## 16.1 INTRODUCTION

Light-gauge steel evolved as a building material in the 1930's and reached large-scale usage only after the Second World War. In comparison with conventional steel construction, where standard hot-rolled shapes are used, the cold-formed light-gauge steel structures are a relatively new development. Light-gauge steel sections are cold-formed in rolls by rolling the material in cold condition or by bending the steel sheets or strips in press brakes; cold-rolling being used for mass production while press brakes are used for economical production of small quantities of special shapes. These are used widely in structures subjected to light or moderate loads or for members of short span lengths. For such structures, the use of conventional hot-rolled shapes is often uneconomical because the stresses developed in the smallest available shape may be very low. Further, a variety of light-gauge members can be formed in the cold state with ease and the material can be used in the most effective manner. Earlier, use of light gauge steel was limited to building construction only, but now it finds application in trucks and trailer bodies, rail coaches, etc. Light gauge members are connected by spot, fillet, plug or slot welds or by screws, rivets, bolts, etc. For specifications of light-gauge steel sections IS 811 and for analysis and design, IS 801 should be referred.

## 16.2 SHAPES

The shapes which can be cold-formed are many and varied. Generally, the shape varies with its applications. Engineers have learned to adopt this versatility to advantage in design of compression members, studs, joists, beams, roof and floor panels and other industrial structural members. In the design of structural sections for framing members the main aim is to develop shapes which combine economy of material (that is a favourable strength–weight ratio) with versatility, ease of mass production, and provision for effective and simple connection to other structural members or to non-structural collateral material or both of them.

The usual shapes of framing members are channels, zees, angles, hat sections, tees, I-Sections and tubular sections and are shown in Fig. 16.1. These sections, 50 to 300 mm in depth can carry substantial loads and are used as primary framing members in residential, commercial and industrial buildings up to two stories in height and roof trusses up to 15 m span. Framing members formed with light-gauge structural steel are cold-formed from steel sheets or strips. Thickness of the framing members ranges from 1.2 to 4.0 mm.

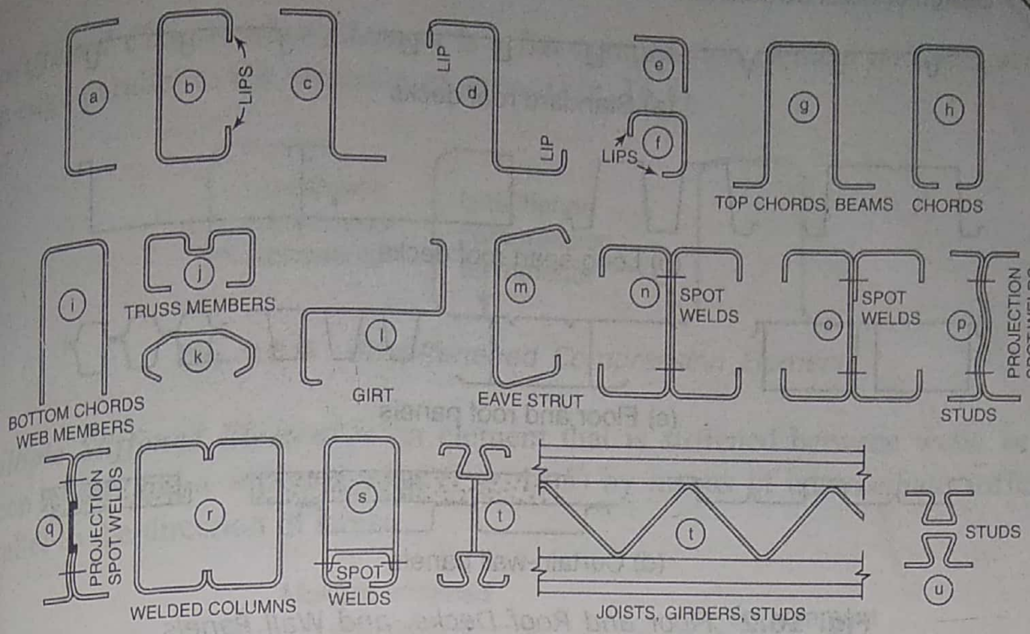



**Fig. 16.1** *Common Shapes of Cold-Formed Light-Gauge Sections*

Shapes *a* to *u* of Fig. 16.1 are similar to hot-rolled shapes, except that in shapes *b*, *d* and *f*, lips are used to stiffen the thin flanges. These shapes are easily produced but have the disadvantage of being unsymmetrical. Shapes *g* to *k* of Fig. 16.1 are to be found only in cold-formed construction; they have the advantage of being symmetrical. Shapes *g*, *h*, *j* and *k* are adapted for use in trusses and latticed girders; these sections are compact, well stiffened and have large radii of gyration in both principal directions. Shape *i* of Fig. 16.1, lacking edge stiffeners on the vertical sides, is better adapted for use as a tension member. Shapes *l* and *m* of Fig. 16.1 are used specifically as girts and eave struts, respectively. In all-metal buildings, shape *l* being the same as shape *d* of Fig. 16.1 which is also, used for purlins. The above members are all one-piece shapes produced merely by cold-forming.

When automatic welding is combined with cold rolling, it is possible to obtain additional shapes. Shapes *n* and *o* of Fig. 16.1 are two varieties of I shapes, the former better adapted for use as studs or columns, the latter for joists or beams. Two of the most successful shapes, namely, shapes *p* and *q* of Fig. 16.1 are further adaptations of shapes *n* and *o*. By deforming the webs and by using projection spot welding, curved slots are formed which provide nailing grooves for connecting collateral material, such as wall boards and wood floors. Shapes *r* and *s* of Fig. 16.1 represent closed sections particularly favourable in compression the former primarily for columns, the latter for compression chords of trusses. Shape *t* of Fig. 16.1 shows one of a variety of open web joists, with chords shaped for nailing, and shape *u* shows a section similar to the chords of shape *t* connected directly to form a nailable stud.

Figure 16.2 shows another category of cold-formed sections, manufactured in panel sizes, used for roof and floor decks, sidings, and walls. Standard roof decks are usually 58 mm deep, with a rib spacing of 130 mm and are used on spans between purlins up to 5 m. Floor and roof panels are made to cover spans from 3 to 10 m. They are usually cellular in shape and permit a wide variety of ancillary uses. The thickness used generally range from 1.2 to 2.5 mm and for standard roof deck and wall cladding from 0.8 to 1.2 mm. The corrugated sheets are the most common use of this category and are in use for decades.

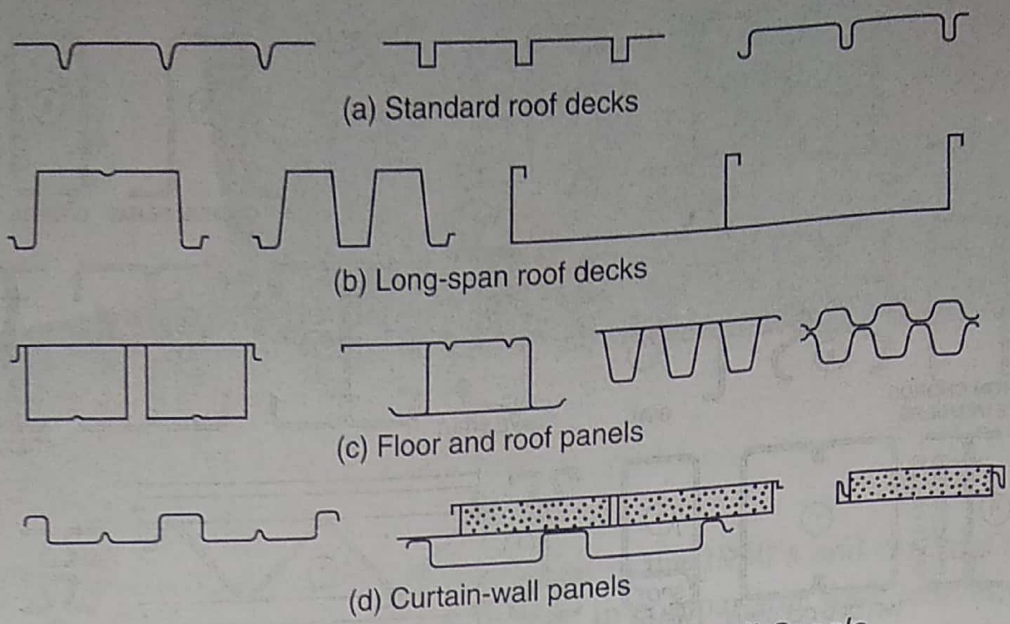



**Fig. 16.2** *Floor and Roof Decks, and Wall Panels*

In the design of decks and panels, structural efficiency is only one of the many criteria since the shape should also be selected to minimize deflections, provide maximum coverage, permit adequate insulation, and accessibility of cells for housing conduits, etc. Optimum strength, that is, optimum strength–weight ratio, therefore, is desired only conditionally, that is, in so far as it is compatible with the other enumerated features. It is evident that the shapes used in light-gauge construction are quite different from, and considerably more varied than, those employed in hot-rolled framing.

Some of the structural sections used for light-gauge construction shown in Figs 16.1 and 16.2 are but a few examples and many more shapes and configurations are possible.

## 16.3 DEFINITIONS

***Stiffened Compression Element*** is a flat compression element of which both edges parallel to the direction of stress are stiffened by another element (Fig. 16.3).

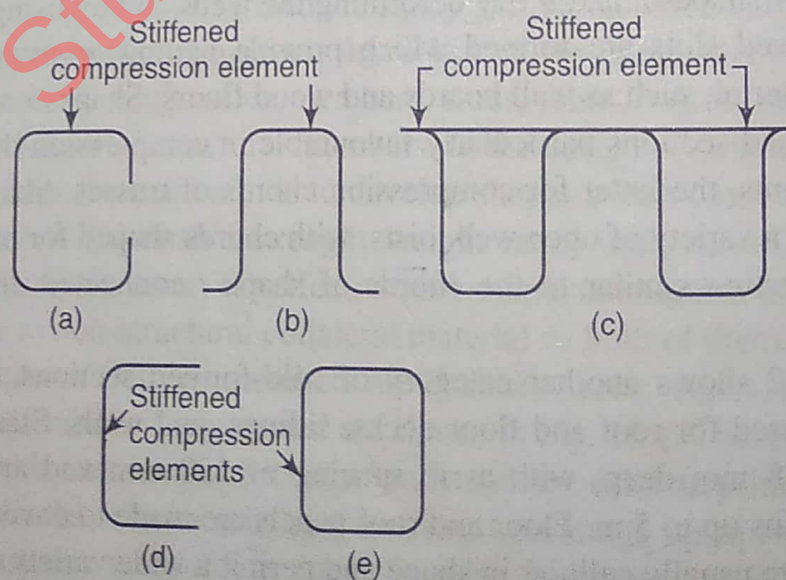



**Fig. 16.3** *Stiffened Compression Elements*

*Unstiffened Compression Element* is a flat compression element stiffened at only one edge parallel to the direction of stress (Fig. 16.4).

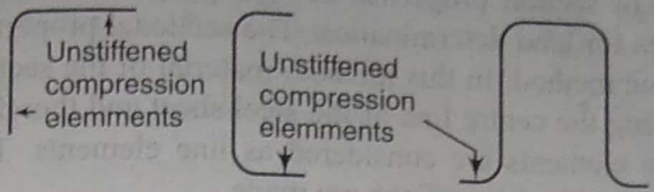



**Fig. 16.4** *Unstiffenened Compression Elements*

*Multiple Stiffened Element* is an element that is stiffened between webs, or between a web and a stiffened edge (Fig. 16.5) by means of intermediate stiffeners parallel to the direction of stress.

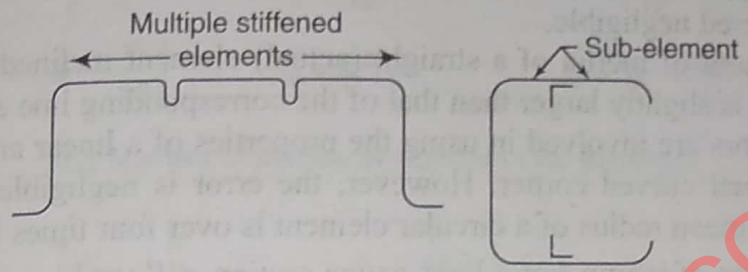



**Fig. 16.5** *Multiple Stiffened Element and Sub-Element*

*Flat Width Ratio* of a single flat element is the ratio of the flat width *w*, exclusive of edge fillets, to the thickness *t* (Fig. 16.6).

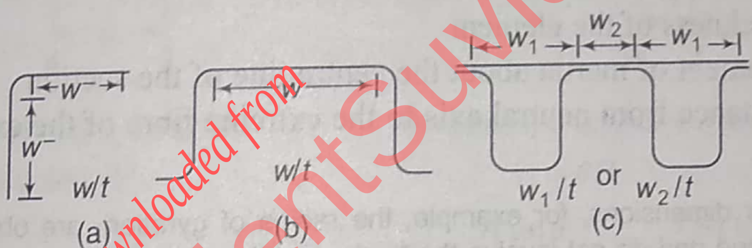



**Fig. 16.6** *Flat-Width Ratio*

*Effective Design Width* of a flat element is its reduced width from design consideration (Fig. 16.7). The portion of the total width which is considered removed to arrive at the effective design width is located symmetrically about the centre line of the element.

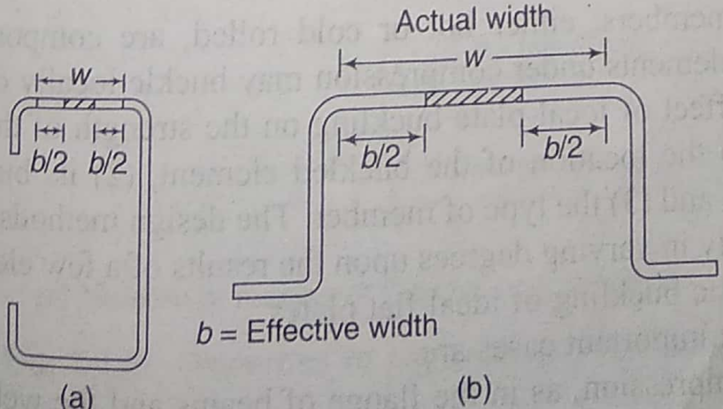



**Fig. 16.7** *Effective Design Width*

## 16.4 PROPERTIES OF SECTIONS

The determination of section properties of light-gauge sections are based on reduced effective area for load determination. The sectional properties may be based on linear or mid-line method. In this method, material of the section is assumed to be concentrated along the centre line of the steel sheet and therefore considered to be a line, and area elements are considered as line elements. The thickness $t$ is introduced after all linear computations are made.

The formulae for line elements are exact, since the line as such has no thickness dimension. However, in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons.

1. The moment of inertia of a straight (actual) element about its longitudinal axis is considered negligible.
2. The moment of inertia of a straight (actual) element inclined to the axis of reference is slightly larger than that of the corresponding line element.
3. Small errors are involved in using the properties of a linear arc to find those of an actual curved corner. However, the error is negligible or disappears when the mean radius of a circular element is over four times its thickness.

$$\text{Moment of inertia } I \text{ of a light-gauge section} = I' \times t$$

$$\text{Total area of a light-gauge section} = L_t \times t$$

$$\text{Section modulus } Z \text{ of the section} = \frac{I}{y}$$

where $L_t$ = total length of the elements

$t$ = thickness of the element

$I'$ = moment of inertia about the centre line of the section

$y$ = distance from neutral axis to the extreme fibre of the extreme element

**Notes**

1. First power dimensions, for example, the radius of gyration, are obtained directly by linear method and do not involve the thickness dimension.
2. When the flat width $w$ of a stiffened compression element is reduced for design purposes, the effective design width is used directly to compute the total effective length $L_e$ of the line elements.

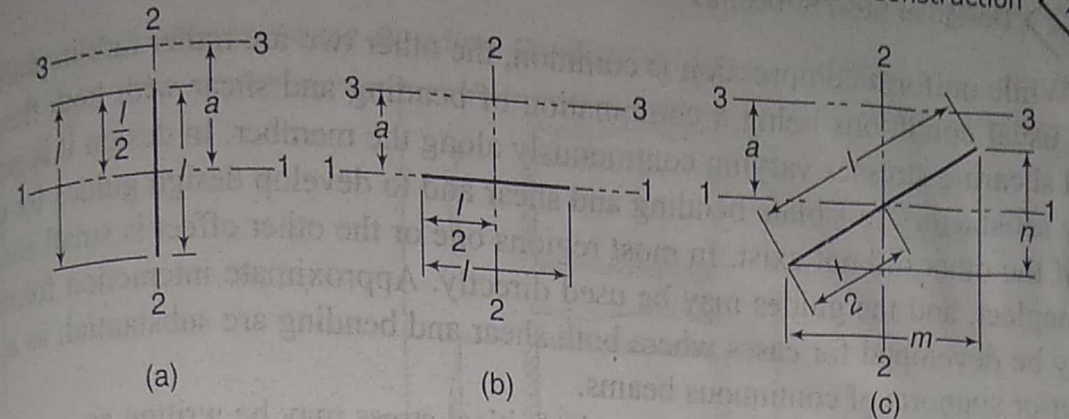
Figure 16.8 shows properties of some of the line elements.

## 16.5 LOCAL BUCKLING OF PLATE ELEMENTS

Most structural members, either hot or cold rolled, are composed of flat plate elements. These elements under compression may buckle locally out of their original planes. The effect of local-plate buckling on the strength of the entire member depends upon (1) the location of the buckled element, (2) its buckling and post-buckling strength, and (3) the type of member. The design methods used in handling these problems rely in varying degrees upon the results of a few elementary cases in the theory of elastic buckling of ideal flat plates.

The three most important cases are

- uniform compression, as in the flange of beams and the webs and flanges of columns,
- pure bending, as in the web of a beam in a region of zero shear, and
- pure shear, as in the web of a beam in a region of zero bending.

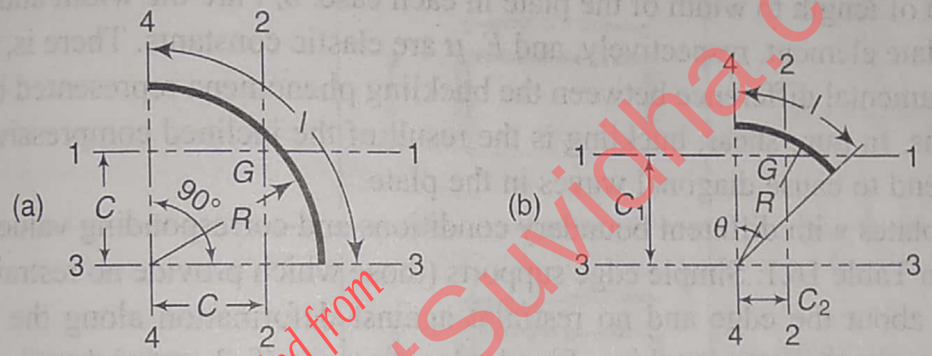



$$I_1 = \frac{l^3}{12} \qquad I_1 = 0 \qquad I_1 = \frac{ln^2}{12}$$

$$I_2 = 0 \qquad I_2 = \frac{l^3}{12} \qquad I_2 = \frac{lm^2}{12}$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right) \qquad I_3 = la^2 \qquad I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$

(a) Moment of Inertias of Common Line Elements




$l = 1.57\ R$; $C = 0.637\ R$

$I_1 = I_2 = 0.149\ R^3$

$I_3 = I_4 = 0.785\ R^3$

G = centre of gravity

θ (radians) = 0.1745 × θ (degrees)

$I_1 = \theta R$

$$C_1 = \frac{R \sin\theta}{\theta}; \quad C_2 = \frac{R(1-\cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2} - \frac{(\sin\theta)^2}{2}\right]R^3$$

$$I_2 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2} - \frac{(1-\cos\theta)^2}{2}\right]R^3$$

$$I_3 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2}\right]R^3$$

$$I_4 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2}\right]R^3$$

(b) Moment of Inertias of Circular Line Elements

**Fig. 16.8** *Properties of Light-Gauge Elements*

While uniform compression is common, the other two are rather rare in practice, the usual conditions being a combination of bending and shear with both flexural and shearing stresses varying continuously along the member. In design it is generally satisfactory to isolate bending and shear and to develop design guides for each as if the other did not exist. In most regions one or the other effect is small enough to neglect, and the guides may be used directly. Approximate interaction formulae may be developed for cases where both shear and bending are substantial, as at the interior supports of continuous beams.

In the first two elementary cases, the critical stress may be written as

$$f_{cr} = \frac{K\pi^2 E}{12(1-\mu^2)(b/t)^2} \tag{16.1}$$

and, in the third,

$$\tau_{cr} = \frac{K\pi^2 E}{12(1-\mu^2)(b/t)^2} \tag{16.2}$$

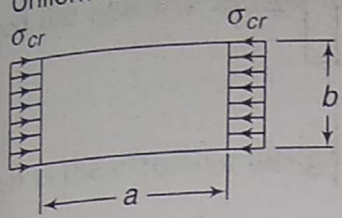
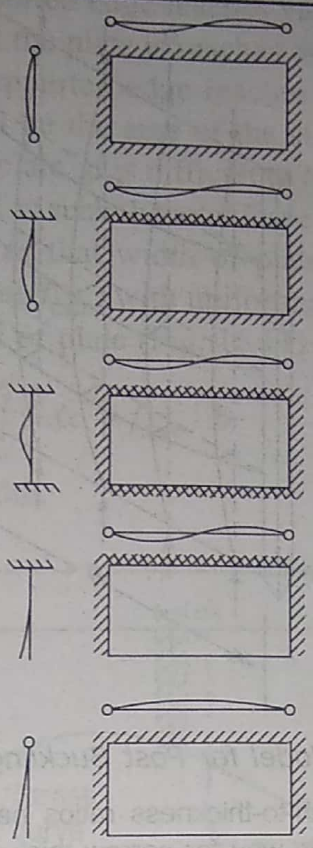
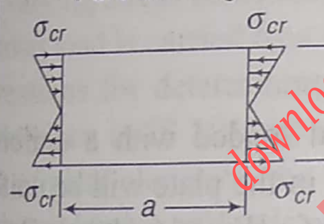
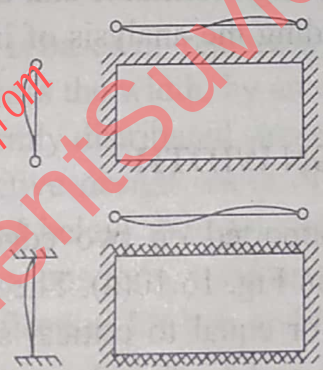
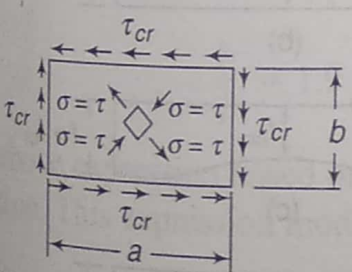
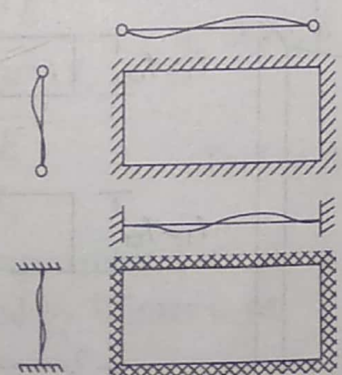
where $K$ is a buckling coefficient which depends upon the support conditions and the ratio of length to width of the plate in each case. $b$, $t$ are the width and thickness of the plate element, respectively, and $E$, $\mu$ are elastic constants. There is, of course, no fundamental difference between the buckling phenomena represented by the two equations. In pure shear, buckling is the result of the inclined compressive stresses which tend to cause diagonal waves in the plate.

Flat plates with different boundary conditions and corresponding values of $K$ are shown in Table 16.1. Simple edge supports (those which provide no restraint against rotation about the edge and no restraint against deformation along the edge) are shown by simple cross-hatching. Fixed-edge supports (full restraint against rotation about the edge but no restraint against deformation along the edge) are indicated by double cross-hatching. Completely free supports are unmarked. The ratio of length to width, $a/b$, is designated $\alpha$, which is called the *aspect ratio*. Deflections of typical sections are also indicated.

For light-gauge steel members the ratio of width to thickness of the plate element is quite large causing the plate element to fail invariably in buckling rather than yielding as is usually for hot-rolled sections; Limits are placed on width to thickness ratio to check that the plate elements do not fail in buckling.

For light-gauge members the critical buckling is generally of a local nature and precedes general buckling of the member. Light-gauge design criteria are based on post-buckling strength of the member after local buckling has occurred. Most design formulae for local-plate buckling boil down to some function of $b/t$ with, where necessary, recognition of the effect of the aspect ratio, the conditions of edge restraint, and the post buckling strength. Due to the post-buckling strength a stiffened light-gauge wide plate element has ultimate strength many times more than the critical stresses. A wide light-gauge plate may be considered to be a grid of longitudinal and transverse bars as shown in Fig. 16.9. Under the loading shown in the figure, longitudinal bars are loaded in compression and hence buckle while the transverse bars stretch like ties. This stretching action of transverse bars restrains the lateral deflection of longitudinal bars and is responsible for post-buckling strength of plates.

**Table 16.1** *Approximate Buckling Coefficients for Flat Plates*

| | | |
|---|---|---|
| Uniform compression  |  | $K = 4.00$ |
| | | $K = 5.42$ |
| | | $K = 6.97$ |
| | | $K = 1.277$ |
| | | $K = 0.425$ |
| Pure bending  |  | $\alpha \geq 0.67$: $K = 24$ |
| | | $\alpha \geq 0.40$: $K = 40$ |
| Pure shear  |  | $\alpha \geq 1.0$: $K = 5.34 + \dfrac{4.0}{\alpha^2}$ |
| | | $\alpha \geq 1.0$: $K = 8.98 + \dfrac{5.6}{\alpha^2}$ |

Note: All k values for uniform compression are for long plates only.

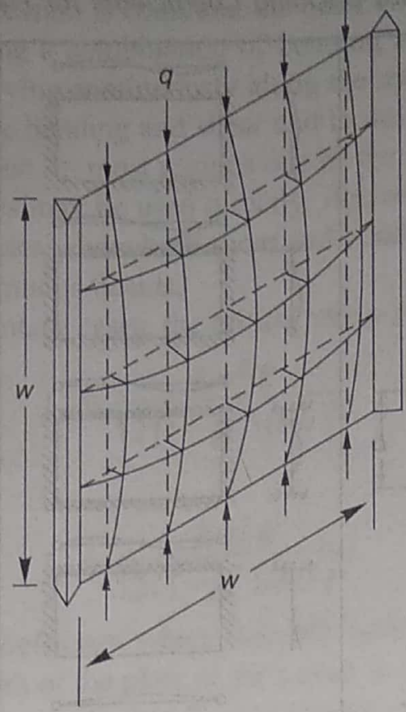



**Fig. 16.9** *Model for Post Buckling Strength of Plate*

*Note* Plates with large width-to-thickness ratios have high post-buckling strengths than those with low *w/t* ratios. That is why for narrow thick plates, failure load never exceeds the critical load.

The development of the design formulae and the necessary definitions and concepts required for understanding the analysis of light-gauge sections are presented in the sections to follow.

## 16.6 EFFECTIVE DESIGN WIDTH

Consider a plate simply supported on two edges and loaded with a uniformly distributed load *p* as shown in Fig. 16.10(a). The stress in the plate will be uniform ($f_1$) for its values less than or equal to critical stress, $f_{cr}$ [Fig. 16.10(b)]. On any further increase of stress beyond $f_{cr}$, the plate at the centre buckles and the stress distribution will be as shown in Fig. 16.10(c). This reveals that the unbuckled

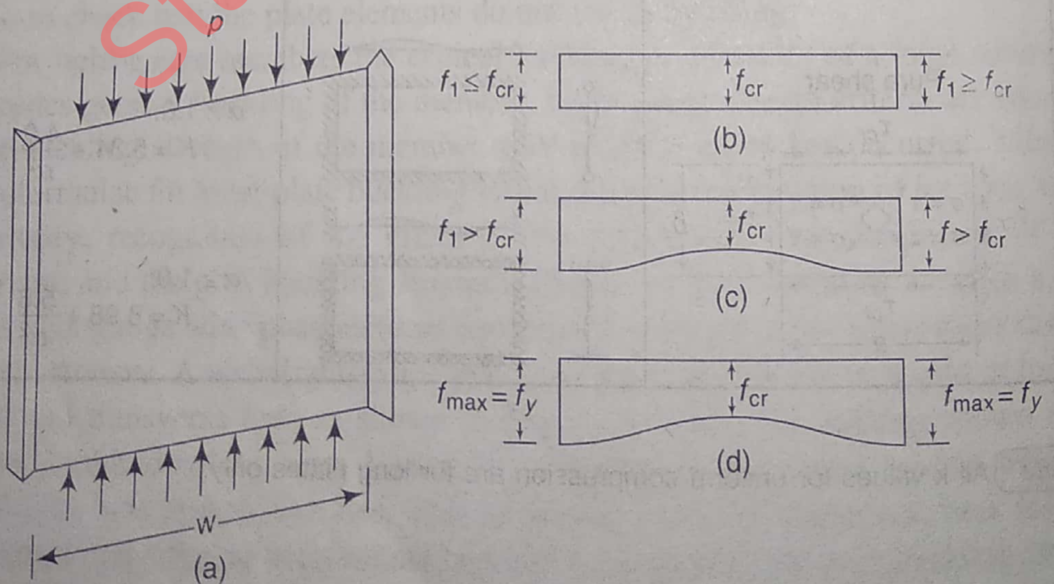



**Fig. 16.10** *Compressive Stress Distributionof Thin Plate Elements*

portion of plate now resists the increased load. On further increase of load, failure occurs when the stress at the supported edge reaches yield stress $f_y$ [Fig. 16.10(d)]. Therefore, the ultimate strength of the plate is reached when the maximum stress in the fibres of the plate near the supported edge reaches the yield stress. The load-carrying capacity of the plate will be the area of the stress curve of Fig. 16.10(d) multiplied by the thickness of the plate. It is difficult to account for the non-uniform stress distribution in the design and as such the total force is assumed to be distributed over lesser width (approximated as that width of plate which just buckles when compressive stress reaches the value $f_{max}$) with uniform stress. This reduced width $b$ is called the *effective design width* of plate (Fig. 16.11) and is given by,

$$\int_0^w f\,dx = f_{max} \cdot b \tag{16.3}$$

where $b$ is the effective design width.

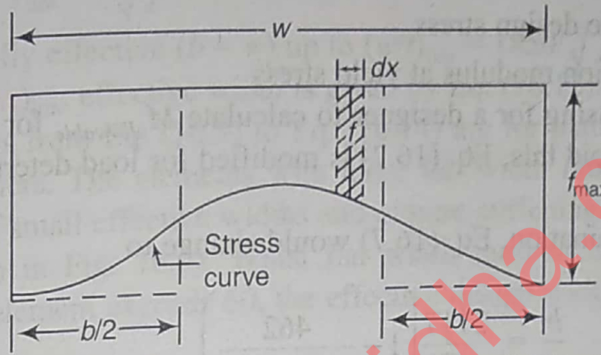



**Fig. 16.11** *Compressive Stress Distribution and Effective Width*

The concept of effective width is based on replacing the plate width $w$, subjected to a varying stress distribution across the width, by an equivalent width $b$, in which the total load is carried by a uniformly distributed stress equal to the edge stress. The expressions for determining effective design width of light-gauge steel plate elements given in IS 801 are detailed below. For determination of loads and deflections of plate elements, the expressions to be used are different. Also, expressions for determination of loads and deflection for box and tubular sections are different than those for plates.

Eq. (16.1) for simply supported plate may be rewritten as

$$\left(\frac{b}{t}\right)^2 = \frac{\pi^2 E}{3(1-\mu^2) f_y} \qquad \text{(Taking } k = 4\text{, Table 16.1)}$$

or

$$\frac{b}{t} = 1.9\sqrt{\frac{E}{f_y}} \qquad \text{(Taking } \mu = 0.3\text{)} \tag{16.4}$$

The above expression based on experimental results is known as the Von–Karman equation. This expression modified by Winter is as

$$\frac{b}{t} = 1.9\sqrt{\frac{E}{f_{max}}}\left[1 - 0.475\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \tag{16.5}$$

IS 801 uses still a modified expression as

$$\frac{b}{t} = 1.9\sqrt{\frac{E}{f_{max}}}\left[1 - 0.415\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \tag{16.6}$$

Taking $E = 2074000$ kg/cm$^2$ (IS 800: 1975), the Eq. (16.6) reduces to

$$\frac{b}{t} = \frac{2736}{\sqrt{f}}\left(1 - \frac{598}{(w/t)\sqrt{f}}\right) \quad (16.7)$$

Equation (16.7) is modified for determination of load-carrying capacity and deflection of the light-gauge sections. The reason for this is as follows.

The ultimate moment of the section,

or $\quad M_{ult} = Z_1 \times 1.67 \times f_b \quad$ (factor of safety = 1.67)

or, $\quad \dfrac{M_{ult}}{1.67} = Z_1 \times f_b$

or, $\quad M_{allowable} = Z_1 \times f_b \quad (16.8)$

where $\quad f_y$ = yield stress

$f_b$ = basic design stress

$Z_1$ = section modulus at yield stress

It may be confusing for a designer to calculate $M_{allowable}$ for section modulus at yield stress. To avoid this, Eq. (16.7) is modified for load determination by replacing $f$ by $1.67\,f$.

For load determination, Eq. (16.7) would change to

$$\frac{b}{t} = \frac{2117}{\sqrt{f}}\left(1 - \frac{462}{(w/t)\sqrt{f}}\right)$$

The above expression after rounding off (in IS 801: 1975) is as follows:

$$\frac{b}{t} = \frac{2120}{\sqrt{f}}\left(1 - \frac{465}{(w/t)\sqrt{f}}\right) \quad (16.9)$$

*In case of small moments ($M < M_{allowable}$)*

If effective width ($b$) = full width ($w$), then Eq. (16.9) can be written as

$$\left(\frac{w}{t}\right)^2 - \frac{2120}{\sqrt{f}}\left(\frac{w}{t}\right) + \frac{95800}{f} = 0$$

$$\frac{w}{t} = \frac{1431}{\sqrt{f}} \quad (16.10)$$

IS 801: 1975 rounds off Eq. (16.10) as under.

$$\left(\frac{w}{t}\right)_{\lim} = \frac{1435}{\sqrt{f}} \quad (16.11)$$

Thus flanges are fully effective ($b = w$) up to $(w/t)_{\lim} = 1435/\sqrt{f}$. For flanges with $(w/t)$ larger than $(w/t)_{\lim}$, effective width is given by Eq. (16.9).

*For deflection determination*

Equation (16.7) has been rounded off (in IS 801: 1975) as

$$\frac{b}{t} = \frac{2710}{\sqrt{f}}\left(1 - \frac{600}{(w/t)\sqrt{f}}\right) \quad (16.12)$$

The value of $(w/t)_{\lim}$ for deflection determination can be obtained as has been done in the case for load determination, and is

$$\frac{w}{t} = \frac{1813}{\sqrt{f}} \quad (16.13)$$

IS 801: 1975 rounds off Eq. (16.13) as under.

$$\left(\frac{w}{t}\right)_{\lim} = \frac{1850}{\sqrt{f}} \quad (16.14)$$

Thus flanges are fully effective ($b = w$) up to $(w/t)_{\lim} = 1850/\sqrt{f}$. For flanges with $(w/t)$ larger than $(w/t)_{\lim}$, effective width is given by Eq. (16.12).

All the equations from Eq. (16.5) to Eq. (16.14) are for plate elements without intermediate stiffeners. The elements with large flat width ratios become uneconomical because of small effective widths and require stiffening (multiple stiffened elements as shown in Fig. 16.5). When flat width ratio of a sub-element of a multiple stiffened element exceeds 60, the effective design width is reduced and is given by

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left(\frac{w}{t} - 60\right) \quad (16.15)$$

The area of stiffeners is reduced to an effective area given by

$$A_{ef} = \alpha A_{st} \quad (16.16)$$

where $\alpha = \left(3 - 2 \times \frac{b_e}{w}\right) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\frac{w}{t} \qquad \left(\frac{w}{t} = 60 \text{ to } 90\right)$

where $A_{st}$ = area of stiffener

and $A_{ef} = \left(\frac{b}{w}\right) A_{st}$ (16.17)

The effective width expressions arrived at as above are not applicable for square and tubular sections. Since square and tubular sections are rolled under strict quality control, a higher value of effective widths are permitted by IS 801 as follows.

For load determination $\frac{b}{t} = \frac{2120}{\sqrt{f}}\left[1 - \frac{420}{(w/t)\sqrt{f}}\right]$ (16.18)

For deflection determination $\frac{b}{t} = \frac{2710}{\sqrt{f}}\left[1 - \frac{545}{(w/t)\sqrt{f}}\right]$ (16.19)

IS 801: 1975 is still in MKS units and is not revised. To maintain SI units throughout the book the formulae given in the code are converted to SI units and are presented in Table 16.2.

**Table 16.2** *Formulae for Effective Width of Stiffened Compression Members Without Intermediate Stiffener*

| *MKS Units* | *SI Units* |
|---|---|
| 1. For compression elements other than tubes | 1. For compression elements other than tubes |
| (a) *For load determination* | (a) *For load determination* |
| For $(w/t)$ up to $(w/t)_{lim} = 1435/\sqrt{f}$, $b = w$ | For $(w/t)$ up to $(w/t)_{lim} = 446/\sqrt{f}$, $b = w$ |
| For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2120}{\sqrt{f}}\left(1 - \frac{465}{(w/t)\sqrt{f}}\right)$ | For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{658}{\sqrt{f}}\left(1 - \frac{145}{(w/t)\sqrt{f}}\right)$ |
| (b) *For deflection determination* | (b) *For deflection determination* |
| For $(w/t)$ up to $(w/t)_{lim} = 1850/\sqrt{f}$, $b = w$ | For $(w/t)$ up to $(w/t)_{lim} = 574/\sqrt{f}$, $b = w$ |
| For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2710}{\sqrt{f}}\left(1 - \frac{600}{(w/t)\sqrt{f}}\right)$ | For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{842}{\sqrt{f}}\left(1 - \frac{186}{(w/t)\sqrt{f}}\right)$ |
| 2. For closed square and rectangular tubes | 2. For closed square and rectangular tubes |
| (a) *For load determination* | (a) *For load determination* |
| For $(w/t)$ up to $(w/t)_{lim} = 1540/\sqrt{f}$, $b = w$ | For $(w/t)$ up to $(w/t)_{lim} = 478/\sqrt{f}$, $b = w$ |
| For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2120}{\sqrt{f}}\left(1 - \frac{420}{(w/t)\sqrt{f}}\right)$ | For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{658}{\sqrt{f}}\left(1 - \frac{130}{(w/t)\sqrt{f}}\right)$ |
| (b) *For deflection determination* | (b) *For deflection determination* |
| For $(w/t)$ up to $(w/t)_{lim} = 1990/\sqrt{f}$, $b = w$ | For $(w/t)$ up to $(w/t)_{lim} = 618/\sqrt{f}$, $b = w$ |
| For $(w/t)$ up to $(w/t)_{lim}$, $\frac{b}{t} = \frac{2710}{\sqrt{f}}\left(1 - \frac{545}{(w/t)\sqrt{f}}\right)$ | For $(w/t)$ up to $(w/t)_{lim}$, $\frac{b}{t} = \frac{842}{\sqrt{f}}\left(1 - \frac{169}{(w/t)\sqrt{f}}\right)$ |

The formulae in SI units can be obtained by dividing the constants of equation, given in IS 801: 1975 in MKS units, by a factor $\sqrt{2074000/200000} = 3.22$, where 2074000 and $2 \times 10^5$ are values of modulus of elasticity in MKS and SI units respectively. Also, the multiplying factor for stress conversion from MKS to SI is taken as $9.81/100 = 0.0981$.

For example, Eq. (16.7) in SI units may be rewritten as

$$\frac{b}{t} = \frac{850}{\sqrt{f}}\left(1 - \frac{185.6}{(w/t)\sqrt{f}}\right) \tag{16.20}$$

where

$\mu$ = Poisson's ratio of steel (0.3)

$f_{max}$ = actual maximum stress in compression element computed on the basis of effective design width

$f$ = actual unit stress in the compression element computed on the basis of effective design width

## 16.7 SPECIFICATIONS

### Stiffeners

Edge stiffeners in the form of web or lip and intermediate stiffeners between webs, or between web and a stiffened edge, are used to stiffen the compression elements.

The minimum moment of inertia of an edge stiffener is given by

$$I_{min} = 1.83t^4\sqrt{\left(\frac{w}{t}\right)^2 - \frac{281200}{f_y}}, \quad \text{but not less than } 9.2t^4 \tag{16.21}$$

where the stiffener consists of a simple lip, the required overall depth of the lip

$$d_{min} = 2.8t\sqrt[6]{\left(\frac{w}{t}\right)^2 - \frac{281200}{f_y}}, \quad \text{but not less than } 4.8t \tag{16.22}$$

*Notes*

1. Equations (16.21) and (16.22) are in MKS units. To change them to SI units the coefficient 281200 may be replaced by 27590.
2. The minimum moment of inertia required by an intermediate stiffener is double of that of the edge stiffener, since it has to stiffen the two compression elements on either side of the intermediate stiffener.
3. For the conditions to be satisfied by an intermediate stiffener, IS 801 may be referred.

### Maximum Allowable Overall Flat-Width Ratios

The maximum allowable overall flat-width ratios $w/t$, disregarding intermediate stiffeners and taking $t$ as the actual thickness of the element are as given in Table 16.3.

**Table 16.3** *Maximum Overall Flat-Width Ratio*

| | |
|---|---|
| (a) Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by: | |
| (i) *Simple lip* | 60 |
| (ii) *Any other kind of stiffener* | 90 |
| (b) Stiffened compression element with both longitudinal edges connected to other stiffened elements | 500 |
| (c) Unstiffened compression element | 60 |

*Notes*

1. Unstiffened compression elements that have *w/t* exceeding approximately 30 and stiffened compression elements that have *w/t* ratios exceeding approximately 250 are likely to develop noticeable deformation at the full allowable working stresses without affecting the ability of the member to carry design loads.
2. Stiffened elements having *w/t* ratios larger than 500 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render the design formulae given in the code inapplicable.

For unusually wide flanges the width is limited by

$$w_f = \sqrt{\frac{126500\,td}{f_{av}}} \times \sqrt[4]{\frac{100\,c_f}{d}} \qquad (16.23)$$

where $w_f$ = the width of flange projection beyond the web for I-beams or similar sections, or half of the distance between webs for box- or U-type beams (cm)

$t$ = flange thickness (cm)

$d$ = depth of beam (cm)

$c_f$ = amount of curling (< 5% not important)

$f_{av}$ = the average stress in the full, unreduced flange width in kg/cm$^2$

### Effective Design Width of Short Span Beams

Where the span of the beam is less than $30w_f$ and it carries one concentrated load or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, should be limited to as given in Table 16.4.

**Table 16.4** *Maximum Allowable Ratio of Effective Design Width to Actual Width*

| $L/w_t$ (1) | *Ratio* (2) | $L/w_t$ (1) | *Ratio* (2) |
|---|---|---|---|
| 30 | 1.00 | 14 | 0.82 |
| 25 | 0.96 | 12 | 0.78 |
| 20 | 0.91 | 10 | 0.73 |
| 18 | 0.89 | 8 | 0.67 |
| 16 | 0.86 | 6 | 0.55 |

where $L$ = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams in cm.

*Note* For flanges of I-beams and similar sections stiffened by lips at the outer edges, $w_f$ should be taken as the sum of the flange projections beyond the web plus the depth of the lip.

### Maximum Allowable Web Depth

The ratio $h/t$ of the webs of flexural members should not exceed the following limitations:

(a) *For members with unstiffened webs:*

$$(h/t)_{max} = 150$$

(b) *For members which are provided with adequate means of transmitting concentrated loads or reactions or both into the web.*

$$(h/t)_{max} = 200$$

where $h$ = clear distance between flanges measured along the plane of web
$t$ = web thickness

*Note* Where the web consists of two or more sheets, the h/t ratio should be computed for individual sheet.

### Deflection of Beams

The maximum deflection of the beam is limited to span/325.

*Notes*

1. Since the effective width of stiffened compression element changes with the stress, the moment of inertia I also changes with the B.M. at that section. Thus, in computing the deflection, a variable moment of inertia must be considered. However, such accuracy is generally not required and one may use the effective section at maximum bending moment for computing the deflection.
2. For beams with unstiffened flanges only gross moment of inertia is used for computing deflection.

## 16.8 BASIC ALLOWABLE DESIGN STRESSES

Stresses in tension, compression and flexural members should not exceed $0.6f_y$, where $f_y$ is the yield stress. Some of the grades of steel with their yield stress and basic allowable design stress as specified in IS: 801: 1975 are given in Table 16.5.

**Table 16.5** *Basic Allowable Design Stress*

| *Minimum Yield Strength of Steel (Kg/cm²)* | *Basic Allowable Design Stress (f) (Kg/cm²)* |
|---|---|
| 2100 | 1250 |
| 2400 | 1450 |
| 3000 | 1800 |
| 3600 | 2160 |

*Note* The allowable stresses may be increased by $33\frac{1}{3}$ % for members and assemblies subjected to wind or earthquake forces.

## 16.9 ALLOWABLE COMPRESSIVE STRESSES IN UNSTIFFENED ELEMENTS

An unstiffened compression element may fail in yielding if it is short and its $w/t$ ratio is less than a certain value. If, however, the $w/t$ ratio is large, it will fail by buckling. For intermediate values of $w/t$ ratios, the failure will be by inelastic buckling. Figure 16.13 shows the failure stresses and allowable stresses for limits 0 < w/t < 60. Compression $f_c$ (in kg/cm$^2$) on flat unstiffened element is given by:

(a) For w/t ratio $\not> 530/\sqrt{f_y}$ :

$$f_c = 0.60 f_y$$

(b) For w/t ratio $> 530/\sqrt{f_y}$ :

but $\not> 1210/\sqrt{f_y^*}$ :

$$f_c = f_y[0.767\ (3.15/10^4)\ (w/t)\sqrt{f_y}\ ]$$

(c) For w/t ratio $> 1210/\sqrt{f_y}$

but $\not> 25$*:

$$f_c = 562\ 000/(w/t)^2 \qquad (16.24)$$

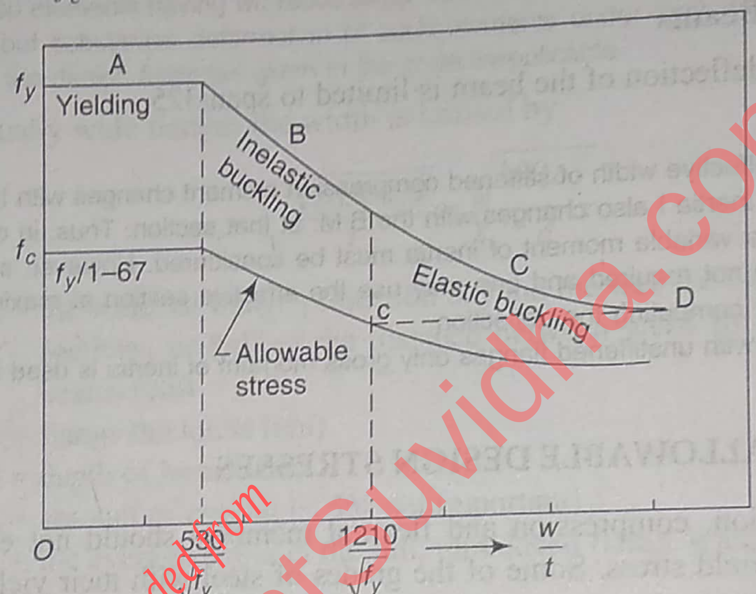



**Fig. 16.12** *Unstiffened Element Failure Stresses and Allowable Stresses for $0 < (w/t) < 60$*

(d) For w/t ratio from 25 to 60†

*For angle struts*: $f_c = 562\ 000/\ (w/t)^2$ (16.26)

*For all other sections*: $f_c = 1390 - 20\ (w/t)$ (16.27)

* When the yield point of steel is less than 2320 kg/cm$^2$ then for w/t ratios between $530/\sqrt{f_y}$ and 25:

$$f_c = 0.6f_y - \frac{\left[\frac{w}{t} - \frac{530}{\sqrt{f_y}}\right](0.6\,f_y - 900)}{25\left[1 - 21.2/\sqrt{f_y}\right]} \qquad (16.25)$$

† Unstiffened compression elements having ratios of w/t exceeding approximately 30 may show noticeable distortion of the free edges under allowable compressive stress without detriment to the ability of the member to support load. For ratios of w/t exceeding approximately 60 distortion of the flanges is likely to be so pronounced as to render the section structurally undesirable unless load and stress are limited to such a degree as to render such use uneconomical.

*Notes*

1. Unstiffened compression elements having $(w/t) > 60$ should not be used.
2. The expressions for compression $f_c$ (in N/mm²) on flat unstiffened elements are given in Table 16.6.

**Table 16.6** *Allowable Stresses in Unstiffened Compression Elements (IS: 801–1975)*

| *MKS Units* | *SI Units* |
|---|---|
| For $w/t \le 530/\sqrt{f_y}$<br>$f_c = 0.60\sqrt{f_y}$ | For $w/t \le 165/\sqrt{f_y}$<br>$f_c = 0.60\sqrt{f_y}$ |
| For $1210/\sqrt{f_y} > w/t > 530/\sqrt{f_y}$<br>$f_c = f_y[0.767 - (3.15/10^4)(w/t)\sqrt{f_y}\,]$ | For $375/\sqrt{f_y} > w/t > 165/\sqrt{f_y}$<br>$f_c = f_y[0.767 - 10^{-3}\,(w/t)\sqrt{f_y}\,]$ |
| $25 > w/t > 1210/\sqrt{f_y}$<br>$f_c = 562\,000/(w/t)^2$ | $25 > w/t > 375/\sqrt{f_y}$<br>$f_c = 54200/(w/t)^2$ |
| For $60 > w/t > 25$<br>(a) *Angle struts:* $f_c = 562\,000/(w/t)^2$<br>(b) *Other sections:* $f_c = 1390 - 20\,(w/t)$ | For $60 > w/t > 25$<br>(a) *Angle struts:* $f_c = 542\,000/(w/t)^2$<br>(b) *Other sections:* $f_c = 134 - 1.93\,(w/t)$ |

## 16.10 COMPRESSION MEMBERS

The column strength of thin plates is defined as the load-carrying capacity of the member controlled by one or a combination of the following four types of failure: (a) crushing, (b) local buckling of thin-plate elements of the section, (c) overall or primary column buckling by lateral bending over the unsupported length of the member, and (d) torsional buckling or twisting of the section about a longitudinal axis.

In cold-formed light-gauge sections, the width–thickness ratios of component elements are large enough to cause local buckling. This necessitates consideration of the local buckling effects in the allowable stress computations. This is done by incorporating a factor $Q$, called *form factor*, in the allowable stress expressions. The form factor is less than one and by substituting $Qf_y$ for $f_y$ in the axial stress expressions, the expressions for allowable compressive stress can be arrived at.

### Determination of Form Factor

*For Members Composed Entirely of Stiffened Elements* The cross sections of two typical light-guage steel columns consisting entirely of stiffened elements are shown in Fig. 16.13. It is easy to see that, when axially compressed, each side of the box section will act as a uniformly compressed plate having a buckling coefficient somewhere between 4.00 and 6.97 and probably much closer to the lower value. The same is true for the web of the $C$ section, but there is a little more uncertainty regarding its flanges. If there were no turned-down lips to support the outer edges of the flanges, they would be free and the buckling coefficient would be somewhere between 1.277 and 0.425, probably much closer to the latter. The lips give some support to these edges, and, if they are sufficiently stiff, their resistance is equivalent to that of an idealized simple support.

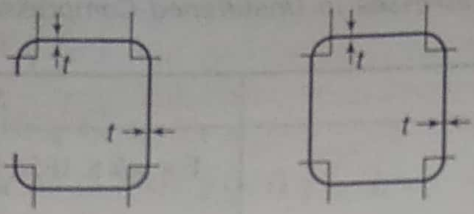


**Fig. 16.13** *Sections with all Stiffened Elements*

We should assume that the stiffening lips on the channel in the figure are adequate and that all major plate elements—the web and the two flanges—may be treated as plates supported along both longitudinal edges. The $Q$ factor for columns of this type is a function of the effective area.

$$P_{ult} = A_{eff} \times f_y$$

where $P_{ult}$ = yield load

$A_{eff}$ = effective area of all stiffened elements computed for basic design stress

$$\text{Yield stress} = \frac{P_{ult}}{A} = \frac{A_{eff}}{A} f_y$$

But, yield stress = $Qf_y$

Hence, $$Q = \frac{A_{eff}}{A} \tag{16.28}$$

That is for a section with entirely stiffened elements, $Q$ is a factor equal to the effective design area of the section divided by the gross area. The ($A_{eff}$) used in the Eq. (16.28) is the effective area at the yield point.

***For Members Composed Entirely of Unstiffened Elements*** Aside from the angle section [Fig. 16.14(a)], it is rather difficult to think of columns made up entirely of unstiffened elements. However, a double channel in which the webs are interconnected or sufficiently thick so that the full area will be effective up to the point of failure is equivalent to one in so far as the computation of $Q$ is concerned [Fig. 16.14(b)]. The $Q$ factor for columns of this type is a function of the ultimate strength of the unstiffened plate.

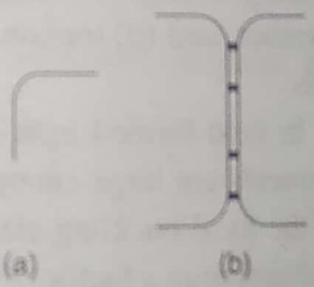


**Fig. 16.14** *Sections with Unstiffened Flanges*

An unstiffened plate having a small width-to-thickness ratio will fail by yielding prior to buckling. The upper limit to this may be generally set at $b/t = 530/\sqrt{f_y}$. Between $b/t = 530/\sqrt{f_y}$ and $b/t = 1210/\sqrt{f_y}$, failure will occur by inelastic buckling with negligible post-buckling reserve.

$$P_{ult} = f_{cr} \times A$$

∴

or
$$\frac{P_{ult}}{A} = \frac{f_{cr}}{f_y} \times f_y = \frac{1.67 f_c}{1.67 f_b} f_y = \frac{f_c}{f_b} f_y$$

where $f_c$ and $f_b$ are the allowable compressive and bending stresses, respectively.

That is for a member composed entirely of unstiffened elements, $Q$ is a factor equal to the allowable compressive stress for the weakest plate of the section, divided by the basic design stress ($0.6f_y$). The weakest plate is, of course, the one with the largest $b/t$ ratio.

But, yield stress = $Qf_y$

Hence,
$$Q = \frac{f_c}{f_b} = \frac{f_c}{f} \qquad (16.29)$$

where $f = f_b$ = basic design stress = $0.6f_y$

The permissible compressive stress $f_c$ is computed by equations given in Table16.6.

*For Members Consisting of both Stiffened and Unstiffened Elements* The cross sections of two light-gauge steel columns consisting of both stiffened and unstiffened elements are shown in Fig. 16.15. The capacity of a member of this type is reached when the weakest unstiffened plate element fails at the stress $\sigma_{cr} = 1.65f_c$, where $f_c$ is computed from the applicable Eq. 16.24. At failure, the effective area of the section is equal to the full area of all unstiffened elements plus the effective area of all stiffened elements at the stress $f_{cr}$ (not $f_y$ as in the case of columns with all stiffened elements). Still calling this total effective area $A_{eff}$, the ultimate capacity of the plate may be written as

$$P_{ult} = A_{eff} f_{cr} \qquad (16.30)$$

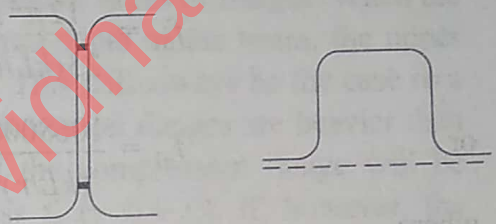


**Fig. 16.15** *Sections with Stiffened and Unstiffened Plates*

The member consisting of both stiffened and unstiffened elements will attain its failure load when the weaker of the unstiffened element buckles at the critical stress ($f_{cr}$). At this stress, $A_{eff}$ will consist of unreduced area of unstiffened elements and effective area of stiffened elements computed for $f_{cr}$ (or value of $f_c$ from Eq. (16.24) is used).

Therefore,
$$P_{ult} = f_{cr} \times A_{eff}$$

This can be arranged as

$$\frac{P_{ult}}{A} = \frac{f_{cr}}{f_y} \times \frac{A_{eff}}{A} f_y = \left(\frac{f_c}{f_b} \times \frac{A_{eff}}{A}\right) f_y$$

But, since yield stress = $Qf_y$

Hence,
$$Q = \frac{f_c}{f_b} \times \frac{A_{eff}}{A} = \frac{A_{eff}}{A} \times \frac{f_c}{f}$$

Thus, $$Q = Q_{\text{unstiffened}} \times Q_{\text{stiffened}} \tag{16.31}$$

## Allowable Stresses for Shapes Not Subjected to Torsional-Flexural Buckling

*For doubly symmetric shapes, closed cross-sectional shapes or cylindrical sections, and any other shapes which can be shown not to be subject to torsional-flexural buckling, and for members braced against twisting. The average axial stress P/A, in compression members should not exceed the following values of* $f_{a1}$.

For $$kL/r < \frac{C_c}{\sqrt{Q}}$$

$$f_{a1} = \frac{12}{23} Q f_y - \frac{3}{23} \times \frac{(Qf_y)^2}{\pi^2 E}\left(\frac{kL}{r}\right)^2 \tag{16.32}$$

or $$f_{a1} = 0.522 Q f_y - \left(\frac{Q f_y\, kL/r}{12500}\right)^2$$

for $$kL/r \geq \frac{C_c}{\sqrt{Q}}$$

$$f_{a1} = \frac{12\pi^2 E}{23(kL/r)^2}$$

or $$f_{a1} = \frac{10680000}{(kL/r)^2} \tag{16.33}$$

where

$C_c = \sqrt{2\pi^2 E/f_y}$

$P$ = total load;

$A$ = full unreduced cross-sectional area of the member

$f_{a1}$ = allowable average compression stress under concentric loading

$E$ = modulus of elasticity = 2074000 kgf/cm²

$k$ = effective length factor*

$L$ = unbraced length of member

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

$f_y$ = yield point of steel

$Q$ = form factor

---

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or by floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses the effective length factor $k$ for the compression members should be taken as unity, unless analysis shows that a smaller value may be used. The effective length $kL$ of compression members in a frame which depends upon its own bending stiffness for lateral stability, should be determined by a rational method and should not be less than the actual unbraced length.

*Note* When the factor Q is equal to unity, the steel is 2.29 mm or more in thickness, and kL/r is less than $C_c$:

$$f_{a_1} = \frac{\left[1 - \frac{(kL/r)^2}{2(C_c)^2}\right] f_y}{\frac{5}{3} + \frac{3}{8}\frac{(kL/r)}{(C_c)} - \frac{(kL/r)^3}{8(C_c)^3}}$$

The factor of safety in the expressions above is 1.92 (i.e. 23/12).

The expressions for allowable stresses for shapes subjected to torsional flexural buckling may be referred from IS 801.

## 16.11 FLEXURAL MEMBERS

The strength of a light-gauge-formed beam is limited by the smallest shear or bending moment that will produce yielding, buckling, or excessive distortion of any of its elements. Where the proper function of the beam depends on its deflection, the useful strength of the beam may be less than the value obtained by shear or moment limitation.

### Flanges of Beams

The maximum bending stress occurs in the most distant fibres of flanges. When the centroid of the effective section is at or below mid-depth of the beam, the upper (compression) fibre will be most highly stressed. This will always be the case in a section symmetrical about the *xx*-axis or in which tension flanges are heavier than the compression flanges; the effective width of the compression flange will be computed on the basis of the basic design stress $f_b$ (= 0.6 $f_y$). If, however, the centroid of the effective section is above mid-depth, as is often the case in the deck type of sections, or for a beam having compression flange heavier than tension flange, the stress at the instance of failure, will be less than $f_y$, the tension flange thus will yield prior to compression flange. Since the actual stress will depend upon the position of neutral axis which will depend on the effective area of compression flange which in turn varies with stress in compression flange, a trial assumption must be made either as to stress or effective width and, possibly, several trial calculations made before convergence is obtained.

*Laterally Unbraced Beams* To prevent lateral buckling, the maximum compressive stress $f_b$ on extreme fibers of laterally unsupported straight flexural members* should not exceed the allowable stress as specified in Section 16.8 or 16.9 nor the following maximum stresses:

(a) When bending is about the centroidal axis perpendicular to the web for either I-shaped sections symmetrical about an axis in the plane of web or symmetrical channel shaped sections:

*These provisions apply to *I*-, *Z*-, or channel-shaped flexural members (not including multiple web deck, *U*- and closed-box type members and curved or arch members). These provisions do not apply to laterally unbraced compression flanges of otherwise laterally stable sections.

For $\dfrac{L^2 S_{xc}}{dI_{yc}}$

$$> \frac{0.36\pi^2 EC_b}{f_y}$$

but $$< \frac{1.8\pi^2 EC_b}{f_y}$$

$$f_b = \frac{2}{3} f_y - \frac{f_y^2}{5.4\pi^2 EC_b}\left(\frac{L^2 S_{xc}}{dI_{yc}}\right) \qquad (1.34)$$

For $\dfrac{L^2 S_{xc}}{dI_{yc}} \geq \dfrac{1.8\pi^2 EC_b}{f_y}$

$$f_b = 0.6\pi^2 EC_b \frac{dI_{yc}}{L^2 S_{xc}} \qquad (1.635)$$

(b) For point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web:

For $\dfrac{L^2 S_{xc}}{dI_{yc}}$

$$> \frac{1.8\pi^2 EC_b}{f_y}$$

but $$< \frac{0.9\pi^2 EC_b}{f_y}$$

$$f_b = \frac{2}{3} f_y - \frac{f_y^2}{2.7\pi^2 EC_b}\left(\frac{L^2 S_{xc}}{dI_{yc}}\right) \qquad (16.36)$$

for $\dfrac{L^2 S_{xc}}{dI_{yc}} \geq \dfrac{0.9\pi^2 EC_b}{f_y}$

$$f_b = 0.3\pi^2 EC_b \frac{dI_{yc}}{L^2 S_{xc}} \qquad (16.37)$$

where

$L$ = the unbraced length of the member

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

$S_{sc}$ = compression section modulus of entire section about major axis $I_x$ divided by distance to extreme compression fibre

$C_b$ = bending coefficient which can conservatively be taken as unity, or calculated from:

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2, \text{ but not more than } 2.3$$

where $M_1$ is the smaller and $M_2$, the larger bending moment at the ends of the unbraced length, taken about the strong axis of the members, and where $M_1/M_2$, the ratio of end moments is positive when $M_1$ and $M_2$ have the same sign ( reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the ratio $M_1/M_2$ should be taken as unity.

*For members subjected to combined axial and bending stress, $C_b$ should be 1.*

$E$ = modulus of elasticity = 2074000 kg/cm²

$d$ = depth of section

### Web of Beams

*Shear Stresses in Webs* The maximum average shear stress $f_{v,cal}$ in kg/cm², on the gross area of a flat web should not exceed:

(a) For $h/t \ngtr 4590/\sqrt{f_y}$ :

$$f_v = \frac{1275\sqrt{f_y}}{h/t} \text{ with a maximum of } 0.40 f_y \tag{16.38}$$

(b) For $h/t > 4590/\sqrt{f_y}$ :

$$f_v = \frac{5850000}{(h/t)^2} \tag{16.39}$$

where

$t$ = web thickness

$h$ = clear distance between flanges measured along the plane of the web

$f_y$ = yield point in kg/cm²

**Note** Where the web consists of two or more sheets, these should be considered as separate members carrying their share of the shear.

*Bending Stresses in Webs* The compressive stress $f_{bw,cal}$ in kg/cm², in the flat web of a beam due to bending in its plane, should not exceed $f_{bw}$ nor should it exceed

$$f_{bw} = \frac{36560000}{(h/t)^2} \text{ kg/cm}^2 \tag{16.40}$$

*Combined Bending and Shear Stresses in Webs* For webs subject to both bending and shear stresses, the member should be so proportioned that such stresses do not exceed the allowable values specified for shear and bending stresses in webs as above and that the quantity $\sqrt{(f_{bw,cal}/f_{bw})^2 + (f_{v,cal}/f_v)^2}$ does not exceed unity.

where

$f_{bw,cal}$ = actual compression stress at junction of flange and web

$$f_{bw} = \frac{36560000}{(h/t)^2} \text{ kg/cm}^2$$

$f_{v,cal}$ = actual average shear stress, that is, shear force per web divided by web area

$f_v$ = allowable shear stress except that the limit of 0.4 $f_y$ should not apply

## Web Crippling of Beams

Web crippling, as described in hot rolled beams (Section 7.9), is also a problem in light-gauge steel beams, and, in these, a new dimension is added. In hot-rolled beams the load is usually applied in the plane of the web, but normally this is not feasible for cold-formed sections because of their make-up. Typical cold-formed members having loads on their flanges are shown in Fig. 16.12. In the two cases, which represent extreme conditions, the radius of bend between flange and web causes the load to be eccentric to the webs. In the first case, the eccentricity has little effect on the web capacity since, because of the symmetry of loading and configuration, there is some end restraint on the webs and since, if they do buckle, they must buckle together. The web which is forced to buckle toward its flange tips furnishes some support for the other. In the second case, however, there is no such restraint, eccentricity aggregates the buckling tendency in this case, and the critical stress becomes a function of the bend radius as well as the other parameters important in hot-rolled members.

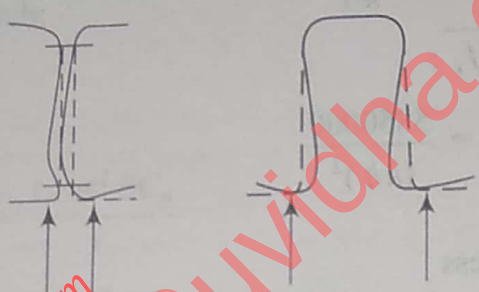


Fig. 16.16 *Web Crippling of Light-Gage Beams*

To avoid crippling of unreinforced beam webs having a flat-width ratio $h/t$ equal to or less than 150, concentrated loads and reactions should not exceed the values of $P_{max}$ given below, where $P_{max}$ is the allowable concentrated load or reaction. Webs of beams for which the ratio $h/t$ greater than 150 should be provided with adequate means of transmitting concentrated loads and reactions directly into the web.

*(a) Beams having Single Unreinforced Webs*

*1. For end Reactions or for Concentrated Loads on Outer Ends of Cantilevers*

For inside corner radius equal to or less than the thickness of sheet

$$P_{max} = 70t^2[98 + 4.20(N/t) - 0.022\,(N/t)\,(h/t) - 0.011\,(h/t)] \times [1.33 - 0.33\,(f_y/2320)]\,(f_y/2320) \quad (16.41)$$

*For other corner radii up to 4t, the value $P_{max}$ given by the above formula should be multiplied by* (1.15 – 0.15 $R/t$)

*2. For Reactions of Interior Supports or for Concentrated Loads Located Anywhere on the Span*

For inside corner radius equal to or less than the thickness of sheet

$$P_{max} = 70t^2[305 + 2.30(N/t) - 0.009(N/t)(h/t) - 0.5(h/t)] \times [1.22 - 0.22(f_y/2320)]\,(f_y/2320) \quad (16.42)$$

*For other corner radii up to 4t, the value $P_{max}$ given by the above formula is to be multiplied by (1.06 – 0.06 R/t).*

*3. For Corner Radii Larger than 4t, Tests shall be Made in Accordance with IS: 801.*

***(b) For I-beams Made of Two Channels Connected Back to Back or for Similar Sections which Provide a High Degree of Restraint Against Rotation of the Web, such as I-Sections Made by Welding Two Angles to a Channel***

*1. For End Reactions or for Concentrated Loads on the Outer Ends of Cantilevers*

$$P_{max} = t^2 f_y (4.44 + 0.558\sqrt{N/t}) \tag{16.43}$$

*2. For Reactions of Interior Supports or for Concentrated Loads Located Anywhere on the Span*

$$P_{max} = t^2 f_y (6.66 + 1.146\sqrt{N/t}) \tag{16.44}$$

where $t$ = web thickness

$N$ = actual length of bearing, expect that in the above formulae the value of $N$ should not be taken greater than $h$

$h$ = clear distance between flanges measured along the plane of web

$f_y$ = yield point

$R$ = inside bend radius

**Notes**

1. In all of the above expressions, $P_{max}$ represents the load or reaction for one solid web sheet connecting top and bottom flanges. For webs consisting of two or more such sheets, $P_{max}$ should be computed for each individual sheet and the results added to obtain the allowable load or reaction for the composite web.
2. For loads located close to ends of beams, provisions of (a)(2) and (b)(2) apply, provided that for cantilevers the distance from the free end to the nearest edge of bearing, and for a load close to an end support the clear distance from edge of end bearing to nearest edge of load bearing is larger than 1.5h. Otherwise provisions of (a)(1) and (b)(2) apply.



## Solved Examples

**Example 16.1** Find the column section properties and allowable load for the column section shown in Fig. Ex. 16.1 (a). The effective length of column is 3.0 m. Take $f_y$ = 235 N/mm$^2$.

*Solution*

***Linear properties***

Radius of corner, $R = 2.4 + 0.8 = 3.2$ mm

Length of corner $= 1.57R = 1.57 \times 3.2 = 5.024$ mm

$C_{xx} = 0.637R = 0.637 \times 3.2 = 2.038$ mm

$L = 2 \times 112 + 2 \times 112 + 4 \times 5.024 = 468.1$ mm

$A = 468.1 \times 1.6 = 748.96$ mm$^2$

Linear $I_{xx} = 2 \times 112 \times 59.2^2 + 2 \times \dfrac{112^3}{12} + 4 \times 5.024 \times 58.038^2 = 1086885.58$ mm$^3$

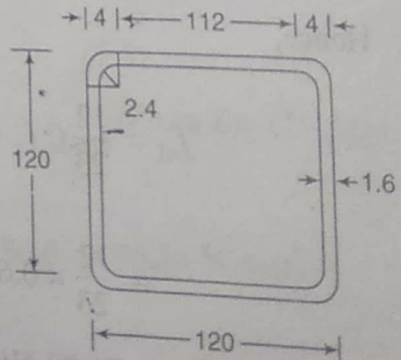



Fig. Ex. 16.1(a)

Actual $I_{xx} = 1086885.58 \times 1.6 = 1739016.92 \text{ mm}^4$

$$r = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{1739016.92}{748.96}} = 48.18 \text{ mm} = 4.8 \text{ cm}$$

$$f = 0.6 f_y = 0.6 \times 235 = 141 \text{ N/mm}^2$$

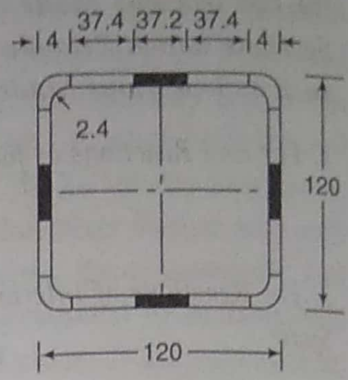


**Fig. Ex. 16.1(b)**

To calculate the form factor, effective width and effective area of the section will be required.

$$\left(\frac{w}{t}\right)_{\lim} = \frac{478}{\sqrt{f}} = \frac{478}{\sqrt{141}} = 40.25$$

$$\frac{w}{t} = \frac{112}{1.6} = 70 > 40.25$$

$$\therefore \quad \frac{b}{t} = \frac{658}{\sqrt{f}}\left(1 - \frac{130}{(w/t)\sqrt{f}}\right) \quad \text{(From Table 16.2)}$$

$$= \frac{658}{\sqrt{141}}\left(1 - \frac{130}{70\sqrt{141}}\right) = 46.75$$

$$\therefore \quad b = 46.75 \times 1.6 = 74.8$$

$$A_{eff} = 749 - 4 \times 37.2 \times 1.6 = 510.92 \text{ mm}^2$$ [refer Fig. Ex. 16.1(b)]

Form factor, $Q = \dfrac{A_{eff}}{A} = \dfrac{510.92}{749} = 0.682$

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} = \sqrt{\frac{2\pi^2 \times 2 \times 10^5}{235}} = 129.61$$

$$\left(\frac{l}{r}\right)_{\lim} = \frac{C_c}{\sqrt{Q}} = \frac{129.61}{\sqrt{0.682}} = 157$$

Actual $\dfrac{l}{r} = \dfrac{3.0 \times 100}{4.8} = 62.5 < 157$

Hence,

$$f_{a1} = \frac{12}{23} Q f_y - \frac{3}{23} \times \frac{(Q f_y)^2}{\pi^2 E}\left(\frac{kL}{r}\right)^2 \quad \text{(From Eq. (16.32))}$$

$$= \frac{12}{23} \times 0.682 \times 235 - \frac{3}{23} \times \frac{(0.682 \times 235)^2}{\pi^2 \times 2 \times 10^5} \times 62.5^2$$

$$= 76.99 \text{ N/mm}^2$$

Allowable load $= f_a \times A = 76.99 \times 748.96 = 57662 \text{ N} = 57.66 \text{ kN}$

**Example 16.2** A hat of 100 mm × 80 mm × 4 mm section with a 25 mm lip as shown in Fig. Ex. 16.2 is to be used as a concentrically loaded column of 3.1 m effective length. Determine the allowable load. Take $f_y = 235$ N/mm$^2$.

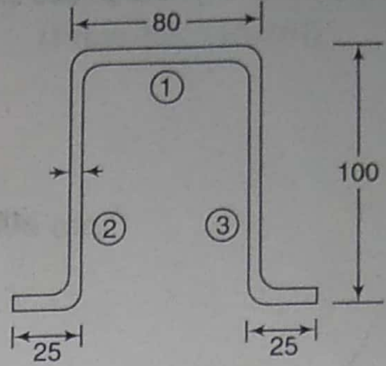



**Fig. Ex. 16.2**

*Solution*

$$f = 0.6 f_y = 0.6 \times 235 = 141 \text{ N/mm}^2$$

$$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{446}{\sqrt{f}} = \frac{446}{\sqrt{141}} = 37.6$$

$$\frac{w_1}{t} = \frac{72}{4} = 18 \quad < 37.6$$

$$\frac{w_2}{t} = \frac{92}{4} = 23 \quad < 37.6$$

Since $w/t$ ratio for both flange and web is less than $(w/t)_{\text{lim}}$, full width will be effective.

i.e., $b = w$ and $Q = 1$.

From IS: 811, the relevant properties of the section are

$$A = 1180 \text{ mm}^2, C_y = 45.1 \text{ mm}$$

$$I_{xx} = 152 \times 10^4 \text{ mm}^4, I_{yy} = 161 \times 10^4 \text{ mm}^4$$

$$r_{xx} = 35.8 \text{ mm}, r_{yy} = 36.9 \text{ mm}$$

$$\frac{l}{r} = \frac{3100}{35.8} = 86.6 \qquad (kL = 3.1 \text{ m})$$

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} = \sqrt{\frac{2\pi^2 \times 2 \times 10^5}{235}} = 129.61$$

$$\left(\frac{l}{r}\right)_{\text{lim}} = \frac{C_c}{\sqrt{Q}} = \frac{129.61}{\sqrt{1}} = 129.61 \quad < 86.6$$

Since $\frac{l}{r} < \left(\frac{l}{r}\right)_{\text{lim}}$

Hence

$$f_{a1} = \frac{12}{23} Q f_y - \frac{3}{23} \times \frac{(Q f_y)^2}{\pi^2 E} \left(\frac{kL}{r}\right)^2 \qquad \text{(From Eq. (16.32))}$$

$$= \frac{12}{23} \times 1 \times 235 - \frac{3}{23} \times \frac{(1 \times 235)^2}{\pi^2 \times 2 \times 10^5} \times 86.6^2 = 95.24 \text{ N/mm}^2$$

Allowable load $= f_a \times A$

$= 95.24 \times 1180 = 112383 \text{ N} = 112.4 \text{ kN}$

**Example 16.3** A square box of 180 mm × 180 mm × 2 mm section as shown in Fig. Ex. 16.3 is to be used as a column of 4 m effective length. It is stiffened on all four sides. Find the maximum load it can carry. Design also the stiffener. Take $f_y$ = 235 N/mm$^2$.

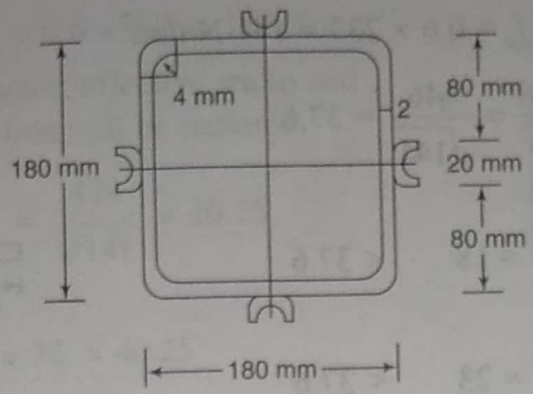



**Fig. Ex. 16.3**

*Solution*

Linear properties

Radius of corner, $R = 4 + 1 = 5$ mm

Length of corner $= 1.57R = 1.57 \times 5 = 7.85$ mm

$$C_{xx} = 0.637R = 0.637 \times 5 = 3.185 \text{ mm}$$

$$L = 2 \times 168 + 2 \times 168 + 4 \times 7.85 = 703.4 \text{ mm}$$

$$A = 703.4 \times 2 = 1406.8 \text{ mm}^2$$

Linear $$I_{xx} = 2 \times 168 \times 89^2 + 2 \times \frac{168^3}{12} + 4 \times 7.85 \times 87.185^2 = 3511397.6$$

Actual $$I_{xx} = 3511397.6 \times 2 = 7022795.22 \text{ mm}^4$$

$$r = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{7022795.22}{1406.8}} = 70.65 \text{ mm} = 7.1 \text{ cm}$$

$$\left(\frac{w}{t}\right)_{\lim} = \frac{478}{\sqrt{f}} = \frac{478}{\sqrt{141}} = 40.25$$

$$\frac{w}{t} = \frac{172}{2} = 86 > 40.25$$

$$\frac{b}{t} = \frac{658}{\sqrt{f}}\left[1 - \frac{130}{\left(\frac{w}{t}\right)\sqrt{f}}\right]$$ (From Table 16.2)

$$= \frac{658}{\sqrt{141}}\left[1 - \frac{130}{86\sqrt{141}}\right] = 48.36$$

When flat width ratio of the sub-element of a multiple stiffened element exceeds 60, the effective design width is reduced and is given by

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left(\frac{w}{t} - 60\right) \qquad \text{(From Eq. (16.16))}$$

$$= 48.36 - 0.10 \times (86 - 60) = 45.76$$

or,

$$b_e = 2 \times 45.76 = 91.52 \text{ mm}$$

$$A_{eff} = 1406.8 - 4 \times (168 - 91.52) \times 2 = 794.96 \text{ mm}^2$$

$$Q = \frac{A_{eff}}{A} = \frac{794.96}{1406.8} = 0.565$$

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} = \sqrt{\frac{2\pi^2 \times 2 \times 10^5}{235}} = 129.61$$

$$\left(\frac{l}{r}\right)_{\text{lim}} = \frac{C_c}{\sqrt{Q}} = \frac{129.61}{\sqrt{0.565}} = 172.43 \qquad (kL = 4.0 \text{ m})$$

$$\text{Actual } \frac{l}{r} = \frac{4 \times 100}{7.1} = 56.34 < 172.43$$

$$f_{a1} = \frac{12}{23} Q f_y - \frac{3}{23} \times \frac{(Q f_y)^2}{\pi^2 E}\left(\frac{kl}{r}\right)^2 \qquad \text{(From Eq. (16.32))}$$

$$= \frac{12}{23} \times 0.565 \times 235 - \frac{3}{23} \times \frac{(0.565 \times 235)^2}{\pi^2 (2 \times 10^5)} (56.34)^2 = 65.57 \text{ N/mm}^2$$

Allowable load $= f_a \times A$

$$= 65.57 \times 1406.8 = 92243.8 \text{ N} = 92.24 \text{ kN}$$

For stiffener,

$$I_{min} = 1.83 t^4 \sqrt{\left(\frac{w}{t}\right)^2 - \frac{27590}{f_y}}$$

$$= 1.83 \times 2^4 \times \sqrt{86^2 - \frac{27590}{235}} = 2498 \text{ mm}^4$$

Use 20 mm × 20 mm × 1.60 mm channels as stiffeners. (From Table 3 of IS: 811)
$I_{xx} = 5540 \text{ mm}^4$, $I_{yy} = 3450 \text{ mm}^4 > 2498 \text{ mm}^4$

**Example 16.4** Design a 4.0 m long column to carry a load of 170 kN. Take $f_y = 235$ N/mm$^2$.

*Solution* Use two channels of 140 mm × 70 mm × 4 mm section having 25 mm lips with a back-to-back distance of 50 mm as shown in Fig. Ex. 16.4.

Area, $A = 2 \times [4 \times 140 + 2 \times (4 \times 62) + 2 \times (4 \times 25)] = 2512 \text{ mm}^2$

$$I_{xx} = 2\times\left[\frac{(140)^3}{12}\times 4 + 2(4\times 62)\times 68^2 + \left\{4\times\frac{(140)^3}{12} - 4\times\frac{(140-2\times 5)^3}{12}\right\}\right]$$

$$= 7759674.67 \text{ mm}^4$$

$$r_x = \sqrt{\frac{I_{xx}}{A}} = 55.58 \text{ mm}$$

$$\frac{l}{r_x} = \frac{4000}{55.58} = 71.97$$

$$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{446}{\sqrt{f}} = \frac{446}{\sqrt{141}} = 37.6$$

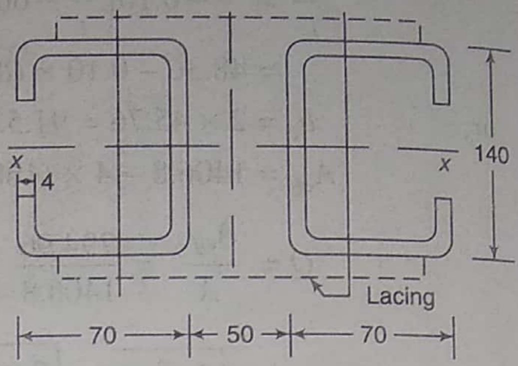


Fig. Ex. 16.4

For web

$$\frac{w}{t} = \frac{132}{4} = 33 < 37.6$$

Hence, $b = w$ and $Q = 1$.

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} = \sqrt{\frac{2\pi^2\times(2\times 10^5)}{235}} = 129.1$$

$$\left(\frac{l}{r}\right)_{\text{lim}} = \frac{C_c}{\sqrt{Q}} = \frac{129.61}{\sqrt{1}} = 129.61$$

Since $\frac{l}{r_x} < \left(\frac{l}{r}\right)_{\text{lim}}$

Hence $f_a = \frac{12}{23}Qf_y - \frac{3}{23}\times\frac{(Qf_y)^2}{\pi^2 E}\left(\frac{kL}{r}\right)^2$ (From Eqn. 16.32)

$$= \frac{12}{23}\times 1\times 235 - \frac{3}{23}\times\frac{(1\times 235)^2}{\pi^2\times 2\times 10^5}\times 71.97^2 = 103.71 \text{ N/mm}^2$$

Allowable load $= f_a \times A$

$= 103.71 \times 2512 = 260519.52$ N $= 260.52$ kN $> 170$ kN

Hence safe.

***Lacing***

Transverse shear per lacing $= \frac{1}{2}\times\frac{2.5}{100}\times 170\times 10^3 = 2125$ N $= 2.125$ kN

Compressive force in lacing bars = 2.125 × cosec 45° = 3.005 kN

Let us consider a weld of 3 mm size and 45 mm length.

Assuming a 5 mm margin on both sides.

Effective spacing = 190 – (2 × 5) = 180 mm

Length of lacing flat, $l$ = 180 cosec 45°

$\frac{l}{r}$ should not exceed 145.

$$r \geq \frac{l}{145} = \frac{180 \operatorname{cosec} 45°}{145} = 1.756 \text{ mm}$$

Since $$r = \frac{t}{\sqrt{12}}$$

Hence $$t \geq 1.756 \times \sqrt{12} = 6.08 \text{ mm}$$

Use 8 mm thick flat of width 20 mm.

$$r = \frac{t}{\sqrt{12}} = \frac{8}{\sqrt{12}} = 2.31 \text{ mm}$$

$$\frac{l}{r} = \frac{254.6}{2.31} = 110.2$$

$$\frac{w}{t} = \frac{20}{5} = 4 < 37.6$$

Hence $b = w$ and $Q = 1$.

$$\therefore \quad f_{a1} = \frac{12}{23} Q f_y - \frac{3}{23} \times \frac{(Q f_y)^2}{\pi^2 E}\left(\frac{kl}{r}\right)^2 \quad \text{(From Eq. (16.32))}$$

$$= \frac{12}{23} \times 1 \times 235 - \frac{3}{23} \times \frac{(1 \times 235)^2}{\pi^2 \times 2 \times 10^5} \times 110.2^2 = 78.29 \text{ N/mm}^2$$

Allowable load = 78.29 × (8 × 20) = 12526.8 N = 12.53 kN > 3.005 kN

**Example 16.5** Two channels of 180 mm × 80 mm section with bent lips as shown in Fig. Ex. 16.5 are connected with webs to act as beam. The thickness of the plate is 2.5 mm and the depth of the lip is 25 mm. The beam has an effective span of 4.1 m. Determine the allowable load per m run on the beam. Take $f_y = 235$ N/mm$^2$.

*Solution* Basic design stress, $f = 0.6 f_y = 0.6 \times 235$
$= 141$ N/mm$^2$

$$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{446}{\sqrt{f}} = \frac{446}{\sqrt{141}} = 37.6$$

$$\frac{w}{t} = \frac{64}{2.5} = 25.6 < 37.6$$

$$\frac{b}{t} = \frac{658}{\sqrt{f}}\left(1 - \frac{145}{(w/t)\sqrt{f}}\right)$$

(From Table 16.2)

$$= \frac{658}{\sqrt{141}}\left(1 - \frac{145}{25.6\sqrt{141}}\right) = 28.98$$

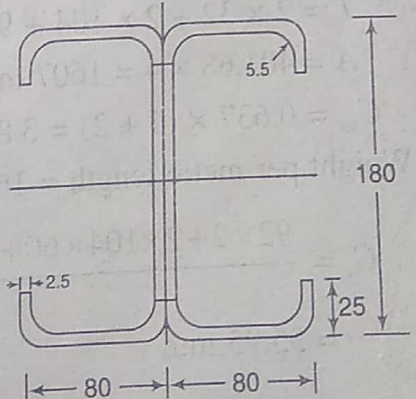



**Fig. Ex. 16.5**

$$b = 28.98 \times 2.5 = 72.4 > 64$$

Hence, $b = w = 64$ mm

$$I_{xx} = 4 \times 80 \times 2.5 \times (90 - 1.25)^2 + \frac{1}{12} \times (2.5 \times 2) \times 172^3 + 4 \times 2.5 \times 22.5 \times (90 - 25 + 11.25)^2$$

$$= 9842486.98 \text{ mm}^4$$

$$Z = \frac{I_{xx}}{y} = \frac{9842486.98}{90} = 109.36 \times 10^3 \text{ mm}^3$$

$$M = fZ$$

$$= 141 \times 109.36 \times 10^3 = 15.42 \times 10^6 \text{ Nmm} = 15.4 \text{ kNm}$$

Let, $w$ be the load in kN/m.

$$\frac{w \times 4.1^2}{8} = 15.4 \text{ kN/m}$$

$$\Rightarrow \quad w = 7.33 \text{ kN/m}$$

**Example 16.6** A top chord member of a roof truss is of hat section as shown in Fig. Ex. 16.6. It is subjected to a compression of 132.5 kN and a moment of 1636 kNm. Check the safety of the section if $f_y = 210$ N/mm$^2$ and the length of the member is 1.70 m.

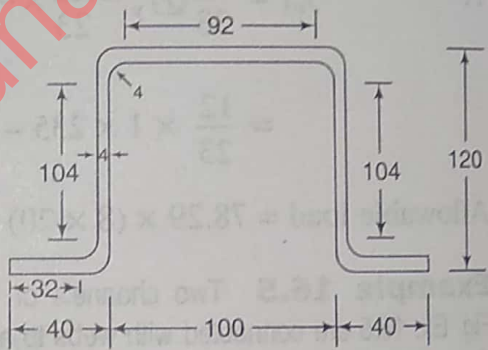


**Fig. Ex. 16.6**

*Solution* For flange

$$\frac{w_1}{t} = \frac{92}{4} = 23 \quad (w_1 = 108 - 16 = 92)$$

For web

$$\frac{w_2}{t} = \frac{104}{4} = 26 \quad (w_2 = 120 - 16 = 104)$$

$$\left(\frac{w}{t}\right)_{lim} = \frac{446}{\sqrt{f}} = \frac{446}{\sqrt{125}} = 40.58$$

Hence, all the elements are fully effective (i.e., $b = w$) and $Q = 1$.

***Linear properties***

$$L = 2 \times 32 + 2 \times 104 + 92 + 4 \times 9.42 = 401.68 \text{ mm}$$

$$A = 401.68 \times 4 = 1607 \text{ mm}^2$$

$$C_{xx} = 0.637 \times (4 + 2) = 3.822$$

Weight per metre length = 1607 × 0.0785 = 126 N

$$C_y = \frac{92 \times 2 + 2 \times 104 \times 60 + 2 \times 32 \times 118 + 2 \times 9.42 \times 4.178 + 2 \times 9.42 \times 115.822}{401.68}$$

$$= 55.95 \text{ mm}$$

Linear $$I_{xx} = 92 \times 54^2 + 2 \times \frac{104^3}{12} + 2 \times 104 \times 4^2 + 2 \times 32 \times 62^2 + 2 \times 9.42 \times 51.822^2 + 2 \times 9.42 \times 59.822^2$$

$$= 823.11 \text{ mm}^3$$

Actual $I_{xx} = 823.11 \times 10^3 \times 4 = 329.24 \times 10^4 \text{ mm}^4$

Linear $I_{yy} = \frac{92^3}{12} + 2 \times \frac{32^3}{12} + 2 \times 32 \times 74^2 + 2 \times 104 \times 52^2 + 2 \times 9.42 \times 49.822^2 + 2 \times 9.42 \times 54.178^2 = 1085.31 \times 10^3 \text{ mm}^3$

Actual $I_{yy} = 1085.31 \times 10^3 \times 4 \text{ mm}^4 = 434.1 \times 10^4 \text{ mm}^4$

$$r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{329.24 \times 10^4}{1607}} = 45.26 \text{ mm}$$

$$\frac{l}{r_{xx}} = \frac{1700}{45.26} = 37.6$$

$$f_{a1} = \frac{12}{23} Qf_y - \frac{3}{23} \times \frac{(Qf_y)^2}{\pi^2 E}\left(\frac{kl}{r}\right)^2 \quad \text{(From Eq. (16.32))}$$

$$= \frac{12}{23} \times 1 \times 235 - \frac{3}{23} \times \frac{(1 \times 235)^2}{\pi^2 \times 2 \times 10^5} \times 37.6^2 = 117.45 \text{ N/mm}^2$$

$$f_{a,cal} = \frac{132.5 \times 10^3}{1607} = 82.45 \text{ N/mm}^2$$

$$f_{b,cal} = \frac{1636 \times 10^3 \times 56}{329.24 \times 10^4} = 27.83 \text{ N/mm}^2$$

$$f_e' = \frac{12}{23} \times \frac{\pi^2 E}{(KL_b / r_b)^2} = \frac{12 \times \pi^2 \times 2 \times 10^5}{23 \times 37.6^2} = 728.46 \text{ N/mm}^2$$

$$C_m = 1.00$$

From the interaction formula (IS 801: 1975)

$$\frac{f_{a,cal}}{f_{a1}} + \frac{f_{b,cal}\, C_m}{f_{b,cal}\left[1 - \frac{f_{a,cal}}{f_e'}\right]} \le 1$$

$$= \frac{82.45}{117.45} + \frac{27.83 \times 1}{125 \times \left[1 - \frac{82.45}{728.46}\right]} = 0.95$$

which is less than 1.

Hence, this section is safe.

## Exercises

16.1 Compute the elastic critical stresses, for the plate shown in Fig. Prob. 16.1, for different edge conditions. Thickness of plate = 2 mm. Take $E = 2 \times 10^5$ N/mm$^2$, $\mu = 0.3$