

TABLE 7.5 Interaction Equations for Combined Bending and Web Crippling

	ASD	LRFD
Shapes having single unreinforced webs	$1.2\left(\frac{P}{P_a}\right)+\left(\frac{M}{M_{axo}}\right)\leq 1.5$ (7.34)	$1.07\left(\frac{P_u}{\phi_w P_n}\right)+\left(\frac{M_u}{\phi_b M_{nxo}}\right)\leq 1.42$ (7.37)
Shapes having multiple unreinforced webs such as I-sections	$1.1\left(\frac{P}{P_a}\right)+\left(\frac{M}{M_{axo}}\right)\leq 1.5$ (7.35)	$0.82\left(\frac{P_u}{\phi_w P_n}\right)+\left(\frac{M_u}{\phi_b M_{nxo}}\right)\leq 1.32$ (7.38)
Support point of two nested Z-shapes	$\frac{M}{M_{no}}+\frac{P}{P_n}\leq 1.0$ (7.36)	$\frac{M_u}{M_{no}}+\frac{P_u}{P_n}\leq 1.68\phi$ (7.39)

Note: The AISI Specification includes some exception clauses, under which the effect of combined bending and web crippling need not be checked.

P = concentrated load or reaction in the presence of bending moment
P_a = allowable concentrated load or reaction in the absence of bending moment
P_n = nominal web crippling strength for concentrated load or reaction in the absence of bending moment (for Equations 7.34, 7.35, 7.37, and 7.38)
P_n = nominal web crippling strength assuming single web interior one-flange loading for the nested Z-sections, i.e., sum of the two webs evaluated individually (for Equations 7.36 and 7.39)
P_u = required strength for the concentrated load or reaction in the presence of bending moment
M = applied bending moment at, or immediately adjacent to, the point of application of the concentrated load or reaction
M_{axo} = allowable moment about the centroidal x-axis determined in accordance with the specification excluding the consideration of lateral buckling
M_{no} = nominal flexural strength for the nested Z-sections, i.e., the sum of the two sections evaluated individually, excluding lateral buckling
M_{nxo} = nominal flexural strength about the centroidal x-axis determined in accordance with the specification excluding the consideration of lateral buckling
M_u = required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction P_u
ϕ = resistance factor = 0.9
ϕ_b = resistance factor for bending
ϕ_w = resistance factor for web crippling

Bracing Requirements

In cold-formed steel design, braces should be designed to restrain lateral bending or twisting of a loaded beam and to avoid local crippling at the points of attachment. When channels and Z-shaped sections are used as beams and loaded in the plane of the web, the AISI Specification [7] provides design requirements to restrain twisting of the beam under the following two conditions: (1) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange, and (2) neither flange is connected to sheathing. In general, braces should be designed to satisfy the strength and stiffness requirements. For beams using symmetrical cross sections, such as I-beams, the AISI Specification does not provide specific requirements for braces. However, the braces may be designed for a capacity of 2% of the force resisted by the compression portion of the beam. This is a frequently used rule of thumb but is a conservative approach, as proven by a rigorous analysis.

7.6.5 Concentrically Loaded Compression Members

Axially loaded cold-formed steel compression members should be designed for the following limit states: (1) yielding, (2) overall column buckling (flexural buckling, torsional buckling, or torsional-flexural buckling), and (3) local buckling of individual elements. The governing failure mode depends on the configuration of the cross-section, thickness of material, unbraced length, and end restraint.

Yielding

A very short, compact column under axial load may fail by yielding. For this case, the nominal axial strength is the yield load, i.e.,

$$P_n = P_y = AF_y \tag{7.40}$$

where A is the full cross-sectional area of the column and F_y is the yield point of steel.

Overall Column Buckling

Overall column buckling may be one of the following three types:

1. Flexural buckling — bending about a principal axis. The elastic flexural buckling stress is

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \tag{7.41}$$

where

E = modulus of elasticity
K = effective length factor for flexural buckling (Figure 7.28)
L = unbraced length of member for flexural buckling
r = radius of gyration of the full section

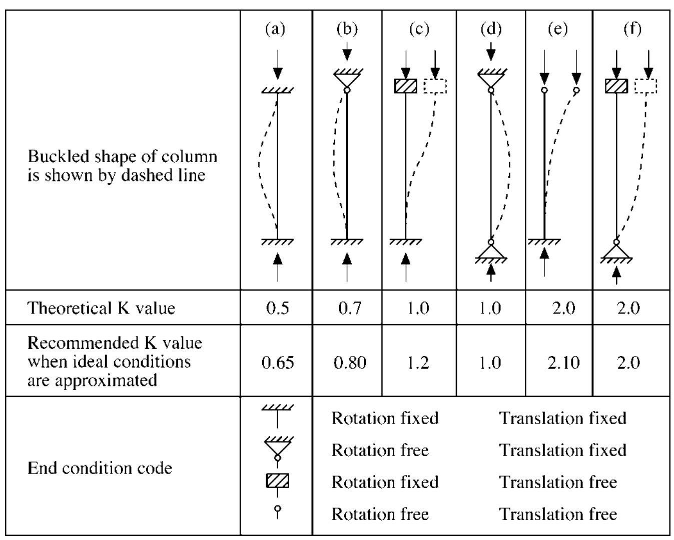


FIGURE 7.28: Effective length factor K for concentrically loaded compression members.

2. Torsional buckling — twisting about shear center. The elastic torsional buckling stress is

$$F_e = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \tag{7.42}$$

where

A = full cross-sectional area

C_w = torsional warping constant of the cross-section
G = shear modulus
J = St. Venant torsion constant of the cross-section
K_t = effective length factor for twisting
L_t = unbraced length of member for twisting
r_0 = polar radius of gyration of the cross-section about shear center

3. Torsional-flexural buckling — bending and twisting simultaneously. The elastic torsional-flexural buckling stress is

$$F_e = [(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}]/2\beta \tag{7.43}$$

where
β = $1 - (x_0/r_0)^2$
σ_{ex} = $\pi^2 E/(K_x L_x/r_x)^2$
σ_t = the same as Equation 7.42
x_0 = distance from shear center to the centroid along the principal x-axis

For doubly-symmetric and point-symmetric shapes (Figure 7.29), the overall column buckling can be either flexural type or torsional type. However, for singly-symmetric shapes (Figure 7.30), the overall column buckling can be either flexural buckling or torsional-flexural buckling.

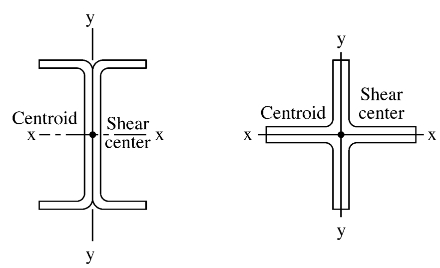


FIGURE 7.29: Doubly-symmetric shapes.

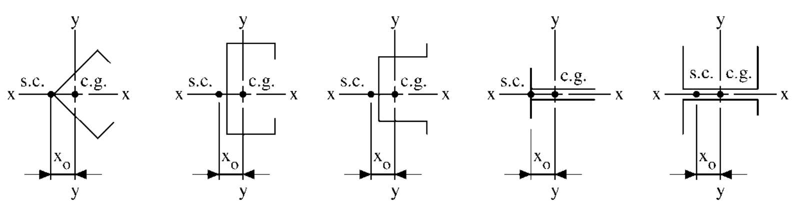


FIGURE 7.30: Singly-symmetric shapes. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

For overall column buckling, the nominal axial strength is determined by Equation 7.44:

$$P_n = A_e F_n \tag{7.44}$$

where

A_e = effective area determined for the stress F_n

F_n = nominal buckling stress determined as follows:

For $\lambda_c \leq 1.5$:

$$F_n = (0.658^{\lambda_c^2})F_y \tag{7.45}$$

For $\lambda_c > 1.5$:

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \tag{7.46}$$

The use of the effective area A_e in Equation 7.44 is to reflect the effect of local buckling on the reduction of column strength. In Equations 7.45 and 7.46,

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

in which F_e is the least of elastic flexural buckling stress (Equation 7.41), torsional buckling stress (Equation 7.42), and torsional-flexural buckling stress (Equation 7.43), whichever is applicable.

For the design of compression members, the slenderness ratio should not exceed 200, except that during construction, KL/r preferably should not exceed 300.

For nonsymmetric shapes whose cross-sections do not have any symmetry, either about an axis or about a point, the elastic torsional-flexural buckling stress should be determined by rational analysis or by tests. See AISI Design Manual [8].

In addition to the above design provisions for the design of axially loaded columns, the AISI Specification also provides design criteria for compression members having one flange through-fastened to deck or sheathing.

EXAMPLE 7.5:

Determine the allowable axial load for the square tubular column shown in Figure 7.31. Assume that $F_y = 40$ ksi, $K_xL_x = K_yL_y = 10$ ft, and the dead-to-live load ratio is 1/5. Use the ASD and LRFD methods.

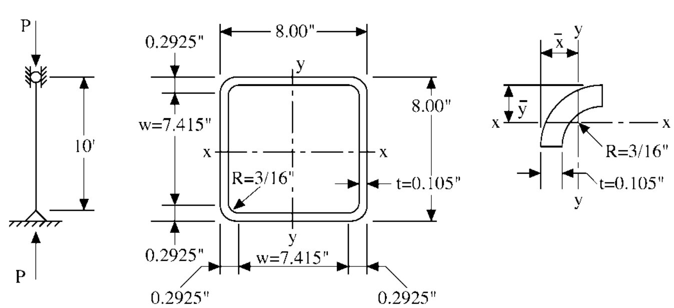


FIGURE 7.31: Example 7.5. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

Solution

(A) ASD Method

Since the square tube is a doubly-symmetric closed section, it will not be subject to torsional-flexural buckling. It can be designed for flexural buckling.

1. Sectional Properties of Full Section

$$
\begin{aligned}
w &= 8.00 - 2(R+t) = 7.415 \quad \text{in.} \\
A &= 4(7.415 \times 0.105 + 0.0396) = 3.273 \quad \text{in.}^2 \\
I_x &= I_y = 2(0.105)[(1/12)(7.415)^3 + 7.415(4 - 0.105/2)^2] + 4(0.0396)(4.0 - 0.1373)^2 \\
&= 33.763 \quad \text{in.}^4 \\
r_x &= r_y = \sqrt{I_x/A} = \sqrt{33.763/3.273} = 3.212 \quad \text{in.}
\end{aligned}
$$

2. Nominal Buckling Stress, F_n. According to Equation 7.41, the elastic flexural buckling stress, F_e, is computed as follows:

$$
\begin{aligned}
\frac{KL}{r} &= \frac{10 \times 12}{3.212} = 37.36 < 200 \quad \text{O.K.} \\
F_e &= \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29500)}{(37.36)^2} = 208.597 \quad \text{ksi} \\
\lambda_c &= \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{40}{208.597}} = 0.438 < 1.5 \\
F_n &= (0.658^{\lambda_c^2})F_y = (0.658^{0.438^2})40 = 36.914 \quad \text{ksi}
\end{aligned}
$$

3. Effective Area, A_e. Because the given square tube is composed of four stiffened elements, the effective width of stiffened elements subjected to uniform compression can be computed from Equations 7.4 through 7.7 by using $k = 4.0$:

$$
\begin{aligned}
w/t &= 7.415/0.105 = 70.619 \\
\lambda &= \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{F_n}{E}} \\
\lambda &= 1.052/\sqrt{4}(70.619)\sqrt{36.914/29{,}500} = 1.314
\end{aligned}
$$

Since $\lambda > 0.673$, from Equation 7.5,

$$b = \rho_w$$

where

$$\rho = (1 - 0.22/\lambda)/\lambda = (1 - 0.22/1.314)/1.314 = 0.634$$

Therefore, $b = (0.634)(7.415) = 4.701$ in.

The effective area is

$$A_e = 3.273 - 4(7.415 - 4.701)(0.105) = 2.133 \quad \text{in.}^2$$

4. Nominal and Allowable Loads. Using Equation 7.44, the nominal load is

$$P_n = A_e F_n = (2.133)(36.914) = 78.738 \quad \text{kips}$$

The allowable load is

$$P_a = P_n/\Omega_c = 78.738/1.80 = 43.74 \quad \text{kips}$$

(B) LRFD Method

In Item (A) above, the nominal axial load, P_n, was computed to be 78.738 kips. The design axial load for the LRFD method is

$$\phi_c P_n = 0.85(78.738) = 66.93 \quad \text{kips.}$$

Based on the load combination of dead and live loads, the required axial load is

$$P_u = 1.2P_D + 1.6P_L = 1.2P_D + 1.6(5P_D) = 9.2P_D$$

where

P_D = axial load due to dead load

P_L = axial load due to live load

By using $P_u = \phi_c P_n$, the values of P_D and P_L are computed as follows:

$$\begin{aligned} P_D &= 66.93/9.2 = 7.28 \quad \text{kips} \\ P_L &= 5P_D = 36.40 \quad \text{kips} \end{aligned}$$

Therefore, the allowable axial load is

$$P_a = P_D + P_L = 43.68 \quad \text{kips}$$

It can be seen that the allowable axial loads determined by the ASD and LRFD methods are practically the same.

7.6.6 Combined Axial Load and Bending

The AISI Specification provides interaction equations for combined axial load and bending.

Combined Tensile Axial Load and Bending

For combined tensile axial load and bending, the required strengths should satisfy the interaction equations presented in Table 7.6. These equations are to prevent yielding of the tension flange and to prevent failure of the compression flange of the member.

Combined Compressive Axial Load and Bending

Cold-formed steel members under combined compressive axial load and bending are usually referred to as beam-columns. Such members are often found in framed structures, trusses, and exterior wall studs. For the design of these members, the required strengths should satisfy the AISI interaction equations presented in Table 7.7.

7.6.7 Cylindrical Tubular Members

Thin-walled cylindrical tubular members are economical sections for compression and torsional members because of their large ratio of radius of gyration to area, the same radius of gyration in all directions, and the large torsional rigidity. The AISI design provisions are limited to the ratio of outside diameter-to-wall thickness, D/t, not greater than 0.441 E/F_y.

TABLE 7.6 Interaction Equations for Combined Tensile Axial Load and Bending

	ASD	LRFD
Check tension flange	$\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} + \frac{\Omega_t T}{T_n} \leq 1.0$ (7.47)	$\frac{M_{ux}}{\phi_b M_{nxt}} + \frac{M_{uy}}{\phi_b M_{nyt}} + \frac{T_u}{\phi_t T_n} \leq 1.0$ (7.49)
Check compression flange	$\frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} + \frac{\Omega_t T}{T_n} \leq 1.0$ (7.48)	$\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} - \frac{T_u}{\phi_t T_n} \leq 1.0$ (7.50)

M_{nx}, M_{ny} = nominal flexural strengths about the centroidal x- and y-axes
M_{nxt}, M_{nyt} = $S_{ft} F_y$
M_{ux}, M_{uy} = required flexural strengths with respect to the centroidal axes
M_x, M_y = required moments with respect to the centroidal axes of the section
S_{ft} = section modulus of the full section for the extreme tension fiber about the appropriate axis
T = required tensile axial load
T_n = nominal tensile axial strength
T_u = required axial strength
ϕ_b = resistance factor for bending
ϕ_t = resistance factor for tension
Ω_b = safety factor for bending
Ω_t = safety factor for tension

TABLE 7.7 Interaction Equations for Combined Compressive Axial Load and Bending

ASD	LRFD
when $\Omega_c P/P_n \leq 0.15$,	when $P_u/\phi_c P_n \leq 0.15$,
$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0$ (7.51)	$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$ (7.54)
when $\Omega_c P/P_n > 0.15$,	when $P_u/\phi_c P_n > 0.15$,
$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \leq 1.0$ (7.52)	$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx}\alpha_x} + \frac{C_{my} M_{uy}}{\phi_b M_{ny}\alpha_y} \leq 1.0$ (7.55)
$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0$ (7.53)	$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$ (7.56)

M_x, M_y = required moments with respect to the centroidal axes of the effective section determined for the required axial strength alone
M_{nx}, M_{ny} = nominal flexural strengths about the centroidal axes
M_{ux}, M_{uy} = required flexural strengths with respect to the centroidal axes of the effective section determined for the required axial strength alone
P = required axial load
P_n = nominal axial strength determined in accordance with Equation 7.44
P_{no} = nominal axial strength determined in accordance with Equation 7.44, for $F_n = F_y$
P_u = required axial strength
α_x = $1 - \Omega_c P/P_{EX}$ (for Equation 7.52)
α_y = $1 - \Omega_c P/P_{EY}$ (for Equation 7.52)
α_x = $1 - P_u/P_{EX}$ (for Equation 7.55)
α_y = $1 - P_u/P_{EY}$ (for Equation 7.55)
P_{EX} = $\pi^2 EI_x/(K_x L_x)^2$
P_{EY} = $\pi^2 EI_y/(K_y L_y)^2$
Ω_b = safety factor for bending
Ω_c = safety factor for concentrically loaded compression
C_{mx}, C_{my} = coefficients whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway) $C_m = 0.85$
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending $C_m = 0.6 - 0.4\,(M_1/M_2)$, where M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature
3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used: (a) for members whose ends are restrained, $C_m = 0.85$; (b) for members whose ends are unrestrained, $C_m = 1.0$

I_x, I_y, L_x, L_y, K_x, and K_y were defined previously.

Bending Strength

For cylindrical tubular members subjected to bending, the nominal flexural strengths are as follows according to the D/t ratio:

1. For $D/t \leq 0.070E/F_y$:

$$M_n = 1.25 F_y S_f \tag{7.57}$$

2. For $0.070E/F_y < D/t \leq 0.319E/F_y$:

$$M_n = [0.970 + 0.020(E/F_y)/(D/t)]F_y S_f \tag{7.58}$$

3. For $0.319E/F_y < D/t \leq 0.441E/F_y$:

$$M_n = [0.328E/(D/t)]S_f \tag{7.59}$$

where

S_f = elastic section modulus of the full, unreduced cross-section

Other symbols were defined previously.

Compressive Strength

When cylindrical tubes are used as concentrically loaded compression members, the nominal axial strength is determined by Equation 7.44, except that (1) the elastic buckling stress, F_e, is determined for flexural buckling by using Equation 7.41 and (2) the effective area, A_e, is calculated by Equation 7.60.

$$A_e = [1 - (1 - R^2)(1 - A_0/A)]A \tag{7.60}$$

where

R = $\sqrt{F_y/2F_e}$

A_0 = $\{0.037/[(DF_y)/(tE)] + 0.667\}A \leq A$

A = area of the unreduced cross-section

In the above equations, the value A_0 is the reduced area due to the effect of local buckling [8, 49].

7.7 Connections and Joints

Welds, bolts, screws, rivets, and other special devices such as metal stitching and adhesives are generally used for cold-formed steel connections. The AISI Specification contains only the design provisions for welded connections, bolted connections, and screw connections. These design equations are based primarily on the experimental data obtained from extensive test programs.

7.7.1 Welded Connections

Welds used for cold-formed steel construction may be classified as arc welds (or fusion welds) and resistance welds. Arc welding is usually used for connecting cold-formed steel members to each other as well as connecting such thin members to heavy, hot-rolled steel framing members. It is used for groove welds, arc spot welds, arc seam welds, fillet welds, and flare groove welds. The AISI design provisions for welded connections are applicable only for cold-formed steel structural members, in which the thickness of the thinnest connected part is 0.18 in. (4.57 mm) or less. Otherwise, when the thickness of connected parts is thicker than 0.18 in. (4.57 mm), the welded connection should be designed according to the AISC Specifications [1, 2]. Additional design information on structural welding of sheet steels can also be found in the AWS Code [16].

Arc Welds

According to the AISI Specification, the nominal strengths of arc welds can be determined from the equations given in Table 7.8. The design strengths can then be computed by using the safety factor or resistance factor provided in Table 7.1.

TABLE 7.8 Nominal Strength Equations for Arc Welds

Type of weld	Type of strength	Nominal strength P_n (kips)
Groove welds (Figure 7.32)	• Tension or compression	$Lt_e F_y$
	• Shear strength of weld	$Lt_e(0.6F_{xx})$
	• Shear strength of connected part	$Lt_e(F_y/\sqrt{3})$
Arc spot welds (Figure 7.33)	• Shear strength	
	Strength of weld	$0.589\, d_e^2 F_{xx}$
	Strength of connected part	
	1. $d_a/t \leq 0.815\sqrt{E/F_u}$	$2.20td_a F_u$
	2. $0.815\sqrt{E/F_u} < (d_a/t) < 1.397\sqrt{E/F_u}$	$0.28[1 + (5.59\sqrt{E/F_u})/(d_a/t)]td_a F_u$
	3. $d_a/t \geq 1.397\sqrt{E/F_u}$	$1.40td_a F_u$
	Shear strength of connected part based on end distance	$eF_u t$
	•Tensile strength	
	Strength of weld	$0.785d_e^2 F_{xx}$
	Strength of connected part	
	1.$F_u/E < 0.00187$	$[6.59 - 3150(F_u/E)](td_a F_u)$ $\leq 1.46td_a F_u$
	2. $F_u/E \geq 0.00187$	$0.70td_a F_u$
Arc seam welds (Figure 7.34)	• Shear strength	$[\pi d_e^2/4 + Ld_e]0.75F_{xx}$
	• Strength of connected part	$2.5tF_u(0.25L + 0.96d_a)$
Fillet welds (Figure 7.35)	• Shear strength of weld (for $t > 0.15$ in.)	$0.75t_w LF_{xx}$
	• Strength of connected part	
	1. Longitudinal loading	
	$L/t < 25$:	$[1-(0.01L/t)]tLF_u$
	$L/t \geq 25$:	$0.75tLF_u$
	2. Transverse loading	tLF_u
Flare groove welds (Figure 7.36)	• Shear strength of weld (for $t > 0.15$ in.)	$0.75t_w LF_{xx}$
	• Strength of connected part	
	1. Transverse loading	$0.833tLF_u$
	2. Longitudinal loading	
	For $t \leq t_w < 2t$ or if lip height $< L$	$0.75tLF_u$
	For $t_w \geq 2\,t$ and lip height $\geq L$	$1.50tLF_u$

d = visible diameter of outer surface of arc spot weld
d_a = average diameter of the arc spot weld at mid-thickness of t
d_a = $(d - t)$ for single sheet
d_a = $(d-2\,t)$ for multiple sheets
d_e = effective diameter of fused area at plane of maximum shear transfer
d_e = $0.7d$ — $1.5\,t \leq 0.55d$
e = 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
F_u = tensile strength of the connected part
F_y = yield point of steel
F_{xx} = filler metal strength designation in AWS electrode classification
L = length of weld
P_n = nominal strength of weld
t = thickness of connected sheet
t_w = effective throat dimension for groove weld, see AISI specification
t_w = effective throat $= 0.707\, w_1$ or $0.707\, w_2$, whichever is smaller
w_1 = leg of weld
w_2 = leg of weld

See AISI Specification for additional design information.

Resistance Welds

The nominal shear strengths of resistance welds are provided in the AISI Specification [7] according to the thickness of the thinnest outside sheet. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.9 oz/ft^2 of sheet or less, and medium carbon and low-alloy steels.

7.7.2 Bolted Connections

Due to the thinness of the connected parts, the design of bolted connections in cold-formed steel construction is somewhat different from that in hot-rolled heavy construction. The AISI design provisions are applicable only to cold-formed members or elements less than 3/16 in. (4.76 mm) in thickness. For materials not less than 3/16 in. (4.76 mm), the bolted connection should be designed in accordance with the AISC Specifications [1, 2].

In the AISI Specification, five types of bolts (A307, A325, A354, A449, and A490) are used for connections in cold-formed steel construction, in which A449 and A354 bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever bolts with smaller than 1/2-in. diameters are required.

On the basis of the failure modes occurring in the tests of bolted connections, the AISI criteria deal with three major design considerations for the connected parts: (1) longitudinal shear failure, (2) tensile failure, and (3) bearing failure. The nominal strength equations are given in Table 7.9.

TABLE 7.9 Nominal Strength Equations for Bolted Connections

Type of strength	Nominal strength, P_n
Shear strength based on spacing and edge distance	teF_u
Tensile strength	
1. With washers under bolt head and nuts	$(1 - 0.9\,r + 3rd/s)F_u A_n \le F_u A_n$
2. No washers or only one washer under bolt head and nuts	$(1 - r + 2.5\,rd/s)F_u A_n \le F_u A_n$
Note: The tensile strength computed above should not exceed $A_n F_y$.	
Bearing strength	
1. With washers under bolt head and nut	
• Inside sheet of double shear connection	
$F_u/F_y \ge 1.08$	$3.33F_u dt$
$F_u/F_y < 1.08$	$3.00F_u dt$
• Single shear and outside sheets of double shear connection	$3.00F_u dt$
2. Without washers under bolt head and nut or with only one washer	
• Inside sheet of double shear connection	$3.00F_u dt$
• Single shear and outside sheets of double shear connection	$2.22F_u dt$

A_n	=	net area of the connected part
d	=	diameter of bolt
e	=	distance measured in the line of force from the center of bolt to the nearest edge of an adjacent hole or to the end of the connected part
F_u	=	tensile strength of the connected part
F_y	=	specified yield point of steel
r	=	force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero
s	=	spacing of bolts perpendicular to line of force
t	=	thickness of thinnest connected part

In addition, design strength equations are provided for shear and tension in bolts. Accordingly, the AISI nominal strength for shear and tension in bolts can be determined as follows:

$$P_n = A_b F$$

where

A_b = gross cross-sectional area of bolt

F = nominal shear or tensile stress given in Table 7.10.

For bolts subjected to the combination of shear and tension, the reduced nominal tension stress is given in Table 7.11.

TABLE 7.10 Nominal Tensile and Shear Stresses for Bolts

Description of bolts	Nominal tensile stress F_{nt}, ksi	Nominal shear stress F_{nv}, ksi
A307 Bolts, Grade A, 1/4 in. $\leq d <$ 1/2 in.	40.5	24.0
A307 Bolts, Grade A, $d \geq$ 1/2 in.	45.0	27.0
A325 Bolts, when threads are not excluded from shear planes	90.0	54.0
A325 Bolts, when threads are excluded from shear planes	90.0	72.0
A354 Grade BD Bolts, 1/4 in. $\leq d <$ 1/2 in., when threads are not excluded from shear planes	101.0	59.0
A354 Grade BD Bolts, 1/4 in. $\leq d <$ 1/2 in. when threads are excluded from shear planes	101.0	90.0
A449 Bolts, 1/4 in. $\leq d <$ 1/2 in., when threads are not excluded from shear planes	81.0	47.0
A449 Bolts, 1/4 in. $\leq d <$ 1/2 in., when threads are excluded from shear planes	81.0	72.0
A 490 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 7.11 Nominal Tension Stresses, F'_{nt} (ksi), for Bolts Subjected to the Combination of Shear and Tension

Description of bolts	Threads not excluded from shear planes	Threads excluded from shear planes
(A) ASD Method		
A325 Bolts	$110 - 3.6f_v \leq 90$	$110 - 2.8f_v \leq 90$
A354 Grade BD Bolts	$122 - 3.6f_v \leq 101$	$122 - 2.8f_v \leq 101$
A449 Bolts	$100 - 3.6f_v \leq 81$	$100 - 2.8f_v \leq 81$
A490 Bolts	$136 - 3.6f_v \leq 112.5$	$136 - 2.8f_v \leq 112.5$
A307 Bolts, Grade A		
When 1/4 in. $\leq d <$ 1/2 in.	$52 - 4f_v \leq 40.5$	$52 - 4f_v \leq 40.5$
When $d \geq$ 1/2 in.	$58.5 - 4f_v \leq 45$	$58.5 - 4f_v \leq 45$
(B) LRFD Method		
A325 Bolts	$113 - 2.4f_v \leq 90$	$113 - 1.9f_v \leq 90$
A354 Grade BD Bolts	$127 - 2.4f_v \leq 101$	$127 - 1.9f_v \leq 101$
A449 Bolts	$101 - 2.4f_v \leq 81$	$101 - 1.9f_v \leq 81$
A490 Bolts	$141 - 2.4f_v \leq 112.5$	$141 - 1.9f_v \leq 112.5$
A307 Bolts, Grade A		
When 1/4 in. $\leq d <$ 1/2 in.	$47 - 2.4f_v \leq 40.5$	$47 - 2.4f_v \leq 40.5$
When $d \geq$ 1/2 in.	$52 - 2.4f_v \leq 45$	$52 - 2.4f_v \leq 45$

d = diameter of bolt

f_v = shear stress based on gross cross-sectional area of bolt

7.7.3 Screw Connections

Screws can provide a rapid and effective means to fasten sheet metal siding and roofing to framing members and to connect individual siding and roofing panels. Design equations are presently given in the AISI Specification for determining the nominal shear strength and the nominal tensile strength of connected parts and screws. These design requirements should be used for self-tapping screws with diameters larger than or equal to 0.08 in. (2.03 mm) but not exceeding 1/4 in. (6.35 mm). The screw can be thread-forming or thread-cutting, with or without drilling point. The spacing between the centers of screws and the distance from the center of a screw to the edge of any part in the direction

TABLE 7.12 Nominal Strength Equations for Screws

Type of strength	Nominal strength
Shear strength	
1. Connection shear	
• For $t_2/t_1 \leq 1.0$: use smallest of three considerations	a. $P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2}$ b. $P_{ns} = 2.7\, t_1 d F_{u1}$ c. $P_{ns} = 2.7\, t_2 d F_{u2}$
• For $t_2/t_1 \geq 2.5$: use smaller of two considerations	a. $P_{ns} = 2.7\, t_1 d F_{u1}$ b. $P_{ns} = 2.7\, t_2 d F_{u2}$
• For $1.0 < t_2/t_1 < 2.5$: use linear interpolation	
2. Shear in screws	$\geq 1.25\, P_{ns}$
Tensile strength	
1. Connection tension	
Pull-out strength	$P_{not} = 0.85\, t_c d F_{u2}$
Pull-over strength	$P_{nov} = 1.5\, t_1 d_w F_{u1}$
2. Tension in screws	$P_{nt} \geq 1.25$ (lesser of P_{not} and P_{nov}

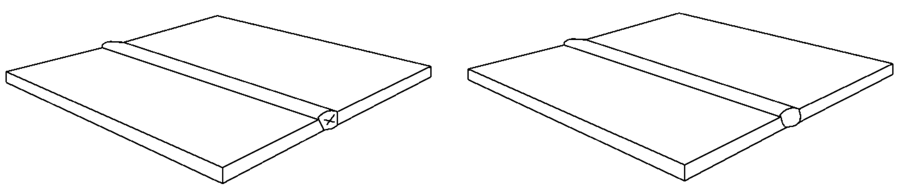
d = diameter of screw
d_w = larger of the screw head diameter or the washer diameter, and should be taken not larger than 1/2 in. (12.7 mm)
F_{u1} = tensile strength of member in contact with the screw head
F_{u2} = tensile strength of member not in contact with the screw head
P_{ns} = nominal shear strength per screw
P_{nt} = nominal tension strength per screw
P_{not} = nominal pull-out strength per screw
P_{nov} = nominal pull-over strength per screw
t_1 = thickness of member in contact with the screw head
t_2 = thickness of member not in contact with the screw head

of the force should not be less than three times the diameter.

According to the AISI Specification, the nominal strength per screw is determined from Table 7.12. See Figures 7.37 and 7.38 for t_1, t_2, F_{u1}, and F_{u2}.

For the convenience of designers, the following table gives the correlation between the common number designation and the nominal diameter for screws.

Number designation	Nominal diameter, d (in.)
0	0.060
1	0.073
2	0.086
3	0.099
4	0.112
5	0.125
6	0.138
7	0.151
8	0.164
10	0.190
12	0.216
1/4	0.250



(a) (b)

FIGURE 7.32: Groove welds.

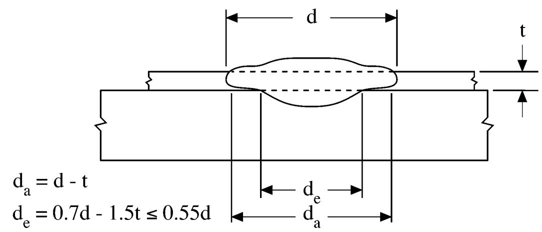


FIGURE 7.33: Arc spot weld — single thickness of sheet.

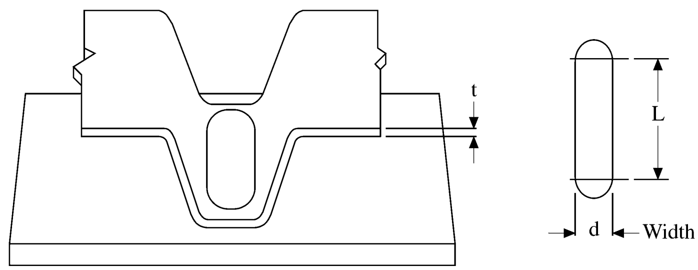


FIGURE 7.34: Arc seam weld.

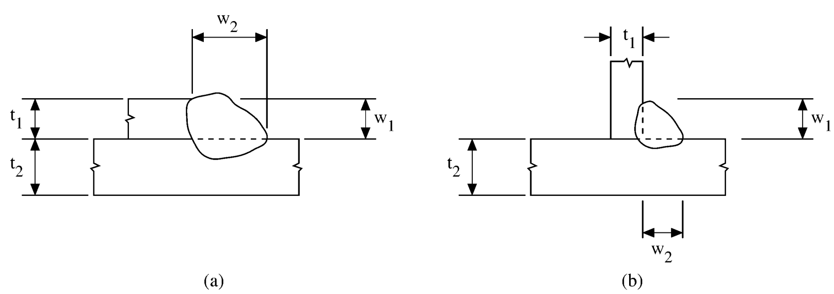


FIGURE 7.35: Fillet welds.

In addition to the design requirements discussed above, the AISI Specification also includes some provisions for spacing of connectors when two channels are connected to form an I-section or when compression elements are joined to other parts of built-up members by intermittent connections.

7.8 Structural Systems and Assemblies

In the past, cold-formed steel components have been used in different structural systems and assemblies such as metal buildings, shear diaphragms, shell roof structures, wall stud assemblies, residential construction, and composite construction.

7.8.1 Metal Buildings

Standardized metal buildings have been widely used in industrial, commercial, and agricultural applications. This type of metal building has also been used for community facilities because it

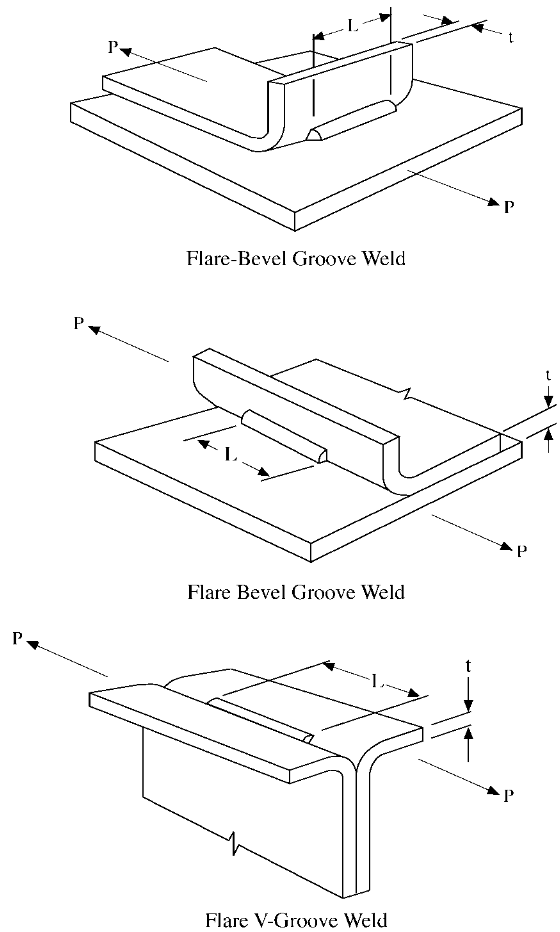


FIGURE 7.36: Flare groove welds.

can provide attractive appearance, fast construction, low maintenance, easy extension, and lower long-term cost.

In general, metal buildings are made of welded rigid frames with cold-formed steel sections used for purlins, girts, roofs, and walls. In the U.S., the design of standardized metal buildings is often based on the Low Rise Building Systems published by the Metal Building Manufacturers Association [33]. This document contains design practices, commentary, common industry practices, guide specifications, and nomenclature for metal building systems. In other countries, many design concepts and building systems have been developed.

7.8.2 Shear Diaphragms

In building construction, it has been a common practice to provide a separate bracing system to resist horizontal loads due to wind load or earthquake. However, steel floor and roof panels, with or without concrete fill, are capable of resisting horizontal loads in addition to the beam strength for gravity loads if they are adequately interconnected to each other and to the supporting frame. For the same reason, wall panels can provide not only enclosure surfaces and support normal loads, but they can also provide diaphragm action in their own planes.

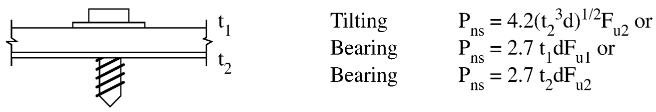


FIGURE 7.37: Screw connection for $t_2/t_1 \leq 1.0$.

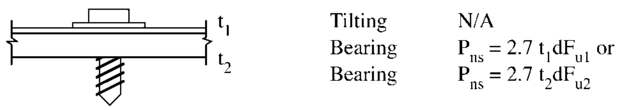


FIGURE 7.38: Screw connection for $t_2/t_1 \geq 2.5$.

The structural performance of a diaphragm construction can be evaluated by either calculations or tests. Several analytical procedures exist, and are summarized in the literature [3, 18, 22, 32]. Tested performance can be measured by the procedures of the Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, ASTM E455 [13]. A general discussion of structural diaphragm behavior is given by Yu [49].

Shear diaphragms should be designed for both strength and stiffness. After the nominal shear strength is established by calculations or tests, the design strength can be determined on the basis of the safety factor or resistance factor given in the Specification. Six cases are classified in the AISI Specification for the design of shear diaphragms according to the type of failure mode, connection, and loading. Because the quality of mechanical connectors is easier to control than welded connections, a relatively smaller safety factor or larger resistance factor is used for mechanical connections. As far as the loading is concerned, the safety factors for earthquake are slightly larger than those for wind due to the ductility demands required by seismic loading.

7.8.3 Shell Roof Structures

Shell roof structures such as folded-plate and hyperbolic paraboloid roofs have been used in building construction for churches, auditoriums, gymnasiums, schools, restaurants, office buildings, and airplane hangars. This is because the effective use of steel panels in roof construction is not only to provide an economical structure but also to make the building architecturally attractive and flexible for future extension. The design methods used in engineering practice are mainly based on the successful investigation of shear diaphragms and the structural research on shell roof structures.

A folded-plate roof structure consists of three major components. They are (1) steel roof panels, (2) fold line members at ridges and valleys, and (3) end frame or end walls as shown in Figure 7.39. Steel roof panels can be designed as simply supported slabs in the transverse direction between fold lines. The reaction of the panels is then applied to fold lines as a line loading, which can be resolved into two components parallel to the two adjacent plates. These load components are carried by an inclined deep girder spanned between end frames or end walls. These deep girders consist of fold line members as flanges and steel panels as a web element. The longitudinal flange force in fold line members can be obtained by dividing the bending moment of the deep girder by its depth. The shear force is resisted by the diaphragm action of the steel roof panels. In addition to the strength, the deflection characteristics of the folded-plate roof should also be investigated, particularly for long-span structures. In the past, it has been found that a method similar to the Williot diaphragm for determining truss deflections can also be used for the prediction of the deflection of a steel folded-plate roof. The in-plane deflection of each plate should be computed as a sum of the deflections due to flexure, shear, and seam slip, considering the plate temporarily separated from the adjacent

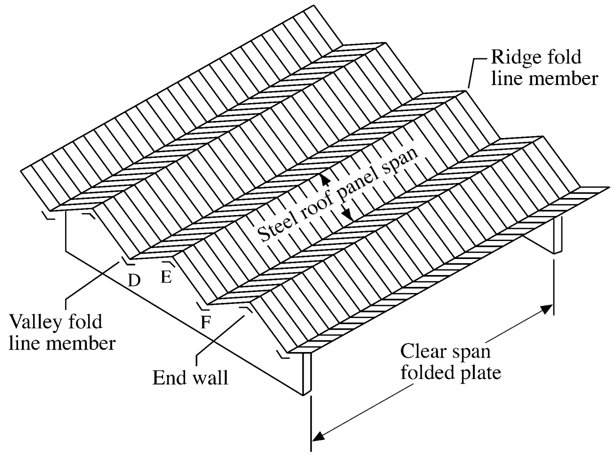


FIGURE 7.39: Folded-plate structure. (From Yu, W.W. 1991. *Cold-Formed Steel Design,* John Wiley & Sons, New York. With permission.)

plates. The true displacement of the fold line can then be determined analytically or graphically by a Williot diagram. The above discussion deals with a simplified method. The finite-element method can provide a more detailed analysis for various types of loading, support, and material.

The hyperbolic paraboloid roof has also gained popularity due to the economical use of materials and its appearance. This type of roof can be built easily with either single- or double-layer standard steel roof deck panels because hyperbolic paraboloid has straight line generators. Figure 7.40 shows four common types of hyperbolic paraboloid roofs which may be modified or varied in other ways to achieve a striking appearance. The method of analysis depends on the curvature of the shell used

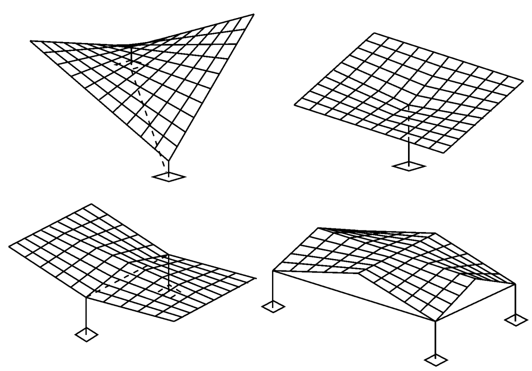


FIGURE 7.40: Types of hyperbolic paraboloid roofs. (From Yu, W.W. 1991. *Cold-Formed Steel Design,* John Wiley & Sons, New York. With permission.)

for the roof. If the uniformly loaded shell is deep, the membrane theory may be used. For the case of a shallow shell or a deep shell subjected to unsymmetrical loading, the finite-element method will provide accurate results. Using the membrane theory, the panel shear for a uniformly loaded hyperbolic paraboloid roof can be determined by $wab/2h$, in which w is the applied load per unit surface area, a and b are horizontal projections, and h is the amount of corner depression of the surface. This panel shear force should be carried by tension and compression framing members. For additional design information, see Yu [49].

7.8.4 Wall Stud Assemblies

Cold-formed steel I-, C-, Z-, or box-type studs are widely used in walls with their webs placed perpendicular to the wall surface. The walls may be made of different materials, such as fiber board, lignocellulosic board, plywood, or gypsum board. If the wall material is strong enough and there is adequate attachment provided between wall material and studs for lateral support of the studs, then the wall material can contribute to the structural economy by increasing the usable strength of the studs substantially.

The AISI Specification provides the requirements for two types of stud design. The first type is "All Steel Design", in which the wall stud is designed as an individual compression member neglecting the structural contribution of the attached sheathing. The second type is "Sheathing Braced Design", in which consideration is given to the bracing action of the sheathing material due to the shear rigidity and the rotational restraint provided by the sheathing. Both solid and perforated webs are permitted. The subsequent discussion deals with the sheathing braced design of wall studs.

Wall Studs in Compression

The AISI design provisions are used to prevent three possible modes of failure. The first requirement is for column buckling between fasteners in the plane of the wall (Figure 7.41). For this case, the limit state may be either (1) flexural buckling, (2) torsional buckling, or (3) torsional-flexural buckling depending on the geometric configuration of the cross-section and the spacing of fasteners. The nominal compressive strength is based on the stud itself without considering any interaction with the sheathing material.

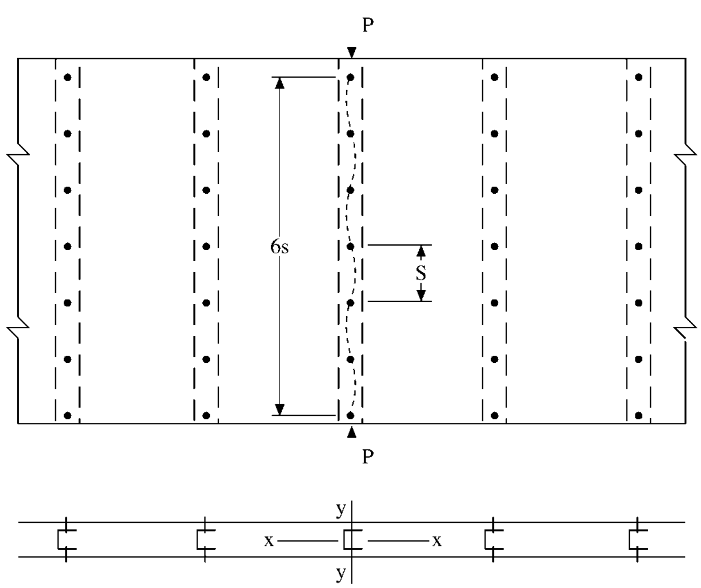


FIGURE 7.41: Buckling of studs between fasteners. (From Yu, W.W. 1991. *Cold-Formed Steel Design*, John Wiley & Sons, New York. With permission.)

The second requirement is for overall column buckling of wall studs braced by shear diaphragms on both flanges (Figure 7.42). For this case, the AISI Specification provides equations for calculating the critical stresses in order to determine the nominal axial strength by considering the shear rigidity of the sheathing material. These lengthy equations can be found in Section D4 of the Specification [7].

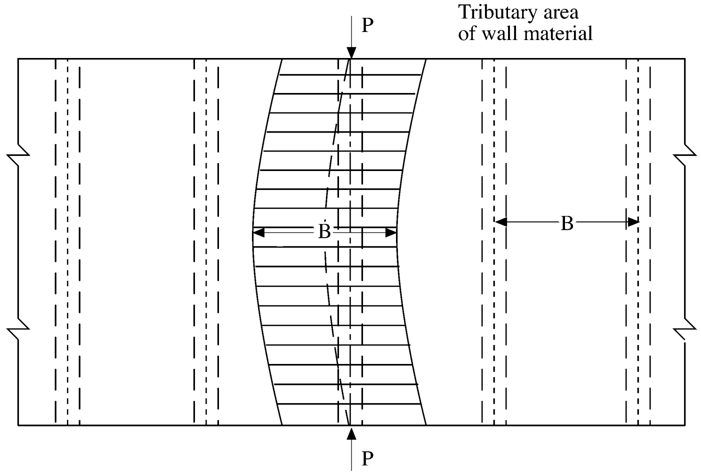


FIGURE 7.42: Overall column buckling of studs. (From Yu, W.W. 1991. *Cold-Formed Steel Design,* John Wiley & Sons, New York. With permission.)

The third requirement is to prevent shear failure of the sheathing by limiting the shear strain within the permissible value for a given sheathing material.

Wall Studs in Bending

The nominal flexural strength of wall studs is determined by the nominal section strength by using the "All Steel Design" approach and neglecting the structural contribution of the attached sheathing material.

Wall Studs with Combined Axial Load and Bending

The AISI interaction equations presented in Table 7.7 are also applicable to wall studs subjected to combined axial load and bending with the exception that the nominal flexural strength be evaluated by excluding lateral buckling considerations.

7.8.5 Residential Construction

During recent years, cold-formed steel members have been increasingly used in residential construction as roof trusses, wall framing, and floor systems (Figure 7.43). Because of the lack of standard sections and design tables, prescriptive standards have recently been developed by the National Association of Home Builders (NAHB) Research Center and the Housing and Urban Development (HUD). The sectional properties and load-span design tables for a selected group of C-sections are calculated in accordance with the AISI Specification [9].

For the design of cold-formed steel trusses and shear walls using steel studs, design guides have been published by the American Iron and Steel Institute [6, 9].

FIGURE 7.43: Steel house using cold-formed members for walls, joists, and trusses.

7.8.6 Composite Construction

Cold-formed steel decks have been used successfully in composite roof and floor construction. For this type of application, the steel deck performs the dual role of serving as a form for the wet concrete during construction and as positive reinforcements for the slab during service.

As far as the design method for the composite slab is concerned, many designs have been based on the SDI Specification for composite steel floor deck [42]. This document contains requirements and recommendations on materials, design, connections, and construction practice. Since 1984, the American Society of Civil Engineers has published a standard specification for the design and construction of composite slabs [11].

When the composite construction is composed of steel beams or girders with cold-formed steel deck, the design should be based on the AISC Specification [1, 2].

7.9 Defining Terms

ASD (allowable stress design): A method of proportioning structural components such that the allowable stress, allowable force, or allowable moment is not exceeded when the structure is subjected to all appropriate combinations of nominal loads.

Beam-column: A structural member subjected to combined compressive axial load and bending.

Buckling load: The load at which a compressed element, member, or frame assumes a deflected position.

Cold-formed steel members: Shapes that are manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets.

Composite slab: A slab in which the load-carrying capacity is provided by the composite action of concrete and steel deck (as reinforcement).

Compression members: Structural members whose primary function is to carry concentric loads along their longitudinal axes.

Design strength: R_n/Ω for ASD or ϕR_n for LRFD (force, moment, as appropriate), provided by the structural component.

Effective design width: Reduced flat width of an element due to local buckling for design purposes. The reduced width is termed the effective width or effective design width.

Effective length: The equivalent length KL used in design equations.

Flexural members (beams): Structural members whose primary function is to carry transverse loads and/or moments.

Flat-width-to-thickness ratio: The flat width of an element measured along its plane, divided by its thickness.

Limit state: A condition at which a structure or component becomes unsafe (strength limit state) or no longer useful for its intended function (serviceability limit state).

Load factor: A factor that accounts for unavoidable deviations of the actual load from the nominal load.

Local buckling: Buckling of elements only within a section, where the line junctions between elements remain straight and angles between elements do not change.

LRFD (load and resistance factor design): A method of proportioning structural components such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

Multiple-stiffened elements: An element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners that are parallel to the direction of stress. A sub-element is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

Nominal loads: The loads specified by the applicable code not including load factors.

Nominal strength: The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions with equations derived from accepted principles of structural mechanics or by tests of scaled models, allowing for modeling effects, and differences between laboratory and field conditions.

Point-symmetric section: A point-symmetric section is a section symmetrical about a point (centroid) such as a Z-section having equal flanges.

Required strength: Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads for LRFD or nominal loads for ASD (using most appropriate critical load combinations).

Resistance factor: A factor that accounts for unavoidable deviations of the actual strength from the nominal value.

Safety factor: A ratio of the stress (or strength) at incipient failure to the computed stress (or strength) at design load (or service load).

Stiffened or partially stiffened compression elements: A stiffened or partially stiffened compression element is a flat compression element with both edges parallel to the direction of stress stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

Stress: Stress as used in this chapter means force per unit area and is expressed in ksi (kips per square inch) for U.S. customary units or MPa for SI units.

Thickness: The thickness of any element or section should be the base steel thickness, exclusive of coatings.

Torsional-flexural buckling: A mode of buckling in which compression members can bend and twist simultaneously without change in cross-sectional shape.

Unstiffened compression elements: A flat compression element which is stiffened at only one edge parallel to the direction of stress.

Yield point: Yield point as used in this chapter means either yield point or yield strength of steel.

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Further Reading

Guide to Stability Design Criteria for Metal Structures, edited by T.V. Galambos, presents general information, interpretation, new ideas, and research results on a full range of structural stability concerns. It was published by John Wiley & Sons in 1988.

Cold-Formed Steel in Tall Buildings, edited by W.W. Yu, R. Baehre, and T. Toma, provides readers with information needed for the design and construction of tall buildings, using cold-formed steel for structural members and/or architectural components. It was published by McGraw-Hill in 1993.

Thin-Walled Structures, edited by J. Rhodes and K.P. Chong, is an international journal which publishes papers on theory, experiment, design, etc. related to cold-formed steel sections, plate and shell structures, and others. It was published by Elsevier Applied Science. A special issue of the Journal on Cold-Formed Steel Structures was edited by J. Rhodes and W.W. Yu, Guest Editor, and published in 1993.

Proceedings of the International Specialty Conference on Cold-Formed Steel Structures, edited by W.W. Yu, J.H. Senne, and R.A. LaBoube, has been published by the University of Missouri-Rolla since 1971. This publication contains technical papers presented at the International Specialty Conferences on Cold-Formed Steel Structures.

"Cold-Formed Steel Structures", by J. Rhodes and N.E. Shanmugan, in *The Civil Engineering Handbook* (W.F. Chen, Editor-in-Chief), presents discussions of cold-formed steel sections, local buckling of plate elements, and the design of cold-formed steel members and connections. It was published by CRC Press in 1995.