19 COLD FORMED STEEL SECTIONS – I

1.0 INTRODUCTION

Thin sheet steel products are extensively used in building industry, and range from purlins to roof sheeting and floor decking. Generally these are available for use as basic building elements for assembly at site or as prefabricated frames or panels. These thin steel sections are ***cold-formed,*** i.e. their manufacturing process involves forming steel sections in a cold state (i.e. without application of heat) from steel sheets of ***uniform*** thickness. These are given the generic title ***Cold Formed Steel Sections***. Sometimes they are also called ***Light Gauge Steel Sections*** or ***Cold Rolled Steel Sections***. The thickness of steel sheet used in cold formed construction is usually *1* to *3* mm. Much thicker material up to *8 mm* can be formed if pre-galvanised material is not required for the particular application. The method of manufacturing is important as it differentiates these products from ***hot rolled steel*** sections. Normally, the yield strength of steel sheets used in cold-formed sections is at least $280\ N/mm^2$, although there is a trend to use steels of higher strengths, and sometimes as low as $230\ N/mm^2$.

Manufacturers of cold formed steel sections purchase steel coils of *1.0* to *1.25 m* width, slit them longitudinally to the correct width appropriate to the section required and then feed them into a series of roll forms. These rolls, containing male and female dies, are arranged in pairs, moving in opposite direction so that as the sheet is fed through them its shape is gradually altered to the required profile. The number of pairs of rolls (called ***stages***) depends on the complexity of the cross sectional shape and varies from *5* to *15*. At the end of the rolling stage a flying shearing machine cuts the member into the desired lengths.

An alternative method of forming is by press - braking which is limited to short lengths of around *6 m* and for relatively simple shapes. In this process short lengths of strip are pressed between a male and a female die to fabricate, one fold at a time and obtain the final required shape of the section. Cold rolling is used when large volume of long products are required and press breaking is used when small volume of short length products are produced.

Galvanizing (or zinc coating) of the preformed coil provides very satisfactory protection against corrosion in internal environments. A coating of $275\ g/m^2$ (total for both faces) is the usual standard for internal environments. This corresponds to zinc coating of 0.04 *mm*. Thicker coatings are essential when moisture is present for long periods of time. Other than galvanising, different methods of pre-rolling and post-rolling corrosion protection measures are also used.

Although the cold rolled products were developed during the First World War, their extensive use worldwide has grown only during the last 20 years because of their versatility and suitability for a range of lighter load bearing applications. Thus the wide range of available products has extended their use to primary beams, floor units, roof trusses and building frames. Indeed it is difficult to think of any industry in which Cold Rolled Steel products do not exist in one form or the other. Besides building industry, they are employed in motor vehicles, railways, aircrafts, ships, agricultural machinery, electrical equipment, storage racks, house hold appliances and so on. In recent years, with the evolution of attractive coatings and the distinctive profiles that can be manufactured, cold formed steel construction has been used for highly pleasing designs in practically every sector of building construction.

In this chapter, the background theory governing the design of cold formed steel elements is presented in a summary form. Design of cold formed steel sections are dealt with in IS: 801-1975 which is currently due under revision. In the absence of a suitable Limit State Code in India, the Code of Practice for Cold Formed Sections in use in the U.K.(BS 5950, Part 5) is employed for illustrating the concepts with suitable modifications appropriate to Indian conditions.

2.0 ADVANTAGES OF COLD FORMED SECTIONS

Cold forming has the effect of increasing the yield strength of steel, the increase being the consequence of cold working well into the strain-hardening range. These increases are predominant in zones where the metal is bent by folding. The effect of cold working is thus to enhance the mean yield stress by *15% - 30%.* For purposes of design, the yield stress may be regarded as having been enhanced by a minimum of *15%.*

Some of the main advantages of cold rolled sections, as compared with their hot-rolled counterparts are as follows:

- Cross sectional shapes are formed to close tolerances and these can be consistently repeated for as long as required.
- Cold rolling can be employed to produce almost any desired shape to any desired length.
- Pre-galvanised or pre-coated metals can be formed, so that high resistance to corrosion, besides an attractive surface finish, can be achieved.
- All conventional jointing methods, (i.e. riveting, bolting, welding and adhesives) can be employed.
- High strength to weight ratio is achieved in cold-rolled products.
- They are usually light making it easy to transport and erect.

It is possible to displace the material far away from the neutral axis in order to enhance the load carrying capacity (particularly in beams).

There is almost no limit to the type of cross section that can be formed. Some typical cold formed section profiles are sketched in Fig.1.

In Table 1 hot rolled and cold formed channel section properties having the same area of cross section are shown. From Table 1, it is obvious that thinner the section walls, the larger will be the corresponding moment of inertia values (I_{xx} and I_{yy}) and hence capable of resisting greater bending moments. The consequent reduction in the weight of steel in general applications produces economies both in steel costs as well as in the costs of handling transportation and erection. This, indeed, is one of the main reasons for the popularity and the consequent growth in the use of cold rolled steel. Also cold form steel is protected against corrosion by proper galvanising or powder coating in the factory itself. Usually a thickness limitation is also imposed, for components like lipped channels.

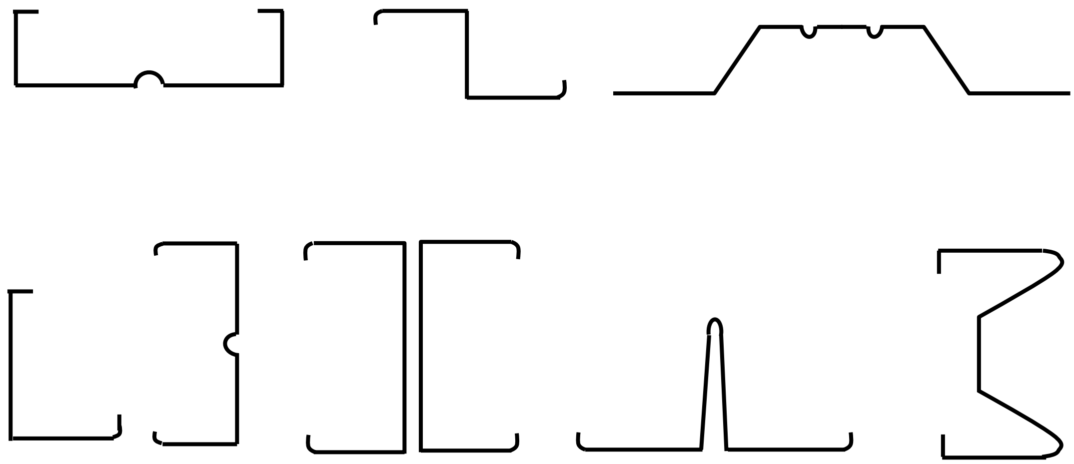
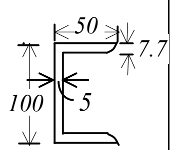
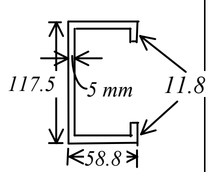
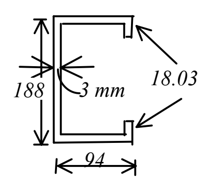
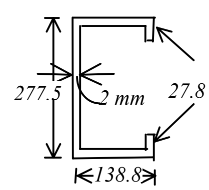


Fig. 1 Typical Cold Formed Steel Profiles

Table - 1 Comparison of Hot Rolled and Cold Rolled sections

	 *Hot rolled channel*	 *Cold rolled channel*	 *Cold rolled channel*	 *Cold rolled channel*
A	$1193\ mm^2$	$1193\ mm^2$	$1193\ mm^2$	$1193\ mm^2$
I_{xx}	$1.9 \times 10^6\ mm^4$	$2.55 \times 10^6\ mm^4$	$6.99 \times 10^6\ mm^4$	$15.53 \times 10^6\ mm^4$
Z_{xx}	$38 \times 10^3\ mm^3$	$43.4 \times 10^3\ mm^3$	$74.3 \times 10^3\ mm^3$	$112 \times 10^3\ mm^3$
I_{yy}	$0.299 \times 10^6\ mm^4$	$0.47 \times 10^6\ mm^4$	$1.39 \times 10^6\ mm^4$	$3.16 \times 10^6\ mm^4$
Z_{yy}	$9.1 \times 10^3\ mm^3$	$11.9 \times 10^3\ mm^3$	$22 \times 10^3\ mm^3$	$33.4 \times 10^3\ mm^3$

While the strength to weight ratios obtained by using thinner material are significantly higher, particular care must be taken to make appropriate design provisions to account for the inevitable buckling problems.

2.1 Types of Stiffened and Unstiffened Elements

As pointed out before, cold formed steel elements are either ***stiffened*** or ***unstiffened***. An element which is supported by webs along both its longitudinal edges is called a ***stiffened*** element. An ***unstiffened*** element is one, which is supported along one longitudinal edge only with the other parallel edge being free to displace. Stiffened and unstiffened elements are shown in Fig. 2.

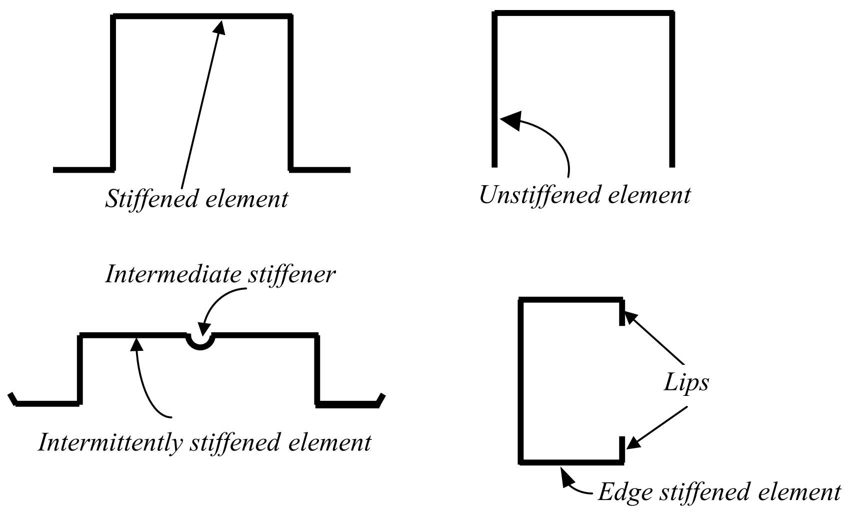


Fig. 2 Stiffened and Unstiffened elements

An ***intermittently stiffened element*** is made of a very wide thin element which has been divided into two or more narrow sub elements by the introduction of intermediate stiffeners, formed during rolling.

In order that a flat compression element be considered as a ***stiffened element***, it should be supported along one longitudinal edge by the web and along the other by a web or lip or other edge stiffener, (eg. a bend) which has sufficient flexural rigidity to maintain straightness of the edge, when the element buckles on loading. A rule of thumb is that the depth of simple “lips” or right angled bends should be at least one-fifth of the adjacent plate width. More exact formulae to assess the adequacy of the stiffeners are provided in Codes of Practice. If the stiffener is adequate, then the edge stiffened element may be treated as having a local buckling coefficient (*K)* value of *4.0*. If the edge stiffener is inadequate (or only partially adequate) its effectiveness is disregarded and the element will be regarded as ***unstiffened***, for purposes of design calculations.

3.0 LOCAL BUCKLING

Local buckling is an extremely important facet of cold formed steel sections on account of the fact that the very thin elements used will invariably buckle before yielding. Thinner the plate, the lower will be the load at which the buckles will form.

3.1 Elastic Buckling of Thin Plates

It has been shown in the chapter on "Introduction to Plate Buckling" that a flat plate simply supported on all edges and loaded in compression (as shown in Fig. 3(a)) will buckle at an elastic critical stress given by

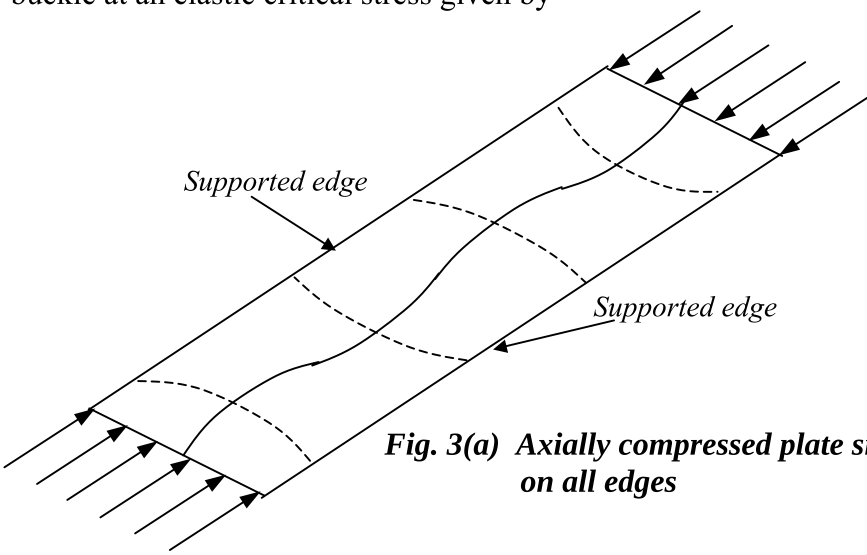


Fig. 3(a) Axially compressed plate simply supported on all edges

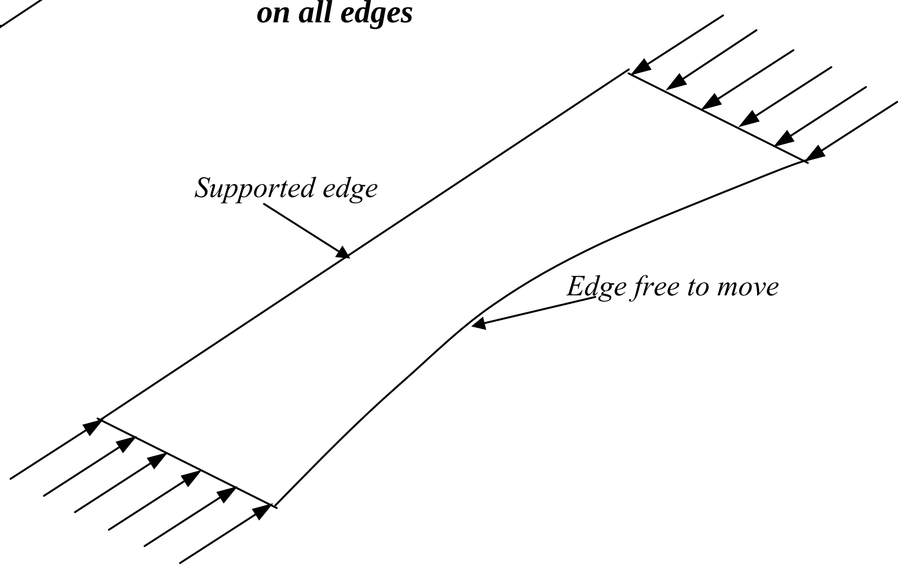


Fig. 3(b) Axially compressed plate with one edge supported and the other edge free to move

$$p_{cr} = \frac{K\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 \tag{1}$$

Substituting the values for π, $\nu = 0.3$ and $E = 205\ kN/mm^2$, we obtain the value of p_{cr} as

$$p_{cr} \approx 185\times 10^3 \times K\left(\frac{t}{b}\right)^2 \text{ with units of } N/mm^2 \tag{1a}$$

The value of *K* is dependent on support conditions. When all the edges are simply supported *K* has a value of *4.0.*

When one of the edges is free to move and the opposite edge is supported, (as shown in Fig. 3b), the plate buckles at a significantly lower load, as *K* reduces dramatically to *0.425.* This shows that plates with free edges do not perform well under local buckling. To counter this difficulty when using cold formed sections, the free edges are provided with a lip so that they will be constrained to remain straight and will not be free to move. This concept of stiffening the elements is illustrated in Fig. 4.

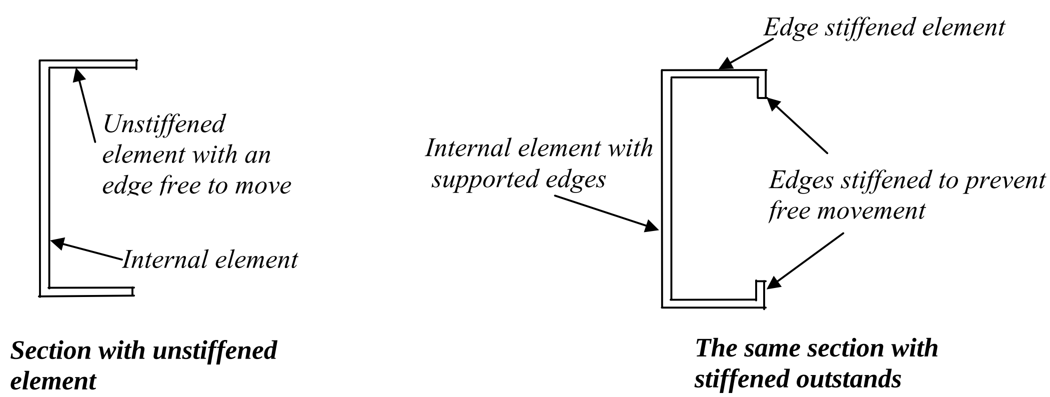
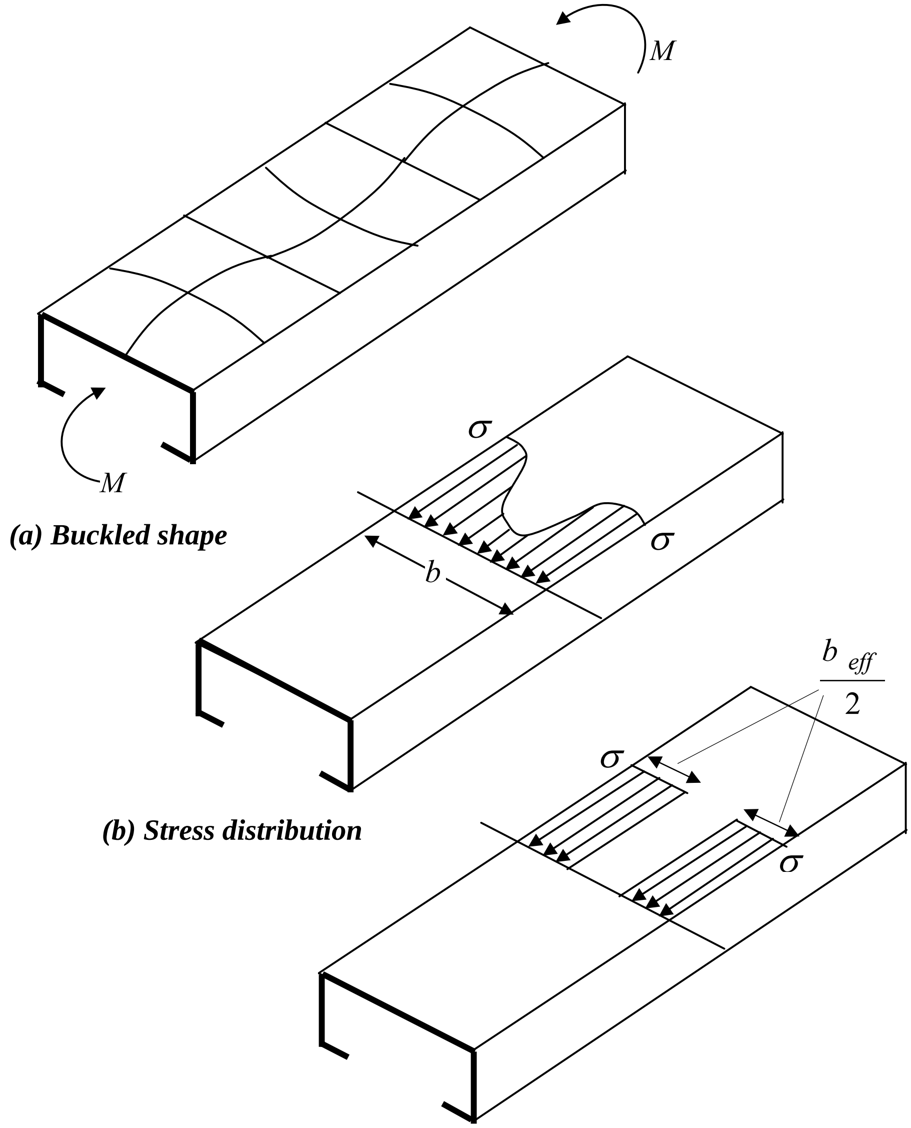


Fig. 4 The technique of stiffening the element

3.2 Post - critical behaviour

Let us consider the channel subjected to a uniform bending by the application of moments at the ends. The thin plate at the top is under flexural compression and will buckle as shown in Fig. 5 (a). This type of buckling is characterised by ripples along the length of the element. The top plate is supported along the edges and its central portion, which is far from the supports, will deflect and shed the load to the stiffer edges. The regions near the edges are prevented from deflecting to the same extent. The stresses are non uniform across the section as shown in Fig. 5 (b). It is obvious that the applied moment is largely resisted by regions near the edges (i.e. elements which carry increased stresses) while the regions near the centre are only lightly stressed and so are less effective in resisting the applied moment.



(a) Buckled shape

(b) Stress distribution

(c) Effective width

Fig. 5 Local Buckling Effects

From a theoretical stand point, flat plates would buckle instantaneously at the elastic critical load. Under incremental loading, plate elements which are not perfectly flat will begin to deform out of plane from the beginning rather than instantaneously at the onset of buckling and fail at a lower load. This means that a non-uniform state of stress exists throughout the loading regime. The variation of mean stress with lateral deflection for flat plates and plates with initial imperfection, under loading are shown in Fig. 6.

This tendency is predominant in plates having *b/t* (breadth/thickness) ratios of *30-60*. For plates having a *b/t* value in excess of *60*, the in-plane tensile stresses or the "membrane stresses" (generated by the stretching of the plates) resist further buckling and cause an increase in the load-carrying capacity of wide plates.

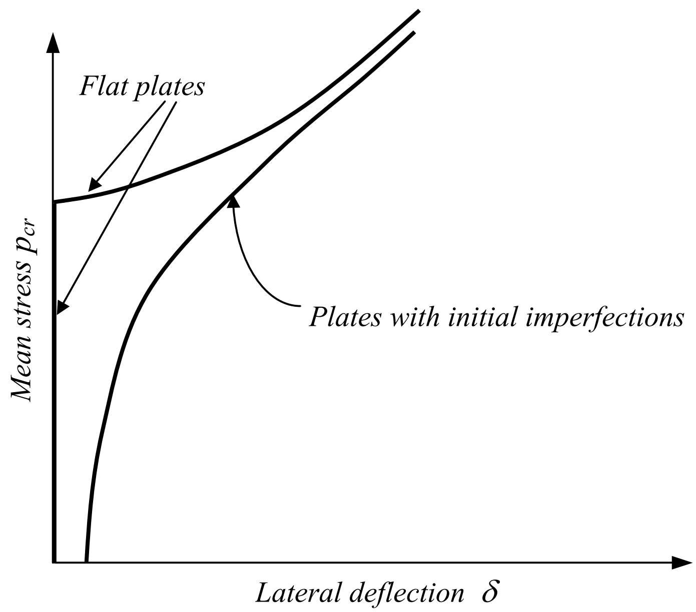


Fig. 6 Mean stress Vs Lateral deflection relation

3.3 Effective Width Concept

The effects of local buckling can be evaluated by using the concept of ***effective width***. Lightly stressed regions at centre are ignored, as these are least effective in resisting the applied stresses. Regions near the supports are far more effective and are taken to be fully effective. The section behaviour is modelled on the basis of the effective width (b_{eff}) sketched in Fig. 5(c).

The effective width, (b_{eff}) multiplied by the edge stress (σ) is the same as the mean stress across the section multiplied by the total width (b) of the compression member.

The ***effective width*** of an element under compression is dependent on the magnitude of the applied stress f_c, the width/thickness ratio of the element and the edge support conditions.

3.4 Code Provisions on "Local Buckling of Compressed Plates"

The effective width concept is usually modified to take into account the effects of yielding and imperfection. For example, BS5950: Part 5 provides a semi-empirical formula for basic effective width, b_{eff}, to conform to extensive experimental data.

When $f_c > 0.123\ p_{cr}$, then

$$\frac{b_{eff}}{b} = \left[1 + 14 \left\{ \left[\frac{f_c}{p_{cr}} \right]^{0.5} - 0.35 \right\}^4 \right]^{-0.2} \qquad (2a)$$

When $f_c < 0.123\ p_{cr}$, then $b_{eff} = b$ (2b)

where

f_c = compressive stress on the effective element, N/mm^2
p_{cr} = local buckling stress given by
$p_{cr} = 185{,}000\ K\ (t/b)^2\ N/mm^2$
K = load buckling coefficient which depends on the element type, section geometry etc.
t = thickness of the element, in *mm*
b = width of the element, in *mm*

The relationship given by eqn. 2 (a) is plotted in Fig.7

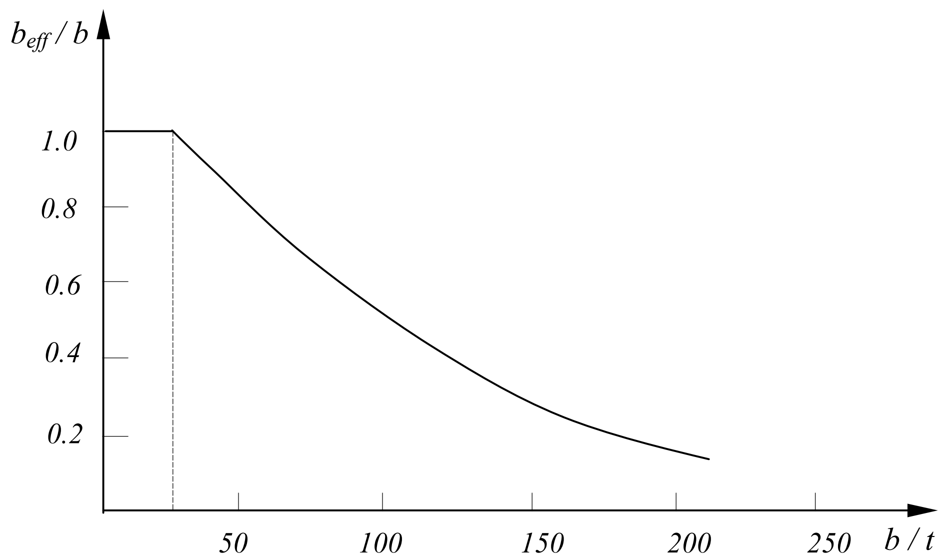


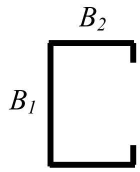
Fig. 7 Ratio of effective width to flat width (f_y = 280 N/mm^2) of compression plate with simple edge supports

It is emphasised that in employing eqn. (2a), the value of K (to compute p_{cr}) could be *4.0* for a stiffened element or *0.425* for an unstiffened element.

BS5950, part 5 provides for a modification for an unstiffened element under uniform compression (Refer clause 4.5.1). The code also provides modifications for elements under combined bending and axial load (ref. Clause 4.5.2). Typical formula given in

BS 5950, Part 5 for computing *K* values for a channel element is given below for illustration. (see BS 5950, Part 5 for a complete list of buckling coefficients).

1. Lipped channel.



The buckling coefficient K_1 for the member having a width of B_1 in a lipped channel of the type shown above is given by

$$K_1 = 7 - \frac{1.8h}{0.15 + h} - 1.43h^3 \qquad (3a)$$

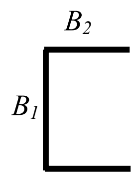
where $h = B_2 / B_1$

For the member having the width of B_2 in the above sketch.

$$K_2 = K_1 h^2 \left(\frac{t_1}{t_2} \right)^2 \qquad (3b)$$

where t_1 and t_2 are the thicknesses of element width B_1 and B_2 respectively. (Note: normally t_1 and t_2 will be equal). The computed values of K_2 should not be less than *4.0* or *0.425* as the case may be.

2) Plain channel (without lips)



The buckling coefficient K_1 for the element of width B_1 is given by

$$K_1 = \frac{2}{\left(1 + 15h^3\right)^{0.5}} + \frac{2 + 4.8h}{\left(1 + 15h^3\right)} \qquad (4)$$

K_2 is computed from eqn.. 3(b) given above.

3.4.1 Maximum width to thickness ratios

IS: 801 and *BS 5950, Part 5* limit the maximum ratios of *(b/t)* for compression elements as follows:

- Stiffened elements with one longitudinal edge connected to a flange or web element and the other stiffened by a simple lip — 60
- Stiffened elements with both longitudinal edges connected to other stiffened elements — 500
- Unstiffened compression elements — 60

However the code also warns against the elements developing very large deformations, when *b/t* values exceed half the values tabulated above.

3.5 Treatment of Elements with Stiffeners

3.5.1 Edge Stiffeners

As stated previously, elements having $b/t \leq 60$ and provided with simple lip having one fifth of the element width may be regarded as a stiffened element. If $b/t > 60$, then the width required for the lip may become too large and the lip itself may have stability problems. Special types of lips (called "compound" lips) are designed in such cases and these are outside the scope of this chapter.

3.5.2 Intermediate stiffeners

A wide and ineffective element may be transformed into a highly effective element by providing suitable intermediate stiffeners (having a minimum moment of inertia (I_{min}) about an axis through the element mid surface). The required minimum moment of inertia of the stiffener about the axis *0-0* in Fig. 8 is given by:

$$I_{min} = 0.2\,t^4 \cdot \left(\frac{w}{t}\right)^2 \cdot \left(\frac{f_y}{280}\right) \qquad (5)$$

where w = larger flat width of the sub element (see Fig. 8) between stiffeners (in *mm*)
t = thickness of the element (*mm*)
f_y = yield stress (N/mm^2)

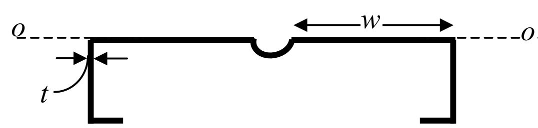


Fig. 8 Intermediate stiffener

If the sub-element width/thickness ratio (*w/t)* does not exceed *60,* the total effective area of the element may be obtained by adding effective areas of the sub-elements to the full areas of stiffeners.

When (*w/t*) is larger than *60,* the effectiveness of the intermediately stiffened elements is somewhat reduced due to shear lag effects. (Refer to BS5950, Part 5, clauses 4.7.2 and 4.7.3) If an element has a number of stiffeners spaced closely ($b/t \leq 30$), *and* then generally all the stiffeners and sub elements can be considered to be effective. To avoid introducing complexities at this stage, shear lag effects are not discussed here.

3.6 Effective Section Properties

In the analysis of member behaviour, the effective section properties are determined by using the effective widths of individual elements. As an example, let us consider the compression member *ABCDEF* shown in Fig.9. The effective portions of the member are shown darkened (i.e. *1-B, B-2, 3-C, C-4, 5-D, D-6, 7-E*, and *E-8*). The parts *A-1, 2-3, 4-5, 6-7* and *8-F* are regarded as being ineffective in resisting compression. As a general rule, the portions located close to the supported edges are effective (see Fig. 5c). Note that in the case of compression members, all elements are subject to reductions in width.

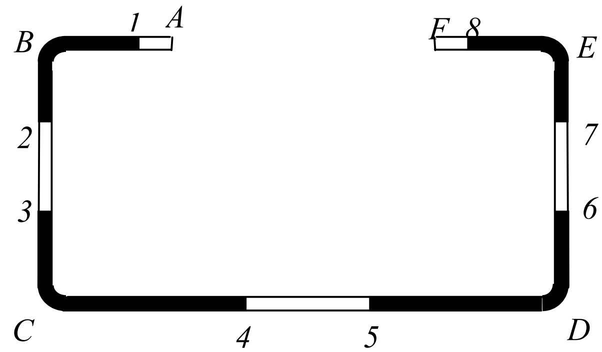


Fig. 9 Effective widths of compression elements

In the case of flexural members, in most cases, only the compression elements are considered to have effective widths. Some typical effective sections of beams are illustrated in Fig.10.

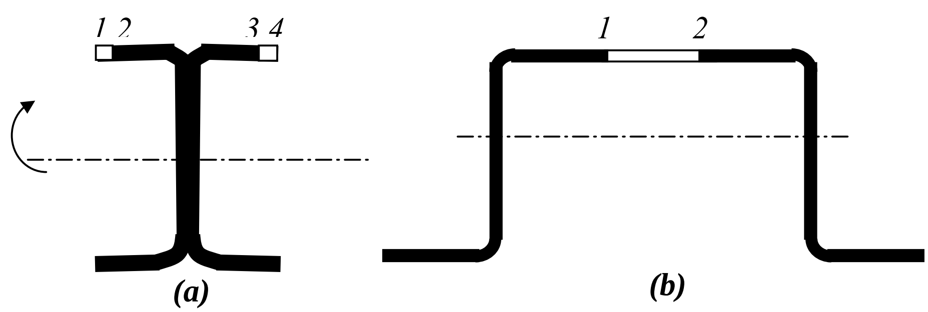


Fig. 10 Effective flexural sections

As in the previous example, fully effective sections in compression elements are darkened in Fig.10. The portions *1-2* and *3-4* in Fig. 10(a) and the portion *1-2* in Fig. 10 (b) are regarded as ineffective in resisting compression. Elements in tension are, of course, not subject to any reduction of width, as the full width will resist tension.

3.7 Proportioning of Stiffeners

The performance of unstiffened elements could be substantially improved by introducing stiffeners (such as a lip). Similarly very wide elements can be divided into two or more narrower sub elements by introducing intermediate stiffeners formed during the rolling process; the sum of the "effective widths" of individual sub elements will enhance the efficiency of the section.
According to BS 5950, Part 5 an unstiffened element (when provided with a lip) can be regarded as a stiffened element, when the lip or the edge stiffener has a moment of inertia about an axis through the plate middle surface equal to or greater than

$$I_{\min} = \frac{b^3\, t}{375} \tag{6}$$

where t and b are the thickness and breadth of the full width of the element to be stiffened.

For elements having a full width b less than or equal to $60\,t$, a simple lip of one fifth of the element width (i.e. $b/5$) can be used safely. For lips with $b > 60\,t$, it would be appropriate to design a lip to ensure that the lip itself does not develop instability.

A maximum b/t ratio of 90 is regarded as the upper limit for load bearing edge stiffeners.

The Indian standard IS: 801-1975 prescribes a minimum moment of inertia for the lip given by

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

where I_{min} = minimum allowable moment of inertia of stiffener about its own centroidal axis parallel to the stiffened element in cm^4

w/t = flat width - thickness ratio of the stiffened element.

F_y = Yield stress in kgf/cm^2

For a simple lip bent at right angles to the stiffened element, the required overall depth d_{min} is given by

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

Note that both the above equations given by the Indian standard are dependent on the units employed.

3.7.1 Intermediate Stiffeners.

Intermediate stiffeners are used to split a wide element into a series of narrower and therefore more effective elements. The minimum moment of inertia about an axis through the element middle surface required for this purpose (according to BS 5950, Part 5) is given in equation (5) above.

The effective widths of each sub element may be determined according to equation 2 (a) and eqn..2 (b) by replacing the sub element width in place of the element width *b*.

When $w / t < 60$, then the total effective area of the element is obtained as the sum of the effective areas of each sub element to the full areas of stiffeners.

When the sub elements having a larger w / t values are employed ($w/ t > 60$), the performance of intermittently stiffened elements will be less efficient. To model this reduced performance , the sub element effective width must be reduced to b_{er} given by,

$$\frac{b_{er}}{t} = \frac{b_{eff}}{t} - 0.1\left(\frac{w}{t} - 60\right) \tag{7}$$

The effective stiffener areas are also reduced when $w / t > 90$ by employing the equation:

$$A_{eff} = A_{st} \cdot \frac{b_{er}}{w} \tag{8}$$

where A_{st} = the full stiffener area and
A_{eff} = effective stiffener area.

For w / t values between 60 and 90, the effective stiffener area varies between A_{st} and A_{eff} as given below:

$$A_{eff} = A_{st}\left[3 - 2\frac{b_{er}}{w} - \frac{1}{30}\left(1 - \frac{b_{er}}{w}\right)\frac{w}{t}\right] \tag{9}$$

It must be noted that when small increases in the areas of intermediate stiffeners are provided , it is possible to obtain large increases in effectiveness and therefore it is advantageous to use a few intermediate stiffeners , so long as the complete element width does not exceed $500\,t$.

When stiffeners are closely spaced , i.e. $w < 30\,t$, the stiffeners and sub elements may be considered to be fully effective. However there is a tendency for the complete element (along with the stiffeners) to buckle locally. In these circumstances, the complete element is replaced for purposes of analysis by an element of width *b* and having a fictitious thickness t_s given by

$$t_s = \left(\frac{12\ I_s}{b}\right)^{1/3} \tag{10}$$

where I_s = Moment of inertia of the complete element including stiffeneres, about its own neutral axis.

IS: 801- 1975 also suggests some simple rules for the design of intermediate stiffeners.

When the flanges of a flexural member is unusually wide, the width of flange projecting beyond the web is limited to

$$w_f = \sqrt{\frac{126500\, t\, d}{f_{av}}} \times \sqrt[4]{\frac{100\, c_f}{d}} \qquad 10(a)$$

where t = flange thickness
d = depth of beam
c_f = the amount of curling
f_{av} = average stress in kgf/cm^2 as specified in IS: 801 – 1975.

The amount of curling should be decided by the designer but will not generally exceed 5 % of the depth of the section.

Equivalent thickness of intermediate stiffener is given by

$$t_s = \sqrt[3]{\frac{12\, I_s}{w_s}} \qquad 10(b)$$

4.0 BEAMS

As stated previously, the effect of local buckling should invariably be taken into account in thin walled members, using methods described already. Laterally stable beams are beams, which do not buckle laterally. Designs may be carried out using simple beam theory, making suitable modifications to take account of local buckling of the webs. This is done by imposing a maximum compressive stress, which may be considered to act on the bending element. The maximum value of the stress is given by

$$p_0 = \left[1.13 - 0.0019\frac{D}{t}\sqrt{\frac{f_y}{280}}\right] p_y \ngeq f_y \qquad (11)$$

where p_o = the limiting value of compressive stress in N/mm^2
D/t = web depth/thickness ratio
f_y = material yield stress in N/mm^2.
p_y = design strength in N/mm^2

For steel with $f_y = 280$ N/mm^2, $p_0 = f_y$ when $(D/t) \le 68$.

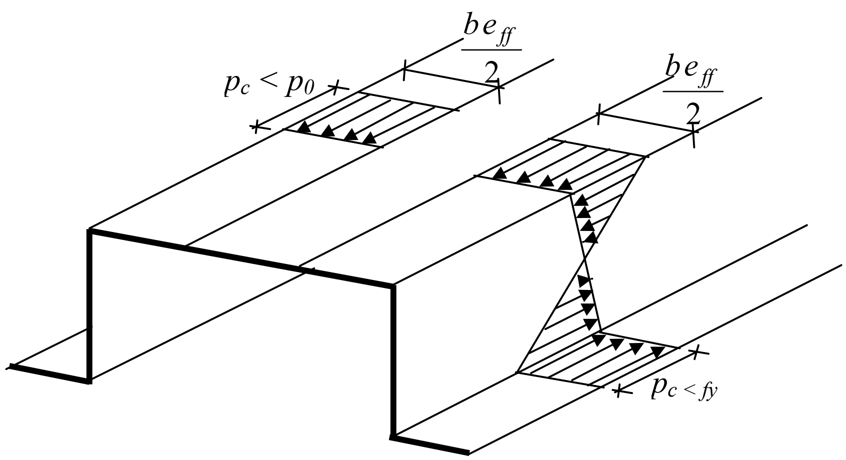
For greater web slenderness values, local web buckling has a detrimental effect. The moment capacity of the cross section is determined by limiting the maximum stress on the web to p_0. The effective width of the compression element is evaluated using this stress and the effective section properties are evaluated. The ultimate moment capacity (M_{ult}) is given by

$$M_{ult} = Z_c . p_0 \qquad (11.a)$$

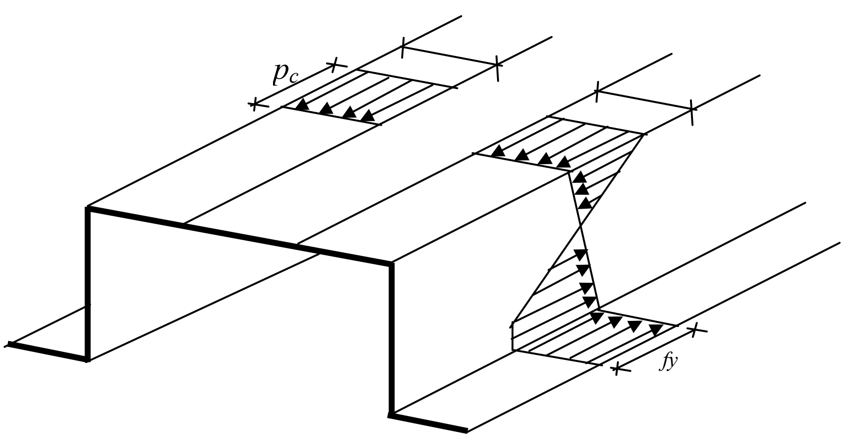
where Z_c = effective compression section modulus

This is subject to the condition that the maximum tensile stress in the section does not exceed f_y (see Fig. 11a).

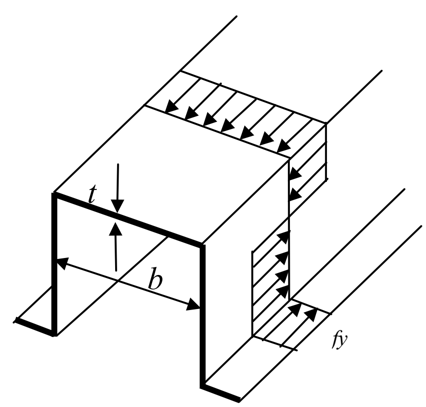
If the neutral axis is such that the tensile stresses reach yield first, then the moment capacity is to be evaluated on the basis of elasto-plastic stress distribution (see Fig.11b). In elements having low (width/thickness) ratios, compressive stress at collapse can equal yield stress (see Fig. 11c). In order to ensure yielding before local buckling, the maximum (width/thickness) ratio of stiffened elements is $\leq 25\sqrt{\frac{280}{f_y}}$ and for unstiffened elements, it is $\leq 8\sqrt{\frac{280}{f_y}}$



(a) Failure by compression
(Tensile stresses elastic)



(b) Tensile stresses reach yield before failure-
(Elasto plastic stress distribution)



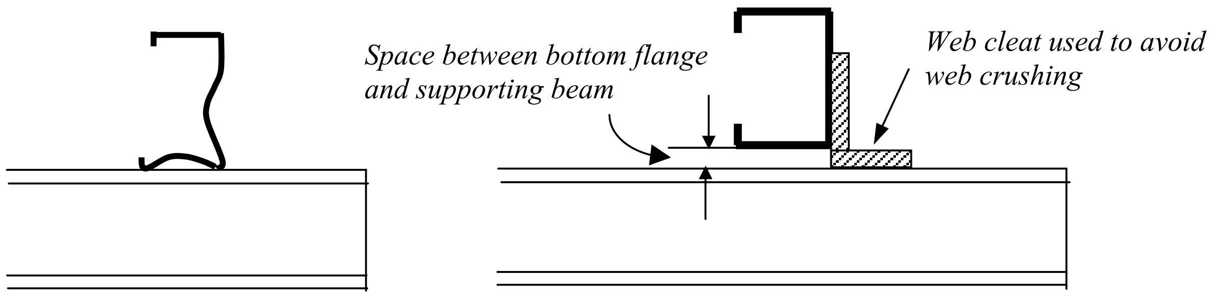
(c) Fully plastic stress distribution
(Thick elements)

Fig. 11 Laterally Stable Beams: Possible stress patterns

4.1 Other Beam Failure Criteria

4.1.1 Web Crushing

This may occur under concentrated loads or at support point when deep slender webs are employed. A widely used method of overcoming web crushing problems is to use web cleats at support points (See Fig.12).



(a) Web crushing *(b) Cleats to avoid web crushing*

Fig.12 Web crushing and how to avoid it

4.1.2 Shear Buckling

The phenomenon of shear buckling of thin webs has been discussed in detail in the chapter on "Plate Girders". Thin webs subjected to predominant shear will buckle as shown in Fig. 13. The maximum shear in a beam web is invariably limited to *0.7* times yield stress in shear. In addition in deep webs, where shear buckling can occur, the average shear stress (p_v) must be less than the value calculated as follows:

$$p_v \leq \left(\frac{1000\ t}{D} \right)^2 \tag{12}$$

where p_v = average shear stress in N/mm^2.
t and D are the web thickness and depth respectively (in *mm*)

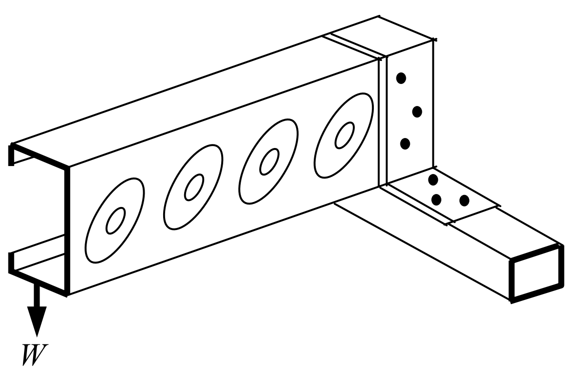


Fig. 13 Web buckling

4.2 Lateral Buckling

The great majority of cold formed beams are (by design) restrained against lateral deflections. This is achieved by connecting them to adjacent elements, roof sheeting or to bracing members. However, there are circumstances where this is not the case and the possibility of lateral buckling has to be considered.

Lateral buckling will not occur if the beam under loading bends only about the minor axis. If the beam is provided with lateral restraints, capable of resisting a lateral force of *3%* of the maximum force in the compression flange, the beam may be regarded as restrained and no lateral buckling will occur.

As described in the chapter on "Unrestrained Beams'", lateral buckling occurs only in "long" beams and is characterised by the beam moving laterally and twisting when a transverse load is applied. This type of buckling is of importance for long beams with low lateral stiffness and low torsional stiffness (See Fig. 14); such beams under loading will bend about the major axis.

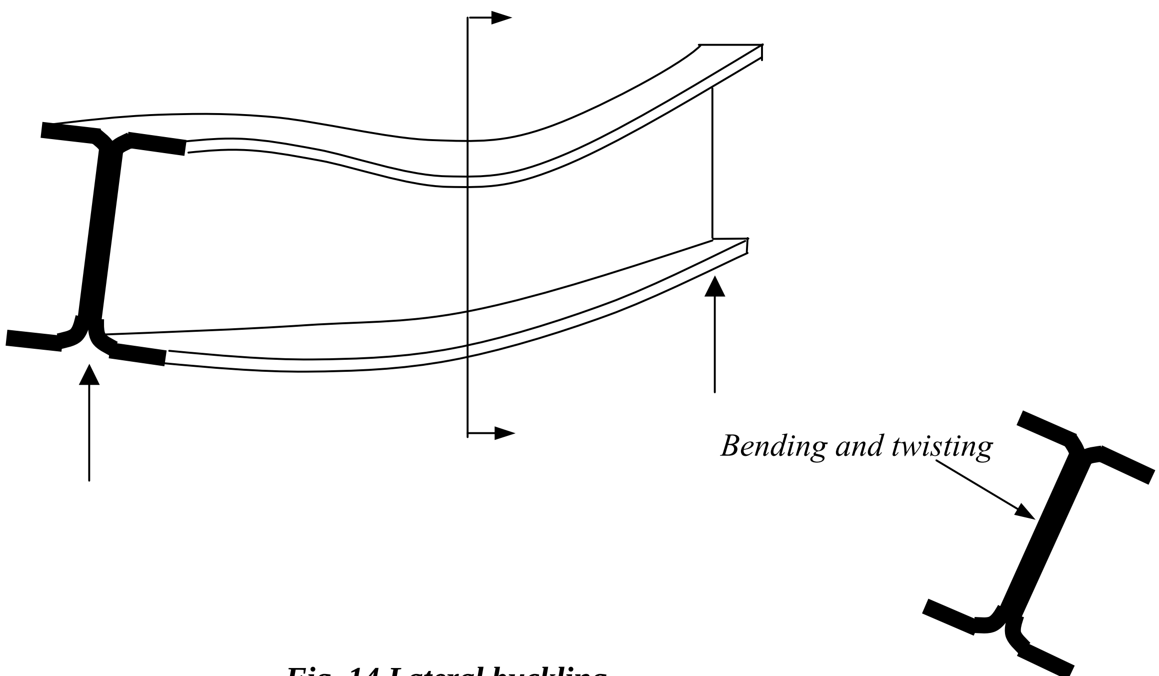


Fig. 14 Lateral buckling

The design approach is based on the "effective length" of the beam for lateral buckling, which is dependent on support and loading conditions. The effective length of beams with both ends supported and having restraints against twisting is taken as *0.9* times the length, provided the load is applied at bottom flange level. If a load is applied to the top flange which is unrestrained laterally, the effective length is increased by *20%.* This is considered to be a "destabilising load", i.e. a load that encourages lateral instability.

The elastic lateral buckling moment capacity is determined next. For an *I* section or symmetrical channel section bent in the plane of the web and loaded through shear centre, this is

$$M_E = \frac{\pi^2 A \cdot E \cdot D}{2\left(\lambda_e / r_y\right)^2} \cdot C_b \sqrt{1 + \frac{1}{20}\left(\frac{\lambda_e}{r_y} \cdot \frac{t}{D}\right)^2} \tag{13}$$

where,

A = cross sectional area, in mm^2
D = web depth, in mm
t = web thickness, in mm
r_y = radius of gyration for the lateral bending of section
$C_b = 1.75 - 1.05\,\beta + 0.3\,\beta^2 \leq 2.3.$

where β = ratio of the smaller end moment to the larger end moment M in an unbraced length of beam. β is taken positive for single curvature bending and negative for double curvature (see Fig. 15)

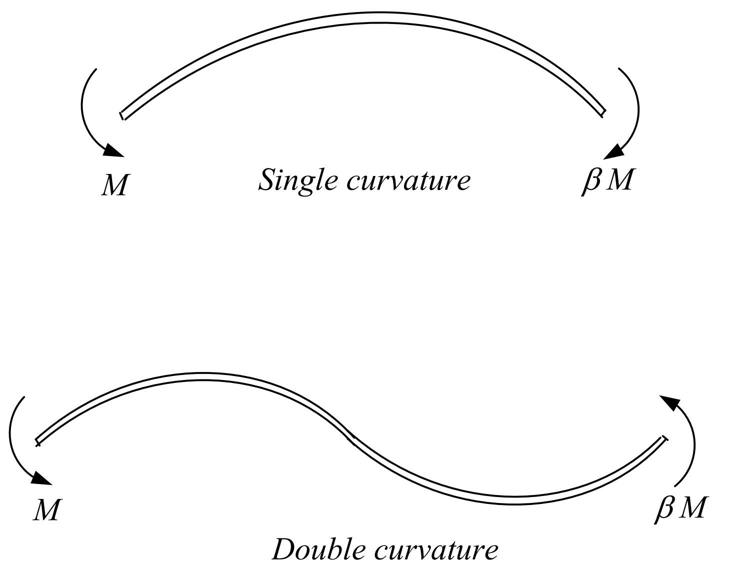


Fig. 15 Single and double curvature bending

To provide for the effects of imperfections, the bending capacity in the plane of loading and other effects, the value of M_E obtained from eq. (13) will need to be modified. The basic concept used is explained in the chapter on Column Buckling where the failure load of a column is obtained by employing the Perry-Robertson equation for evaluating the collapse load of a column from a knowledge of the yield load and Euler buckling load.

A similar Perry-Robertson type equation is employed for evaluating the Moment Resistance of the beam

$$M_b = \frac{1}{2}\left[\left\{M_y + (1+\eta)\,M_E\right\} - \sqrt{\left[M_y + (1+\eta)M_E\right]^2 - 4\,M_y \cdot M_E}\;\right] \tag{14}$$

M_y = First yield moment given by the product of yield stress (f_y) and the Elastic Modulus (Z_c) of the gross section.

M_E = Elastic lateral buckling resistance moment given by equation (13)

η = Perry coefficient, given by

$$When\ \frac{\lambda_e}{r_y} < 40\,C_b,\ \ \eta = 0.$$

$$When\ \frac{\lambda_e}{r_y} > 40\,C_b,\ \ \eta = 0.002\left(\frac{\lambda_e}{r_y} - 40\,C_b\right)$$

λ_e = effective length
r_y = radius of gyration of the section about the y - axis.

When the calculated value of M_b exceed M_{ult} calculated by using equation (11.a), then M_b is limited to M_{ult}. This will happen when the beams are "short".

5.0 CONCLUDING REMARKS

In this chapter the difference between cold rolled steel and hot rolled steel has been discussed and the merits of the former are outlined. The concepts of "effective width" and "effective section" employed in the analysis and design of cold rolled section have been explained. The difference between "stiffened" and "unstiffened" elements has been explained. Considerations in the design of cold rolled beams have been explained and formulae employed for the limit state design of beams made of cold rolled sections have been provided.

6.0 REFERENCES

1. BS5950, Part 5: Structural Use of Steelwork in Building, British Standards Institution, London 1987.
2. J. Rhodes and R.M. Lawson "Design of Structures using Cold Formed Steel Sections, SCI Publication 089, The Steel Construction Institute, U.K. 1992.

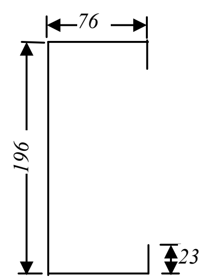
Structural Steel Design Project	Job No.	Sheet ***1*** of ***5***	Rev.
	Job title: ***Section Properties Calculation***		
	Worked Example. ***1***		
CALCULATION SHEET		Made by ***RSP***	Date ***April 2000***
		Checked by ***RN***	Date ***April 2000***

ANALYSIS OF EFFECTIVE SECTION UNDER COMPRESSION

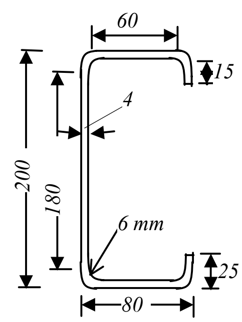
To illustrate the evaluation of reduced section properties of a section under axial compression.

Section : $200 \times 80 \times 25 \times 4.0$ *mm*

Using mid-line dimensions for simplicity. Internal radius of the corners is 1.5t.



Mid-line dimension



Exact dimension

Effective breadth of web (flat element)

$$h = B_2 / B_1 = 60 / 180 = 0.33$$

$$K_1 = 7 - \frac{1.8\ h}{0.15 + h} - 1.43\ h^3$$

App. B Fig . 13 BS 5950: Part 5

$$= 7 - \frac{1.8 \times 0.33}{0.15 + 0.33} - 1.43 \times 0.33^3 = 5.71 \text{ or } 4 \text{ (minimum)}$$

$$= 5.71$$

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$$p_{cr} = 185000\, K_1\, (t / b)^2$$

$$= 185000 \times 5.71 \times (4 / 180)^2 = 521.7\ N/mm^2$$

$$\frac{f_{cr}}{p_{cr} \times \gamma_m} = \frac{240}{521.7 \times 1.15} = 0.4 > 0.123$$

$$\frac{b_{eff}}{b} = \left[1 + 14 \left\{ \sqrt{f_{cr} / (p_{cr} \times \gamma_m)} - 0.35 \right\}^4 \right]^{-0.2}$$

$$= \left[1 + 14 \left\{ \sqrt{0.4} - 0.35 \right\}^4 \right]^{-0.2} = 0.983$$

or $b_{eff} = 0.983 \times 180 = 176.94\ mm$

<u>Effective width of flanges (flat element)</u>

$$K_2 = K_1 h^2 (t_1 / t_2)^2$$

$$= K_1 h^2 \qquad (\because t_1 = t_2)$$

$$= 5.71 \times 0.33^2 = 0.633 \text{ or } 4 \text{ (minimum)}$$

$$= 4$$

$$p_{cr} = 185000 \times 4 \times (4 / 60)^2 = 3289\ N/mm^2$$

$$\frac{f_c}{p_{cr} \times \gamma_m} = \frac{240}{3289 \times 1.15} = 0.063 > 0.123$$

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$$\therefore \quad \frac{b_{eff}}{b} = 1$$

$$b_{eff} = 60 \text{ mm}$$

<u>Effective width of lips (flat element)</u>

K = 0.425 (conservative for unstiffened elements)

$$p_{cr} = 185000 \times 0.425 \times (4/15)^2 = 5591 \text{ N/mm}^2$$

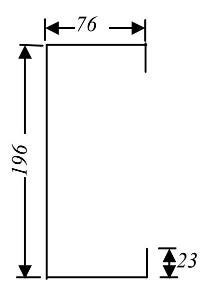
$$\frac{f_c}{p_{cr} \times \gamma_m} = \frac{240}{5591 \times 1.15} = 0.04 < 0.123$$

$$\therefore \quad \frac{b_{eff}}{b} = 1$$

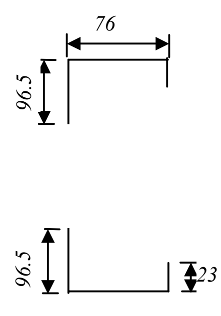
$$b_{eff} = 15 \text{ mm}$$

<u>Effective section in mid-line dimension</u>

As the corners are fully effective, they may be included into the effective width of the flat elements to establish the effective section.



Gross section



Reduced section

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The calculation for the area of gross section is tabulated below:

	A_i (mm^2)
Lips	$2 \times 23 \times 4 = 184$
Flanges	$2 \times 76 \times 4 = 608$
Web	$196 \times 4 = 784$
Total	*1576*

The area of the gross section, A $= 1576\ mm^2$

The calculation of the area of the reduced section is tabulated below:

	A_i (mm^2)
Lips	$2 \times 15 \times 4 = 120$
Corners	$4 \times 45.6 = 182.4$
Flanges	$2 \times 60 \times 4 = 480$
Web	$176.94 \times 4 = 707.8$
Total	*1490.2*

The area of the effective section , $A_{eff} = 1490.2\ mm^2$

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Therefore, the factor defining the effectiveness of the section under compression,

$$Q = \frac{A_{eff}}{A} = \frac{1490}{1576} = 0.95$$

The compressive strength of the member $= Q\,A\,f_y / \gamma_m$

$$= 0.95 \times 1576 \times 240 / 1.15$$

$$= \underline{\ 313\ kN}$$

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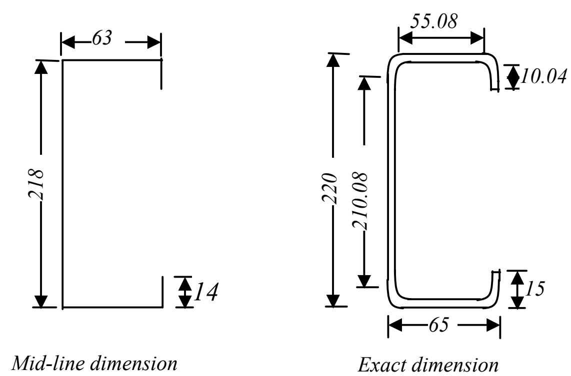
ANALYSIS OF EFFECTIVE SECTION UNDER BENDING

To illustrate the evaluation of the effective section modulus of a section in bending.

We use section : 220 × 65 × 2.0 mm Z28 Generic lipped Channel (from "Building Design using Cold Formed Steel Sections", Worked Examples to BS 5950: Part 5, SCI PUBLICATION P125)

Only the compression flange is subject to local buckling.

Using mid-line dimensions for simplicity. Internal radius of the corners is 1.5t. *Cl.5.2.2.3 BS 5950: Part 5*



Thickness of steel(ignoring galvanizing), t = 2 - 0.04 = 1.96 mm
Intenal radius of the corners = 1.5 × 2 = 3 mm

App. B Fig. 14 BS 5950: Part 5

<u>Limiting stress for stiffened web in bending</u>

$$p_0 = \left\{ 1.13 - 0.0019 \frac{D}{t} \sqrt{\frac{f_y}{280}} \right\} p_y$$

$$\text{and } p_y = 280 / 1.15 = 243.5 \ N/mm^2$$

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$$p_0 = \left\{ 1.13 - 0.0019 \times \frac{220}{1.96}\sqrt{\frac{280}{280}} \right\} \frac{280}{1.15}$$

$$= 223.2\ N/mm^2$$

which is equal to the maximum stress in the compression flange, i.e.,

$$f_c = 223.2\ N/mm^2$$

<u>Effective width of compression flange</u>

$$h = B_2 / B_1 = 210.08 / 55.08 = 3.8$$

$$K_1 = 5.4 - \frac{1.4\,h}{0.6 + h} - 0.02\,h^3$$

$$= 5.4 - \frac{1.4 \times 3.8}{0.6 + 3.8} - 0.02 \times 3.8^3$$

$$= 3.08 \text{ or } 4\ (\text{minimum}) = 4$$

$$p_{cr} = 185000 \times 4 \times \left(\frac{1.96}{55.08}\right)^2 = 937\ N/mm^2$$

$$\frac{f_c}{p_{cr}} = \frac{223.2}{937} = 0.24 > 0.123$$

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

$$= \left[1 + 14 \left\{ \sqrt{0.24} - 0.35 \right\}^4 \right]^{-0.2} = 0.998$$

$$b_{eff} = 0.99 \times 55 = 54.5$$

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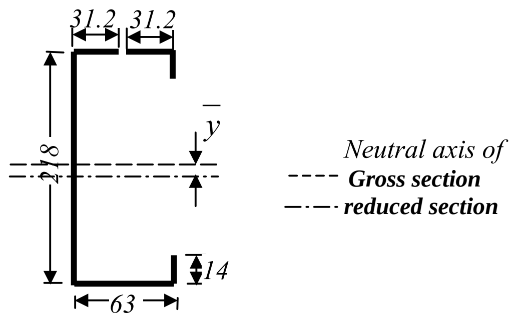
Effective section in mid-line dimension:

The equivalent length of the corners is $2.0 \times 2.0 = 4$ *mm*

The effective width of the compression flange $= 54.5 + 2 \times 4 = 62.5$



The calculation of the effective section modulus is tabulated as below:

Elements	A_i (mm^2)	y_i (mm)	$A_i y_i$ (mm^3)	$I_g + A_i y_i^2$ (mm^4)
Top lip	27.44	102	2799	448 + 285498
Compression flange	122.5	109	13352.5	39.2 + 1455422.5
Web	427.3	0	0	1692171.2 + 0
Tension flange	123.5	-109	-13459.3	39.5 + 1467064
Bottom lip	27.4	-102	-2799	448 + 285498
Total	728.2		-106.8	5186628.4

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The vertical shift of the neutral axis is

$$\bar{y} = \frac{-106.8}{728.2} = -0.15\,mm$$

The second moment of area of the effective section is

$$I_{xr} = (5186628.4 + 728.2 \times 0.15^2) \times 10^{-4}$$

$$= 518.7\,cm^4 \quad at\ p_0 = 223.2\,N/mm^2$$

$$or \quad = 518.7 \times \frac{223.2 \times 1.15}{280} = 475.5\,cm^4 \quad at\ p_y = 280/1.15\,N/mm^2$$

The effective section modulus is,

$$Z_{xr} = \frac{475.5}{(109 + 0.15)/10} = 43.56\,cm^3$$

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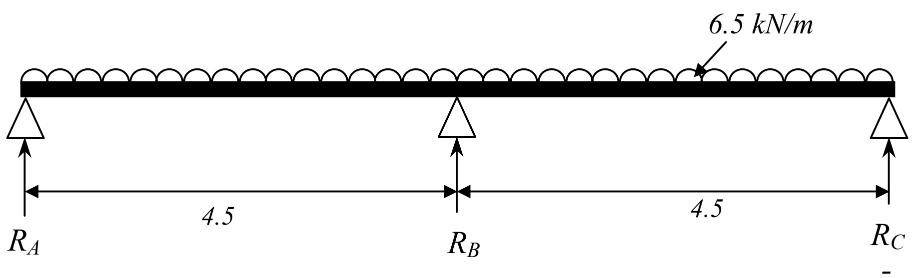
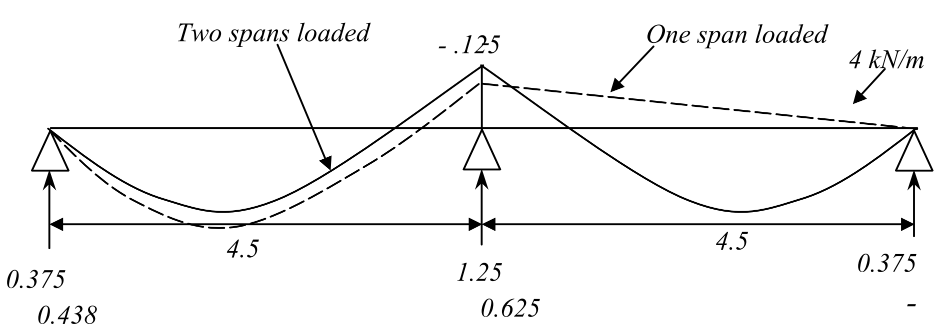
Design a two span continuous beam of span 4.5 m subject to a UDL of 4kN/m as shown in Fig.1.



Factored load on each span $= 6.5 \times 4.5 = 29.3$ *kN*

<u>Bending Moment</u>



Coefficients for reactions

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Maximum hogging moment $= 0.125 \times 29.3 \times 4.5 = 16.5$ *kNm*

Maximum sagging moment $= 0.096 \times 29.3 \times 4.5 = 12.7$ *kNm*

Shear Force

Two spans loaded : $R_A = 0.375 \times 29.3 = 11$ *kN*

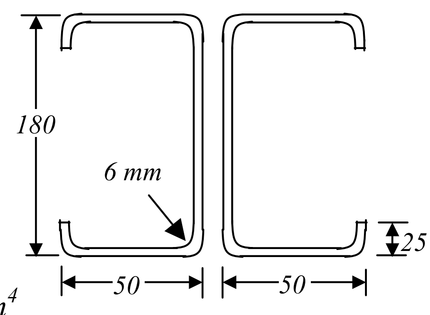
$R_B = 1.25 \times 29.3 = 36.6$ *kN*

One span loaded : $R_A = 0.438 \times 29.3 = 12.8$ *kN*

$\therefore$ *Maximum reaction at end support,* $F_{w,max} = 12.8$ *kN*

Maximum shear force , $F_{v,max} = 29.3 - 11 = 18.3$ *kN*

Try $180 \times 50 \times 25 \times 4$ *mm Double section (placed back to back)*



Material Properties : $E = 205$ kN/mm^2

$p_y = 240 / 1.15$

$= 208.7$ N/mm^2

Section Properties : $t = 4.0$ *mm*

$D = 180$ *mm*

$r_{yy} = 17.8$ *mm*

$I_{xx} = 2 \times 518 \times 10^4$ mm^4

$Z_{xx} = 115.1 \times 10^3$ mm^3

Only the compression flange is subject to local buckling

Limiting stress for stiffened web in bending

$$p_0 = \left\{ 1.13 - 0.0019 \frac{D}{t} \sqrt{\frac{f_y}{280}} \right\} p_y$$

and $p_y = 240 / 1.15 = 208.7$ N/mm^2

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$$p_0 = \left\{ 1.13 - 0.0019 \times \frac{180}{4} \sqrt{\frac{240}{280}} \right\} \times 208.7$$

$$= 219.3\ N/mm^2$$

which is equal to the maximum stress in the compression flange, i.e.,

$$f_c = 219.3\ N/mm^2$$

Effective width of compression flange

$$h = B_2 / B_1 = 160/30 = 5.3$$

$$K_1 = 5.4 - \frac{1.4\,h}{0.6 + h} - 0.02\,h^3$$

$$= 5.4 - \frac{1.4 \times 5.3}{0.6 + 5.3} - 0.02 \times 5.3^3$$

$$= 1.1 \text{ or } 4 \text{ (minimum)} = 4$$

$$p_{cr} = 185000 \times 4 \times \left(\frac{4}{30}\right)^2 = 13155\ N/mm^2$$

$$\frac{f_c}{p_{cr}} = \frac{219.3}{13155} = 0.017 < 0.123$$

$$\frac{b_{eff}}{b} = 1$$

$$b_{eff} = 30\ mm$$

i.e. the full section is effective in bending.

$$I_{xr} = 2 \times 518 \times 10^4\ mm^4$$

$$Z_{xr} = 115.1 \times 10^3\ mm^3$$

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Moment Resistance

The compression flange is fully restrained over the sagging moment region but it is unrestrained over the hogging moment region, that is, over the internal support. However unrestrained length is very short and lateral torsional buckling is not critical.

The moment resistance of the restrained beam is:

$$M_{cx} = Z_{xr}\, p_y$$

$$= 115.1 \times 10^3 \times (240 / 1.15)\, 10^{-6} = 24 \text{ kNm} > 16.5 \text{ kNm}$$

$\therefore$ *O.K*

Shear Resistance

Shear yield strength,

$$p_v = 0.6\, p_y = 0.6 \times 240 / 1.15 = 125.2 \text{ N/mm}^2$$

BS 5950: Part 5 Cl. 5.4.3

Shear buckling strength, $q_{cr} = \left(\dfrac{1000\, t}{D}\right)^2 = \left(\dfrac{1000 \times 4}{180}\right)^2$

$$= 493.8 \text{ N/mm}^2$$

Maximum shear force, $F_{v,max} = 18.3$ *kN*

Shear area $= 180 \times 4 = 720 \text{ mm}^2$

Average shear stress $f_v = \dfrac{18.3 \times 10^3}{720} = 25.4 \text{ N/mm}^2 < q_{cr}$

$\therefore$ *O.K*

Web crushing at end supports

Check the limits of the formulae.

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$$\frac{D}{t} = \frac{180}{4} = 45 \le 200 \qquad \therefore \ O.K$$

$$\frac{r}{t} = \frac{6}{4} = 1.5 \le 6 \qquad \therefore \ O.K$$

At the end supports, the bearing length, N is 50 mm (taking conservatively as the flange width of a single section)

For c=0, $N/t = 50/4 = 12.5$ *and restrained section .*
C is the distance from the end of the beam to the load or reaction.

Use (Table 8 BS 5950: Part 5)

$$P_w = 2 \times t^2 C_7 \frac{f_y}{\gamma_m} \left\{ 8.8 + 1.11 \sqrt{N/t} \right\}$$

$$C_7 = 1 + \frac{D/t}{750}$$

$$= 1 + \frac{45}{750} = 1.06$$

$$P_w = 2 \times 4^2 \times 1.06 \times \frac{240}{1.15} \left\{ 8.8 + 1.11 \sqrt{12.5} \right\} 10^{-3}$$

$$= 89.8 \ kN > R_A \ (= 12.8 \ kN) \qquad \therefore \ O.K$$

Web Crushing at internal support

At the internal support, the bearing length, N, is 100mm (taken as the flange width of a double section)

For $c > 1.5D$, $N/t = 100/4 = 25$ *and restrained section.* (Table 8 BS 5950: Part 5)

$$P_w = t^2 C_5 C_6 \frac{f_y}{\gamma_m} \left\{ 13.2 + 1.63 \sqrt{N/t} \right\}$$

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$$k = \frac{f_y}{228 \times \gamma_m} = \frac{240}{1.15 \times 228} = 0.92$$

$$C_5 = (1.49 - 0.53\,k) = 1.49 - 0.53 \times 0.92 = 1.0 > 0.6$$

$$C_6 = (0.88 - 0.12\,m)$$

$$m = t / 1.9 = 4 / 1.9 = 2.1$$

$$C_6 = 0.88 - 0.12 \times 2.1 = 0.63$$

$$\therefore P_w = 2 \times 4^2 \times 1 \times 0.63 \times \frac{240}{1.15}\left\{13.2 + 1.63\sqrt{25}\right\}10^{-3}$$

$$= 89.8\ kN > R_B\ (= 36\ kN)$$

Deflection Check

A coefficient of $\frac{3}{384}$ *is used to take in account of unequal loading on a double span. Total unfactored imposed load is used for deflection calculation.*

$$\delta_{\max} = \frac{3}{384}\,\frac{W L^3}{E I_{av}}$$

$$I_{av} = \frac{I_{xx} + I_{xr}}{2} = \frac{1036 + 1036}{2} = 1036 \times 10^4\ mm^4$$

$$W = 29.3 / 1.5 = 19.5\ kN$$

$$\delta_{\max} = \frac{3}{384}\,\frac{19.5 \times 10^3 \times 4500^3}{205 \times 10^3 \times 1036 \times 10^4} = 6.53 mm$$

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Deflection limit $= L / 360$ *for imposed load*

$= 4500 / 360 = 12.5 \text{ mm} > 6.53 \text{ mm} \quad \therefore O.K$

$\therefore$ *In the double span construction :*

Use double section $180 \times 50 \times 25 \times 4.0$ *mm lipped channel placed back to back.*