

# 49

## Cold Formed Steel Structures

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### 49.1 Introduction to Cold-Formed Steel Sections

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Cold-formed steel products find extensive application in modern construction in both low-rise and high-rise steel buildings. Primary as well as secondary framing members in low-rise construction are fabricated using cold-formed steel sections, while in tall buildings, roof and floor decks, steel joists, wall panels, door and window frames, and sandwich panel partitions built out of cold-formed steel sections have been successfully used. In addition, these products are used in car bodies, railway coaches, storage racks, grain bins, highway products, and transmission towers. Although the uses of these products are many and varied, a multiplicity of widely different products, with a tremendous diversity of shapes, sizes, and applications, are produced in steel using the cold-forming process. This chapter is primarily concerned with the design of cold-formed steel members for use in building construction. However, the general design philosophies developed in the chapter are applicable in many cases over a wide range of other uses. More detailed information on cold-formed steel structures are available in books by Yu (1991), Rhodes (1991), and Hancock (1988).

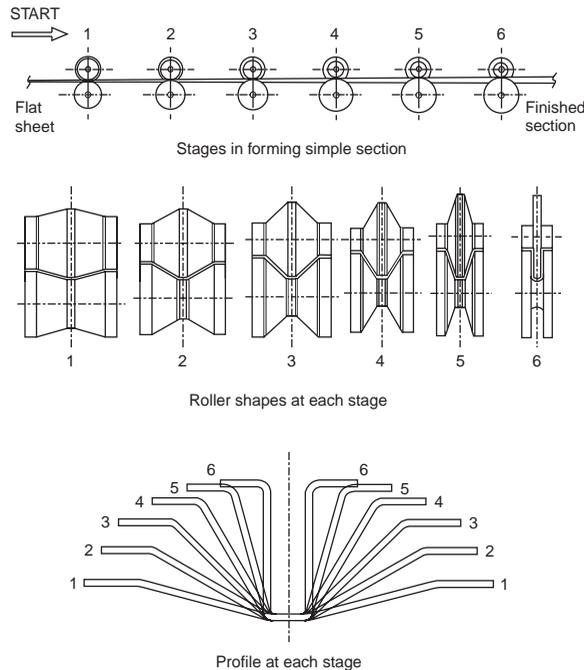


FIGURE 49.1 Stages in roll forming a simple section.

## Manufacturing Methods

*Cold forming* is the term used to describe the manufacture of products by forming material in the cold state from a strip or sheet of uniform thickness. There is a variety of different methods of forming used for cold-formed products in general, but in the case of structural sections, the main methods used are folding, press-braking, and rolling.

*Folding* is the simplest process, in which specimens of short length and of simple geometry are produced from a sheet of material by folding a series of bends. This process has very limited application.

*Press-braking* is more widely used, and a greater variety of cross-sectional forms can be produced by this process. Here a section is formed from a length of strip by pressing the strip between shaped dies to form the profile shape. Usually each bend is formed separately. This process has limitations on the profile geometry that can be formed and, more importantly, on the lengths of sections that can be produced.

The major cold-forming process used for large-volume production is *cold rolling*. In this process strip material is formed into the desired profile shape by feeding it continuously through successive pairs of rolls. Each pair of rolls brings the form of the strip progressively closer to the final profile shape, as illustrated in Fig. 49.1. The number of pairs of rolls, or “stages,” required depends on the thickness of the material and the complexity of the profile to be formed.

The cold-rolling process can be used to produce prismatic sections of virtually any profile, from a wide range of materials, with a high degree of consistency and accuracy to any desired length. Sections are rolled at speeds varying from about 10 m per min up to about 100 m per min, depending on the complexity of the profile, material being formed, equipment used, etc. Holes, notches, and cutouts can be produced in a member during the rolling process, and a pregalvanized or precoated steel strip is often used to eliminate corrosion and to produce aesthetically pleasing finished products. Typical profiles produced are shown in Fig. 49.2.

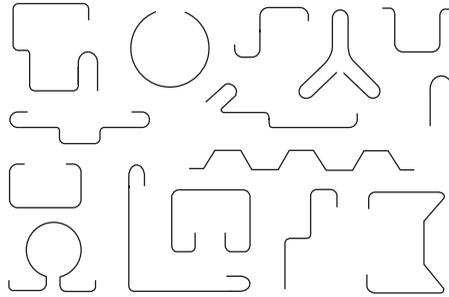


FIGURE 49.2 Typical shapes produced by roll forming.

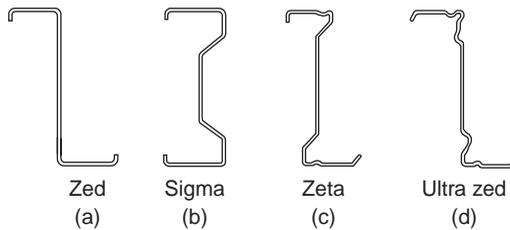


FIGURE 49.3 Types of roof purlin in common use at the present time.

## Applications of Cold-Formed Steel

Trapezoidal profiles and the many variations on the trapezoidal shape are now widely used for industrial and commercial buildings, sports arenas, hotels, restaurants, and many other types of building construction. A variety of advances have been made in the field of profiled sheeting in relatively recent times. The use of fixing systems, which ensure water tightness of roofs, has been an area of development, as has the incorporation of stiffeners in the profiles. The use of foam-filled sandwich panels in which the roof or wall covering is combined with the insulation to give superior structural performance, in addition to other advantages, is an area of rapid growth. Composite steel–concrete flooring is another area in which there has been rapid growth in the last few years. Steel decks acting compositely with concrete have been used in the United Kingdom and United States for a long period.

Roof purlins, which are ideally suited for production, as cold-rolled sections account for a substantial proportion of cold-formed steel usage in buildings. Two basic shapes are used for purlins in the United Kingdom, the zeta (Z) shape (Fig. 49.3a), which was introduced from the U.S., and the sigma shape (Fig. 49.3b). Both of these shapes are very efficient in acting in conjunction with the sheeting to produce a high structural performance. Recent research and development efforts have lead to refinements in the Z-shape (Fig. 49.3c) and UltraZED-shape (Fig. 49.3d) sections. Purlin thicknesses used range from about 0.047 in. (1.2 mm) to about 0.126 in. (3.2 mm), and material of a yield strength of 50 ksi (350 N/mm<sup>2</sup>) is becoming widely used in the production of purlins.

Storage platforms and mezzanine floor systems form another area of growing use of cold-formed steel members. In these systems the columns are often hot-rolled sections such as square hollow sections or I sections, and the beams are cold-formed sections. In lattice beam construction the boom members are generally cold-formed sections of hat or similar shape, and the lattice members may be tubular or made from round-bar or other cold-formed shapes. The concept of preengineered buildings, made largely from cold-formed steel sections in the factory and erected on site, has been a constantly recurring theme in the development of the cold-rolled sections industry, and the use of steel stud wall systems is a further step in this direction. Storage racking is another area that forms a significant outlet for cold-formed steel products. This accounts for perhaps 20% of all the constructional use of cold-formed sections and utilizes

a substantial proportion of perforated members. Storage installations range from relatively small shelving systems to extremely large and sophisticated pallet racking systems.

## Advantages of Cold-Formed Steel

Cold-formed steel products have several advantages over hot-rolled steel sections. The main attractions of cold-formed steel sections are their lightness, high strength and stiffness, ease of fabrication and mass production, fast and easy erection and installation, substantial elimination of delays due to weather, more accurate detailing, nonshrinking and noncreeping at ambient temperatures, absence of formwork, protection from termites and rot, uniform quality, economy in transportation and handling, and noncombustibility. The combination of these advantages can result in cost savings during construction (Yu, 1991).

## Design Codes and Specifications

Since the late 1970s cold-formed steel has taken on a new importance in Europe, and there has been a period of substantial activity in research and in the development of new design codes. This began with the publication of a new Swedish design specification in 1982 (National Swedish Committee on Regulations for Steel Structures, 1982), followed by European recommendations at various stages. Insofar as the design method is concerned, some specifications use the allowable stress design approach, whereas others are based on a limit state design. The American Iron and Steel Institute (AISI) includes both allowable stress design (ASD) and load and resistance factor design (LRFD). In the United Kingdom, British Standard (BS) 5950, Part 5 (British Standards Institution, 1987), deals with the design of cold-formed steel members. This code had some amendments added in 1996. Eurocode 3: "Design of Steel Structures," Part 1.3: "General Rules, Supplementary Rules for Cold-Formed Thin Gauge Members and Sheeting," was published as a European prestandard in 1996 and is having substantial and increasing effect on cold-formed steel design throughout Europe. Both Canada (Canadian Standards Association, 1989) and Australia (Hancock, 1988) have developed their own codes, and in the United States a new version of the AISI code was published in 1996.

New design codes have also been produced in the past few years to deal with some associated topics. For example, stainless steel, dealt with by an ASCE specification in the U.S. (1990) was the subject of another new European prestandard, Eurocode 3, Part 1.4: "General Rules, Supplementary Rules for Stainless Steels," in 1996, and this was followed by new South African (1997) and Australian (2001) standards.

## Range of Thicknesses

The provisions of codes apply primarily to steel sections with thickness not more than 0.33 in. (8 mm), although the use of thicker material is not precluded. Minimum thicknesses for specific applications are set by practical considerations, such as damage tolerance during handling, etc., and of course by the economics of the particular applications. With regard to the maximum thickness, 0.33 in. (8 mm) is about the limiting thickness normally rolled, although sections of up to about 0.8 in. (20 mm) can be rolled for specific applications.

## Properties of Steel

The design strength of the steel used should be taken as the yield strength of the material provided that the steel has an ultimate tensile strength about 20% or more greater than the yield strength. To ensure that, if this is not the case, the design strength is reduced accordingly in some codes. In the case of steels that have no clearly defined yield strength, either the 0.2% proof stress or the stress at 0.5% total elongation in a tensile test may be taken as the design strength. The yield points of steels listed in the AISI specification range from 25 to 70 ksi (172 to 483 MPa).

The strength of members that fail by buckling is also a function of the modulus of elasticity  $E$ , the value of which is recommended as 29,500 ksi (203 kN/mm<sup>2</sup>) by AISI in its specification for design purposes. Poisson's ratio is taken as 0.3.

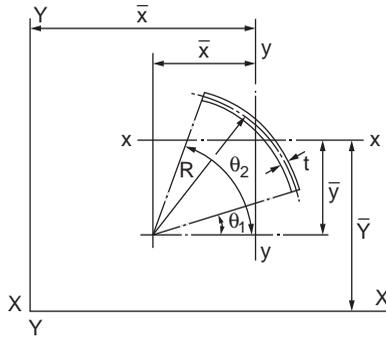


FIGURE 49.4 Geometry of a corner.

### Effects of Cold Forming

Cold forming increases the yield and ultimate tensile strengths of the material being formed in the vicinity of the bend areas. Experiments suggest that the average increase in yield strength of a section is dependent on the number of bends, the area of the section, the material thickness, and the difference between the ultimate and yield strengths of the material. Significant enhancement of the yield strength due to cold forming is found only for sections composed largely of corners with small radii and having small width-to-thickness ratios. For more slender cross-sections the benefits of cold forming on enhancement of yield become substantially reduced. If a member is subjected to tension only, then it is quite permissible to use the increased yield strength for the complete cross-section. If, however, the member is subjected to compression or combinations of compression and bending, then each element should be considered separately after the initial determination of the average yield strength of a formed section. In determination of the compression yield strength of an individual element the width-to-thickness ratio of the element is important. Since there is no conclusive evidence to support the design use of enhanced yield strength in the presence of local buckling, which occurs for slender elements, enhancement of the yield strength due to cold forming should not be taken into account for slender elements subject to local buckling behavior.

Since any operation on the formed material that introduces heat, such as welding, annealing, galvanizing, etc., will affect the material properties, the use of the enhanced yield strength is prohibited if any such operation is carried out.

### Calculation of Section Properties

Since many cold-formed steel sections have thin walls and small radii, the determination of section properties in many cases can be simplified by assuming that the material is concentrated at the centerline of the section and the area of elements are replaced by straight or curved “line elements.” The thickness dimension  $t$  is introduced after the linear computations have been completed.

### Properties of Corners

For a corner element of the geometry shown in Fig. 49.4 the properties are as follows, with the angles given in radian measure:

$$A = Rt(\theta_2 - \theta_1), \quad \bar{x} = \frac{R(\sin \theta_2 - \sin \theta_1)}{(\theta_2 - \theta_1)}, \quad \bar{y} = \frac{R(\cos \theta_1 - \cos \theta_2)}{(\theta_2 - \theta_1)} \quad (49.1)$$

$$I_{xx} = R^3 t \left\{ \frac{1}{2}(\theta_2 - \theta_1) - \frac{1}{4}(\sin 2\theta_2 - \sin 2\theta_1) - \frac{(\cos \theta_1 - \cos \theta_2)^2}{(\theta_2 - \theta_1)} \right\} \quad (49.2)$$

$$I_{yy} = R^3 t \left\{ \frac{1}{2}(\theta_2 - \theta_1) + \frac{1}{4}(\sin 2\theta_2 - \sin 2\theta_1) - \frac{(\sin \theta_2 - \sin \theta_1)^2}{(\theta_2 - \theta_1)} \right\} \quad (49.3)$$

$$I_{xy} = R^3 t \left\{ \frac{1}{4}(\cos 2\theta_1 - \cos 2\theta_2) - \frac{\sin(\theta_1 + \theta_2) - \frac{1}{2}(\sin 2\theta_1 + \sin 2\theta_2)}{(\theta_2 - \theta_1)} \right\} \quad (49.4)$$

In the particular case of a right angled corner with  $\theta_1 = 0$  and  $\theta_2 = \pi/4$  the properties may be written as follows:

$$A = \frac{\pi R t}{2} = 1.57 R t, \quad \bar{y} = \frac{2R}{\pi} = 0.637 R, \quad \bar{x} = \frac{2R}{\pi} = 0.637 R \quad (49.5)$$

$$I_{xx} = I_{yy} = R^3 t \left( \frac{\pi}{4} - \frac{2}{\pi} \right) = 0.149 R^3 t \quad (49.6)$$

$$I_{xy} = R^3 t \left( \frac{1}{2} - \frac{2}{\pi} \right) = -0.137 R^3 t \quad (49.7)$$

Formulas for the bending properties of a number of common cross-sections, based on centerline dimensions, are given in [Table 49.1](#).

### Effects of Holes

In evaluation of the section properties of members in bending or compression, holes made specifically for fasteners such as screws, bolts, etc. may be neglected on the basis that the hole is filled with materials in any case. However, for any other openings or holes the reduction in cross-sectional area and cross-sectional properties caused by these holes or openings should be taken into account. If the section properties are to be evaluated analytically they should be calculated considering the net cross-section that has the most detrimental arrangement of holes that are not specifically for fasteners. This is not necessarily the same cross-section for bending analysis and compression analysis. This is illustrated in [Fig. 49.5](#), where for the channel section shown the net cross-section A-A has a smaller area than cross-section B-B and is therefore critical with regard to purely compressional behavior. The second moment of area about x-x and minimum section modulus of cross-section B-B with regard to axis, however, are less than those of section A-A, and for bending strength section B-B is critical.

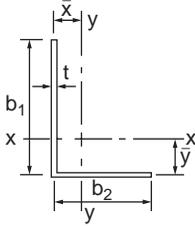
In the case of tension members, fasteners do not themselves effectively resist the tension loading, which is tending to open the fastener holes; holes made for fasteners must also be taken into consideration for tension loading. In determining the net area of a tension member, the cross-section that has the largest area of holes should be considered. The area that should be deducted from the gross cross-sectional area is the total cross-sectional areas of all holes in the cross section. In deducting the area of fastener holes the nominal hole diameter should be used. In the case of countersunk holes the countersunk area should also be deducted. In a tension member that has staggered holes, the weakening effects of holes that are not in the same cross-section, but close enough to interact with the holes in a given cross-section, should be taken into account. If two lines of holes are far apart, then one line of holes does not have any effect on the strength of the section at the position of the other line of holes. If the lines are close, however, then each line of holes affects the other.

## 49.2 Local Buckling of Plate Elements

A major advantage of cold-formed steel sections over hot-rolled sections is to be found in the relative thinness of the material from which the sections are often formed. This can lead to highly efficient and

**TABLE 49.1** Formulas for Bending Properties of Typical Sections

Unequal angle



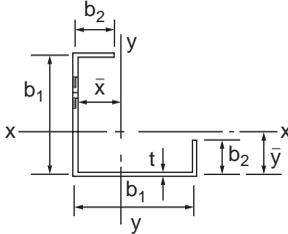
$$A = t(b_1 + b_2) \quad \bar{y} = \frac{b_1^2}{2(b_1 + b_2)} \quad \bar{x} = \frac{b_2^2}{2(b_1 + b_2)}$$

$$I_{xx} = t \frac{b_1^3(b_1 + 4b_2)}{12(b_1 + b_2)} \quad I_{yy} = t \frac{b_2^3(4b_1 + b_2)}{12(b_1 + b_2)}$$

$$I_{xy} = -\frac{tb_1^2b_2^2}{4(b_1 - b_2)}$$

$$y = \frac{1}{2} \tan^{-2} \frac{6b_1^2b_2^2}{b_1^4 + 4b_1^3b_2 - 4b_1b_2^2 - b_2^4}$$

Lipped angle



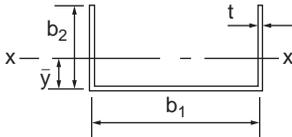
$$A = 2(b_1 + b_2) \quad \bar{y} = \bar{x} = \frac{(b_1 + b_2)}{4}$$

$$I_{xx} = I_{yy} = \frac{t}{24} (5b_1^2 + 5b_2^2 + 15b_1^2b_2 - 5b_1b_2^2)$$

$$I_{xy} = \frac{t}{8} (5b_2b_2^2 - b_1^3 - b_2^3 - 3b_1^2b_2)$$

$$I_{xx} = \frac{t}{12} (b_1 + b_2)^2 \quad I_{yy} = \frac{t}{3} (2b_1^3 - (b_1 - b_2)^3)$$

Plain channel



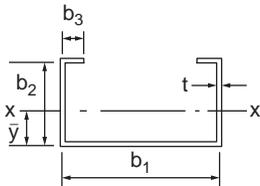
$$A = t(b_1 + 2b_2) \quad \bar{y} = \frac{b_2^2}{2(b_1 + b_2)}$$

$$I_{xx} = \frac{tb_2^3(2b_1 + b_2)}{3(b_1 + 2b_2)} \quad I_{yy} = \frac{tb_1^3}{12} \left( 1 + 6 \frac{b_2}{b_1} \right)$$

$$z_{x1} = \frac{tb_2^3}{3} (2b_1 + b_2) \quad z_{x2} = \frac{tb_2^3}{3} \frac{(2b_3 + b_2)}{(b_1 + b_2)}$$

$$z_y = \frac{tb_1^2}{6} \left( 1 + 6 \frac{b_2}{b_1} \right)$$

Lipped channel



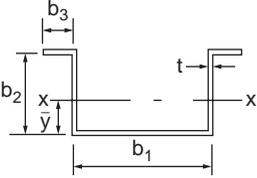
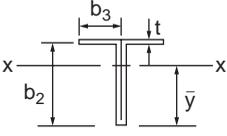
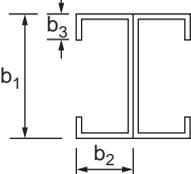
$$A = t(b_1 + 2b_2 + 2b_3) \quad \bar{y} = \frac{b_2(b_2 + 2b_3)}{(b_1 + 2b_2 + 2b_3)}$$

$$I_{xx} = \frac{b_2^3 t}{3} \frac{\left( b_2 + 4b_3 + b_1 \left( 2 + 6 \frac{b_3}{b_2} \right) \right)}{(b_1 + 2b_2 + 2b_3)}$$

$$I_{yy} = \frac{t}{12} \left[ 2b_1^3 + 6b_2b_1^2 - (b_1 - 2b_3)^3 \right]$$

$$z_{x1} = \frac{b_1^2 t}{3} \frac{\left[ b_2 + 4b_3 + b_1 \left( 2 + 6 \frac{b_3}{b_2} \right) \right]}{(2b_1 + b_2)}$$

TABLE 49.1 (continued) Formulas for Bending Properties of Typical Sections

<p>Top hat section</p> 	$z_{xz} = \frac{b_2^2 t}{3} \left[ \frac{b_2 + 4b_3 + b_1 \left( 2 + 6 \frac{b_3}{b_1} \right)}{(b_1 + b_2)} \right]$
	$z_y = 2 \frac{x_{yy}}{b_1}$
	<p>For A, <math>\bar{y}</math>, <math>I_{xx}</math>, <math>z_{x1}</math>, <math>z_{z1}</math> see lipid channel</p>
	$I_{yy} = \frac{5}{12} \left[ (b_2 + 2b_3)^3 + 6b_1^2 b_2 \right]$
	$Z_y = 2 \frac{t_{yy}}{(b_2 - 2b_3)}$
<p>Tee section</p> 	<p>Use Top Hat equations with <math>b_1 = 0</math></p>
<p>I Section</p> 	$A = 2t(b_1 + 2b_2 + 2b_3)$
	$I_{yy} = b_2^2 t \left( \frac{2}{3} b_2 + 4b_3 \right)$
	$I_{xx} = \frac{1}{6} \left[ 2b_1^3 + 6b_2 b_1^2 - (b_1 - 2b_3)^3 \right]$
	$z_x = 2 \frac{I_{xx}}{b_1}$
	$z_y = 2 \frac{I_{yy}}{b_2}$

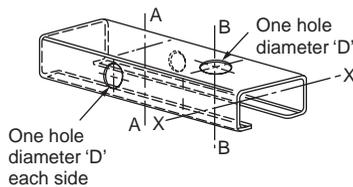


FIGURE 49.5 Channel section with holes.

weight-effective members and structures. However, the potential advantages of the thin walls can be only partially obtained, and to obtain these advantages the designer must be aware of the phenomena associated with thin-walled members and their effects on design analysis. Perhaps the most important of these phenomena is *local buckling*.

## Local Buckling of Plates

When a thin plate is loaded in compression the possibility of local buckling arises. This type of buckling is so called because the length of buckles that form is similar to the dimensions of the cross-section rather than the length of the structure, as is normally the case with other types of buckling. Elastic local buckling in a member is characterized by a number of ripples, or buckles, becoming evident in the component plates, as illustrated in Fig. 49.6. This local buckling has substantial bearing on the stiffness and strength of the member, and to gain some insight into the local buckling phenomenon, we shall now examine local buckling of plate elements.

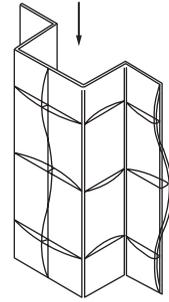


FIGURE 49.6 Local buckling in a thin-walled section.

### Local Buckling Analysis

Consider the plate shown in Fig. 49.7, supported on all four edges and compressed uniformly on its longitudinal edges to produce a displacement  $u$ , as shown in the figure. Due to the loading we shall assume that out-of-plane deflections  $w$  occur as shown. We can examine the local buckling situation from a consideration of the strain energy in the plate. The strain energy due to bending  $U_B$  can be written in terms of the deflections as

$$U_B = \frac{D}{2} \iint_{\text{area}} \left\{ \left( \frac{\delta^2 w}{\delta x^2} + \frac{\delta^2 w}{\delta y^2} \right) - 2(1-\nu) \left[ \frac{\delta^2 w}{\delta x^2} \frac{\delta^2 w}{\delta y^2} - \left( \frac{\delta^2 w}{\delta x \delta y} \right)^2 \right] \right\} dx dy \quad (49.8a)$$

The strain energy in the plate due to the membrane actions is given by

$$U_D = \frac{1}{2} \int_{\text{vol}} p_x \epsilon_x d(\text{vol}) = \frac{E t a}{2} \int \left( \bar{\epsilon} - \frac{1}{2a} \left( \frac{\partial w}{\partial x} \right)^2 \right)^2 dy \quad (49.8b)$$

where  $\bar{\epsilon}$  = the “nominal” applied strain, equal to  $u/a$   
 $E$  = the modulus of elasticity

The total strain energy stored in the plate is given by the sum of bending and membrane energies. Since the displacement of the plate ends is prescribed, the principle of minimum potential energy requires that the strain energy is a minimum. The simplest case of local buckling, very often considered in design, is that of a plate simply supported on all four edges. In this case the deflections  $w$  at buckling are given by the expression

$$w = w_c \sin \left( \frac{n \pi x}{a} \right) \sin \frac{\pi y}{b} \quad (49.9)$$

in which  $n$  indicates the number of half-sine waves into which the plate buckles in the  $x$  direction. Substituting for  $w$  in  $U (= U_D + U_B)$  and performing the integrations gives the strain energy in terms

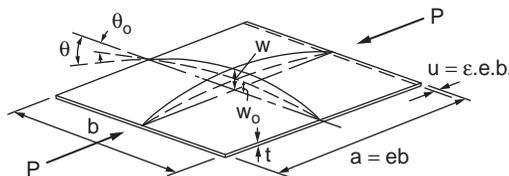


FIGURE 49.7 Uniformly compressed plates.

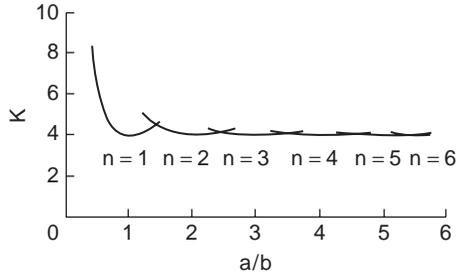


FIGURE 49.8 Variation of buckling coefficient with length for a simply supported plate.

of the deflection magnitude coefficient  $w_c$ . Using the principle of minimum potential energy it can be shown that the critical stress ( $p_{cr}$ ) to cause local buckling is given as

$$p_{cr} = E \bar{\epsilon}_{cr} = \frac{\pi^2 D}{b^2 t} \left[ \frac{nb}{a} + \frac{a}{nb} \right]^2 = \frac{K \pi^2 E}{12(1-\nu^2)} \left( \frac{t}{b} \right)^2 \quad (49.10)$$

The coefficient  $K$  is called the buckling coefficient and, for the case in question, is given by

$$K = \left[ \frac{nb}{a} + \frac{a}{nb} \right]^2 \quad (49.11)$$

The variation of  $K$  with the variation in the plate length-to-width ratio  $a/b$  is shown in Fig. 49.8 for various numbers of buckles  $n$ . As may be observed from this figure, the minimum value of  $K$  is 4; this occurs when the length of the plate is equal to  $n$  times the plate width. For long plates, the number of buckle half-waves that occur is approximately the same as the ratio of the plate's length to its width, and the buckling coefficient is very close to 4. For plates with different support conditions the value of the buckling coefficient becomes different from 4.

Thus the value of the stress required theoretically to produce local buckling varies inversely as the square of the plate width-to-thickness ratio. Plates with lower width-to-thickness ratios will theoretically yield before local buckling, and plates with higher width-to-thickness ratios will buckle before yielding. This statement holds true only for perfect plates. In the practical situation imperfections are always present, and the effects of local buckling are generally to be observed at stresses less than the theoretical buckling stress. Furthermore, local buckling does not necessarily signify the attainment of the full load capacity of a plate. For very thin plates, local buckling occurs at low stress levels and such plates can sustain loads greatly in excess of the buckling load. It is this capacity to carry loads beyond the local buckling load that provides the means for advantageous use of thin plates, but also requires knowledge of the adverse effects of local buckling to ensure safe design.

### Postbuckling Analysis

If the plate has buckled, the magnitude of the local buckles is related to the compression magnitude. The variation of stresses with the variation in strains after buckling can be shown to be

$$p_x = E \left[ \bar{\epsilon} - \frac{4}{3} (\bar{\epsilon} - \bar{\epsilon}_{cr}) \sin^2 \frac{\pi y}{b} \right] \quad (49.12)$$

Thus the average stress varies across the plate as indicated in Fig. 49.9. Strips of plate near the supports are relatively unaffected by local buckling and carry increased loading as further compression is applied, while strips of plate near the center shed the load and offer very little resistance to further compression. The plate may be thought of as consisting of a series of slender columns linked together, with those near

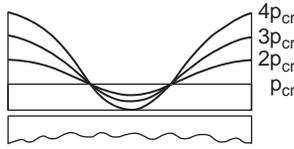


FIGURE 49.9 Variation of stress across a plate after local buckling.

the edges largely prevented from buckling and those near the center buckling relatively freely. The plate does not behave completely like a column and lose all its compression resistance after local buckling, but its resistance after buckling is confined to portions of the plate near the supported edges.

The load on the plate at any end compression is obtained by summing the stresses across the plate, i.e.,

$$p = t \int p_x dy = Etb \left[ \bar{\epsilon} - \frac{2}{3} (\bar{\epsilon} - \bar{\epsilon}_{cr}) \right] = \frac{Etb}{3} [\bar{\epsilon} + 2\bar{\epsilon}_{cr}] \quad (49.13)$$

The plate load grows after buckling with increasing compression, but the rate of growth is substantially reduced relative to that before buckling.

The analysis shows that after buckling the plate stiffness reduces, the edge stresses increase more quickly with the load than before buckling, and the plate center becomes inefficient at resisting the load, witnessed by the load shedding in this area. The plating could only be considered effective in resisting compression over a short width adjacent to the supports. As a rule of thumb, in ship design it was considered that a width of plate equal to 25 times the thickness could be considered to effectively resist compression adjacent to each support. Thus for a plate with supports on each edge a total width of 50t was considered to be effective in resisting compression. This was the origin of the effective width concept, now used widely in design analysis.

### The Effective Width Concept

The effective width concept, as generally used in design, assumes that the portions of a plate element near the supports are fully effective in resisting load and the remainder of the element is completely ineffective. This is illustrated in Fig. 49.10, in which the varying stress distribution across an element is idealized into a constant stress acting over the two effective portions and the center part of the plate is considered completely ineffective and stress-free.

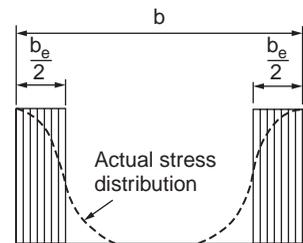


FIGURE 49.10 Effective width idealization.

In 1932 von Karman et al. produced the first theoretical explanation of the effective width concept, and in the years that followed many investigators produced further insight into this field. At the present time there is a variety of methods available for rigorous analysis of plates under many conditions of loading and support, ranging from approaches that have been set up for relatively easy use by the reader to the numerical approaches using finite elements or finite differences.

Research into cold-formed steel began largely in the U.S., and much of the original work was carried out at Cornell University, Ithaca, New York by George Winter. In dealing with local buckling, Winter (1947) produced an empirical variation of the von Karman effective width expression that, with minor modifications, has been accepted in the U.S. and in many other countries for analysis of local buckling. This expression, in the form widely used at the present time, is as follows

$$\frac{b_{eff}}{b} = \sqrt{\frac{P_{cr}}{P_{max}}} \left( 1 - 0.22 \sqrt{\frac{P_{cr}}{P_{max}}} \right) \quad (49.14)$$

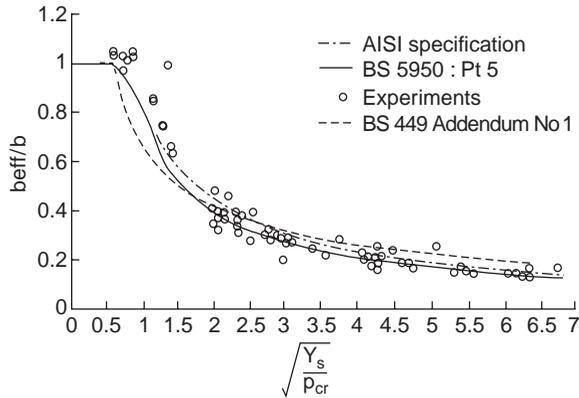


FIGURE 49.11 Effective widths from design codes and tests from various sources.

in which  $p_{\max}$  = the maximum edge stress in the plate  
 $b_{\text{eff}}$  = the effective width of the compression element

As per AISI recommendation,  $b_{\text{eff}} = b$  for  $\sqrt{\frac{p_{\max}}{p_{\text{cr}}}} < 0.673$ , and the  $b_{\text{eff}}$  is calculated from Eq. (49.14) for  $\sqrt{\frac{p_{\max}}{p_{\text{cr}}}} > 0.673$ .

The basic effective width expression developed for BS 5950, Part 5 (British Standards Institution, 1987), is as follows:

$$\frac{b_{\text{eff}}}{b} = \left[ 1 + 14 \left( \sqrt{\frac{f_c}{p_{\text{cr}}}} - 0.35 \right)^4 \right]^{-0.2} \quad (49.15)$$

in which  $f_c$  is the edge stress corresponding to the yield stress when failure is said to occur.

This expression is a little more cumbersome than that of Eq. (49.14), but still perfectly usable on a calculator or a small microcomputer.

Figure 49.11 shows comparison of the basic effective width expression (Eq. (49.15)) with the AISI expression (Eq. (49.14)), the CL factors of addendum 1 (British Standards Institution, 1975) to BS 449, and experiments from various sources. The experiments shown here are all on simply supported plates, and even so, a significant degree of scatter is noticeable.

## Classification of Elements

In using the effective width approach to deal with elements of sections, the different types of element used in such sections need to be taken into consideration. These may be classified into four groups: stiffened elements, unstiffened elements, edge-stiffened elements, and intermediately stiffened elements. Examples of each type of element are shown in Fig. 49.12.

*Stiffened elements* are elements that are supported (or stiffened) by having a substantial element on both longitudinal edges. If the supporting elements are themselves stiffened, or edge stiffened, elements, then this type of element can have a width-to-thickness ratio of up to 500. For such an element the minimum K factor applied is 4.

*Unstiffened elements* are elements that are supported along only one longitudinal edge. In this case local buckling arises much more quickly than it does for stiffened elements, and because of this, the width-to-thickness ratio that can be covered by the code is considerably reduced. The relevant maximum width-to-thickness ratio is 60, and the minimum K factor is 0.425.

Since unstiffened elements are severely affected by local buckling, it is common practice to convert these into stiffened elements by folding the free edge of these elements to produce a lip, or a similar edge

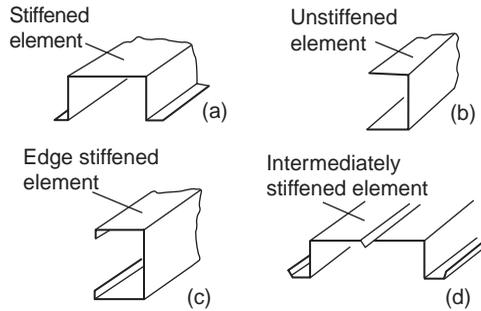


FIGURE 49.12 Types of elements found in cold-formed sections.

stiffener, to support this edge and prevent local buckling of the edge. If the edge stiffener satisfies the requirements of the code, then such an element may be treated as a stiffened element. For such an *edge-stiffened element*, however, the maximum width-to-thickness ratios are severely curtailed by the codes. In the case of an element stiffened by a simple lip, the maximum allowable width-to-thickness ratio is 60, while if any other type of edge stiffener is used, the width-to-thickness ratio can be increased to 90.

In order to improve the behavior of stiffened elements, intermediate stiffeners are often employed. This type of stiffener is usually formed during the rolling process and has the effect of transforming a slender, high  $b/t$  ratio element into two or more relatively compact subelements. This can substantially increase the effectiveness of the element. The total width of *intermediately stiffened elements* is limited to 500 times the material thickness.

## Stiffened Elements

### Buckling Coefficients

Stiffened elements under uniform compression are considered to be governed by effective width expression. The minimum  $K$  factor used for such elements is 4, but if higher values can be justified, then these may be used. The  $K$  factors applicable to compact elements, low  $b/t$ , can be much less than those for slender elements in the same section. In general, the elements in a section that are restrained from buckling at their natural stress induce premature buckling in the elements that restrain them, and if one element has a buckling coefficient greater than 4, others will have buckling coefficients less than 4. This mathematically correct, but rather pessimistic, view has been incorporated in some design codes, and invariably leads to lower design loads than would occur if all elements were considered simply supported.

However, it has been observed (Rhodes, 1987) that for restraining elements the effects of premature local buckling are negligible. Only when the applied loading is sufficient to attain the natural (simply supported) buckling stress of such elements do these elements suffer the effects of local buckling to any substantial degree.

### Stiffened Elements under Eccentric Compression

For some stiffened elements, such as webs of beams, the loading is not pure compression, but some combination of axial loading and in-plane bending. In cases where this type of loading occurs the codes treat the situation in one of two ways, depending on the degree of bending involved. For beam webs in which the stress changes from compression to tension across the element, the effective width approach is replaced by a limiting stress approach.

For cases in which an element is subjected to a combination of compression and in-plane bending in which the stress is compressive on both unloaded edges, sufficiently accurate analysis may be obtained as follows. If the stress varies from  $f_{c1}$  at one edge to  $f_{c2}$  at the other edge, as illustrated in Fig. 49.13, then the mean value of these stresses should be taken as the stress on

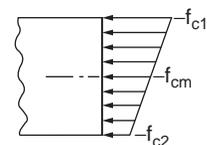


FIGURE 49.13 Element with varying stress distribution.

the element. Using this stress together with the critical stress, based on a K factor of 4, the effective width may be found as it is for uniformly compressed elements. The effective portions of the element are considered to be equally distributed adjacent to each supported edge, as for uniformly compressed elements, and the ultimate load on the element may be assumed to occur when the maximum stress reaches yield. This rather rough-and-ready method has some theoretical backing and gives reasonable and conservative results when compared with tests.

### Unstiffened Elements

In open sections there are normally unstiffened elements. As these elements buckle much more quickly than stiffened elements, they constitute a rather unsatisfactory type of element with regard to load capacity, and because of this the cold-formed counterparts of common hot-rolled sections such as channels, angles, and hat sections are often lipped to increase the resistance to local buckling.

Unstiffened elements can also be analyzed using the approach employed earlier for stiffened elements. If we assume that the out-of-plane deflections of an unstiffened element under load, as shown in Fig. 49.14, are given by the expression

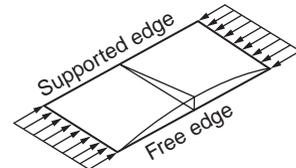


FIGURE 49.14 Unstiffened element under compression.

$$w = w_{\max} \frac{y}{b} \sin \frac{\pi x}{a} \tag{49.16}$$

then the same type of analysis gives the following results

$$K = \left(\frac{b}{a}\right)^2 + 0.425 \tag{49.17}$$

and the relative stiffness before and after buckling is

$$\frac{E^*}{E} = \frac{4}{9} \tag{49.18}$$

The variation of K with the variation in the element length-to-width ratio is shown in Fig. 49.15. In this case the element buckles into a single half wavelength, regardless of its length, according to the analysis used. This is only true in cases where the supported edge is simply supported, such as elements of angle sections. For unstiffened elements that have some restraint on their supported edge a number of buckles are found if the element is long. However, the simply supported free condition gives a lower

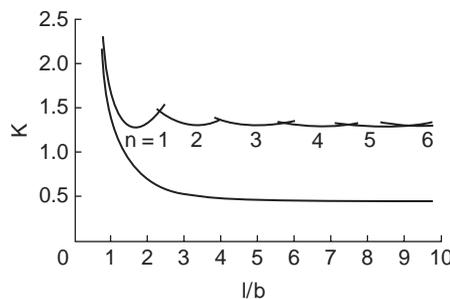


FIGURE 49.15 Variation of buckling coefficients for unstiffened elements with element length.

bound to the strength and stiffness of more restrained elements and leads to safe design. The buckling coefficients applicable to a fully fixed supported edge are also shown in Fig. 49.15 for comparison purposes.

The minimum value of  $K$  in this case is 0.425 for an element with simple support on the supported edge, based on a Poisson's ratio of 0.3. Thus the buckling load of an unstiffened element is reduced by a factor of  $4/0.425$ , i.e., 9.4, from that of a stiffened element.

### Effective Widths

Returning to our approximate analysis, if we examine the load compression behavior of a perfect plate after buckling and plot the relevant curves for stiffened and unstiffened elements, normalized with respect to the buckling point, we find that the postbuckling stiffness of the unstiffened element is greater than that of a stiffened element at a given multiple of the buckling stress. Because the effective width expressions contain the critical stress, it follows that the same expression will be rather conservative for unstiffened elements.

In the British code the effective widths so obtained are increased for unstiffened elements. The method used is to initially determine the effective width,  $b_{\text{eff}}$ , as for a stiffened element, but using the buckling coefficient applicable to the unstiffened element under examination. This is then converted to an enhanced effective width for an unstiffened element using the following equation:

$$b_{\text{eu}} = 0.89b_{\text{eff}} + 0.11b \quad (49.19)$$

## Stiffeners

### Edge Stiffeners

Because of the low buckling resistance of unstiffened elements and the rather unfortunate consequences that can arise, the benefits of incorporating edge stiffeners are plain to see. Adequately edge-stiffened elements can be substantially stronger than the unstiffened counterparts. In general an edge stiffener is required to eliminate, or at least minimize, any tendency for the otherwise unsupported edge of an element to displace out of plane. If a stiffener is adequate, the stiffened element will not incur deflections at the stiffened edge, and the element can be treated as a stiffened element. If, however, the stiffener does not have sufficient flexural rigidity to prevent out-of-plane deflections of the edge, the stiffener is said to be inadequate.

The precise requirements for adequacy are even now undergoing change. In various cold-formed steel design codes in the past it was accepted that for adequacy, a stiffener should prevent buckling of the element edge until the element buckled as a stiffened plate with a  $K$  factor of 4. This was shown (Desmond et al., 1981) to be an insufficient requirement, as the edge stiffener must also be able to prevent the edge of the element from buckling even after local buckling has occurred, indeed until it fails as a stiffened element if it is to do its job correctly. The requirements for adequacy in the latest AISI code and in the European recommendations have been based on this premise.

However, this is not the only difficulty in assessing stiffener adequacy. Rigorous analysis shows that the required rigidity of an edge stiffener depends not only on the geometry of the element to be stiffened, but on the geometry of the section as a whole and indeed on the geometry of the stiffener itself (Rhodes, 1983). Thus obtaining a single formula that covers all these variables with accuracy is a daunting task.

### Intermediate Stiffeners

Intermediate stiffeners are becoming more widely used in cold-formed steel members. The advantages of replacing slender elements by more effective subelements of relatively compact proportions at the expense of a little extra material for the stiffener are apparent. As with edge stiffeners, the intermediate stiffeners used must have adequate rigidity to prevent deflection in the element in the region of the stiffener. Adequate and inadequate stiffeners of this type are illustrated in Fig. 49.16. The geometry of an element with a single intermediate stiffener is shown in Fig. 49.17a.

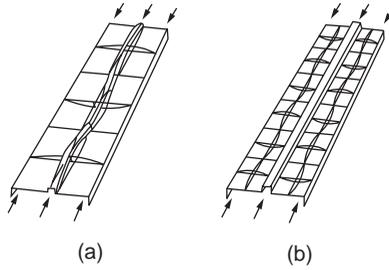


FIGURE 49.16 Intermediate stiffeners: (a) inadequate stiffeners, (b) adequate stiffeners.

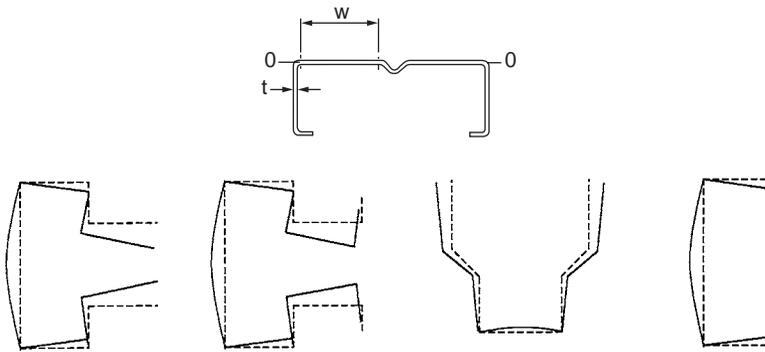


FIGURE 49.17 (a) Geometry of an element with a single intermediate stiffener. (b) Examples of distortional buckling.

If the rigidity requirement is attained by an intermediate stiffener, the effective area of the stiffened element may, under certain conditions, be evaluated from the sum of the effective areas of each individual subelement, analyzed as stiffened elements, and the stiffener area. The condition under which this is allowable is that the width-to-thickness ratios of the subelements are less than 60. Since in this case the out-of-plane deflections are eliminated near the stiffener, the hypothetical situation is that under uniform compression here, there is no shedding of load near the stiffener and this area of the element becomes fully stressed. Each subelement is therefore stressed in the same manner as a stiffened element of width  $w$ . The stiffener is also fully stressed and plays its full part in resisting the load.

### Reductions in Capacity for High Width-to-Thickness Ratios

If the subelement width-to-thickness ratios are greater than 60, reductions in the subelement effective area and in the effective stiffener area must be introduced. The main reasons for this are to be found in the examination of beam behavior. Beams that have very wide, slender flanges, either in tension or compression, suffer from the tendency of these flanges more toward the neutral axis of the section under loading.

BS 5950, Part 5, takes the adverse effects into account in a rather simple way, based on the AISI specification prior to the current version. If the subelement width-to-thickness ratio,  $w/t$ , is greater than 60, the effective width of the subelement,  $b_{\text{eff}}$ , is replaced by a reduced effective width,  $b_{\text{cr}}$ , determined from the expression

$$\frac{b_{\text{cr}}}{t} = \frac{b_{\text{eff}}}{t} - 0.1 \left( \frac{w}{t} - 60 \right) \quad (49.20)$$

The effective stiffener area is also reduced. For  $w/t$  less than 60, the stiffener is taken as fully effective. For  $w/t$  greater than 90, the ratio of effective stiffener area,  $A_{\text{eff}}$ , to full stiffener area,  $A_{\text{st}}$ , is taken as the same as that of the ratio of the effective subelement area to the full subelement area, i.e.,

$$A_{\text{eff}} = A_{\text{st}} \frac{b_{\text{cr}}}{w} \quad (49.21)$$

For  $w/t$  values between 60 and 90 a linear interpolation formula is used to obtain the effective stiffener area. This is

$$A_{\text{eff}} = A_{\text{st}} \left( 3 - 2 \frac{b_{\text{cr}}}{w} + \frac{1}{30} \left( 1 - \frac{b_{\text{cr}}}{w} \right) \frac{w}{t} \right) \quad (49.22)$$

This expression gives the stiffener effective area varying linearly from  $A_{\text{st}}$  at  $w/t = 60$  to  $A_{\text{st}} b_{\text{cr}}/w$  at  $w/t = 90$ .

### Multiple Intermediate Stiffeners

If an element has many intermediate stiffeners that are spaced closely enough to eliminate significant local buckling, i.e.,  $w/t$  is less than 30, then all stiffeners may be considered effective. However, in such a case local buckling that involves the complete element, with all stiffeners participating in the buckling, has to be guarded against. This is accomplished in a rather simple way by considering the complete element as a stiffened element without intermediate stiffeners, but having a fictitious equivalent thickness. The fictitious thickness is arranged so that the flexural rigidity of the stiffened element is the same as that of the multiply stiffened element. To accomplish this, the equivalent thickness,  $t_s$ , is taken as

$$t_s = \left( \frac{12I_s}{w_s} \right)^{1/3}$$

where  $I_s$  = the second moment of area of the full multiply stiffened element, including the intermediate stiffeners, about its own neutral axis

$w_s$  = the complete width of the element between two webs

### Distorsional Buckling

If a lip or edge stiffener, or indeed an intermediate stiffener, is not adequate, then the buckling mode that arises involves in-plane movement of the stiffener together with out-of-plane distortion of the stiffened elements. Hancock (1988) observed that this kind of behavior was evident in some storage racking members in which the edge stiffeners were not so clearly identifiable as lips and coined the term “distorsional buckling” to cover this type of behavior. The term is now widely used to describe such behavior, and at the same time the design treatment of stiffeners has been the subject of substantial change in some recent specifications. Distorsional buckling, as illustrated in Eurocode 3, Part 1.3, is shown in Fig. 49.17b for some cross-sections. In Eurocode 3, Part 1.3, distorsional buckling is specified as a design consideration that must be taken into account. For edge and intermediate stiffeners recourse can be made to the relevant rules for these elements, which are extremely demanding in calculation time and which result in significant diminution of the potential capacity of a stiffened element. Indeed, in the design of Z- or C-section beams to Eurocode 3, Part 1.3, the capacity is very substantially governed by the edge stiffener.

#### Example 49.1

Compute the effective width of the compression (top) flange of the beam shown in Fig. 49.18; assume that the compressive stress in the flange is 25 ksi.  $E = 29,500$  ksi and  $\nu = 0.3$ .

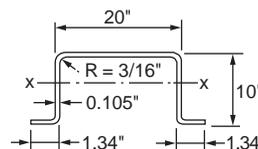


FIGURE 49.18 Example 49.1.

**Solution:**

The following solution is based on the AISI design code:

As the first step, compute  $p_{\max}/p_{cr}$ .

$$K = 4.0$$

$$b = 20 - 2(R + t) \\ = 20 - 2(0.1875 + 0.105) = 19.415 \text{ in.}$$

$$\frac{b}{t} = \frac{19.415}{0.105} = 184.9$$

$$P_{cr} = \frac{4\pi^2 E}{12(1-\nu^2)} \frac{1}{(b/t)^2} \\ = 3.12 \text{ ksi}$$

$$P_{\max} = 25 \text{ ksi}$$

$$\sqrt{\frac{P_{\max}}{P_{cr}}} = 2.83 > 0.673$$

$$\therefore b_{\text{eff}} = b \sqrt{\frac{P_{cr}}{P_{\max}}} \left[ 1 - 0.22 \sqrt{\frac{P_{cr}}{P_{\max}}} \right] \\ = 6.518 \text{ in.}$$

**Example 49.2**

Calculate the effective width of the compression flange of the box section (Fig. 49.19) to be used as a beam bending about the x axis. Use  $p_{\max} = 33$  ksi. Assume that the beam webs are fully effective and that the bending moment is based on initiation of yielding.  $E = 29,500$  ksi and  $\nu = 0.3$ .

**Solution:**

The solution is in accordance with the AISI code.

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 can be computed by using Eq. (49.14) with  $K = 4.0$ .

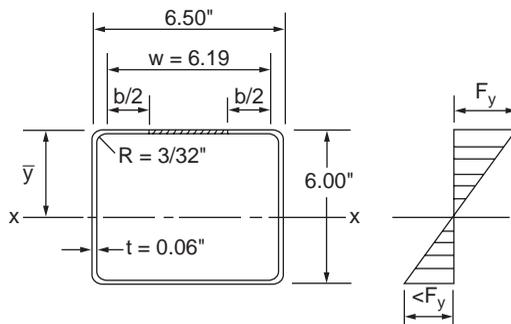


FIGURE 49.19 Example 49.2.

Given that the bending strength of the section is based on *initiation of yielding*

$$\begin{aligned}\bar{y} &= \geq 3 \text{ in.} \\ \frac{b}{t} &= \frac{6.1924}{0.06} = 103.21 \\ P_{cr} &= \frac{4\pi^2 E}{12(1-\nu^2)} \frac{1}{(b/t)^2} = 10.01 \text{ ksi} \\ \sqrt{\frac{P_{max}}{P_{cr}}} &= \sqrt{\frac{33}{10.01}} = 1.816 > 0.673 \\ \therefore b_{eff} &= b \sqrt{\frac{P_{cr}}{P_{max}}} \left[ 1 - 0.22 \sqrt{\frac{P_{cr}}{P_{max}}} \right] \\ &= 2.997 \text{ in.} \\ &\approx 3 \text{ in.}\end{aligned}$$

### 49.3 Members Subject to Bending

Because of the thin-walled nature of cold-formed steel sections, the effects of local buckling must, in most cases, be taken into account in determining the moment capacity and flexibility of beams. In the design of beam webs, the capacity of webs to withstand concentrated loads or support reactions must be ensured and the interaction of different effects must be taken into account. In the case of unbraced beams, the possibility of lateral torsional buckling arises, and the designer must be able to guard against this phenomenon. There are certain circumstances when the use of the simple bending theory cannot give realistic estimates of beam behavior. The most obvious of these circumstances arises in the design analysis of beams having unsymmetrical cross-sections. If such beams are not restrained continuously along their lengths, they must be analyzed taking into account the unsymmetrical nature of the behavior.

#### Bending of Unsymmetrical Cross Sections

Consider a thin-walled beam having a general nonsymmetrical cross-section, as shown in Fig. 49.20. If the x-x and y-y axes shown in the figure are not the principal axes of the cross-section, the application of a moment about either one of the axes will cause the beam to bend about both axes, i.e., a moment  $M$  applied about axis x-x will cause bending about both the x-x and y-y axes. There are several approaches to take account of this behavior, one of these being the use of effective moments,  $M_x^*$  and  $M_y^*$ . In this method the stresses and deflections occurring in the beam under the action of moments are dealt with as though x-x and y-y were principal axes, but with the actual moments about these axes replaced by the effective moment (Megson, 1975).

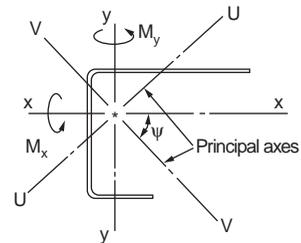


FIGURE 49.20 Beam of unsymmetrical cross-section.

#### Laterally Stable Beams

A laterally stable beam is a beam that has no tendency to displace in a direction perpendicular to the direction of loading. A beam may be laterally stable by virtue of its shape, or a beam may be considered

laterally stable if it is braced sufficiently to prevent potential displacements out of the plane of loading. For laterally stable beams local buckling is the major weakening effect. In the analysis of laterally stable beams according to BS 5950, Part 5, limiting web stress is used to take into account the possibility of local buckling in the webs and the effective width approach is used to take account of local buckling in the compression elements.

### Limiting Web Stress

The effects of local buckling due to varying bending stresses in thin webs of beams can be quite substantial. The buckling stress in a web is generally much greater than in a compression element of the same geometry, but if the web depth-to-thickness ratio is large, local buckling of the web can still have a significant influence on the beam strength. For webs under pure bending the minimum buckling coefficient is approximately 23.9, compared with 4 for a uniformly compressed plate.

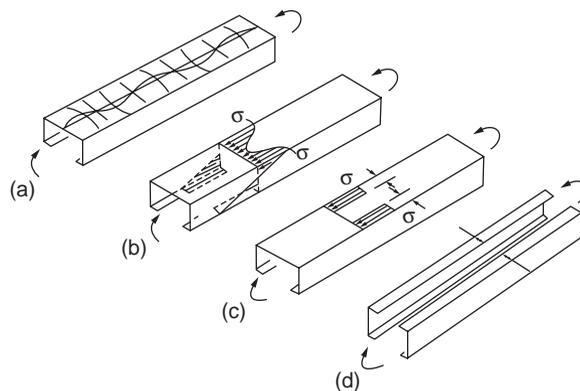
The effects of local buckling on web strength are not so easily taken into consideration, as in the case of elements under uniform compression. Effective width approaches have been investigated to take local buckling of webs into account and have been found to accomplish this task very well, but at the expense of adding further complexity to the analysis. This is mainly due to the necessity to position the effective and ineffective portions correctly. A number of design codes, including the AISI code, use the effective width approach. If a web has intermediate stiffeners, they will assist the web in resisting local buckling. There has been substantial research into this topic.

### Effective Width of Compression Elements

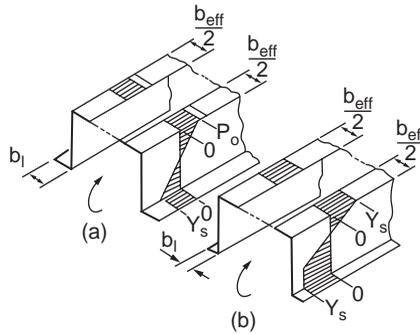
In the case of beams the elements under compression are considered to have effective widths less than their actual widths, while all other elements are considered to have their actual dimensions. A typical effective cross section is shown in Fig. 49.21. The effective width of the compression element or elements is evaluated using effective width expression. For such elements the buckling coefficients in the case of beams are in most circumstances greater than the minimum values, since buckling of the compression elements is generally (but not always) restrained by the adjacent webs.

### Moment Capacity

The moment capacity of the cross-section is determined on the basis that the maximum compressive stress on the section is  $p_0$ . This leads to two possible types of failure analysis, depending on the situation on the tension side of the cross-section, as indicated in Fig. 49.22. If the geometry of the effective cross-section is such that the compressive stress reaches  $p_0$  before the maximum tensile stress reaches the yield



**FIGURE 49.21** Effective width concept applied to laterally stable beams: (a) local buckles, (b) stress distribution, (c) idealized stress distribution on compressive element, (d) effective section.



**FIGURE 49.22** Failure criteria for laterally stable beams: (a) failure by compression yield — tensile stress elastic, (b) tensile stresses reach yield before failure — elastic-plastic stress distribution.

stress, as in Fig. 49.22a, then the moment capacity is evaluated using the product of compression section modulus and  $p_0$ , i.e.,

$$M_c = p_0 \times Z_c \quad (49.23)$$

where  $Z_c$  is the compression section modulus of the effective cross-section. This situation will occur in the case of members that have wide tension elements or substantially ineffective compression elements.

If, on the other hand, the tensile stresses reach yield before  $p_0$  is attained on the compression side, as in Fig. 49.22b, the designer is allowed to take advantage of the plastic redistribution of tension stresses and thus obtain higher predictions of moment capacity than would be the case if the first yield were taken as the criterion. This necessitates an increase in the complexity of the analysis if the added capacity is to be obtained. If simplicity of analysis is more important than the requirement to obtain the most beneficial estimate of capacity, then the moment capacity can also be obtained using the product of the yield stress and tension modulus of the effective cross section. This may well, however, lead to significant underestimates of the member capacity.

### Determination of Deflections

Determination of deflections is often required to satisfy deflection limitations at the working load, and in such a case the use of the fully reduced properties will often give overconservative results. This occurs because the effective section properties reduce progressively, and for thin-walled cross-sections these will be greater at the working load than at the ultimate load. In general the use of the fully reduced section properties overestimates deflections at loads below ultimate, whereas the use of the full section properties underestimates these deflections.

### Plastic Bending Capacity

The potentiality of local buckling and its adverse effects is not always present for cold-formed steel sections. Since material thicknesses of up to 8 mm are covered primarily by codes and greater thicknesses are not precluded, it is possible to have very compact cross-sections, even in the case of large members, in which local buckling does not take place. In such cases the potential for fully plastic design cannot be ignored. Local buckling is the major source of impairment of the capacity of a laterally stable member to function adequately in the plastic range, and when this phenomenon is eliminated by virtue of the compactness of the cross-section, the member can behave plastically.

For compact elements the cross-section not only can withstand the fully plastic moment, but also can provide sufficient rotation capacity at the point of maximum moment to allow plastic redistribution of the moments in statically indeterminate beams. The limiting width-to-thickness ratios for compression elements of the plastic cross-section are specified in codes. Sections whose compression elements have

b/t ratios less than the limiting values may be designed using the principle of plastic analysis, providing that the following qualifying features are complied with:

1. The member is laterally stable.
2. The virgin yield strength of the material is used, and the enhanced yield due to cold-forming effects is neglected.
3. The depth-to-thickness ratio of the compression portion of the web is less than the value specified in the codes.
4. The maximum shear force is less than the value limited by the code.
5. The angle between any web and the loading plane does not exceed 20°.
6. The ratio of ultimate-to-yield strength is at least 1.08, and the total elongation at failure in a tensile test is not less than 10% over a 2-in. gauge length.

These qualifications are imposed largely on the basis of engineering judgment to avoid any possibility of underdesign through the use of plastic analysis.

## Web Crushing

An important effect that must be avoided in the use of cold-formed steel beams is local crushing at support points or points of concentrated load. The thinness of the web material makes cold-formed sections susceptible to such behavior if they are supported directly on the bottom elements over a short support length. Web crushing is characterized by localized buckling in the immediate vicinity of the concentrated load or support point, as illustrated in Fig. 49.23. This type of buckling signifies the limit of the load capacity of a beam and must be avoided.

In the most commonly used cold-formed beams, i.e., roof purlins, web crushing is avoided by the use of cleats that support the beam using bolts fixed through the web, thus eliminating the high compressive stresses that would be incurred if the beam was supported through its bottom flange. The use of cleats is illustrated in Fig. 49.24 and is a most effective way of overcoming the problem of web crushing.

If cleats are not to be used, then the main method of ensuring that web crushing does not occur is to make the length of support sufficiently large to avoid the possibility. The capacity of a beam web to withstand concentrated loading is dependent on the web  $D/t$  ratio, the material yield strength, the length over which the load or support takes place, the corner radius of the supported flange, the web angle, the general geometry of the cross section, and the position of the load or support point on the member.

If concentrated loads are applied close to the ends of a member, the capacity of the web to resist these loads is less than that for loads applied far from the ends, since it is easier for the web to buckle out of plane if it has material only on one side of the support to resist buckling. In BS 5950, Part 5, the rules governing web crushing were adapted from the 1980 AISI specification, which is based largely on tests carried out at Cornell University (Winter and Pian, 1946; Zetlin, 1955), with refinements produced by further testing at the University of Missouri–Rolla (Hetrakul and Yu, 1980). A more detailed consideration of the web crushing problem and the set up of the AISI design rules is given in Yu (1991). In the recent past attempts have been made by a number of researchers, e.g., Rhodes et al. (1999) and Hoffmeyer et al. (2000), with some success, to produce design methods based to a greater extent on analysis than was the case in the past, and it is possible that the current highly empirical approach to this problem may be replaced by alternative, more analytically based methods in the future.

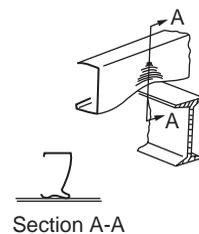


FIGURE 49.23 Web crushing at support.

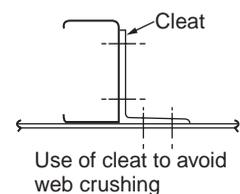


FIGURE 49.24 Use of cleat to avoid web crushing.

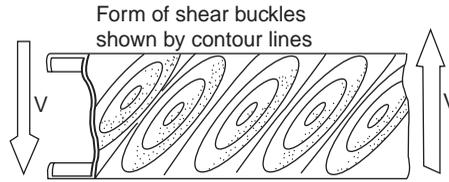


FIGURE 49.25 Shear buckling of thin web.

## Shear in Webs

The primary functions of webs are to keep the flanges apart and to carry the shear loadings. It is necessary for safe design to ensure that the shear stresses in the webs do not become unacceptably large. In thin webs there are two potential sources of danger regarding the behavior of the web in shear. The first is the possibility of shear stresses, including yield of the material, and the second is the possibility of shear buckling in the web.

### Material Yielding in Shear

With regard to material yielding, the von Mises yield criterion predicts that in the case of a material under pure shear yielding will occur when the shear stress reaches 0.577 times the yield stress in simple tension. In AISI with a factor of safety equal to 1.44, shear stresses equal to 0.4 times the yield stress are allowed at the strength limit state. In the determination of the shear stress limitations with regard to yield resistance two different provisions are given, one applying to the maximum shear stress in the web and the other applying to the average shear stress in the web. The average shear stress in the web is obtained by dividing the shear force by the web area. The use of the average shear stress in design calculations is simple and expedient, but it should be borne in mind that the shear stress is not normally constant across the web, and the maximum shear stress should also be checked.

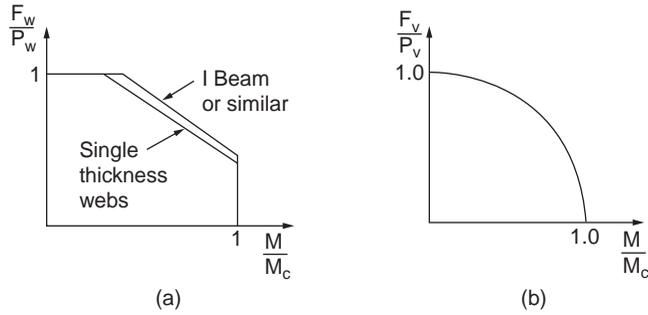
### Web Buckling Due to Shear

In short, deep beams with thin webs, as illustrated in Fig. 49.25, local buckling due to shear becomes a potential problem. Under shear loading the form of the local buckles is rather different from that produced by direct stresses. The main difference lies in the fact that shear buckles are orientated at some angle to the axis of the web, as indicated in the figure. The degree of orientation depends on the relative magnitudes of the direct stresses and shear stresses in the webs, and becomes a maximum of  $45^\circ$  when only shear is present. This type of buckling has been investigated by many researchers (Rockey, 1967; Allen and Bulson, 1980). After buckling, the web can withstand further loading due to tensile stresses that arise to resist shear deformation. This resistance is known as *tension field action*, and for hot-rolled sections tension field action may be used in design to improve the design capacity.

In the case of cold-formed steel sections the variety of possible sections that must be covered by the design rules preclude the use of tension field action in the general case in the light of present-day knowledge, and shear buckling is taken as the limiting factor. However, in view of the underlying sources of increased safety the reductions in buckling resistance due to imperfections, etc., which are taken into account for other forms of buckling, are disregarded in the case of web buckling due to shear. In determining the shear buckling resistance, the worst case of shear on a web is considered.

## Combined Effects

When different load actions take place on a member simultaneously, each action affects the general behavior, and the resistance of a member to one type of load is dependent on the magnitude of all the load actions on the member. Since in general beams are subjected to shear, bending, and support loading at the same time, the interactions of each different loading type should be checked out. Ideally in assessing



**FIGURE 49.26** (a) Interaction diagram for combined web crushing and shear. (b) Interaction diagram for combined bending and shear.

the capacity of a member subject to a variety of different actions, all actions that contribute to failure should be incorporated in the assessment. This is rather difficult, however, because of the complexities of taking many actions into account at the same time, and in practice the main actions are included only in the interaction equations. Design codes give interaction equations dealing with the combinations of bending and web crushing and web crushing and shear.

### Combined Bending and Web Crushing

Since the web crushing provisions were adapted from the AISI specification, the rules in BS 5950, Part 5, governing the load capacity of beams under combined bending and web crushing were naturally taken from the same source. The AISI rules were based on a series of tests carried out at the University of Missouri–Rolla (Hetrakul and Yu, 1980). Two different interaction formulas are given in BS 5950, Part 5: one for single-thickness webs and the other for I beams made from channels connected back-to-back.

For single-thickness webs the relevant interaction equation is

$$1.2 \left( \frac{F_w}{P_w} \right) + \left( \frac{M}{M_c} \right) \leq 1.5 \quad (49.24)$$

For I beams, or for any section where the web is provided with a high degree of rotational restraint at its junction with the flange, the relevant interaction equation is

$$1.1 \left( \frac{F_w}{P_w} \right) + \left( \frac{M}{M_c} \right) = 1.5 \quad (49.25)$$

In both of these equations  $F_w$  is the concentrated web load or reaction,  $M$  is the applied bending moment at the point of application of the web load, and  $P_w$  and  $M_c$  are the web crushing capacity and moment capacity, respectively, of the member. These equations are, of course, subject to the overriding conditions that  $P$  cannot be greater than  $P_w$  and  $M$  cannot be greater than  $M_c$ . The interaction diagrams are shown in Fig. 49.26a, which indicates that single-thickness webs are considered to be affected to a somewhat greater extent by the combination of effects than I-beam webs.

### Combined Bending and Shear

The interaction of shear force and bending is covered in BS 5950, Part 5, by the equation

$$\left( \frac{F_v}{P_v} \right)^2 + \left( \frac{M}{M_c} \right)^2 \leq 1 \quad (49.26)$$

where  $F_v$  and  $P_v$  are the shear force and shear capacity, respectively. This equation is illustrated in Fig. 49.26b and is the same as that used in the AISI specification. The AISI specification also has further provisions for webs fitted with transverse stiffeners at the load points.

## Lateral Buckling

Lateral buckling, sometimes called lateral torsional buckling, generally occurs when a beam that is bent about its major axis develops a tendency to displace laterally, i.e., perpendicularly to the direction of loading, and twist. Many, if not most, beams used in cold-formed construction are restrained against lateral movement, in many cases continuously restrained by roof or wall cladding. In other cases restraint is afforded by other members connected to the beam in question or by bracing such as antisag bars. Such restraints reduce the potentiality of lateral buckling, but do not necessarily eliminate the problem. For example, roof purlins are generally restrained against lateral displacement by the cladding, but under wind uplift, which induces compression in the unrestrained flange, lateral buckling is still a common cause of failure. This occurs due to the flexibility of the restraining cladding and to the distortional flexibility of the purlin itself, which permits lateral movement to occur in the compression flange, even if the other flange is supported.

A further point that should be noted is that, contrary to the statement made in BS 5950, Part 5, it is not a necessary condition for lateral torsional buckling that bending take place about the major axis. In some cases beams that are bent about the minor axis may undergo this type of buckling behavior. In general, lateral torsional buckling is closely related to torsional flexural buckling in columns. Any cross-section that is susceptible to torsional flexural buckling may also have lateral buckling tendencies.

### Elastic Lateral Buckling Resistance Moment

In the case of an I beam, theoretical analysis (Allen and Bulson, 1980) shows that the elastic critical moment,  $M_E$ , for a beam of length  $L$  bent in the plane of the web is given by the expression

$$M_E = \frac{\pi}{L} \left[ \frac{EI_1}{\gamma} \left( GJ + EC_w \frac{\pi^2}{L^2} \right) \right]^{1/2} \quad (49.27)$$

where  $I_1$  = the second moment of area about an axis through the web  
 $C_w$  = the warping constant,  $G$  is the shear modulus  
 $J$  = the torsion constant  
 $\gamma = 1 - I_1/I_2$ ,  $I_2$  being the second moment of area about the neutral axis perpendicular to the web

Using the relationships

$$I_1 \approx \frac{B^3 t}{6} = Ar_y^2 \quad C_w \approx I_1 \times \frac{D^2}{4}$$

where  $B$  = the flange width  
 $D$  = the beam depth  
 $A$  = the area of cross section  
 $r_y$  = the the radius of gyration about the  $y$  axis,

$$G = \frac{E}{2(1+\nu)} = \frac{E}{2.6} \quad J = \frac{At^2}{3}$$

This equation can then be rearranged to give

$$M_E = \frac{\pi^2 A E D}{2(L/r_y)^2} \left[ 1 + \frac{4}{7.8 \pi^2} \left( \frac{L t}{r_y D} \right)^2 \right]^{1/2} \quad (49.28)$$

The term  $4/7.8\pi^2$  is very close to  $1/20$ , and this forms the basis of the elastic lateral buckling resistance moment used in the AISI specification and BS 5950, Part 5, for I sections. Analysis of channels gives very similar results, and these can be dealt with using the same equation. In this equation the *effective length*,  $L_E$ , is used instead of  $L$ , and a coefficient  $C_b$ , which accounts for the variation in moment along a beam, is also incorporated.

In the case of Z-section beams, it is rather difficult to envisage such beams being used completely unrestrained against lateral movement, as the unsymmetrical behavior would make them highly flexible. The vast majority of Z sections are used as purlins, with a high degree of lateral restraint from roof cladding, and even types of cladding classified as nonrestraining offer sufficient restraint to enable these purlins to function more or less as laterally braced members if lateral buckling is not considered. In the case of Z sections that are not restrained laterally or that have very light restraint, the lateral buckling resistance is taken in the AISI specification and BS 5950, Part 5, as half of that calculated for a channel or I section. This recommendation is based on tests in the U.S. by Winter (1947).

### Variation in Moment along a Beam

The coefficient  $C_b$  is used to take account of the variation in moment along a beam. Without this coefficient the buckling resistance is calculated on the basis of a uniform moment acting all along the beam, which is a most severe condition. If the moment varies along the beam, then the maximum moment to cause lateral buckling will be greater than that analyzed on the basis of the pure moment, and this is taken into account by the  $C_b$  factor.

$C_b$  acts as a multiplying factor, and if the elastic lateral buckling moment derived for pure bending is multiplied by this factor, the resulting values of  $M_E$  become good approximations to the elastic lateral buckling moments for the case of the linearly varying bending moment along a beam. These coefficients were derived on the basis of a linearly varying bending moment distribution, but within limits they may also be used in the case of nonlinearly varying moments.

The  $C_b$  factors used in the AISI specification and BS 5950, Part 5, are, with reference to Fig. 49.27,

$$C_b = 1.75 - 1.05\beta + 0.3\beta^2 \leq 2.3 \quad (49.29)$$

in which  $\beta$  is the ratio of the end moments.

If the maximum moment within the beam span between supports is less than the larger of the end moments, Eq. (49.29) can be used to determine  $C_b$ . If the maximum moment within the span is greater than the larger end moment,  $C_b$  must be taken as unity.

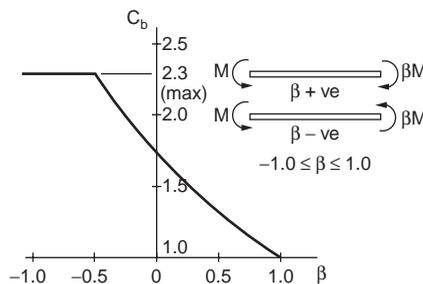


FIGURE 49.27 Variation of  $C_b$  factor with distribution of moment over the span.

## Effective Lengths

The restraints afforded by the supports can have a substantial effect on the lateral buckling resistance of beams. The expressions given for  $M_E$  assume that no resistance to warping is afforded by the support. If the supports can resist torsion, however, increases in buckling resistance can be obtained. These increases in buckling resistance are derived using the well-known effective length concept, in which it is assumed that the beam has an effective length different from its actual length for the purpose of determining buckling resistance.

In estimating the effective length with regard to lateral buckling the engineer is required to exercise a degree of judgment. The effective lengths are directly affected by the degree of restraint on rotation of the beam at the supports or bracing points, and the rotations that require examination occur about three perpendicular axes, as shown in Fig. 49.28. Some assessment must be made regarding the degree of restraint afforded by the support about each axis. If it is considered that no restraint is provided against rotation about any axis, then for safe design the effective length should be taken as 1.1 times the actual span between supports or bracing members.

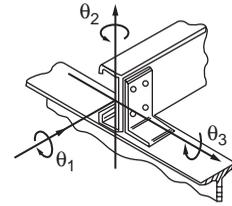


FIGURE 49.28 Support conditions with regard to effective length determination.

## Destabilizing Loads

The elastic buckling resistance moment was determined initially on the basis of pure moment loading on simply supported beams. This was then modified to take account of moment variation via the  $C_b$  factors and to take account of the support restraints via the use of effective lengths. One further factor that must be taken into account concerns the position of the loading on the cross-section. If we consider the I section shown in Fig. 49.29, any twisting of the section reduces the vertical distance between the shear center and the web flange junctions. Thus during lateral buckling a load applied to the upper flange will displace further than a load applied at the shear center, while a load applied to the lower flange will displace by a lesser amount. Thus the work done by the load during buckling is greatest if applied to the upper flange and least if applied to the lower flange, and the values of the buckling stresses are dependent on this effect. For loads applied above the shear center the buckling resistance decreases, while for loads applied below the shear center the buckling resistance increases.

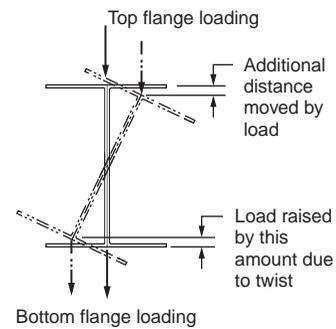


FIGURE 49.29 Stabilizing and destabilizing loads.

## Example 49.3

Figure 49.30a shows a hat section that is subjected to bending about the x axis. Assuming the yield point of steel as 50 ksi, determine the allowable bending moment in accordance with AISI specifications.

### Solution: Calculation of Sectional Properties

Midline dimensions shown in Fig. 49.30b are used for the calculation.

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

Arc length of the corner element:

$$L = 1.57R' = 0.3768 \text{ in.}$$

$$c = 0.637R' = 0.1529 \text{ in.}$$



$$P_{cr} = \frac{4\pi^2 E}{12(1-\nu^2)} \frac{1}{(w/t)^2}$$

$$= \frac{4 \times \pi^2 \times 29,500}{12 \times (1-0.3^2)} \frac{1}{(108.71)^2} = 9.02 \text{ ksi}$$

$$p_{max} = 50 \text{ ksi}$$

$$\sqrt{\frac{p_{max}}{P_{cr}}} = \sqrt{\frac{50}{9.02}} = 2.35 > 0.673$$

$$b = w \sqrt{\frac{P_{cr}}{P_{max}}} \left[ 1 - 0.22 \sqrt{\frac{P_{cr}}{P_{max}}} \right]$$

$$= 4.396 \text{ in.}$$

The neutral axis of the effective section with a reduced width of the compression flange equal to 4.396 in. can be determined as given below. The webs are assumed to be fully effective.

Element	Effective Length L (in.)	Distance from Top Fiber y (in.)	Ly (in. <sup>2</sup> )
1	$2 \times 0.9575 = 1.9150$	7.9475	15.22
2	$2 \times 0.3768 = 0.7536$	7.8604	5.92
3	$2 \times 7.4150 = 14.8300$	4.0000	59.32
4	$2 \times 0.3768 = 0.7536$	0.1396	0.1052
5	4.3960	0.0525	0.2310
Total	22.6482		80.7962

$$y_{cg} = \frac{\Sigma(Ly)}{\Sigma L} = \frac{80.7962}{22.6482} = 3.567 \text{ in.}$$

Because the distance  $y_{cg}$  is less than the half-depth of 4.0 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:

$$f = 50 \left( \frac{3.567}{8 - 3.567} \right) = 40.23 \text{ ksi}$$

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

Second approximation:

Assuming that  $f = 40.23$  ksi,

$$p_{max} = 40.23 \text{ ksi}$$

$$\sqrt{\frac{p_{max}}{P_{cr}}} = \sqrt{\frac{40.32}{9.02}} = 2.11 > 0.673$$

$$b = w \sqrt{\frac{P_{cr}}{P_{max}}} \left[ 1 - 0.22 \sqrt{\frac{P_{cr}}{P_{max}}} \right] = 4.842 \text{ in.}$$

Element	Effective Length L (in.)	Distance from		
		Top Fiber y (in.)	Ly (in. <sup>2</sup> )	Ly <sup>2</sup> (in. <sup>3</sup> )
1	1.9150	7.9475	15.2200	120.961
2	0.7536	7.8609	5.9200	46.537
3	14.8300	4.0000	59.3200	237.280
4	0.7536	0.1396	0.1052	0.015
5	4.8420	0.0525	0.2542	0.013
Total	23.0942		80.8194	404.806

$$y_{cg} = \frac{80.8194}{23.0942} = 3.5 \text{ in.}$$

*Check the Effectiveness of the Web* — Using Section B2.3 of the AISI specification, the effectiveness of the web element can be checked as follows:

From Fig. 49.30c,

$$f_1 = 50(3.2075/4.50) = 35.64 \text{ ksi (Compression)}$$

$$f_2 = -50(4.2075/4.50) = -46.75 \text{ (tension)}$$

$$= f_2/f_1 = -1.312$$

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

$$= 33.341$$

$$\frac{h}{t} = \frac{7.415}{0.105} = 70.62 < 200 \text{ OK}$$

$$\lambda = \frac{1.052}{\sqrt{33.341}} (70.62) \sqrt{\frac{35.64}{29,500}}$$

$$= 0.447 < 0.673$$

$$b_e = h = 7.415 \text{ in.}$$

$$b_1 = b_e / (3 - \psi) = 1.72 \text{ in.}$$

Since  $\psi < -0.236$ ,

$$b_2 = b_e / 2 = 3.7075 \text{ in.}$$

$$b_1 + b_2 = 5.4275 \text{ in.}$$

Because the computed value of  $(b_1 + b_2)$  is greater than the compression portion of the web (3.2075 in.), the web element is fully effective.

*Moment of Inertia and Section Modulus* — The moment of inertia based on line elements is

$$2I'_3 = 2 \left( \frac{1}{12} \right) (7.415)^3 = 67.95$$

$$\Sigma(Ly^2) = 404.806$$

$$I'_z = 67.95 + 404.806 = 472.756 \text{ in.}^3$$

$$-(\Sigma L)(y_{cg})^2 = 23.0942(3.5)^2 = 282.9 \text{ in.}^3$$

$$I'_x = 189.85 \text{ in.}^3$$

The actual moment of inertia is

$$I_x = I'_x t = 189.85(0.105) = 19.93 \text{ in.}^4$$

The section modulus relative to the extreme tension fiber is

$$S_x = 19.93/4.50 = 4.43 \text{ in.}^3$$

*Nominal and Allowable Moments* — The nominal moment for section strength is

$$M_n = S_e F_y = S_x F_y = (4.43)(50) = 221.50 \text{ in-kips}$$

The allowable moment is

$$M_a = M_n / \Omega_f = 221.50 / 1.67 = 132.63 \text{ in-kips.}$$

## 49.4 Members Subject to Axial Load

Axial loading is a very common and very important type of loading, and the requirements to deal with this type of loading in cold-formed steel members vary according to the type of loading, tension or compression, and geometry and use of the member. Due to the thinness of the walls in cold-formed steel sections and the variety of different cross-sectional shapes that can be produced, types of behavior not commonly found in traditional hot-rolled members can occur, and these must be recognized and taken into account in design. Codes provide design methods to deal with the various phenomena associated with thin-walled sections in a fairly simple manner.

### Short Struts

Local buckling must be taken into consideration in the analysis of members in compression. We have seen in previous chapters how individual elements are dealt with in this regard. In the case of complete sections subjected to compression we must take into account the possibility of local buckling in all elements of a cross section. To do so we consider initially a short length of member that is acted upon by compressive loads, as shown in Fig. 49.31.

Due to the compressive loads, each element of the cross-section can suffer local buckling. We therefore consider each flat element in turn, find the effective width — and hence effective area — of the element, and sum these, together with the areas of the corners, to obtain the total effective area,  $A_{eff}$ . The ratio of the effective cross-sectional area to the full cross-sectional area,  $A$ , is denoted as  $Q$ , i.e.,

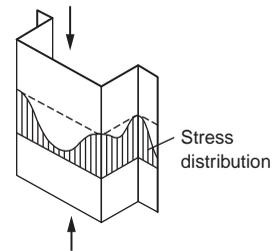


FIGURE 49.31 Short length of compressed member.

$$Q = \frac{A_{eff}}{A} \quad (49.30)$$

The factor  $Q$  was adopted partly because it described more realistically the actual situation in a cross-section, e.g., effective and ineffective portions.

The load capacity of a short strut under uniform compression is given by the product of the effective area ( $A_{\text{eff}}$ ) and the yield stress ( $Y_s$ ), i.e.,

$$P_{cs} = QAY_s \quad (49.31)$$

## Flexural Buckling

### Euler Buckling

We have seen how short uniformly compressed members behave and how the effects of local buckling must be taken into account in design analysis. For long members under compression, different modes of failure arise, due to overall buckling. We shall first consider buckling due to flexure, or Euler buckling. Euler buckling occurs when a long, slender member, i.e., a column, is compressed. The elastic buckling load, or Euler load, for such a column under pinned-end conditions is well known as

$$P_E = \frac{\pi^2 EI}{\ell^2} \quad (49.32)$$

where  $I$  = the relevant second moment of area  
 $E$  = the elasticity modulus  
 $\ell$  = the column length

By writing  $I = Ar^2$ , where  $r$  is the radius of gyration of the cross section corresponding to  $I$ , Eq. (49.32) can be put in terms of the critical, or Euler buckling, stress,  $p_E$ , as follows:

$$p_E = \frac{\pi^2 E}{(\ell/r)^2} \quad (49.33)$$

As the length of the column increases, the critical stress to cause Euler buckling decreases, so that for a very long column Euler buckling occurs at extremely low stress levels. In the case of local buckling we have seen that the local buckling stress is relatively unaffected by length. Thus for long columns the effects of local buckling do not arise, and in determining the Euler load for such a column we do not need to take local buckling into account.

### Effective Lengths

If the ends of a column are not pinned, but subject to some other degree of fixity, Eqs. (49.32) and (49.33) do not apply directly, but must be modified to take the actual end conditions into account. In design, this is often accomplished using the effective length concept, in which the actual column length  $L$  is replaced by an effective length  $L_E$  in the equations. The effective length of a column is normally taken as the distance between the points of contraflexure in a buckled column. Values for the effective length as a proportion of the actual length between supports are given in the AISI specification and BS 5950, Part 5, for a number of conditions of column support.

The ratio of effective length to the relevant radius of gyration of a column is termed the *slenderness ratio*. Maximum permitted values of the slenderness ratios of columns are given in codes for different types of members. For members that normally act in tension, but may be subject to load reversal due to the action of wind, high slenderness ratios are permitted. For members subjected to loads other than wind loads, the maximum slenderness ratio is given as 180 in BS 5950, and AISI stipulates a maximum value of 200.

In the design analysis of columns the complete range of slenderness ratios must be catered to. We have seen that for short columns local buckling is important and Euler buckling is of little consequence, while for long columns Euler buckling assumes the highest significance and local buckling has little effect. For

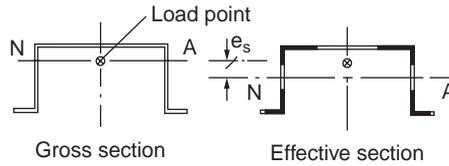


FIGURE 49.32 Neutral axis shift for locally buckled cross-section.

short members that are fully effective failure occurs when the load reaches the squash load, i.e.,  $Y_s \times A$ . If local buckling is present, this load is modified, due to the local buckling effects, to  $Y_s \times A_{\text{eff}}$ , or  $QY_s A$ . If the slenderness ratio is greater than a fixed value, Euler buckling occurs and the failure load reduces with an increase in the slenderness ratio.

Real columns are, of course, not perfect, and column imperfections cause some bending to occur even in very short members, thus hastening yield in these members and causing failure at loads less than the Euler load. It is imperative that the effects of imperfections are accounted for in the design analysis.

### Effects of Neutral Axis Shift

If we examine the gross cross section and the effective cross section together, as illustrated in Fig. 49.32, we can see that the effects of local buckling have been not only to alter its effective area, but also to change the geometry, since some elements have become more ineffective than others. Because of this the neutral axis of the effective cross-section moves from its original position as local buckling progresses. If the loading is applied at the centroid of the full cross section, it becomes eccentric to the centroid of the effective cross-section, thus inducing bending in the member.

It is therefore evident that any section that is not doubly symmetric and that is subject to loads inducing local buckling effects is likely to incur bending in addition to axial load if the loading is applied through its centroid. The degree of bending incurred depends on the distance that the effective neutral axis is displaced from its initial position, and this in turn depends on the degree of local buckling undergone by the member. Since this bending has the effect of reducing the column load capacity, and since the magnitude of the neutral axis shift increases with load, it should make for conservative estimates of load capacity if the neutral axis shift is determined on the basis of the short strut load,  $P_{cs}$ .

If the neutral axis of the effective section is displaced by an amount  $e_s$  from that of the gross cross section, the moment produced by a load applied through the original neutral axis is the product of load  $P$  and displacement  $e_s$ . To take the combination of axial load and moment into account a simple linear interaction formula is used:

$$\frac{M}{M_c} + \frac{P}{P_c} = 1 \quad (49.34)$$

where  $M_c$  is the moment capacity in the absence of axial load, determined as illustrated in the previous chapter, and  $P_c$  is the failure load of the column under uniform compression. At the ultimate load of the member,  $P$ , the moment acting is  $P \times e_s$ . Eq. (49.34) becomes

$$\frac{P' e_s}{M_c} + \frac{P'}{P_c} = 1 \quad (49.35)$$

The full effects of neutral axis shift will not be incurred in practice for columns that are not, in fact, pinned end. If the effective length of a column is less than the full length between supports, any accurate assessment of the effects of neutral axis shift is complex, and there is as yet no satisfactory solution to this question. Experimental results suggest that for completely fixed ends the effects of neutral axis shift may be completely neglected in assessing the column capacity.

# Torsional Flexural Buckling

## Theoretical Basis

Apart from local buckling, perhaps the major difference in behavior between hot-rolled steel and cold-formed steel structural members is to be found in the relative susceptibility of the latter to *torsional flexural buckling*. Designers in hot-rolled steel do not come across this phenomenon to a great extent, partly because hot-rolled steel sections are generally thicker and more compact than cold-formed steel sections, but more generally because of the greater variety of sectional shapes that are designed in cold-formed steel. When dealing with members that are of arbitrary cross-section, a more general theoretical approach must be adopted than that used in the earlier sections of this chapter.

Consider a member having a generally unsymmetrical cross section, as depicted in Fig. 49.33. If this member is loaded in compression, it is not possible to determine by inspection the direction in which the cross-section will move during buckling. For such a cross section, on the basis of classical theory, which is detailed in Murray (1984) and Allen and Bulson (1980), the deflections of the member will have components in the x and y directions and twisting will also occur about the shear center, or center of twist. Indeed, if precise analysis of the situation were to be carried out, it would be found that distortion, or change in shape of the cross-section, is also a distinct possibility in thin-walled sections, but this complicates the analysis considerably.

The application of classical theory to deal with cold-formed steel sections has been researched extensively at Cornell University in the U.S. (Chajes and Winter, 1965; Peköz and Celebi, 1969), and Yu (1991) gives a thorough summary of the design approach based on this work. If we consider that the section of Fig. 49.33 is loaded through its centroid, and axes x-x and y-y are the principal axes, then the buckling load, which in the general case is due to a combination of biaxial flexure and twisting, and thus denoted  $P_{TF}$  may be obtained from the following equation:

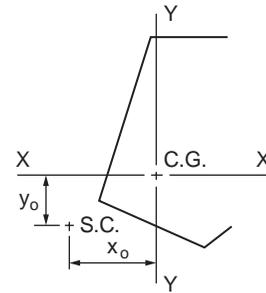


FIGURE 49.33 Generally unsymmetrical cross-section.

$$\frac{I_c}{A} (P_{TF} - P_{EY})(P_{TF} - P_{EX})(P_{TF} - P_T) - P_{TF}^2 \{ y_0^2 (P_{TF} - P_{EX}) + x_0^2 (P_{TF} - P_{EY}) \} = 0 \quad (49.36)$$

In this equation  $I_c$  is the polar second moment of area with respect to the shear center of the section;  $P_{EX}$  and  $P_{EY}$  are the critical loads for buckling about the x-x and y-y axes, respectively; and  $P_T$  is the torsional buckling load. The dimensions  $x_0$  and  $y_0$  are the distances between the centroid of the section and its shear center measured in the x and y directions, respectively. The smallest root of the equation gives the value of  $P_{TF}$  of interest, and this is always less than or equal to the smallest value of the individual critical loads.

If the member has simple support conditions, as normally defined, at its ends, then  $P_{EX}$  and  $P_{EY}$  are simply the Euler loads for buckling about the x-x and y-y axes, respectively. The torsional buckling load,  $P_T$ , however, is not fully described by the commonly accepted simple support conditions; closer examination of the support conditions must be carried out to define this load.

$P_T$  is defined by the following equation:

$$P_T = \frac{1}{r_0^2} \left( GJ + k \pi^2 E \frac{C_w}{L^2} \right) \quad (49.37)$$

where  $G$  = the shear modulus for the material  
 $J$  = the torsion constant for the section

$C_w$  = the *warping constant* for the section  
 $r_o$  = the polar radius of gyration about the shear center, given by

$$r_o = \left( r_x^2 + r_y^2 + x_o^2 + y_o^2 \right)^{1/2} \quad (49.38)$$

where  $r_x$  and  $r_y$  are radii of gyration corresponding to the  $x$  and  $y$  axes, respectively.

The constant  $k$  is dependent on the degree of warping restraint afforded by the end connections of the column. If warping is completely unrestrained, then  $k = 1$ . There is, as can be seen, an analogy with the Euler load for flexural buckling, in which a fully fixed column has a buckling load four times as great as the corresponding simply supported column.

### Warping Restraint

We can see that a column that is nominally simply supported, or simply supported with regard to flexural buckling, may exhibit a wide range of variation in buckling load under the more general torsional flexural buckling situation. If the walls of such a column are very thin, then for column lengths of commercial applicability the torsion constant,  $J$ , which is equal to  $Sbt^3/3$ , can become very small in comparison with  $C_w/L_E^2$ , and the degree of warping restraint becomes very important.

The degree of warping restraint by different types of end connections is not easy to quantify, as it depends on a wide range of factors. However, warping of the ends is often prevented by the end plates, and the effects of torsional flexural buckling are often minimized. This assumption of full warping restraint is therefore optimistic for the general case.

In the AISI specification, and in the various design codes based on the AISI specification, the opposite view is taken. Here it is assumed that for safe design no account should be taken of the effects of connections on warping restraint, and so torsional flexural buckling is based on  $k = 1$ . This gives much lower design loads for many cases and results in torsional flexural buckling being the governing design criterion for a much wider range of section shapes than is the case. Eurocode 3, Part 1.3, considers a column to have an effective length with regard to torsional flexural buckling, and this effective length may be different from that for flexural buckling and depends on the degree of torsion and warping restraint at the column ends. This code gives some indicative drawings of the connections for which restraint may be considered sufficient to take the restraints applied into account in estimating the effective length.

### Members under Combined Bending and Compression

Under practical conditions structural members are very often subjected to combinations of bending and axial loading. The interaction of the different effects must be taken into account when this occurs. When the axial loading is compressive there are possibilities that the different types of buckling that can occur for both beams and columns may interact with each other, and this must be guarded against. Even in the case of members subjected to combinations of bending and tension there are possibilities that buckling of some form may occur, and this must be taken into consideration. In this section, however, we shall consider combinations of compressive forces and bending.

Under hypothetical simplified conditions in which buckling effects are absent and a member behaves perfectly elastically until failure occurs when the maximum stress reaches yield, the effects of each different type of loading that acts on a member can simply be added together to produce an equation governing the capacity under simultaneous applications of moment,  $M$ , and axial load,  $P$ :

$$\frac{P}{P_s} + \frac{M}{M_y} = 1 \quad (49.39)$$

where  $P_s$  = the load capacity in the absence of moments  
 $M_y$  = the moment capacity in the absence of axial loads

If buckling possibilities are negligible and if the material has any postyield capacity, this type of equation tends to give conservative results of the load-carrying capacity of the member under combined loadings. If, on the other hand, there is any tendency for the member to undergo buckling, then this type of equation can underestimate the degree of interaction and give nonconservative estimates of carrying capacity.

The AISI recommends that the axial force and bending moments satisfy the following interaction equations:

$$\frac{P}{P_a} + \frac{C_{mx} M_x}{M_{ax} \alpha_x} + \frac{C_{my} M_y}{M_{ay} \alpha_y} \leq 1.0 \quad (49.40)$$

$$\frac{P}{P_{a0}} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1.0 \quad (49.41)$$

When  $P/P_a \leq 0.15$ , the following formula may be used in lieu of the above two formulas:

$$\frac{P}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1.0 \quad (49.42)$$

where

$P$  = the applied axial load

$M_x$  and  $M_y$  = the applied moments with respect to the centroidal axes of the effective section determined for the axial load alone

$P_a$  = the allowable axial load determined in accordance with Section C4

$P_{a0}$  = the allowable axial load determined in accordance with Section C4, with  $F_n = F_y$

$M_{ax}$  and  $M_{ay}$  = the allowable moments about the centroidal axes determined in accordance with Section C3

$C_{mx}$  and  $C_{my}$  = the coefficients, whose values shall be taken from the AISI specifications

$1/\alpha_x$  and  $1/\alpha_y$  = the magnification factors,  $1/[1 - (O_c P/P_{cr})]$ , in which  $O_c$  is the factor of safety used in determining  $P_a$  and

$$P_{cr} = \frac{\pi^2 EI_b}{(K_b L_b)^2} \quad 46$$

For the above equation,  $I_b$  is the moment of inertia of the full, unreduced cross-section about the axis of bending;  $L_b$  is the actual unbraced length in the plane of bending; and  $K_b$  is the effective length factor in the plane of bending.

BS 5950 considers two different possibilities that should be examined in dealing with the interaction of axial compression and bending:

1. The capacity of the member may be exceeded locally at discrete points at which the maximum loadings occur.
2. The overall capacity of the member to resist buckling due to the action of the combined loadings may be exceeded.

The first possibility is likely to occur in members subjected largely to bending with some additional axial loading. At points of maximum moment, as indicated in Fig. 49.34, the effects of the axial loading on the ultimate condition must be considered. This situation can be adequately covered by a linear interaction equation, and in BS 5950, Part 5, the following expression is used to check

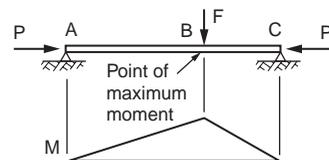


FIGURE 49.34 Moment variation along a member with axial and transverse loading.

the local capacity of a member subjected to axial load  $F_c$  and bending moments  $M_x$  and  $M_y$  about the  $x$  and  $y$  axes, respectively:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \quad (49.43)$$

where  $P_{cs}$  is the axial load capacity in the absence of moments and  $M_{cx}$  and  $M_{cy}$  are the moment capacities if the member is subjected only to bending about the relevant axis in the absence of all other load actions.

This equation should be satisfied at discrete points on the member where the local load or moment magnitudes are at their peak. It is worthy of mention here that this equation takes account of local buckling effects, since the calculated load and moment capacities are determined on the basis of the methods already described in this book.

The second possibility that must be considered is that the overall buckling capacity of the member may be attained by a combination of loads. The types of buckling, in addition to local buckling, that are possible are flexural buckling or torsional flexural buckling of members loaded largely in compression and lateral torsional buckling of members loaded largely in bending. A suitable overall buckling capacity check should consider all of these possibilities. BS 5950, Part 5, prescribes two interaction equations to deal with the overall buckling capacity check; the particular equation to be used in any given situation is dependent upon whether or not lateral torsional buckling is a possibility. Eurocode 3, Part 1.3, uses a single interaction equation for combined bending and compression (although a modified equation must also be satisfied for members susceptible to lateral torsional buckling), and this equation is substantially different from those of the AISI code or the U.K. code. Comparisons of all three codes with experimental results have not been conclusive to date and suggest that despite the differences in setup the various methods are reasonably accurate and safe.

## Members under Combined Bending and Tension

In the case of members subjected to combinations of bending and tension there is not, in general, any need for an overall buckling capacity check, as the application of tensile loads has no tendency to increase the possibility of overall buckling. There are, of course, possibilities that the effects of local buckling may be present in the member, since not all elements of a cross-section will be in tension under the combination of loads.

In BS 5950, Part 5, an interaction equation of the same form as Eq. (49.39) is used to check the local capacity at discrete points on a member. The relevant equation is

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \quad (49.44)$$

with

$$\frac{M_x}{M_{cx}} \leq 1 \quad \frac{M_y}{M_{cy}} \leq 1$$

In this equation  $F_t$  and  $P_t$  are the applied tensile load and the tensile capacity of the member, respectively;  $M_{cx}$  and  $M_{cy}$  are the moment capacities computed in the absence of any other loading. Since  $M_{cx}$ , for example, is evaluated on the basis of an effective cross-section, local buckling is automatically taken into account in the interaction equation.

### Example 49.4

Compute the allowable axial load for the square tubular column shown in Fig. 49.35. Assume that the yield point of steel is 40 ksi and  $K_x L_x = K_y L_y = 10$  ft.

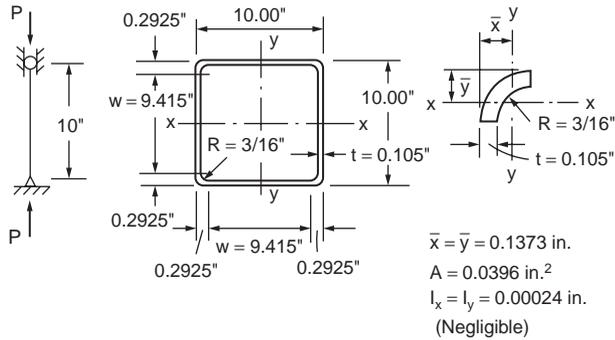


FIGURE 49.35 Example 49.4.

**Solution:**

The solution given below is based on the AISI code.

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

*Sectional Properties of Full Section*

$$b = 10.00 - 2(R + t) = 10.00 - 2(0.1875 + 0.105) = 9.415 \text{ in.}$$

$$A = 4(9.415 \times 0.105 + 0.0396) = 4.113 \text{ in.}^2$$

$$\begin{aligned}
 I_x = I_y &= 2(0.105) \left[ \frac{1}{12} (9.415)^3 + 9.415 \left( 5 - \frac{0.105}{2} \right)^2 \right] + 4(0.0396) (5.0 - 0.1373)^2 \\
 &= 66.75 \text{ in.}^4
 \end{aligned}$$

$$r_x = r_y = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{66.75}{4.113}} = 4.029 \text{ in.}$$

*Nominal Buckling Stress,  $F_n$*  — The elastic flexural buckling stress,  $F_e$ , is computed as follows:

$$\frac{KL}{r} = \frac{10 \times 12}{4.029} = 29.78 < 200 \quad \text{O.K.}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29,500)}{(29.78)^2} = 328.3 \text{ ksi}$$

Since  $F_e > F_y/2 = 20 \text{ ksi}$ ,

$$\begin{aligned}
 F_n &= F_y \left( 1 - F_y / 4F_e \right) \\
 &= 40 \left[ 1 - 40 / (4 \times 328.3) \right] \\
 &= 38.78 \text{ ksi}
 \end{aligned}$$

*Effective Area,  $A_{eff}$*  — Because the given square tube is composed of four stiffened elements, the effective width of stiffened elements subjected to uniform compression can be computed by using  $k = 4.0$ :

$$b/t = 9.415/0.105 = 89.67$$

$$P_{cr} = \frac{4\pi^2 E}{12(1-\nu^2)} \frac{1}{(b/t)^2} = 13.26 \text{ ksi}$$

$$\sqrt{\frac{f_{max}}{P_{cr}}} = \sqrt{\frac{40}{13.26}} = 1.737$$

Since  $\sqrt{\frac{f_{max}}{P_{cr}}} > 0.63755$ ,

$$b_{eff} = b \sqrt{\frac{P_{cr}}{f_{max}}} \left[ 1 - 0.22 \sqrt{\frac{P_{cr}}{f_{max}}} \right]$$

$$= 4.73 \text{ in.}$$

The effective area is  $(4.73 \times 0.105 + 0.0396) = 2.145 \text{ in.}^2$

Nominal and Allowable Loads:

$$P_n = A_e F_n = (2.145)(38.78) = 83.18 \text{ kips}$$

$$P_a = P_n / \Omega_c = 83.18 / 1.92 = 43.32 \text{ kips}$$

The use of  $\Omega_c = 1.92$  is because the section is not fully effective.

## 49.5 Connections for Cold-Formed Steelwork

All of the connection methods applicable to hot-rolled sections, such as bolting and welding, are also applicable to cold-formed steel sections at the thicker end of the range. In the case of thinner cold-formed steel sections an extremely wide assortment of proprietary fasteners and fastening techniques exists. This wide range raises problems in setting realistic and reliable approaches to defining connection strength, and evaluation of the connection properties on the basis of testing is generally undertaken.

### Types of Fastener

Davies (Rhodes, 1991) listed fasteners in three main groupings, as shown in [Table 49.2](#)

The selection of the most suitable type of fastener for a given application is governed by several factors, notably:

1. load-bearing requirements viz. strength, stiffness, and deformation capacity
2. economic requirements viz. number of fasteners required, cost of labor and materials, skill required in fabrication, design life, maintenance, and ability to be dismantled

**TABLE 49.2** Typical Applications of Different Types of Fastener

Thin to Thin	Thin to Thick or Thin to Hot Rolled	Thick to Thick or Thick to Hot Rolled
Self-drilling self-tapping screws	Self-drilling self-tapping screws	Bolts
Blind rivets	Fired pins	Arc welds
Single-flare V welds	Bolts	
Spot welds	Arc puddle welds	
Lock seaming		

**TABLE 49.3** Permissible Bearing Stress in Bolted Connections

Code	Bearing Stress	Basis
Addendum 1 (April 1975) to BS 449	$1.4U_s$	Permissible stress
BS 5950, Part 1 (hot-rolled)	$0.64(U_s + Y_s)$	Limit state
BS 5950, Part 5 (cold formed) <sup>a</sup>		Limit state
(a) $t \leq 1$ mm	$2.1Y_s$	
(b) $1 \text{ mm} \leq t \leq 3$ mm	$(1.65 + 0.45t)Y_s$	
(c) $3 \text{ mm} \leq t \leq 8$ mm	$3.0Y_s$	
AISI specification (August 1986)		
(a) with washers		
(b) without washers		Permissible stress
European recommendations		
(a) $t \leq 1$ mm	$1.35U_s$ to $1.50U_s$	
(b) $3 \text{ mm} \leq t \leq 6$ mm	$1.00U_s$ to $1.35U_s$	
Eurocode 3, Part 1.3		Limit state
	$2.1Y_s$	
	$4.3Y_s$	
	$2.5 U_s$	Limit state

Note:  $Y_s$  = yield stress of fastened sheet,  $U_s$  = ultimate stress of fastened sheet.

<sup>a</sup> Values in BS 5950, Part 5, are reduced by 25% unless two washers are used.

3. durability viz. resistance to aggressive environments
4. watertightness
5. appearance

Although structural engineers tend to be primarily concerned with the most economical way of meeting the load-bearing requirements, in many applications other factors may be equally important.

### Bolts

The use of bolts to connect cold-formed steel components follows a practice similar to that of hot-rolled construction; however, because of the thinness of the material and the relatively small size of the components, the main design considerations tend to be end distance and bearing. With regard to bearing it is worth noting that hot-rolled steel design codes tend to have significantly different design treatments of this than cold-formed steel codes. This may be partially due to the different behavior of thin material, but is mainly due to the adoption of different philosophies by the writers of cold-formed steel and hot-rolled steel codes. In British codes in particular, the cold-formed steel rules are based on strength design, while the hot-rolled steel rules are based on limiting slip. [Table 49.3](#), from Davies (Rhodes, 1991), shows the comparison between permissible bearing stresses in bolted connections according to various codes.

### Self-Tapping Screws

Self-tapping screws fall into two distinct types, depending on whether or not they require a predrilled hole. Conventional self-tapping screws require a predrilled hole and fall into a number of subgroups, depending on the type of thread, head, and washer. Self-drilling self-tapping screws drill their own hole and form their own thread in a single operation. There are two basic types, depending on the thickness of the base material. Both of these screws are usually combined with washers that serve to increase the load-bearing capacity or sealing ability.

### Blind Rivets

Blind rivets are normally used for fastening two thin sheets of material together when access is available from one side only. These are installed in a single operation, for example, by pulling a mandrel that forms a head on the blind side of the rivet and expands the rivet shank. Fastening is completed when the mandrel either pulls through or breaks off. This type of fastener generally requires strength considerably in excess of the sheet material to minimize possibilities of brittle failure.

## Fired Pins

These pins, as the name suggests, are fired through thin material into thicker base material to form a connection. Two different methods of firing the pins are commonly used: powder actuation and pneumatic actuation. In the former the pins are fired from a tool that contains an explosive cartridge, and in the latter compressed air is used as the firing agent. Fired pins generally provide a very tight grip and a very good sealing capability.

## Spot Welds

Electrical resistance spot welds are a widely used method of connecting thin sheets. In the United Kingdom these are governed by BS 1140, General Requirements for Spot Welding of Light Assemblies in Mild Steel, and recommendations regarding weld sizes and capacities are provided in BS 5950, Part 5, and the AISI specification. Further information on loads in resistance spot welds has been given by Baehre and Berggren (1973).

## Arc Welds

Conventional fillet and butt welds are applicable to cold-formed steel sections if the material is relatively thick, and in such cases design considerations are the same as those for hot-rolled sections. In the case of thinner material a wide range of special weld types are used in, but not outside of, the U.S. . Guidance on making these special welds has been given by Blodgett (1978). Expressions for the evaluation of the ultimate loads for various types of these connections are available from a large testing and analysis program carried out by Peköz and McGuire (1979).

## Clinch Joints and Rosette Joints

A relatively new method of connecting components together is *mechanical clinching*. This technique was originally developed for use in the automobile industry, but in recent years has found a number of applications in cold-formed steel construction and has been the subject of substantial research (e.g., Davies et al., 1996). In making clinched joints, hydraulic tools are used as a punch-and-die set that shears and deforms the material to be connected to produce a connection in which material from one sheet bears on material from another, and the connection is dependent only on the parent material.

Another type of connection that is uniquely suited to light-gauge material and requires no additional fixings is the Rosette joint. In this joint holes are punched into the two adjoining strips of material at the connection point, with one of the sheets having a reduced diameter to form a collar. In the joining process the collar is snapped into the hole in the adjoining strip, and the Rosette tool penetrates the hole and collar, expands, and is pulled back to crimp the collar to the opposite side of the hole, thus producing a secure connection. Details of Rosette joints may be found, for example, in Makelainen et al. (1999).

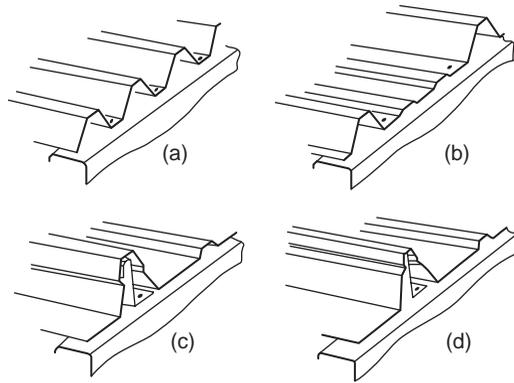
## Assemblies of Fasteners

The available information on the the behavior of assemblies of fasteners in light-gauge steelwork suggests that the performance of such assemblies is not in any way inferior to that of similar connection assemblies in hot-rolled members. Connections in cold-formed members, however, do tend to be more flexible and ductile than in hot-rolled members, and herein lies a bonus that is well appreciated in some areas of cold-formed steel usage. The ductility afforded by such connections can provide the “plastic plateau” behavior that often occurs in hot-rolled sections, thus allowing redistribution of moments in the postyield range. A prime example of this in the United Kingdom is the wide use of “sleeves” in purlin design. These sleeves are used to connect purlins in a semirigid fashion, which produces a moment distribution close to the ideal at the point of failure.

## 49.6 Sheeting and Decking

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There has been in the recent past a substantial increase in the usage of profiled sheeting and decking, and this has been accompanied by a corresponding development of the design principles applicable to



**FIGURE 49.36** Typical roof sheeting profiles.

this type of construction. Although only a few years ago 3-in. corrugated profiles or simple trapezoidal profiles were the norm, today there is a multiplicity of different shapes available. Profiles can be obtained in a wide variety of colors, surface coatings, etc., with a life to first maintenance of up to 25 years. There have been a large number of developments of structural improvements, involving the use of thinner material, of increased yield strength in highly stiffened profiles. It is most probable, however, that in this area improvements in aesthetics, utility, and durability outweigh the structural improvements.

### Profiles for Roof Sheeting

*Cold roof construction* is the term applied to construction in which profiled steel sheeting is used as the outer waterproof skin of a roof, with insulation placed inside this skin. It is important that the skin should provide the optimum resistance to moisture penetration, and the side laps should occur near the crests of the corrugations. With modern fasteners, e.g., self-drilling self-tapping screws with neoprene washers, it is quite permissible to fix the sheeting to the roof purlins through the troughs. These fasteners satisfactorily prevent moisture penetration, while providing various structural benefits in the fixity provided to the purlins over through-crest fixings.

Modern profiles can be grouped into four main types, as illustrated in Fig. 49.36. For the trapezoidal profile shown in Fig. 49.36a, it is found that to obtain the most economic use of material for a given span and loading, using standard analysis procedures, the thickness of material and web slope angle that result are impractical, the thickness usually being very small and the web slope very shallow. Fairly wide variations in the trapezoidal geometry result in relatively small variations in the economy, and the design of this type of sheeting is generally governed by practical considerations. In the profile of Fig. 49.36b, again a fairly widely used shape of profile, the trapezoidal shape is altered to incorporate intermediate stiffeners in the lower flanges. In order to reduce or eliminate the requirement to penetrate the skin with fasteners, concealed fixed or standing seam sheeting has been developed over a number of years (Fig. 49.36c and d). The fixing in these systems is generally by means of clips that connect mechanically to the sheeting and by screws to the purlin, with the mechanical connections to the sheeting often incorporated in the standing seam, eliminating penetration of the sheeting. In this type of system the purlin–sheeting connections are less useful in providing stability to the purlin than the more direct screw fixings, and may necessitate close consideration of purlin design in such circumstances.

### Profiles for Roof Decking

Profiled steel decking, supporting a built-up finish, including insulation and waterproofing, is often termed *warm roof construction*. In this type of construction there is a tendency toward longer spans, more complex profiles, and the use of higher strength steels. A typical profile resulting from these tendencies is shown in Fig. 49.37. The profile depth is of the order of 4 in., and the material yield strength is of the

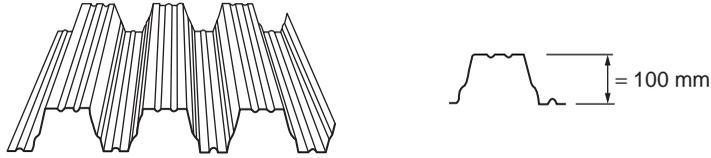


FIGURE 49.37 Typical long-span decking profiles.

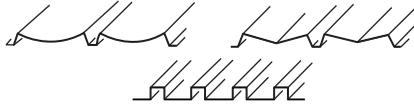


FIGURE 49.38 Typical wall cladding profiles.

order of 50 ksi. These profiles are suitable for spans up to 30 ft and can thus eliminate the need for purlins. In general, stiffeners are rolled into both the flanges and the webs, as flanges and webs are slender and require stiffeners for efficiency.

There are nowadays a large number of different design specifications dealing with roof decking. In Europe calculation procedures for complex shapes, as shown in Fig. 49.37, have been developed in Sweden for incorporation into the Swedish code of practice (Höglund, 1980), and these procedures form the basis of the European recommendations (ECCS, 1983), as well as the national standards of some European countries, for example, DIN 18807 (1987). These procedures have been verified by a very substantial number of tests (Baehre and Fick, 1982).

## Wall Coverings

Wall cladding carries significant wind loading and must satisfy structural considerations. However, the aesthetic considerations here are so strong, since the appearance of a building is very substantially dependent on the sheeting, that aesthetics are often the primary consideration. Typical wall cladding profiles are shown in Fig. 49.38. By virtue of the variety of profiles available, the wide range of coatings and colors, and the possibility of orienting the cladding in any desired direction, there is substantial scope for imagination and artistry in the use of modern wall coverings in building design.

## Composite Panels

Composite panels, having outer faces of thin steel (or other material such as aluminum) and an internal core of foamed plastic (such as polyurethane or expanded polystyrene), are now widely used in construction (Fig. 49.39). The design criteria for composite panels are complex and are not discussed here.

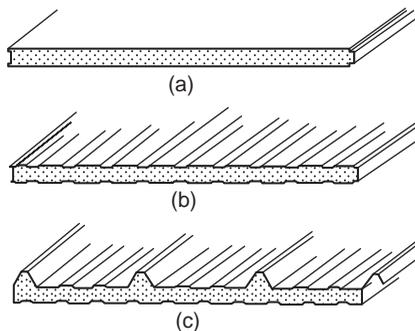


FIGURE 49.39 Typical sandwich panels: (a) panel with flat faces, (b) panel with profiled faces, (c) panel with profiled face.

However, this type of construction must be mentioned because of its current growth and future potentialities. Foam-filled composite construction utilizes the favorable attributes of skin and core materials in combination. The core connects the metal skins, while ensuring that they are kept a distance apart to provide flexural capacity for the panel, and also provides a degree of resistance to local buckling of the skins. The skins protect the core from accidental damage and from the elements. Foam-filled panels combine a high load-carrying capacity with low weight, attractive appearance, and other made-to-order attributes, such as an extremely high thermal insulation capacity, to provide very good solutions to a variety of construction problems.

## 49.7 Storage Racking

The term *storage racking* covers an extremely wide range of products that have as their purpose the storage of material in a secure and easily accessible manner. Storage racking systems range from small shelving systems to extremely large rack-clad buildings. The components used in storage racking are largely of cold-formed steel, and the storage racking industry is one of the major users of cold-formed steel.

The forerunners of the modern storage structures were slotted-angle products first introduced in the 1930s. These consisted of cold-formed angle sections with perforations to provide a means whereby the designer could utilize the simplicity and flexibility of connections thus permitted to produce storage systems in a variety of shapes and configurations. By this means simple shelving could be provided for warehouses of any size. From these beginnings natural evolution led to the present-day situation. It was found that for many systems bolted connections could be beneficially replaced by clips or other proprietary connections, and a wide variety of alternative connection methods were developed for different systems. Although perforated angles had much in their favor for some purposes, particularly for smaller storage systems, the requirements for bracing in these torsionally weak members meant that for many purposes other shapes of cross-sections were superior. Nowadays slotted angles are still widely used for shelving systems, but in larger storage racking systems other perforated members, generally monosymmetric, are used as columns, while normally unperforated members are used as beams.

### Components

#### Beams

In pallet racking the most common types of beams used are boxed beams, made from two interlocking lipped channels (Fig. 49.40a), and open-section beams (Fig. 49.40b), which may be stepped to receive the edge of a shelf (Fig. 49.40c). The boxed beam is generally more structurally efficient than the open beam, but for lighter loads the economical aspects can favor the open section. Over the normal range of spans used in storage racking the limiting factor for the beams used is most often the bending capacity. The most common configuration of beam loading is that shown in Fig. 49.41 with two pallets in place.

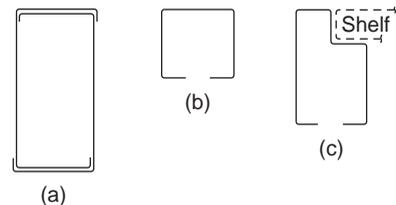


FIGURE 49.40 Typical beam profiles: (a) box, (b) open, (c) stepped open.

#### Uprights

Figure 49.42 shows typical perforated upright profiles used in storage racking at the present time. These are, in the main, developments from the simple lipped channel section. The developments generally involve the addition of more bends, which can be used to assist the attachment of bracing members, as well as to enhance the structural capacity of the uprights. The bracing members, often of channel section,

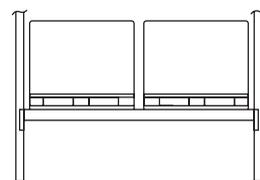


FIGURE 49.41 Two-pallet beam loading.

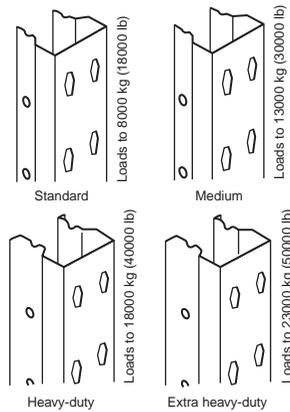


FIGURE 49.42 Typical upright profiles.

connect through the rear circular holes of the section, while the beams are connected, via beam end connectors, through the perforations on the front face. The loads carried by the uprights are spread into the floor through base plates. These are normally thin plates, of about 3 mm thick, which are bolted to the upright and to the floor.

### Beam End Connectors

The beams used in storage racking are generally connected to the uprights via beam end connectors. These are generally made from hot-rolled material and are welded to the ends of the beam to provide a means of connecting the beam to the upright at any of a large number of positions through the perforations in the uprights. The connectors have hooks that fit through the upright perforations to provide a semirigid connection between the beam and upright. In evaluation of the behavior of beams and uprights the connection behavior is very important. The moment rotation behavior of beam end connectors is normally obtained on the basis of testing, and the strength and stiffness values so obtained are used in design analysis of the structure.

### Design Codes of Practice

In Europe storage racking systems are generally considered to lie outside the scope of the normal codes of practice for structural design. This is largely because of the presence of perforations in uprights and other components. In the United Kingdom the Storage Equipment Manufacturers Association (SEMA) devised and published its own code of practice (SEMA, 1980). In Europe the Federation Europeene de la Manutention (FEM) has in recent years prepared a new code of practice (FEM, 1986) that is at completion stage and will have currency throughout Europe, eventually superseding the SEMA code in the United Kingdom and similar codes in other countries. This code is written in limit state to be compatible with Eurocode 3.

### Defining Terms

**Beam** — A straight or curved structural member, primarily supporting loads applied at right angles to the longitudinal axis.

**Beam-column** — A beam that also functions to transmit compressive axial force.

**Bifurcation** — A term relating to the load-deflection behavior of a perfectly straight and perfectly centered compression element at critical load.

**Blind rivet** — Mechanical fastener capable of joining workpieces together where access to the assembly is limited to one side only.

**Buckle** — To kink, wrinkle, bulge, or otherwise lose original shape as a result of elastic or inelastic strain.

**Buckling load** — The load at which a compressed element, member, or frame collapses in service or buckles in a loading test.

**Cladding** — Profiled sheet for walls.

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

**Composite slab** — Floor in which the structural bearing capacity is formed by the cooperation of concrete and floor decking (as reinforcement).

**Critical load** — The load at which bifurcation occurs, as determined by a theoretical stability analysis.

**Effective length** — The equivalent or effective length that, in the buckling formula for a hinged-end column, results in the same elastic critical load as that for the framed member or other compression element under consideration at its theoretical critical load.

**Effective width** — Reduced width of a compression element in a flexure or compression members computed to account for local buckling when the flat width-to-thickness ratio exceeds a certain limit.

**Flat width** — Width of the straight portion of the element, excluding the bent portion of the section.

**Flat width-to-thickness ratio** — Ratio of the flat width of an element to its thickness.

**Flexural buckling** — Buckling of a column due to flexure.

**Initial imperfection** — Deviation from perfect geometry, for example, initial crookedness of a member or initial out-of-flatness of a plate.

**Limit state** — Condition beyond which a structure would cease to be fit for its intended use.

**Local buckling** — Buckling of the walls of elements of a section characterized by a formation of waves or ripples along the member. It modifies the capacity of cross-sections.

**Multiple stiffened element** — An element stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress.

**Residual stress** — The stresses that exist in an unloaded member after it has been formed into a finished product.

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

**Self-drilling screw** — Screw that drills its own hole and forms its mating thread in one operation.

**Self-tapping screw** — Screw that taps a counterthread in a prepared hole.

**Sheeting** — Profiled sheet for floors, roofs, or walls.

**Stiffened or partially stiffened element** — A flat compression element in which both edges parallel to the direction of stress are stiffened either by a web, flange, stiffening lip, or intermediate stiffener.

**Tension field action** — Postbuckling behavior of a plate girder panel under shear force, during which diagonal compressive stresses cause the web to form diagonal waves.

**Thickness** — Thickness of base steel in cold-formed sections.

**Torsional buckling** — Buckling of a column by twisting.

**Torsional flexural buckling** — A mode of buckling in which compression members can bend and twist simultaneously. This type of buckling mode is critical, in particular when the shear center of the section does not coincide with the centroid.

**Unstiffened element** — A flat compression element that is stiffened at one of the two edges parallel to the direction of stress.

**Web crippling** — Failure mode of the web of a beam caused by the combination of a bending moment and a concentrated load.

**Yield point** — The maximum stress recorded in a tensile or compressive test of steel specimen prior to entering the plastic range.

**Yield strength** — In a tension or compression test, the stress at which there is a specified amount of measured deviation from an extension of the initial linear stress–strain plot.

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## Further Information

The *Guide to Stability Design Criteria for Metal Structures*, by Theodore V. Galambos, is the most comprehensive treatment of stability design of metal structures. It covers a wide range of topics, including the postbuckling strength of unstiffened and stiffened plates, plate girders, and thin-walled metal girders.

*Cold-Formed Steel in Tall Buildings*, edited by W.W. Yu et al., deals with the state-of-the-art design guide devoted to the most efficient and economical use of cold-formed steel in high-rise buildings.

*Background to Buckling*, by H.G. Allen and P.S. Bulson, provides a coherent account of the buckling problem, including an analytical treatment for studying the stability of plates and shells. Provides a valuable aid to forging the vital links between research, analysis, and design.

*Plated Structures: Stability and Strength*, by R. Narayanan, deals with various aspects related to stability problems of plated structures.

*The Stressed Skin Design of Steel Buildings*, by E.R. Bryan, and *Manual of Stressed Skin Diaphragm Design*, by E.R. Bryan and J.M. Davies, provide design methods and recommendations for roof decking and wall cladding profiles.

*The Proceedings of the International Conference on Cold-Formed Steel Structures*, edited by W.W. Yu et al. and available in a number of volumes, extensively covers all aspects of cold-formed steel structures.