DSS-MOD-III-COMPRESSION MEMBER

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COMPRESSION MEMBER

In concrete members we use different terms for compression members like, Column, (Short, Long) and others. Easy for design of Compression members in RCC.

For steel compression members difficult to design due to slenderness ratio.

In steel structure building, compression members is named as stanchion, the compressive member in a roof truss or bracing is called strut. Similarly, the principal compression in a crane is called boom.

Failure of compression members

Type-1. Squashing

Squashing basically happens when the length of the compression member is quite less compared to its transverse direction cross section area. Just like short column, the crushing will come into the picture and full strength will attain at its yield strength and therefore the failure load can be calculated in strength into the cross sectional area.

Type-2 Local Buckling

Another one is the local buckling. This is due to individual or part of structure may buckle. For a channel section, due to compression either web flange may buckle individually or some other part of the member may buckle which is called local buckling.

Type-3. Overall flexural buckling.

When the member length along its axis is quite high compared to its cross sectional dimension then such type of buckling occurs which is called flexural buckling. For a long column means compared to its lateral dimension, so in such cases, it may buckle in this way. So before going to fail due to crushing it may fail due to buckling.

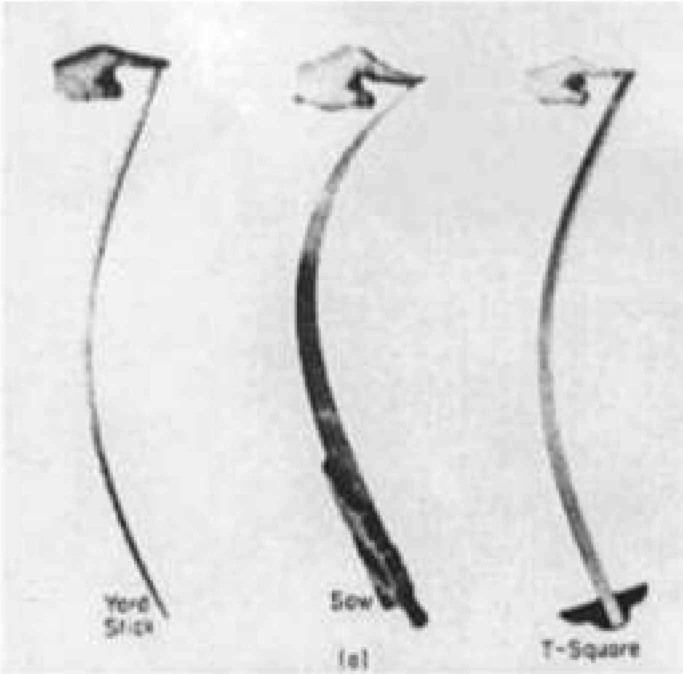
Type-4 Torsional Buckling.

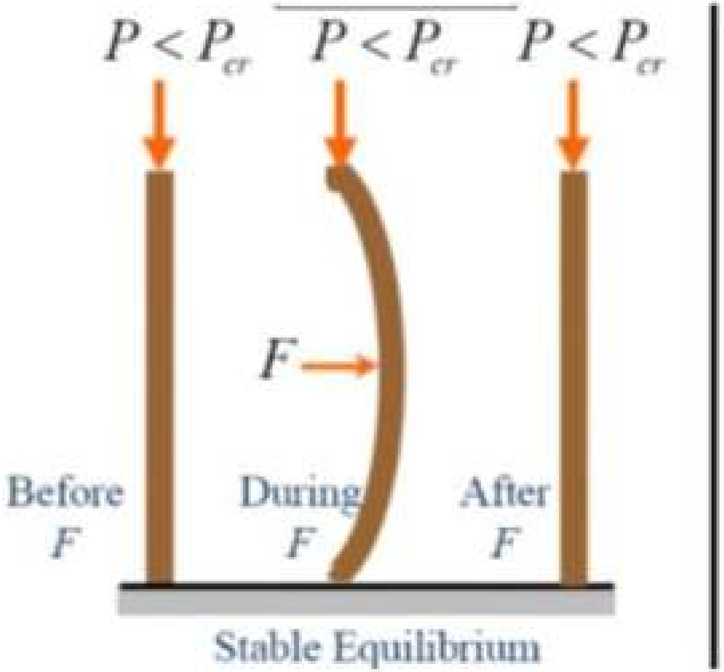
Torsional buckling failure occurs due to torsional moment, the member gets twisted about the shear centre in the longitudinal axis. So torsional buckling may occur may be in case of angle section or channel section depending on the type of load acting in a particular place.

Type-5- flexural-torsional buckling.

The buckling which occurs when the member bends and twists simultaneously. Such type of failure happens generally in case of unsymmetrical cross section. So unsymmetrical cross section may be channel section it is symmetrical in one direction but it is unsymmetrical in another direction.

Example



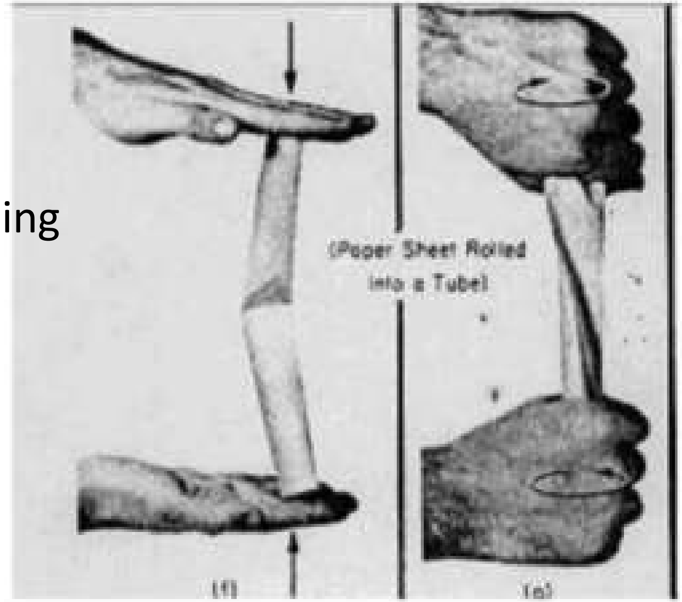


Elastic Buckling

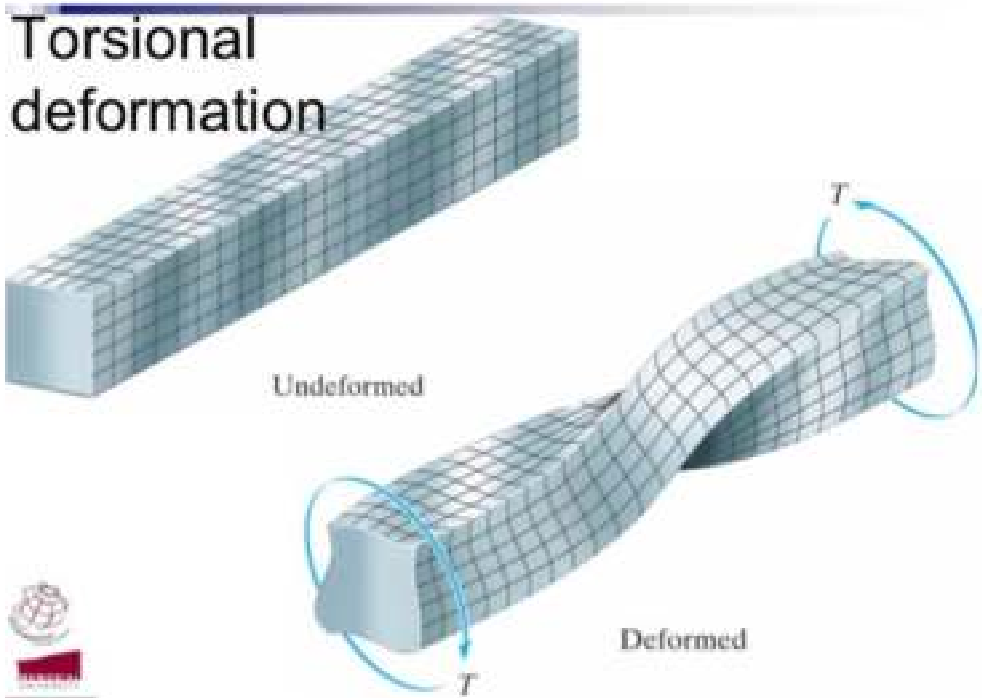
Example

Twisting

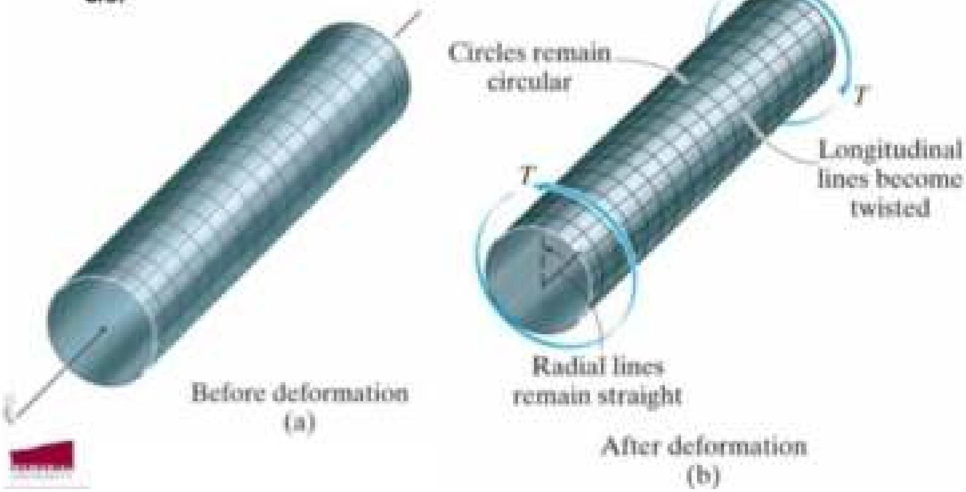
bending



Torsional deformation



Torque is a moment that causes twisting along the length of a bar. The twist is also the torsional deformation. For a circular shaft, the torque (or torsional moment) rotates each c/s relative to the nearby c/s.



Before After

(Localized Buckling)

(h)

The Factor governing failure of Compression Member:

The Length to lateral dimension:

Short compression member: Length is quite short compared to its width and thickness, L by R ratio is quite low in that case the failure stress will be equal to the yield stress and there will be no buckling, buckling will not happen in this case. So failure will be due to yielding of the material .

Long compression member- In this case, stress will occur due to buckling as length to lateral dimension is too high. So in this case buckling may happen before yielding of the stress that is why we need to consider whether it is long compression member or not .

Intermediate compression, member failure occur due to the combined effect of crushing and buckling. Intermediate compression means in practice most of the members are considered as intermediate compression member because in this case, the member will undergo both the stress, one is

Elastic Buckling Theory:

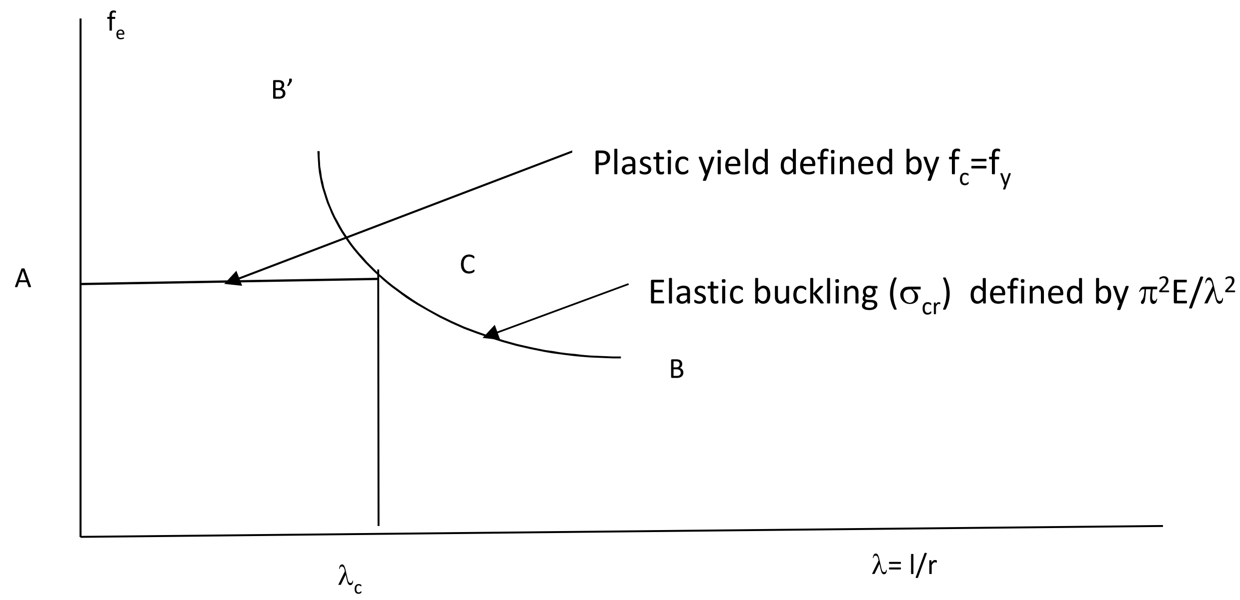
The governing equation is: $$\frac{d^2y}{dx^2} + \frac{P_{cr}}{EI} y = 0$$

The lowest value of critical load $P_{cr} = \dfrac{\pi^2 EI}{l^2}$

Stress:

$$\sigma_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 EI}{Al^2} = \frac{\pi^2 Er^2}{l^2} = \frac{\pi^2 Er^2}{(\frac{l}{r})^2} = \frac{\pi^2 E}{\lambda^2}$$

Strength Curve for an ideal strut



AC= Failure by yielding (Low slenderness ratio)
CB= Failure by buckling $\lambda > \lambda_c$ (High slenderness ratio)
AC= Failure by yielding (Low slenderness ratio)

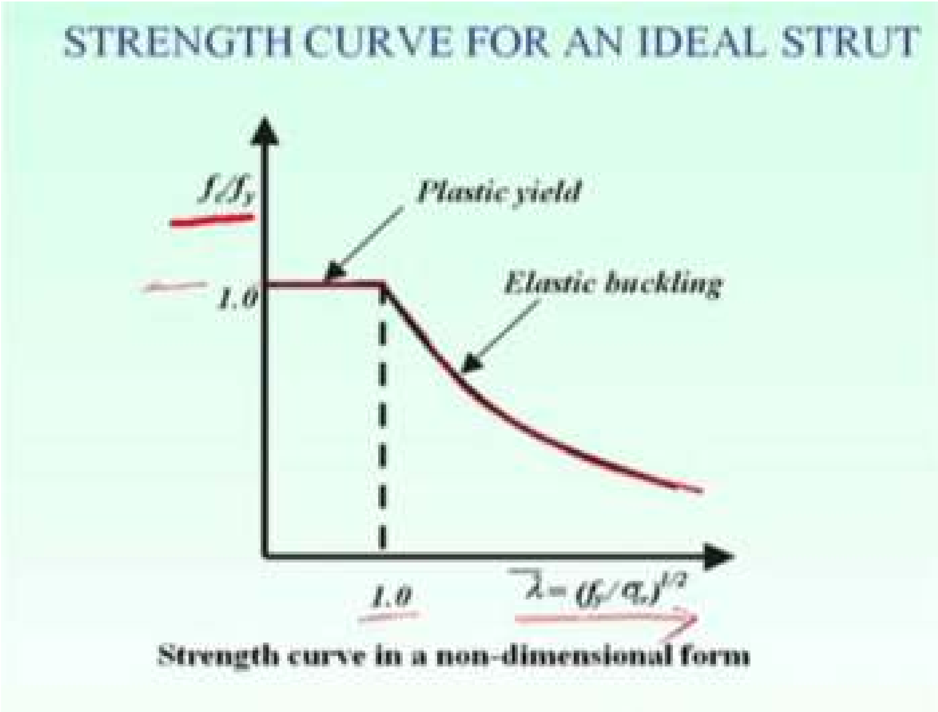
The strength curve of a column can be derived if the strut is axially loaded and initially straight with pin-ended .Taking x axis will be the slenderness ratio that is l/r and y axis will be the compressive strength of the of the material. The path is varying from A to C and then C to B. Column fails when the compressive strength is greater than or equal to the values defined by ACB. if the column stress is going to be greater than the stress defined by this path ACB then, column is going to fail and this AC is basically failure by yielding and for low slenderness ratio then failure may happen due to yielding and failure may happen due to buckling for high slenderness ratio if λ is greater than λc.

plastic yield defined by $f_c=f_y$ *and this is defined by the elastic buckling stress.*

$f_c=\sigma_{cr}=f_y=250\ MPa$

$\lambda_c=\pi * SQRT(E/f_y)$=88.85 for mild steel

From this curve that if the slenderness ratio value becomes more than 88.85 then it will fail by elastic buckling and if it is less than that it will fail by plastic yield.



Euler’s formulae is a theoretical one. Its practical application is affected by certain parameters.

1. The material property of the member.
2. The length of the member .
3. Cross sectional configuration ,in case of steel member we use different type of built up section say for example built up section or rolled section.
4. Support Condition
5. Imperfection
6. Residual Strength

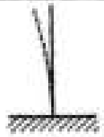
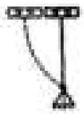
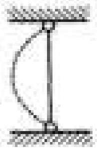
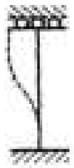
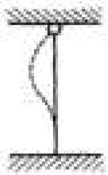
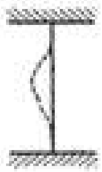
- .

Effective Length Factor for Centrally Loaded Columns with various End (Table 11 of IS 800 2007)

End Conditions	Theoretical K value	IS-800 provisions
Columns with both ends pinned	1.0	1.0
Columns with both ends fixed	0.5	0.65
Columns with one end fixed and other end pinned	0.7	0.8
Columns with one end fixed and other end free	2.0	2.0
Columns partially restrained at each end	1.0	1.2
Columns with one end unrestrained and other end rotation partially restrained	2.0	2.0

Table 11 Effective Length of Prismatic Compression Members

(*Clause* 7.2.2)

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

ANNEX D

(*Clause* 7.2.2)

DETERMINATION OF EFFECTIVE LENGTH OF COLUMNS

D-1 METHOD FOR DETERMINING EFFECTIVE LENGTH OF COLUMNS IN FRAMES

In the absence of a more exact analysis, the effective length of columns in framed structures may be obtained by multiplying the actual length of the column between the centres of laterally supporting members (beams) given in Fig. 27 and Fig. 28 with the effective length factor K, calculated by using the equations given below, provided the connection between beam and column is rigid type:

a) *Non-sway Frames (Braced Frame)* [(*see* **4.1.2**(a)]

A frame is designated as non-sway frame if the relative displacement between the two adjacent floors is restrained by bracings or shear walls (*see* **4.1.2**). The effective length factor, K, of column in non-sway frames is given by (*see* Fig. 27):

$$K = \frac{\left[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2\right]}{\left[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2\right]}$$

b) *Sway Frames (Moment Resisting Frames)* [*see* **4.1.2**(b)]

The effective length factor K, of column in sway frames is given by (*see* Fig. 28):

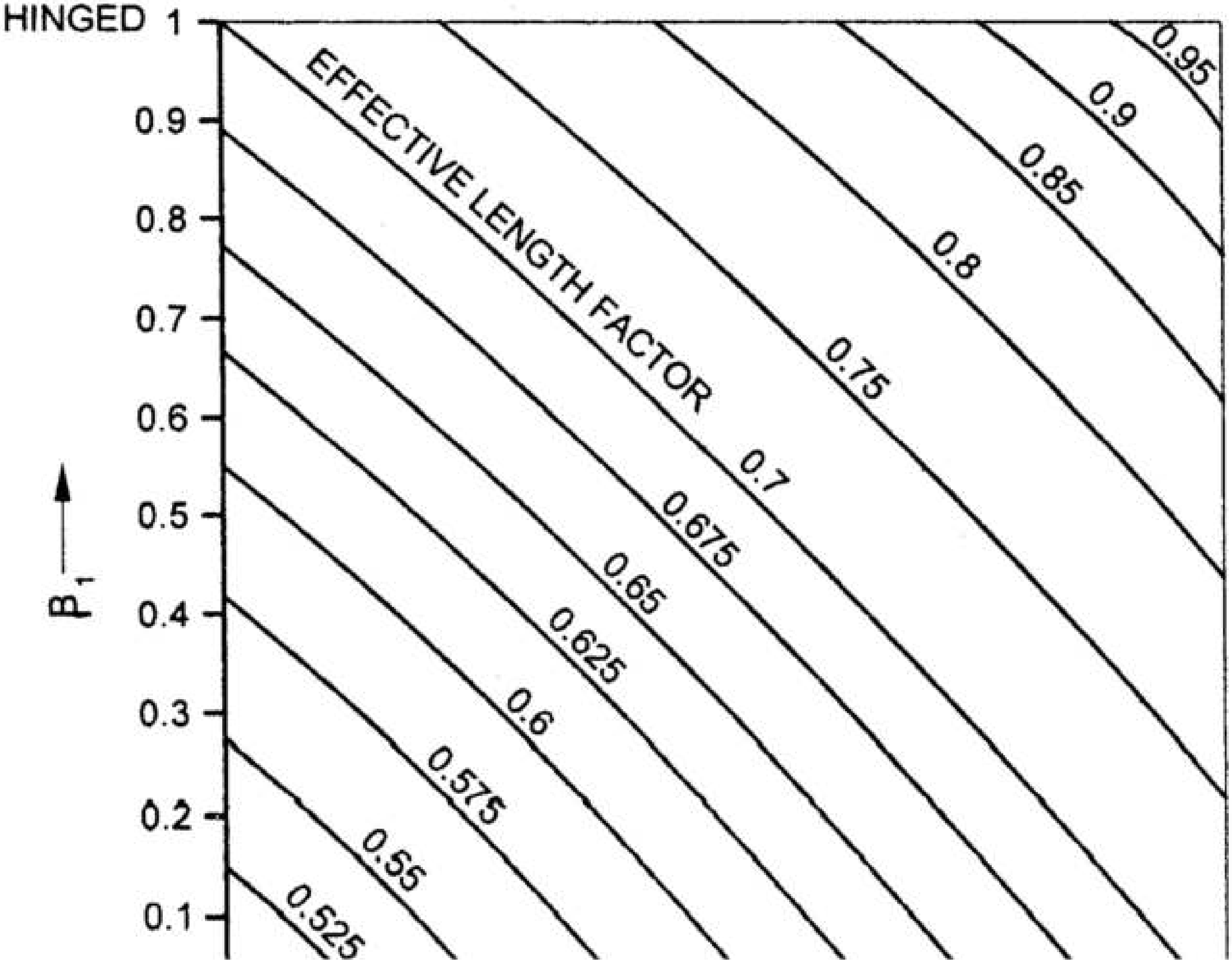
$$K = \left[\frac{1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2}{1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2}\right]^{0.5}$$

where

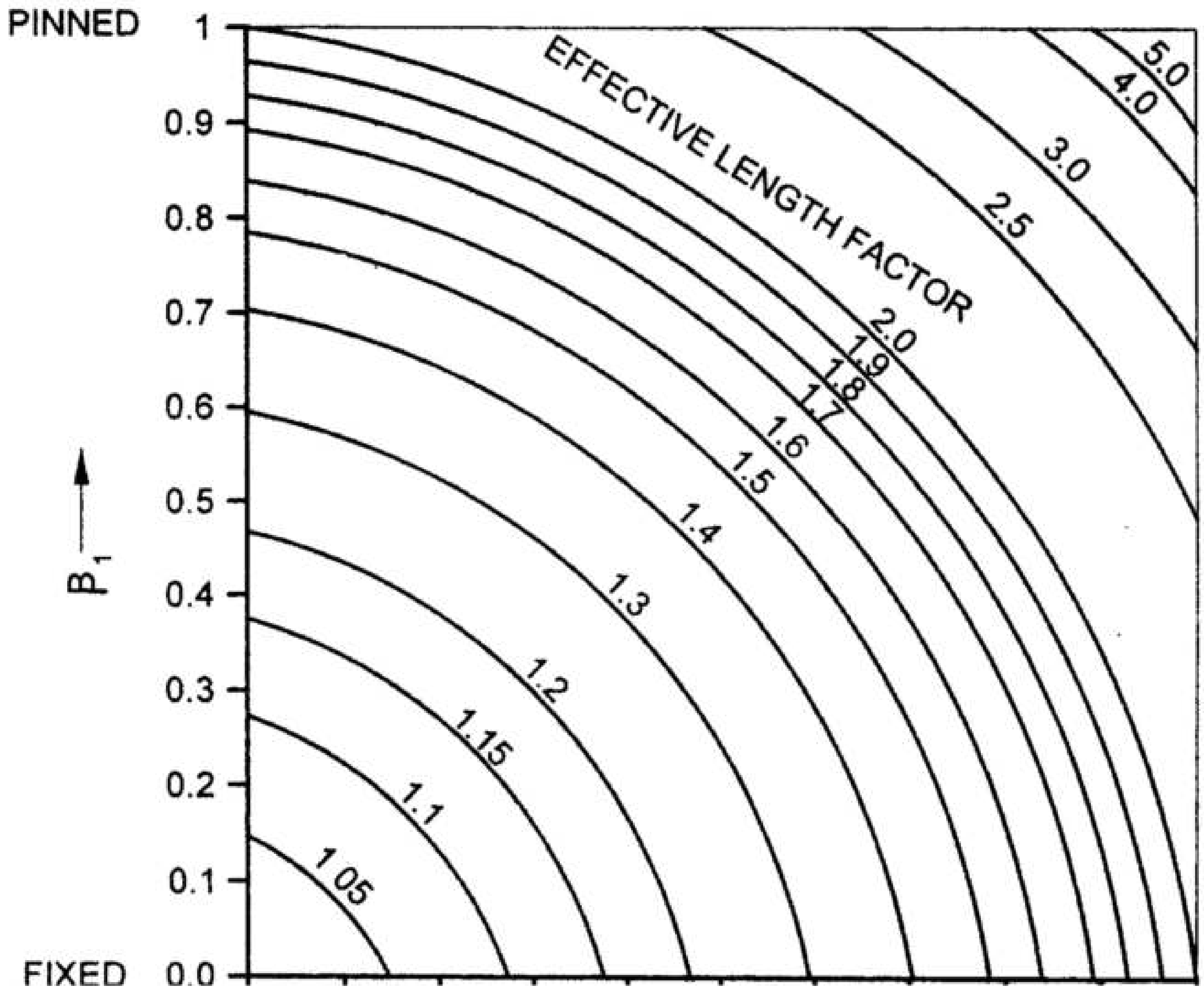
β_1, β_2 are given, $\beta = \dfrac{\sum K}{\sum K_C + \sum K_b}$

K_c, K_b = effective flexural stiffness of the columns and beams meeting at the joint at the ends of the columns and rigidly connected at the joints, and these are calculated by:

$$K = C\,(I / L)$$



HINGED
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
β1
EFFECTIVE LENGTH FACTOR
0.95
0.9
0.85
0.8
0.75
0.7
0.675
0.65
0.625
0.6
0.575
0.55
0.525


PINNED
FIXED
β_1
EFFECTIVE LENGTH FACTOR
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
5.0
4.0
3.0
2.5
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.15
1.1
1.05

Design compressive strength

7.1.2 The design compressive strength P_d, of a member is given by:

$$P < P_d$$

where

$P_d = A_e f_{cd}$

where

A_e = effective sectional area as defined in **7.3.2**, and

f_{cd} = design compressive stress, obtained as per **7.1.2.1**.

Clause 7.3.2

7.3.2 *Effective Sectional Area*, A_e

Except as modified in **3.7.2** (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

CALCULATION OF COMPRESSIVE STRENGTH

The design strength of compression member, depends on different factors .

The main three parameters which are effecting on the on the compressive strength of a member, one is the material strength of the member that means what is the yield strength of the member .

Next factor is the slenderness ratio, the Euler's critical load and Euler's buckling formula that the compressive strength varies inversely with the slenderness ratio.

Another aspects which are of the importance we have to give that is the local buckling because one is crushing, another is overall buckling, and another is local buckling. So because of the configuration of the member cross section of the member the local buckling of the flange or web may happen.

- So four different approaches have been considered for finding the design column formula. So one is the formula based on the maximum strength, this is one approach in which people have tried.
- Another is formula based on the yield limit, which is called Perry-Robertson formula and basically this approach is considered by our Indian Code the IS 800:2007 has also adapted the multiple column curves based on the Perry Robertson formula and this is basically similar to the British code BS 5950 (part-1) 2000.
- This the formula which have been derived is similar to the British code and the formula was prescribed by Perry-Robertson who has proposed. So this has been adapted.
- Another two formula are also adapted to establish column design formula that is formula based on tangent modulus theory and Empirical formula such as Merchant-Rankine formula. So these four basic approach are observed to establish column design formula and we may recall the earlier code that is IS 800:1984, which was established as per Merchant-Rankine formula.

- The curve corresponding to different buckling class are presented in non-dimensional form as shown in the figure below. Using this curve one can find the value of fcd (design compressive stress) corresponding to non- dimensional effective slenderness ratio λ (page 35)

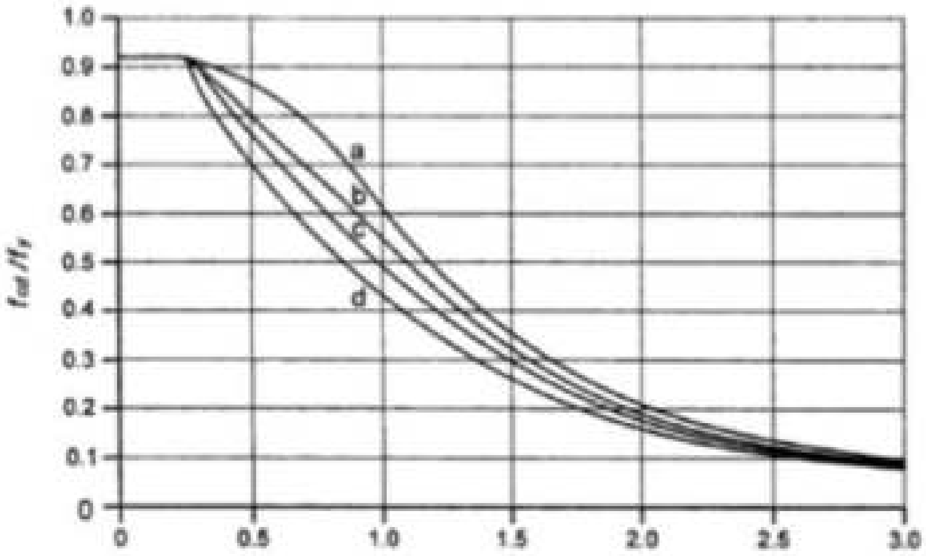


FIG. 8 COLUMN BUCKLING CURVES

λ=sqrt(f_y/f_{cr})

Perry-Robertson formula the multiple design curve has been adapted by the IS code, in IS code figure 8

CLASSIFICATION OF CROSS-SECTIONS

- Plate elements of a cross-section may buckle locally due to compressive stresses. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross-section subjected to compression due to axial force, moment or shear. When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling, to enable the redistribution of bending moment required before formation of the failure mechanism. When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling. On basis of the above, Cl. 3.7 of IS 800:200 7 categorizes the sections in to four classes as follows.
- When different elements of a cross-section fall under different classes, the section shall be classified as governed by the most critical element. The maximum value of limiting width to thickness ratios of elements for different classifications of sections are given in Table 2 of IS 800:2007

CLASSIFICATION OF CROSS-SECTIONS

- **Class 1 (Plastic)**

  Cross-sections which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism fall under this category. The width to thickness ratio of plate elements shall be less than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

- **Class 2 (Compact)**

  Cross-sections which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling come under this class. The width to thickness ratio of plate elements shall be less than that specified under Class 2 (Compact), but greater than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

- **Class 3 (Semi-compact)**

  Cross-sections in which the extreme fiber in compression can reach yield stress but can not develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact), in Table 2 of IS 800:2007.

- **Class 4 (Slender)**

  Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under Class 3 (Semicompact), in Table 2 of IS 800:2007. In such cases, the effective sections for design shall be calculated either by following the provisions of IS 801 to account for the post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

TYPES OF ELEMENTS

- IS 800:2007 classifies elements in to three types, as per Cl. 3.7.3., as follows.
- **Internal elements**

  These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section.
- **Outside elements or outstands**
- These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane, for example flange overhang of an I-section, stem of T section and legs of an angle section.
- **Tapered elements**

  These maybe treated as flat elements having average thickness as defined in SP 6 (Part 1).
- **MAXIMUM EFFECTIVE SLENDERNESS RATIO**

  The maximum effective slenderness ratio, as per Cl. 3.8 of IS 800:2007, KL/r values of a beam, strut or tension member shall not exceed those given in Table 3 of IS 800:2007. 'KL' is the effective length of the member and 'r' is appropriate radius of gyration based on the effective section as defined in Cl. 3.6.1 of IS 800:2007. This data is reproduced here in Table .

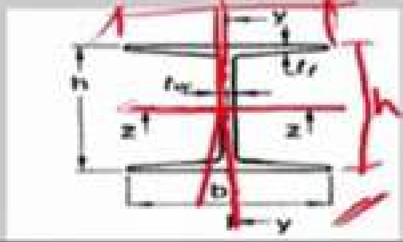
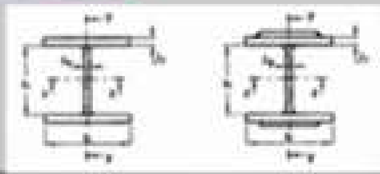
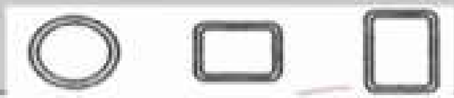
Member	Maximum Effective Slenderness Ratio (KL/r)
A member carrying compressive loads resulting from dead loads and imposed loads	180
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compression forces resulting only from combination with wind/earthquake actions, provided the deformation of such member does not adversely affect tbe stress in any part of the structure	250
Compression flange of a beam against lateral torsional buckling	300
A member normally acting m a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake forces	350
Members always under tension (other than pre-tensioned members)	400

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4)

Compression Element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)			(2)	(3)	(4)	(5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\leq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

Buckling Class of Cross Sections (Table 10 IS 800 :2007)……

Cross Section	Limits	Buckling about Axis	Buckling Class
Rolled I Section 	$h/b_f > 1.2$ $t_f \leq 40$ mm	z-z	a
		y-y	b
	$40 \text{ mm} \leq t_f \leq 100$ mm	z-z	b
		y-y	c
	$h/b_f \leq 1.2$; $t_f \leq 100$ mm	z-z	b
		y-y	c
	$t_f > 100$ mm	z-z	d
		y-y	d
Welded I Section 	$t_f \leq 40$ mm	z-z	b
		y-y	c
	$t_f > 40$mm	z-z	c
		y-y	d
Hollow Section 	hot rolled	any	a
	cold formed	any	b

IS 800: 2007 Clause 7.1.2.1

7.1.2.1 The design compressive stress, f_{cd} of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \le f_y / \gamma_{m0}$$

where

ϕ = $0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 \Big/ \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r;

α = imperfection factor given in Table 7;

χ = stress reduction factor (*see* Table 8) for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{\left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right]}$$

λ_{m0} = partial safety factor for material strength.

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Design steps

- For beginners , for an average column size of 3-5 m the slenderness ratio of 40 to 60 is selected. For very long column a λ of 60 may be assumed. For column with very heavy factored load a smaller value of slenderness ratio should be assumed.
- Choose a trial section by assuming an appropriate slenderness ratio from following table

Type of member	slenderness ratio
Single angle	100-50
Single channel	90-110
Double angles	80-120
Double channels	40-80
Single I -Section	80-100
Double I - section	30-60

Determine the design axial load on the column section ISMB 400, given that the height of the column is 3.5 m and that it is pin-ended. Also assume the following: f_y = 250 N/mm^2, f_u = 410 N/mm^2; E = 2×105 N/mm^2

Properties of ISMB 400 [Table I SP: 6(1)-1964]

Depth of section, h = 400 mm

Flange thickness, t_f = 16 mm

Thickness of web, t_w= 8.9 mm

Flange width, b = 140 mm

Cross-sectional area, A = 7846 mm^2

r_z= 161.5 mm, r_y= 28.2 mm

a) Buckling curve classification (Table 10, IS 800 :2007):

h/b=400/140=2.86>1.2 ; t_f=16mm<40mm

Hence, we should use buckling curve 'a' about z-z axis and 'b' about y-y axis.

So about z-z axis, it is class a, about y-y axis it is class b, as per the table 10 definition.

- **b) Effective length:**

Since both ends are pinned effective length,

$K^* L_y = K^* L_z = 3.5m$

c) Non-dimensional slenderness ratio:(7.1.2.1 of IS 800 :2007)

About z-z axis: *$\alpha=0.21$ [**Table 7, IS 800:2007]***

$$\lambda_z = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{f_y\left(\frac{kL_z}{r_z}\right)^2 / (\pi^2 E)}$$

$$= \sqrt{250\left(\frac{3500}{161.5}\right)^2 / (\pi^2 * 2 * 10^5)} = 0.2439$$

$$\Phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$

$$= 0.5[1 + 0.21(0.2439 - 0.2) + 0.2439^2]$$

$$= 0.534$$

$$\mathrm{f_{cd}} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

$$= \frac{250/1.1}{0.534 + [0.534^2 - 0.2439^2]^{0.5}}$$

$$= 225.2\ N/mm^2$$

- About y-y axis: α = 0.34 [Table 7, IS 800:2007]

$$\lambda_y = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{f_y\left(\frac{kL_y}{r_y}\right)^2/(\pi^2 E)}$$

$$= \sqrt{250\left(\frac{3500}{28.2}\right)^2/(\pi^2 * 2 * 10^5)} = 1.3968$$

$$\Phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$
$$= 0.5[1 + 0.34(1.3968 - 0.2) + 1.3968^2]$$
$$= 1.697$$

$$\mathrm{f_{cd}} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

$$= \frac{250/1.1}{1.697 + [1.697^2 - 1.3968^2]^{0.5}}$$
$$= 87.06\ N/mm^2$$

$P_d = 7846 \times 87.06 \times 10^{-3} = 683.07\ kN$

CALCULATION FROM TABLE-9

about y-y axis it is class b Table

$K^* L_y=K^* L_z=3.5m,$

$(K^* L_y/r_y)=3500/28.2=124.11$

For $(K^* L_y/r_y)=120$, $f_{cd}=91.7$

For $(K^* L_y/r_y)=130$, $f_{cd}=81.0$

For $(K^* L_y/r_y)=124.11$, $f_{cd}=91.7+[(81.0-91.7)/(130-120)]*(124.11-120)=87.30$

$Pcd=fcd*Ae=87.30*7846$

about z-z axis it is class a

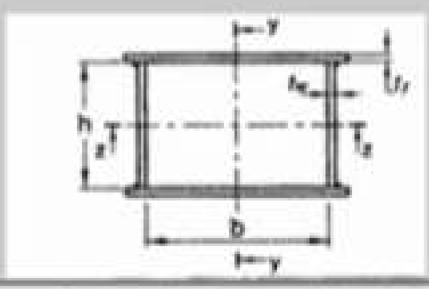
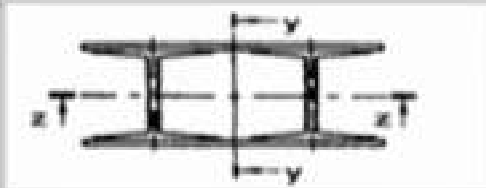
$K^* L_z=K^* L_z=3.5m,$

$(K^* L_z/r_z)=3500/161.5=21.67$

In Z direction fcd value will be more than 220

- 120= 91.7
- 130=81
- 124.11=91.7+[(81.0-91.7)/(130-120)]*(124.11-120)=87.30

Buckling Class of Cross Sections (Table 10 IS 800 :2007)

Cross Section	Limits	Buckling about Axis	Buckling Class
Welded Box Section 	Generally (except as below)	any	b
	thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z	c
		y-y	c
Channel, Angle, T and Solid Sections 		any	c
Built-up Member 		any	c

Angle Struts
(Cl. 7.5 of IS 800 :2007)

1. Single Angle Struts (Cl. 7.5.1 IS 800 :2007):

The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentrically by one of its leg to a gusset or adjacent member.

- **Concentric loading:** (Cl. 7.5.1.1 IS 800 2007)

When a single angle is concentrically loaded in compression, the design strength may be evaluated as per clause 7.1.2 of IS 800 :2007.

When the angle is concentrically loaded this can be calculated through the clause 7.1.2 of IS 800:2007. This is like concentric loading which may be channel section, maybe I section, maybe other type of section.

- **Loaded through one leg (Cl. 7.5.1.2 of IS 800 :2007)**

When the single angle is loaded concentrically through one leg of its legs, the flexural torsional buckling strength may be evaluated using an equivalent slenderness ratio λ_e given by

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

Where k_1, k_2, k_3 = constants depending upon the end condition as per Table 12 of IS 800-2007.

$$\lambda_{vv} = \frac{l/r_{vv}}{\varepsilon\sqrt{\pi^2 E/250}}$$

$$\lambda_{\phi} = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\pi^2 E/250}}$$

Where, *l = centre to centre length of the supporting member*
r_{vv} = radius of gyration about minor axis
b1 ,b = width of two legs of the angle
t = thickness of the leg
ε = yield stress ratio, *ε=SQRT(250/f_y)*

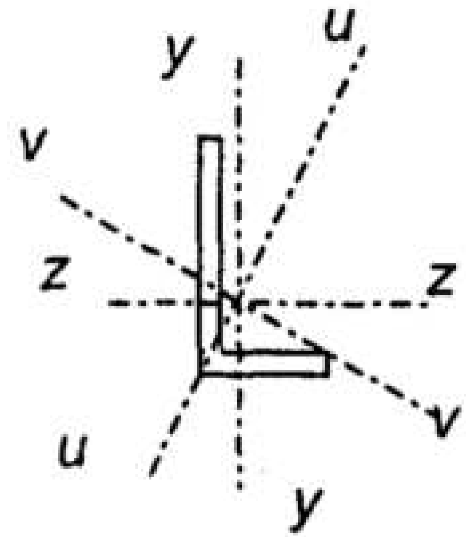
Where *k1 , k2 , k3=. constants depending upon the end condition as per Table 12 of IS 800-* 2007.

r_{vv} *in the previous expression which is the radius of gyration about* minor axis, which will be the minimum radius of gyration about the minor.
So for a particular angle section we can find out the properties from the geometry, from SP: 6
we can find out the value of r_{vv} for a particular angle section, then we know what is the width of the legs of the angle b_1, b_2 and thickness of the leg and also if we know the grade of steel, then we can find out the yield stress ratio ε that also we can find out.
And the constants k_1, k_2, k_3 that can be found in table 12 of IS 800:2007. Now this value of k1, k2, k3 depends on number of bolts at the end of the member as well as the connecting member fixity that means gusset or connecting member fixity means what type of fixity is there whether it is fixed, or hinged depending on that and whether number of bolts are more than or equal to 2 or 1. So depending on that we can find out k1, k2, k3.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

[1] Stiffeness of in-plane rotational restraint provided by the gusset/connecting member.
For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.



Example: An ISA 150×150×12 used as a strut has the effective length as 3 m. Calculate the

strength when it is connected by

a) One bolt at each end

b) Two bolts at each end

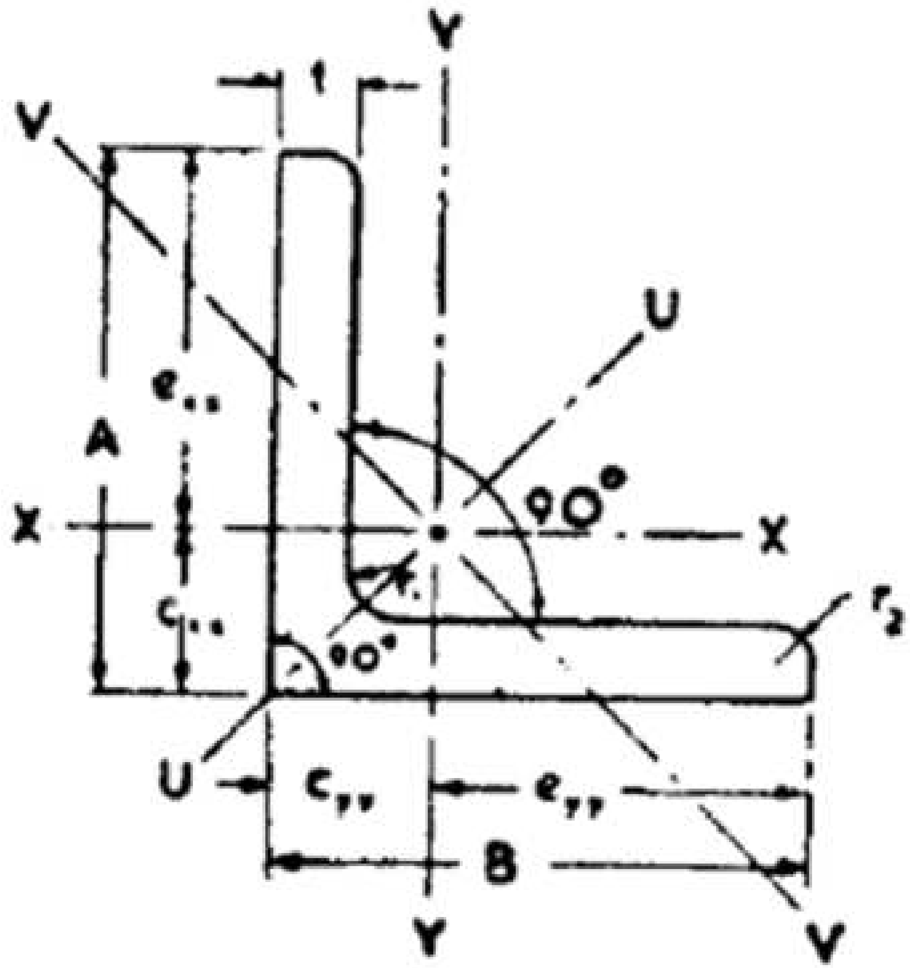
c) Welded at each end

For ISA 150×150×12, $A = 3459\ mm^2$, **[Table III, SP:6(1)-1964]**

$r_{vv} = 29.3\ mm$

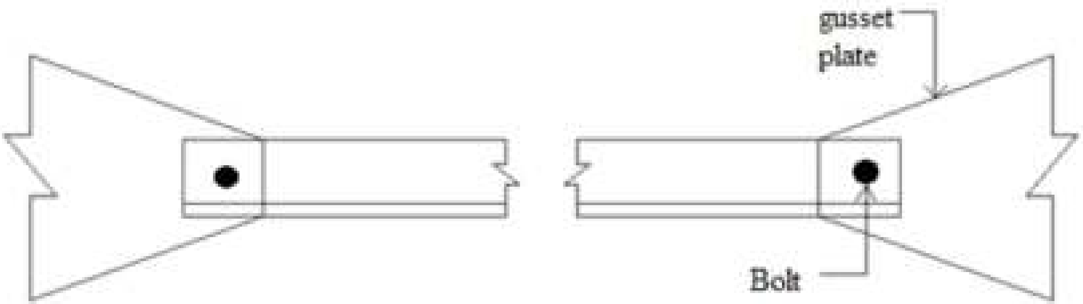
For angle sections, Buckling curve **'c'** is used. **[Table 10, IS 800:2007]**

Imperfection factor, $\alpha = 0.49$ **[Table 7, IS 800:2007]**



Y
t
V
U
e
A
X
90°
X
c
r
90°
r
U
c
e
B
Y
V

a) ***Connected by one bolt at each end:***



Assuming fixed conditions, for one bolt at each end,

$k_1 = 0.75$, $k_2 = 0.35$, $k_3 = 20$ **[Table 12, IS 800: 2007]**

c/c length, $l = 3000$ mm

Yield stress ratio, $\varepsilon = \sqrt{250/f_y} = \sqrt{250/250} = 1$ **[cl.7.5.1.2 of IS 800: 2007]**

$$\lambda_{vv} = \frac{l/r_{vv}}{\varepsilon\sqrt{\pi^2 E/250}} = \frac{3000/29.3}{1 \times \sqrt{\pi^2 \times 2 \times 10^5/250}} = 1.1523$$

$$\lambda_{\phi} = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\pi^2 E/250}} \qquad \frac{\dfrac{150+150}{2 \times 12}}{1 \times \sqrt{\pi^2 \times 2 \times 10^5/250}} = 0.1407$$

Hence, $\lambda_e = \sqrt{k_1 + k_2 \lambda_w^2 + k_3 \lambda_\phi^2}$

¿ $\sqrt{0.75 + 0.35 \times 1.1523^2 + 20 \times 0.1407^2}$

$= 1.2692$

Now $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$

¿ $0.5[1 + 0.49 \times (1.2692 - 0.2) + 1.2692^2]$

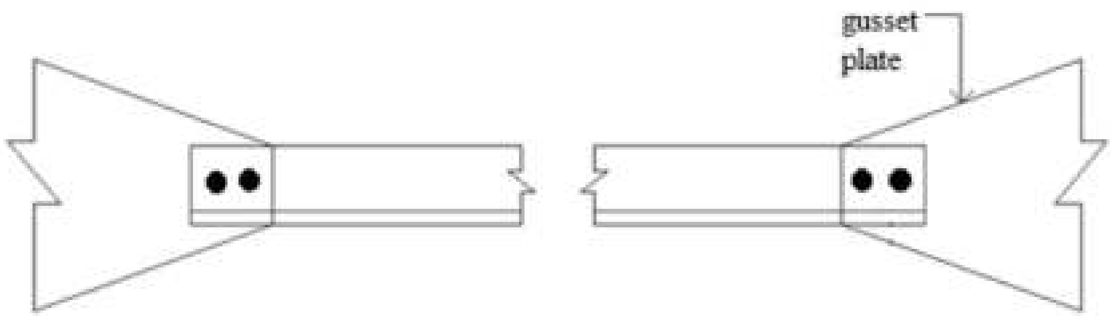
$= 1.5674$

Design compressive stress, $f_{cd} = \dfrac{f_y / \gamma_{m0}}{\varnothing + [\varnothing^2 - \lambda^2]^{0.5}} \quad \dfrac{250/1.1}{1.5674 + [1.5674^2 - 1.2692^2]^{0.5}}$

$= 91.38 \text{ N/mm}^2$

$P_d = A_e f_{cd} \quad = 3459 \times 91.38 \quad \times 10^{-3} \quad = \mathbf{316.1\ kN}$

b) ***Connected by two bolts at each end:***



Now the same calculations will be done when the two sides are calculated by two bolt for the same case and let us see how the strength is going to vary from earlier case.

Assuming fixed conditions, for two bolts at each end,

$k_1 = 0.2,\ k_2 = 0.35,\ k_3 = 20$ **[Table 12, IS 800:2007]**

Effective length, $l = 3000$ mm ; $\varepsilon = 1$

$\lambda_{vv} = 1.1523$ λ_{ϕ}=1407

$$\lambda_e = \sqrt{0.2 + 0.35 \times 1.1523^2 + 20 \times 0.1407^2}$$ λ_e=1.03

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] \quad 0.5\left[1 + 0.49 \times (1.03 - 0.2) + 1.03^2\right] \quad 1.2338$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

$$\frac{250/1.1}{1.2338 + [1.2338^2 - 1.03^2]^{0.5}} = 118.8 \text{ N/mm}^2$$

$$P_d = A_e f_{cd} = 3459 \times 118.8 \times 10^{-3} = \mathbf{410.9\ kN}$$

c) ***Connected by weld at each end:***

Now coming to third case when it is connected by weld at each end, so for this case what we can consider that this will be similar to the earlier. So in case of weld connection we can assume it will be fixed at both the end and as we have calculated the k1, k2, k3 value considering two bolts for this case also will become same.

This case will be exactly similar to earlier case, i.e., Connected by two bolts at each end.

Therefore, *Pd=.* ***410.9 kN***

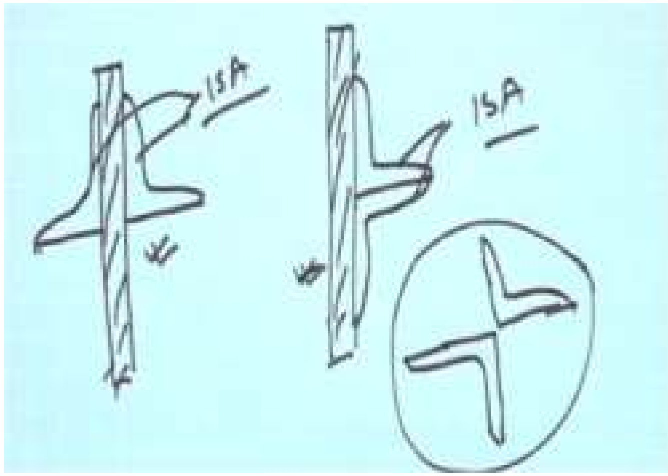
COMPRESSIVE STRENGTH OF DOUBLE ANGLES

Design compressive strength of a double angles section.

Double angles section is often used because many cases appear when the single angle is not capable of taking that much load and also in case of single angle the radius of gyration about the minor axis is very low compared to its X axis, or Y axis, or major axis. Therefore as radius of gyration is quite low about in minor axis, so strength of the angle section is quite low because it may buckle about its minor axis.

To calculate the strength of a double angle section, follow this codal provision 7.5.2 to find out the effective length and thereafter the slenderness ratio and then compressive stress, the allowable compressive stress.

The effective length of the double angle section means whether it is according to the degree of restraint means whether the length is in the plane perpendicular to that of end gusset or other one.



Effective length for Double Angle Struts:(Cl. 7.5.2 IS 800:2007)

The effective length, KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections.

the effective length of the double angles which is given in clause 7.5.2. So the codal provision says that depending on the degree of restraint provided the effective length will be consider as 0.7 to 0.85 times the distance between intersections and the effective length KL, in the plane perpendicular to that of the end gusset shall be taken as equal to the distance between centre of intersections.

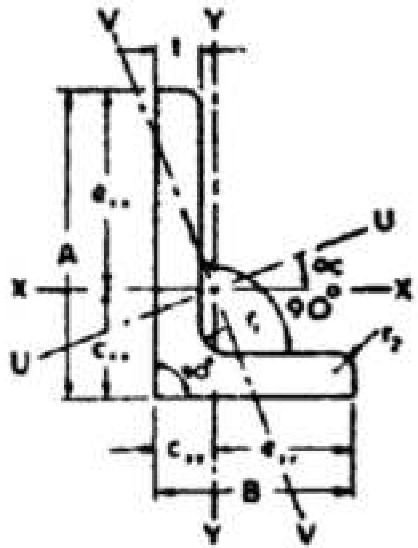
For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections.

Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed in accordance with 7.5.1 and the angles shall be connected together over their lengths so as to satisfy the requirements of 7.8 and 10.2.5.

Example: A discontinuous strut of length 4 m consists of two unequal angles ISA 100×75×8 and is connected to a 10 mm thick gusset plate by its longer leg. Determine the strength if it is connected on the:

i) Opposite side of the gusset plate

ii) Same side of the gusset plate



Solution:

Properties of ISA 100×75×8 : **[Table IV, SP:6(1)-1964]**

A= 1336 mm²

r_x = 31.4 mm r_y = 21.8 mm

r_u = 34.8 mm r_v = 15.9 mm

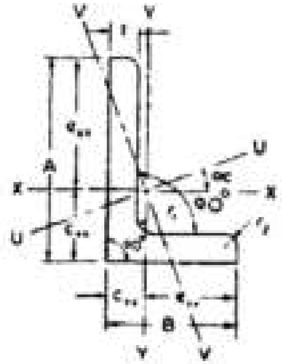
C_x = 31.0 mm C_y = 18.7 mm

I_x= 131.6×10⁴ mm⁴ I_y = 63.3×10⁴ mm⁴

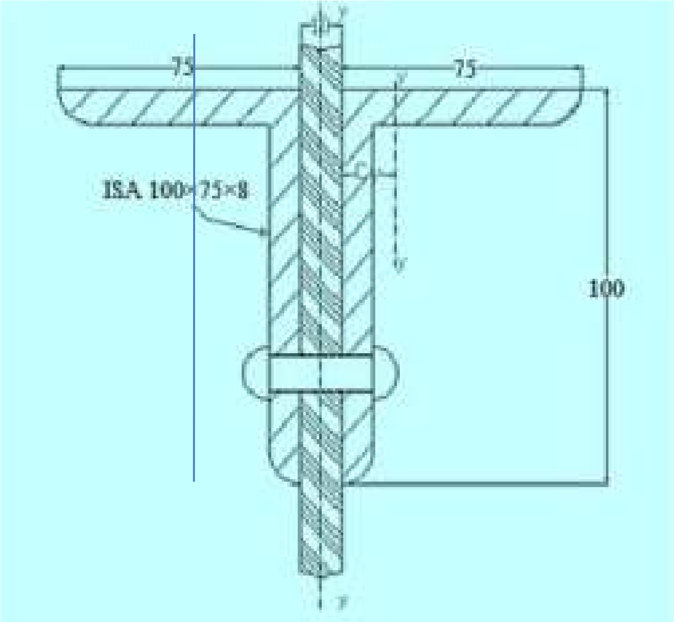
TABLE IV ROLLED STEEL UNEQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)



Designation	Size	Thickness	Sectional Area	Weight per Metre	Centre of Gravity		Distance of Extreme Fibre		Moments of Inertia			
	$A \times B$	t	a	w	C_{zz}	C_{yy}	e_{zz}	e_{yy}	I_{xx}	I_{yy}	I_{uu}	I_{vv}
	mm.mm	mm	cm^2	kg	cm	cm	cm	cm	cm^4	cm^4	cm^4	cm^4
ISA 9060	90 × 60	6·0	8·65	6·8	2·87	1·39	6·13	4·61	70·6	25·2	81·5	14·3
		8·0	11·37	8·9	2·96	1·48	6·04	4·52	91·5	32·4	105·3	18·6
		10·0	14·01	11·0	3·04	1·55	5·96	4·45	110·9	39·1	127·3	22·8
		12·0	16·57	13·0	3·12	1·63	5·88	4·37	129·1	45·2	147·5	26·8
ISA 10065	100 × 65	6·0	9·55	7·5	3·19	1·47	6·81	5·03	96·7	32·4	110·6	18·6
		8·0	12·57	9·9	3·28	1·55	6·72	4·93	125·9	41·9	143·6	24·2
		10·0	15·51	12·2	3·37	1·63	6·63	4·87	153·2	50·7	174·2	29·7
ISA 10075	100 × 75	6·0	10·14	8·0	3·01	1·78	6·99	5·72	100·9	48·7	124·0	25·6
		8·0	13·36	10·5	3·10	1·87	6·90	5·63	131·6	63·3	161·3	33·6
		10·0	16·50	13·0	3·19	1·95	6·81	5·55	160·4	76·9	196·1	41·2
		12·0	19·56	15·4	3·27	2·03	6·73	5·47	187·5	89·5	228·4	48·6

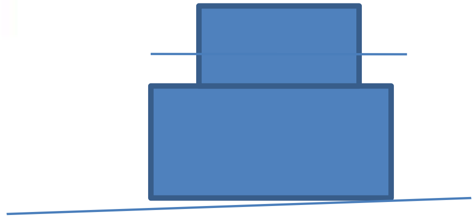


For double angle struts

$$A' = 2\times1336 = 2672\ \text{mm}^2$$

$$r'_x = 31.4\ \text{mm (same as for single angle)}$$

$$I'_y = 2\left[I_y + A\left(c_y + t_g/2\right)^2\right]$$



$$I'_y = 2\left[I_y + A\left(c_y + t_g/2\right)^2\right]$$

¿ $2\times\left[63.3\times 104 + 1336\times\left(18.7 + 10/2\right)2\right]$

¿ $276.68\times 104\ mm4$

Therefore, $r'_y = \sqrt{\dfrac{I'_y}{A'}} = \sqrt{\dfrac{276.68\times10^4}{2672}} = 32.18\ mm$

r_{min}=*min of* $(r_{x'}\ ,\ r_{y'})$=*31.4mm*

Now effective Length, *le=0.85 l=0.85×4=3.4m*

Slenderness Ratio, $\lambda = l_e/r_{min} = 3.4\times 1000/31.4 = 108.28 < 180$ **[Table 3, IS 800 2007]**

Hence, the section is ok.

Buckling class for angle section – 'c' **[Table 10, IS 800: 2007]**

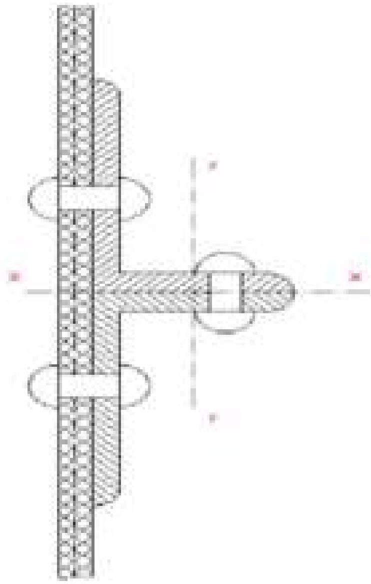
For $f_y = 250$ *MPa and* $\lambda = 108.28$ *and buckling class* ***c,***

using Table 9(c) of IS 800:**2007, we have**

$$f_{cd} = 107 - \frac{(107 - 94.6)}{10} \times 8.28 = 96.73 \quad \text{MPa}$$

Hence strength of the member= $P_d = 96.73\times 2672\times 10^{-3} = 258.46\ kN$

ii) Angle placed on the same side of the gusset plate



$A' = 2672\ mm^2$

$r'_y = 21.8$ mm (same as for single angle)

$I_{yy}/A = 2*63.3*10^4/(2*1336) = 21.8$ mm

$I_x' = 2\ [I_x + A\ Cx^2] = 2\times[131.6\times10^4 + 1336\times31^2] = 519.98\times10^4\ mm4$

Therefore $r_x = SQRT(Ix'/A') = SQRT(519.98\times104/2672) = 44.\ 11mm$

Hence, $r_{min} = min\ of\ (r_x'\ \ and\ \ r_y') = 21.8mm$

Effective Length, *le=0.85 l=0.85×4=3.4m* ***[cl. 7.5.2.1, IS 800: 2007]***

Slenderness Ratio, λ =*le/r min=* 3.4×*1000/*21.8=155.96<180 **[Table 3, IS 800: 2007]**

Hence, the section is ok.

Buckling class for angle section – 'c' **[table 10, IS 800: 2007]**

For f_y*=250 MPa and λ=155.96 and buckling class* ***c, using table 9(c) of IS 800:*****2007**

λ-=150, fcd=59.2, for λ= 160, fcd=53.03

f_{cd}*=59.2+[(53.3- 59.2)/*10]**×(155.96-150)=55.68 MPa*

Hence strength of the member 55.68×*2672×10*$^{-3}$=. 148.78 *kN*

- ***Strength of the member when Angles placed on***

(i) Opposite sides of the gusset plate: 258.46 *k N*

(ii) Same side of the gusset plate: 148.78 *k N*

for the same angle if it is placed opposite side to the gusset plate its strength is quite high compared to this that means we will try to prefer always the angles to be placed opposite side of the gusset plate. So that the strength can be achieved more compared to the earlier case means when angles are placed same side of the gusset plate.

DESIGN OF COMPRESSION MEMBER

The design compressive force Pd $=A_e*f_{cd}$ Both A_e and f_{cd} are unknown.

The allowable compressive stress (f_{cd}) of the member because it depends on the slenderness ratio which is depends on the radius of gyration. Now the radius of gyration will be depending on the dimension of the section.

f_{cd} generally considered f_{cd} as 0.4fy to 0.6fy. P=fcd*A

1) **Assume a suitable design compression stress (*f cd*) *as 0.4f_y to 0.6 f_y.***
2) **Effective sectional area required is, *$Ae=P_d/f_{cd}$***
3) **A section is to be selected which gives effective area required and then calculate *r_{min}*.**
4) **Determine effective length, knowing the end conditions and by deciding the type of connection.**
5) **Determine the slenderness ratio and hence design stress *f_{cd} and load carrying capacity P_d.***
6) **Modify the section if calculated *Pd differs significantly from the design load.***

Slenderness ratio to be assumed while selecting the trial section:

Type of member	*Slenderness Ratio (l/r)*
Single angles	100-150
Single channels	90-150
Double angles	80-120
Double channels	40-80
Single I-sections	80-150
Double I-sections	30-60

- **Compute KL/r for the section selected & check for slenderness ratio.**

Design a single angle section to carry a compression load 100 kN.C/C distance between end connection is 2 m. with two bolts . Design end connection also. Steel Grade E250.

- For angle section buckling class is C and f_{cd} varies 227MPa to24.3 MPa.
- Assumed value =150 MPa to 100 MPa.
- Assume f_{cd}=120 MPa.
- Required section =Factored Load/ f_{cd}=1.5*100*1000/120=1250 mm2.
- Trial section =80*80*8 mm
- A= 1220 mm^2.
- $r_z=r_y$=24.4 mm r_u=30.8 mm r_v=15.5 mm
- L/rv=2000/15.5=129 (b1+b2)/2t=(80+80)/2*8=10
- ε=1 $\varepsilon(\pi^2E/250)^{0.5}$=88.86
- $\lambda v=(L/r_v)/\{\varepsilon(\pi^2E/250)^{0.5}\}$=129/88.86=1.4588.86

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\pi^2 E/250}}$$

- λ_ϕ=10/88.86=0.1125

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\pi^2 E/250}}$$

- Assuming gusset plate fixed and two bolts on each end
- K_1=0.20, k_2=0.35 and k_3=20
- λ_e=(0.2+0.35*1.45*1.45+20*0.1125^2)=1.09

$$\lambda_e = \sqrt{k_1 + k_2\lambda_w^2 + k_3\lambda_\phi^2}$$

- ϕ=0.5[1+0.49*(1.09-0.2)+1.09^2]=1.824

$$\phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2]$$

Design compressive stress, $f_{cd} = \dfrac{f_y/\gamma_{m0}}{\varnothing + [\varnothing^2 - \lambda^2]^{0.5}}$

- f_{cd}=(250/1.1)/[1.824+{1.824^2-1.09^2}$^{0.5}$]=111.3 Mpa
- Pd=111.3* 1220=135.75 kN< 150 kN

- Redesign
- Take 80*80*10 mm A=1500 mm
- $r_z=r_y=24.1$ mm $r_u=30.4$ mm $r_v=15.5$ mm
- L/rv=2000/15.5=129 (b1+b2)/2t=(80+80)/2*10=8
- $\varepsilon=1$ $\varepsilon(\pi^2E/250)^{0.5}=88.86$
- $\lambda v=(L/r_v)/\{\varepsilon(\pi^2E/250)^{0.5}\}=129/88.86=1.4588$

$$\lambda_\phi=\frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\pi^2 E/250}}$$

- $\lambda_\phi=8/88.86=0.09$

$$\lambda_e=\sqrt{k_1+k_2\lambda_w^2+k_3\lambda_\phi^2}$$

- $K_1=0.20$, $k_2=0.35$ and $k_3=20$

$$\phi=0.5[1+\alpha(\lambda-0.2)+\lambda^2]$$

- $\lambda_e=(0.2+0.35*1.45*1.45+20*0.09^2)=1.048$
- $\phi=0.5[1+0.49*(1.048-0.2)+1.048^2]=1.2569$

Design compressive stress, $f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$

f_{cd}=(250/1.1)/[1.2569+{1.2569^2-1.048^2}$^{0.5}$]=116.5 Mpa

Pd=116.5* 1500=174.75 kN< 150 kN

Design of End connection:

Bolt Property Class M20, 4.6

Taking shear plane out of thread

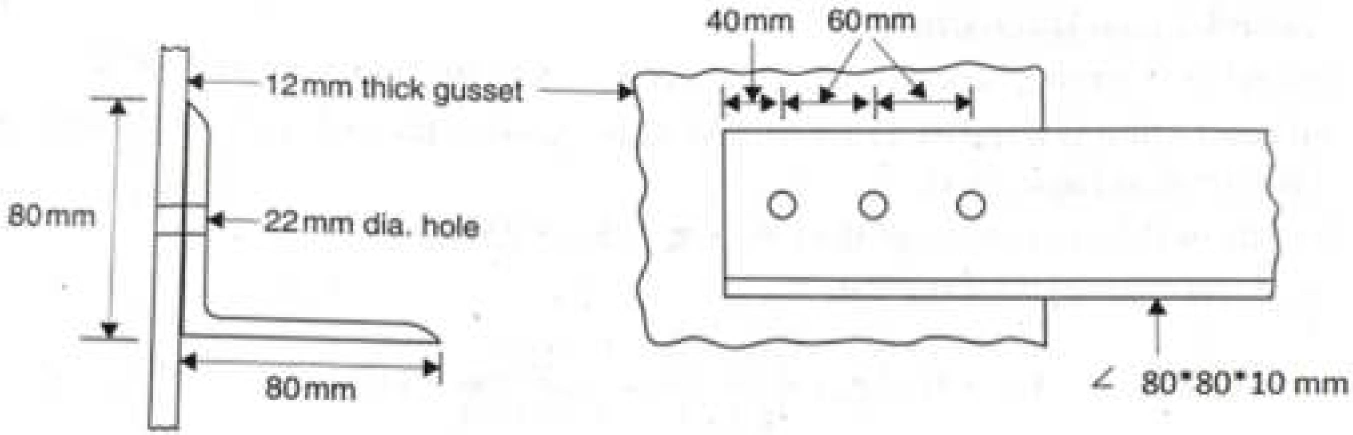
Bolt value in shear= 400*314/[sqrt(3)*1.25]=58.04 kN

Bolt value in bearing e=40 mm,p=60 mm

Kb=0.606

Bolt value in bearing=2.5*0.606*20*10*410/1.25=99.34 Kn

No.of bolts=150/58.04= say 3 bolts



Example: Design a compression member carrying an axial load of 250kN. The effective length of the member is 3 m. Design the member with 2 equal angles in star orientation as shown in the figure below.

Solution:

Assuming $fcd=0.5\ fy=0.5\times 250=125MPa$

Required area . 250×$1000/1\ 25=2000mm2$

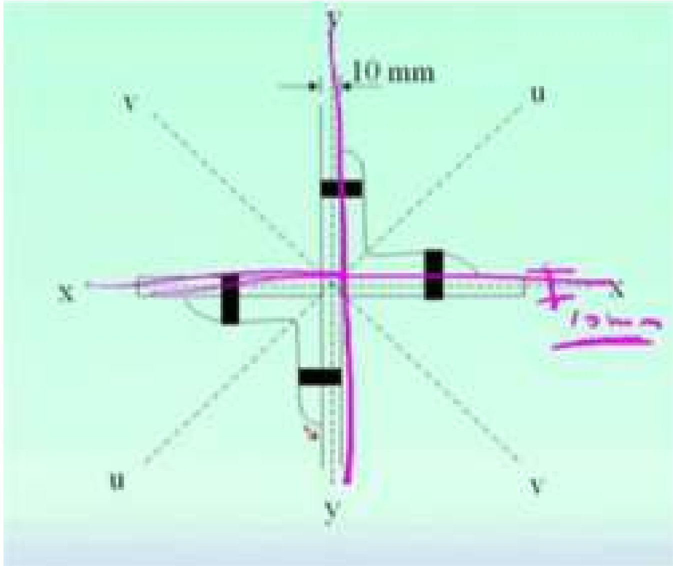
Selecting 2 ISA 90×90×6 with the following properties **[Table III, SP: 6(1)-1964]**

$A=1047\ mm^2$

$c_x=c_y=24.2mm\ r_x=r_y=27.7mm$

$r_u=35mm\ r_v=17.5mm$

To find out the property of the combined section.



So property of the combined section can be found as follows

$$A' = 2\times 1047 = 2094\,mm2$$

Assuming 10 mm gusset plate.

$$We\ know,\ I_x' = 2\left(I_x + A\left(C_x + \frac{t_g}{2}\right)^2\right)$$

A’=2*1047=2094 mm2

Gusset plate thickness=10 mm

$I'_x=2[I_x+A(c_x+t_g/2)^2]$

r'_x=SQRT(I’x/2A)=SQRT{$2[I_x+A(c_x+t_g/2)^2]/2A$}=SQRT[Ix/A+$(cx+tg/2)^2$]

=SQRT[$r^2x+(c_x+t_g/2)^2$]

$$r'_x=r'_y=\sqrt{r_x^2+\left(c_x+\frac{t_g}{2}\right)^2} \qquad \sqrt{27.7^2+\left(24.2+\frac{10}{2}\right)^2}=40.25\,mm$$

$$r_v' = \sqrt{r_v^2 + 2\left(c_y + \frac{t_g}{2}\right)^2}$$

$$= \sqrt{(17.5)^2 + 2\times\left(24.2 + \frac{10}{2}\right)^2}$$

$$= 44.85\text{ mm.}$$

$$r_u' = r_u = 35\text{ mm.}$$

ru ' =ru=35mm , Hence, rmin=35mm

Effective Length, *le=0.85 l=0.85×3000*

[cl. 7.5.2.1, IS 800]=2550*mm*

$r_x{}' = r_y{}' = SQRT[(r_x{}^2+(c_x+t_g/2)^2\ ]=SQRT[(27.7^2+(24.2+10/2)^2]=40.25mm$

$r_v{}' = SQRT[(r_x{}^2+(c_x+t_g/2)^2]=SQRT[17.5^2+2\times(24.2+ 10/2)]^2=44.85mm$

$r_u{}' = r_u=35mm$, Hence, $r_{min}=35mm$

So $\lambda=\frac{l_e}{r_{min}}=\frac{2550}{35}=72.86<180$ hence safe.

For $f\,y=250$ MPa and $\lambda=72.86$ and buckling class **c**, using table **9(c) of IS 800 : 2007**, we have

$$f_{cd}=152-\frac{(152-136)}{10}\times 2.86=147.42 \quad \text{MPa}$$

$P_d = 147.42 \times 2094$

$= \underline{308.7\ kN.} > \underline{250}\ kN$

- **Tack welding:**

Tack welding should be provided along the length to avoid local buckling of each of the elements

λ e≤0.6 λ=0.6×72.86=43.716 or 40, whichever is less (clause 7.8.1)

So, *λe=S/r v=40*

Hence, spacing between welds, *S=40×rv (Min r of the individual member)*

40×*17.5=700mm*

Welding is designed to resist a transverse load (P) of 2.5% of axial load

2.5×*250/100=6.25 kN*

Using 5 mm weld size (shop weld)

$$\text{Hence length of weld} = \frac{P}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{6.25 \times 10^3}{\frac{0.7 \times 5 \times 410}{\sqrt{3} \times 1.25}} = 9.43 \text{ mm}$$

Hence provide a 5mm tack welding of 10 mm length at 700 mm spacing.

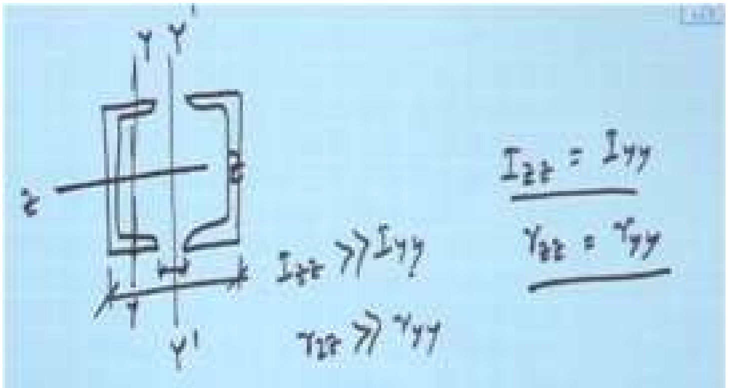
DESIGN OF BUILT UP COMPRESSION MEMBER

A built up compression member are those which consist of two or more rolled steel sections. The reasons for built-up columns are follows: -

1. The built up sections provide large cross-sectional area which cannot be furnished by single rolled steel section.

2. The built up sections provide special shape & depth. The special shape & depth facilitate connections between different members.

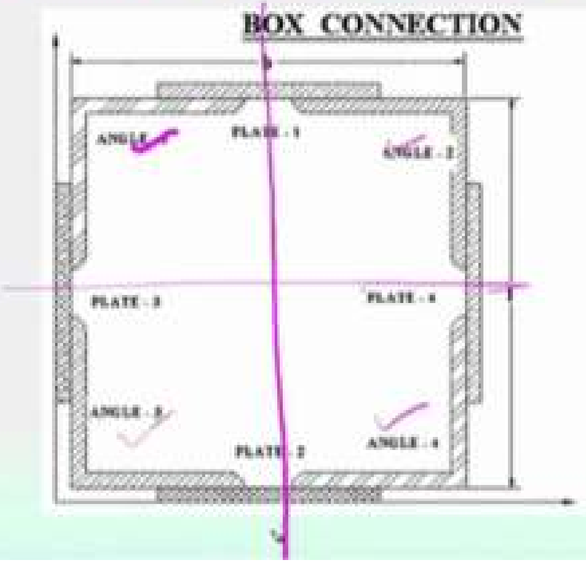
3. The built up sections provide sufficient large radius of gyration in two different directions.

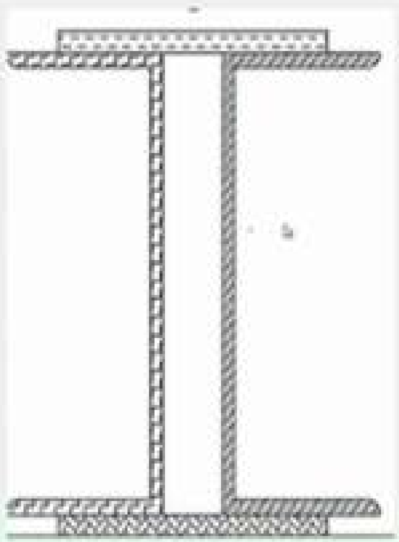


Say for example if we consider a channel section as a compression member as shown in the figure, as we can see the Izz is much higher than Iyy, right or rzz is much higher than ryy.

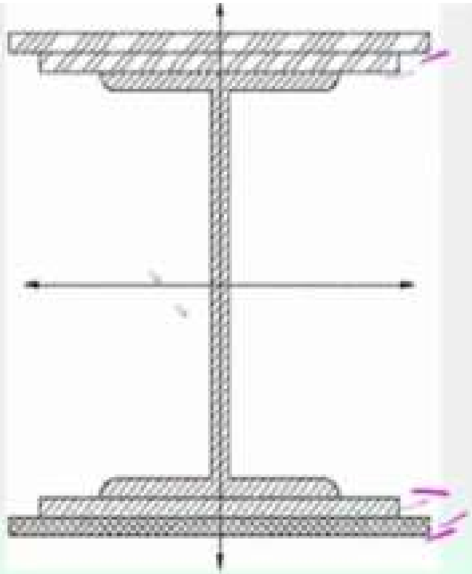
Therefore the chances of buckling about y-y axis will be much earlier than about z-z axis. So if we provide built up member then we can increase the radius of gyration by providing another member and with a certain spacing.

Examples of Built-up Sections

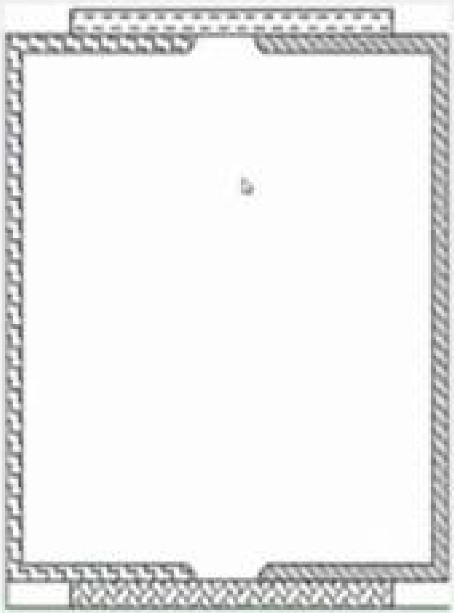




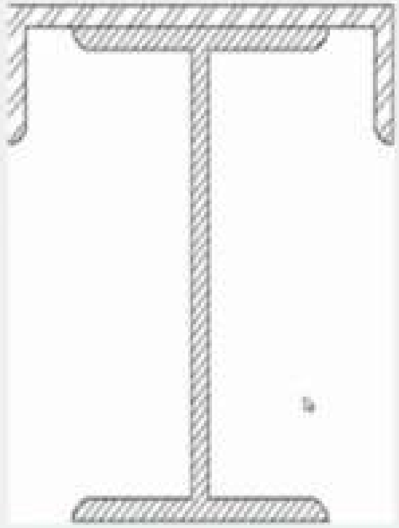
Channel Back to Back



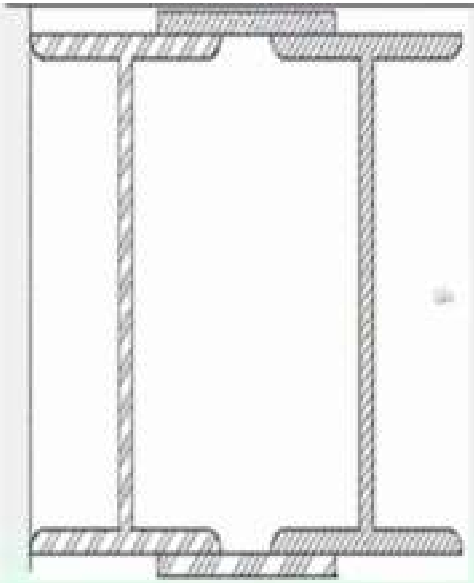
I Section with Plates



Channel Face to Face



I Section with Chanel at Top



Double I Section

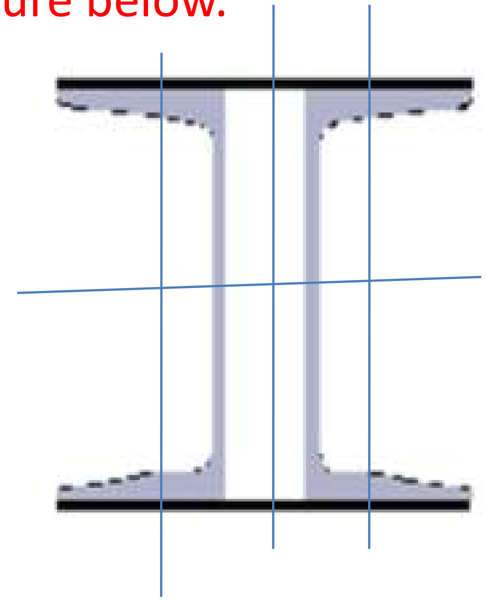
1. In the first step for the design of built-up compression member we find out the effective length taking consideration of end condition.
2. Then in 2nd step we generally assume certain value of slenderness ratio λ as 30 to 60 for built up section. We generally consider less value of λ because of the built up section the radius of gyration is quite high and therefore the slenderness ratio we can consider quite less means it may be from 30 to 60 which will be sufficient.
3. Then in step 3 we find the compressive stress fcd from table 9c because the buckling class for built-up member is c, therefore we can use table c and corresponding to table c for a particular value of λ whatever we consider we can find out the fcd value for a given grade of steel. So once we get the fcd value,
4. In step-4 we can find out the required cross-sectional area (A) which is P by fcd, where P is the axial compressive force which is acting on the member.

5. In step-5 we can choose a built-up section as per the requirement it may be channel section back to back, it may be channel section face to face, it may be I section. As per the requirement we have to decide what type of sections we are going to provide and what will be the arrangement. So accordingly we will find out the area from that considered section and then we will arrange the section in such a way may be if we use two channel section back to back, then we will arrange the section in such a way that the I_{xx} or I_{zz} become I_{yy}, so that we can find out the value of S, where S is the spacing between two section.Once this S is found we can find out the rmin value which will be practically more or less equal r_{zz} and r_{yy}.

6. In step-6, from the minimum value of the radius of gyration we can calculate the λ, the slenderness ratio which is *le/r.*

7. In step-7 we can find out the value of fcd corresponding to particular λ and grade of steel, right. So once we get fcd value we can find out the design compressive strength Pd which is $A_e \times f_{cd}$. So design compressive strength we can find out and if we see the design compressive strength is more than the axial compressive strength acting externally then it is okay, otherwise we can go for a higher section and we can repeat from step 5 to step 7.

Design a laced column 10.5 m long to carry factored axial load of 1000 kN. The column is restrained in position but not in direction at both the ends. Use 2 channel section placed as back to back as shown in the figure below.



Solution:

For steel of grade Fe 410:*f u=410 MPa, f y=250 MPa*

Design of column:

P=1000 kN =1000×10³ N

Le=1.0×10.5=10.5 m

Now let the design axial compressive stress for the column $=0.6*f_y= 150$ MPa

Required area = $1000\times 10^3/150=6666.67$ mm2

Let us try two ISMC 250 @ 298.2 N/m.

Relevant properties of ISMC 250 [Table II SP 6 (1): 1964]

$A=3867$ mm2, $r_{zz}=99.4$ mm, $r_{yy}=23.8$ mm $t_f=14.1$ mm

$I_{zz}=3816.8\times 10^4$ mm^4 , $I_{yy}=219.1\times 10^4$ mm^4

$c_{yy}=23$ mm $b=80$ mm, Area provided $2\times 3867=7734$ mm^2

In the design of built-up column with two sections, the sections are so spaced that the least radius of gyration of the built-up section becomes as large a value as possible. Therefore, the radius of gyration about *y-y axis is increased so that it becomes equal to or more than the* radius of gyration about *z-z axis. This can be achieved by spacing the sections in such a way* that r_{zz} *becomes* r_{min} *. Let us first check the safety of the section.*

Now, $L/rzz = 10.5\times10^3/99.4 = 105.63$

As per clause **7.6.1.5 of IS 800:2007** The effective slenderness ratio, $(KL/r)_e$, *of laced* columns shall be taken as 1.05 times the $(KL/r)o$, *the* actual maximum slenderness ratio, in order to account for shear deformation effects.

The effective slenderness ratio, $(KL/r)_e = 1.05\times105.63 = 110.91 < 180$

For $(KL/r)e == 110.91$, *fy=250 MPa and buckling class c,*

the design compressive stress, from Table 9c of IS 800: 2007

$f_{cd} = 94.6 - [(94.6 - 83.7)/10]\times0.91 = 93.61$ *MPa*

Therefore load carrying capacity =Pd= $A_e * f_{cd} = 7734\times93.61\times10^{-3} =$ 723.98 kN < 1000 kN, Which is not safe.

Try two ISMC 300 @ 351.2 N/m

Relevant properties of ISMC 300 [Table II SP 6 (1): 1964]

$A = 4564$ *mm2*, $r_{zz} = 118.1$ *mm*, $r_{yy} = 26.1$ *mm* $t_f = 13.6$ *mm*, $I_{zz} = 6362.6\times10^4\ mm^4$

$I_{yy} = 310.8\times104\ mm^4$

$c_{yy} = 23.6$ *mm* $b = 90$ *mm*

Area provided $2\times4564 = 9128\ mm^2$

$L/r_{zz} = 10.5\times10^3/118.1 = 88.91$

- As per clause 7.6.1.5 of IS 800:2007,

The effective slenderness ratio, $(KL/r)_e = 1.05 \times 88.91 = 93.35 < 180$

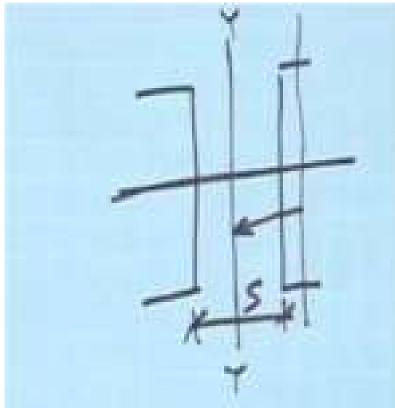
For $(KL/r)e = 93.35$, $f_y = 250$ *MPa and buckling class c,*

the design compressive stress, from Table 9c of IS 800: 2007

$f_{cd} = 121 + [(107-121)/10] \times 3.35 = 116.31$ *MPa*

Therefore load carrying capacity =Pd= $A_e * f_{cd} = 9128 \times 116.31 \times 10^{-3} = 1061.68$ kN > 1000 kN, Which is safe.

As KL/r in ZZ direction is taken as minimum, KL/r in Y direction should be more or at least equal to 93.35



So $r_{yy} > r_{zz}$ if spacing between two channels are s

$r_{zz} = 2 * Izz/(2 * A) = r_{yy} = I_{yy}/(2 * A)$

So, $2 * Izz = I_y$

$$2I_{zz} = 2\left[I_{yy} + A\left(\frac{S}{2} + C_{yy}\right)^2\right]$$

$$2 \times 6362.6 \times 104 = 2 \times \left[310.8 \times 104 + 4564\left(\frac{S}{2} + 23.6\right)^2\right]$$

$$\left(\frac{S}{2} + 23.6\right)^2 = 13259.86 \qquad \Rightarrow S = 183.1 \quad \text{mm}$$

Take S=184 mm

Lacing Systems

Lacing systems are used basically to keep the built up sections throughout its length and we need to tie them to make them parallel and to make them equidistant and to make them act as a monolithically, so that as a whole the built up section works.

Lacing which are basically some inclined member between the two vertical members. We can also use batten system instead of lacing system. Batten system is basically is a horizontal plate which are connected with the two main members. Therefore especially for eccentric loading we generally prefer lacing system.

Lacing system the lacing members are generally flat plate, it may be angle section, it may be light channel section, or may be circular section means tubular section, so in different way we can provide.

7.6 Laced Columns

7.6.1 *General*

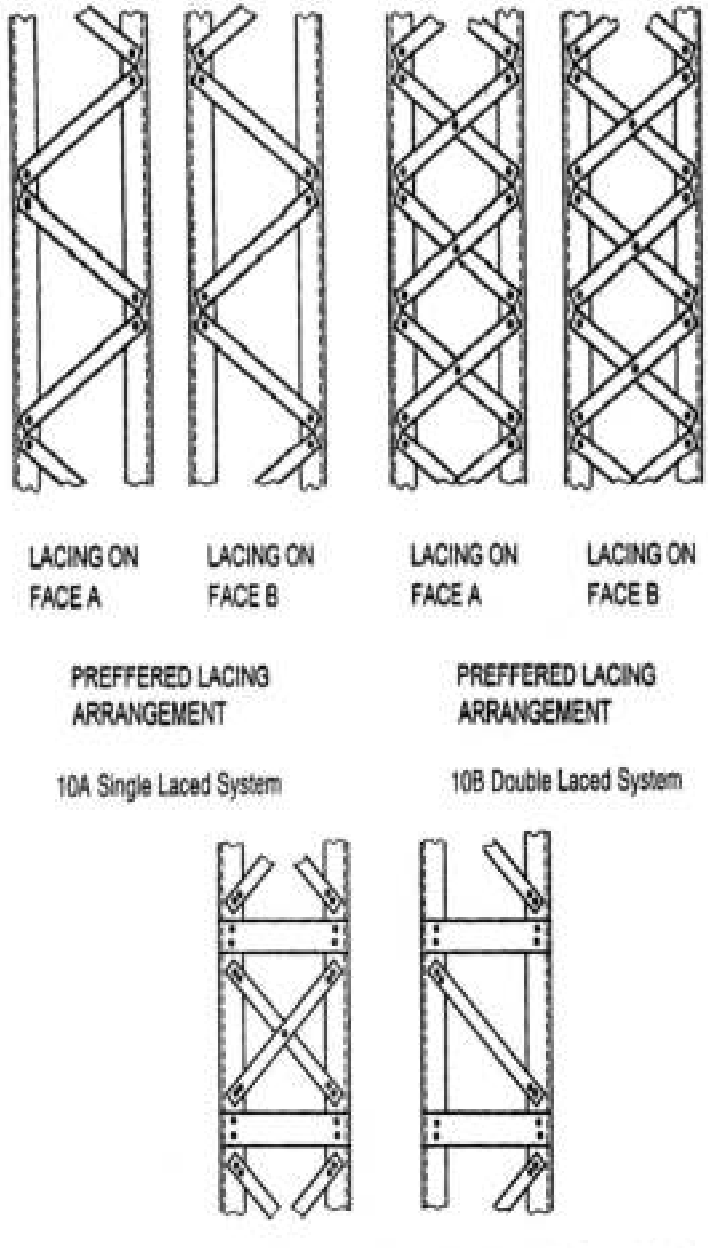
7.6.1.1 Members comprising two main components laced and tied, should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing (*see* Fig. 10A and 10B).

7.6.1.2 As far as practicable, the lacing system shall be uniform throughout the length of the column.

7.6.1.3 Except for tie plates as specified in **7.7**, double laced systems (*see* Fig. 10B) and single laced systems (*see* Fig. 10A) on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut (*see* Fig. 10C), unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

7.6.1.4 Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.

7.6.1.5 The effective slenderness ratio, $(KL/r)_e$, of laced columns shall be taken as 1.05 times the $(KL/r)_0$, the actual maximum slenderness ratio, in order to account for shear deformation effects.



10C Double Laced and Single Laced System Combined with Cross Numbers

FIG. 10 LACED COLUMNS

7.6.2 *Width of Lacing Bars*

In bolted/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolt/rivet.

7.6.3 *Thickness of Lacing Bars*

The thickness of flat lacing bars shall not be less than one-fortieth of its effective length for single lacings and one-sixtieth of the effective length for double lacings.

7.6.3.1 Rolled sections or tubes of equivalent strength may be permitted instead of flats, for lacings.

7.6.4 *Angle of Inclination*

Lacing bars, whether in double or single systems, shall be inclined at an angle not less than 40° nor more than 70° to the axis of the built-up member.

7.6.5 *Spacing*

7.6.5.1 The maximum spacing of lacing bars, whether connected by bolting, riveting or welding, shall also be such that the maximum slenderness ratio of the components of the main member (a_1/r_1), between consecutive lacing connections is not greater than 50 or 0.7 times the most unfavourable slenderness ratio of the member as a whole, whichever is less, where a_1 is the unsupported length of the individual member between lacing points, and r_1 is the minimum radius of gyration of the individual member being laced together

7.6.5.2 Where lacing bars are not lapped to form the connection to the components of the members, they shall be so connected that there is no appreciable interruption in the triangulation of the system.

7.6.6 *Design of Lacings*

7.6.6.1 The lacing shall be proportioned to resist a total transverse shear, V_t at any point in the member, equal to at least 2.5 percent of the axial force in the member and shall be divided equally among all transverse-lacing systems in parallel planes.

7.6.6.2 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the actual shear due to bending, in addition to that specified in **7.6.6.1**.

7.6.6.3 The slenderness ratio, KL/r, of the lacing bars shall not exceed 145. In bolted/riveted construction, the effective length of lacing bars for the determination of the design strength shall be taken as the length between the inner end fastener of the bars for single lacing, and as 0.7 of this length for double lacings effectively connected at intersections. In welded construction, the effective lengths shall be taken as 0.7 times the distance between the inner ends of welds connecting the single lacing bars to the members.

NOTE — The required section for lacing bars as compression/tension members shall be determined by using the appropriate design stresses, f_{cd} subject to the requirements given in **7.6.3**, to **7.6.6** and T_d in **6.1**.

7.6.7 *Attachment to Main Members*

The bolting, riveting or welding of lacing bars to the main members shall be sufficient to transmit the force calculated in the bars. Where welded lacing bars overlap the main members, the amount of lap measured along either edge of the lacing bar shall be not less than four times the thickness of the bar or the thickness of the element of the members to which it is connected, whichever is less. The welding should be sufficient to transmit the load in the bar and shall, in any case, be provided along each side of the bar for the full length of lap.

7.6.8 *End Tie Plates*

Laced compression members shall be provided with tie plates as per **7.7** at the ends of lacing systems and at intersection with other members/stays and at points where the lacing systems are interrupted.

For single lacing system of two parallel faces the force on each bar:

$$F = \frac{V}{n\sin\theta} = \frac{V}{2\sin\theta}$$

n = No. of transverse system in parallel plane

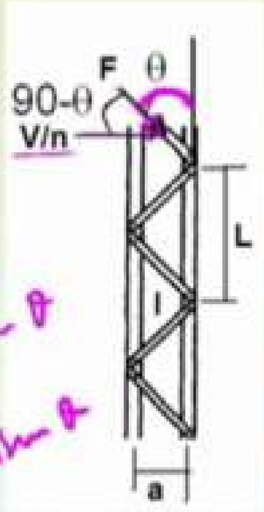
$\frac{V}{n} = F\sin\theta$

$F = \frac{V}{n\sin\theta}$

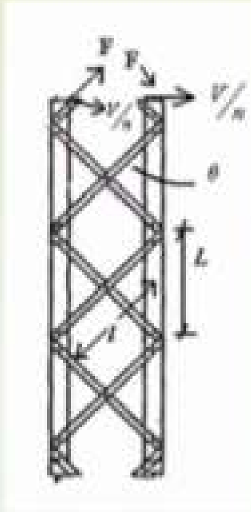
For double lacing:

$$F = \frac{V}{4\sin\theta}$$

This force will be tensile in one lacing bar & compressive in the other



Single lacing



Double lacing

The slenderness ratio l_e/r of the lacing bars should not exceed 145. The effective length of lacing bars should be taken as follows:-

Types of welding	Effective length
Single lacing (bolted at ends)	Length between inner end bolts on lacing bar *i.e.* ($l_e = l$)
Double lacing, (bolted at ends & at intersections)	0.7 times the length between inner end bolts on lacing bars *i.e.* ($l_e = 0.7l$)
Welded lacing	0.7 time the distance between inner ends of lengths of welds at ends *i.e.* ($l_e = 0.7l$)

Now if we use flat bars as lacings then the slenderness ratio is calculated as follows:

$$\lambda = \frac{l_e}{r} = \frac{l_e}{\sqrt{I/A}} = \frac{l_e}{\sqrt{\frac{bt^3/12}{bt}}} = \frac{l_e\sqrt{12}}{t}$$

$$\therefore \lambda_{lacing} = \frac{l_e\sqrt{12}}{t} < 145$$

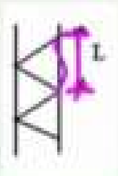
5. Spacing: The maximum spacing of lacing bars should be such that minimum slenderness ratio will be,

$$\frac{L}{r^{c}_{min}} = 0.7\lambda_{max}$$
$$= 50 \qquad \text{which ever is minimum}$$

Where, λ_{max} is the maximum slenderness ratio of the compression member as a whole

L = Distance between centers of connections of the lattice bars to each component

r^{c}_{min} = Minimum radius of gyration of the component of compression members



6. **<u>Attachment to the main members:</u>**

The riveting, bolting or welding of the lacing bars to the main member should be sufficient to transmit the load in the bar.

Welded Connection:

Where welded lacing bars overlap the main members, the amount of lap measured along either edge of the lacing bar shall be not less than four times the thickness of the bar or the members, whichever is less.

The welding should be sufficient to transmit the load in the bar and shall, in any case, be provided along each side of the bar for the full length of lap.

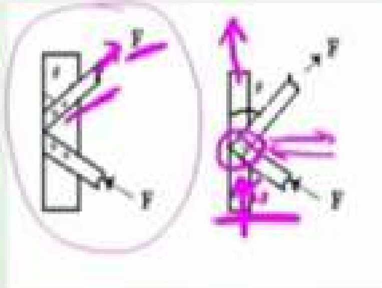
Where lacing bars are fitted between the main members, they shall be connected to each member by fillet welds on each side of the bar or by full penetration butt welds. The lacing bars shall be so placed as to be generally opposite the flange or stiffening elements of the main member.

Bolted connection:

For first case, the numbers of bolt, $n = \frac{F}{R}$

For second case, the numbers of bolt,

$$n = \frac{Resultant\ Force}{Bolt\ Value} = \frac{2F\cos\theta}{R}$$



Where, F = Force in lacing bar

θ = Inclination

DESIGN PROCEDURE

1. Choose lacing system either single single lacing or double lacing.
2. Assume angle of inclination (θ) with the axis of the compression member, theta should vary from 40 to 70 degree and generally we try to keep theta from 40 to 45 degree to get maximum efficiency of the lacing system.

   Find a, g *$a = 2g + S$,* Now the distance between the bolt centres is calculated as follows

   $$L = \frac{2a}{\tan\theta} \quad \text{For single lacing}$$

   $L = a/\tan\theta$ For double lacing

   Decide the spacing (S) so that I_{xx} and I_{yy} become same.

3. *Find* out the slenderness ratio of each component and check for slenderness ratio. So slenderness ratio is calculated as minimum of the followings

$$\frac{L}{r^c_{min}} = 0.7\,\lambda_{max} \qquad \frac{L}{r^c_{min}} = 50$$

$t > \frac{l}{40}$ for single lacing

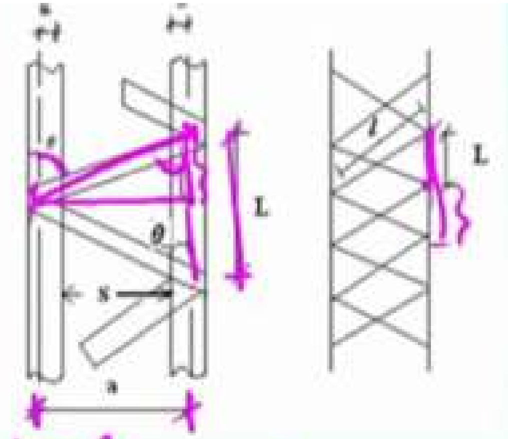
Thus according to the figure,

$a = 2g + S$

S = Clear spacing

$L = \frac{2a}{\tan\theta}$ For single lacing

$= \frac{a}{\tan\theta}$ For double lacing



$\frac{a}{L/2} = \tan\theta$

$\frac{L}{2} = \frac{a}{\tan\theta}$

$l = \frac{a}{\sin\theta}$ For both single and double lacing

$L = \frac{2a}{\tan\theta}$

4. Find the effective length. Effective length is calculated as follows
   For single lacing system (bolted ends), *le = l*
   Double lacing system (bolted ends), *le = 0.7l*
   For welded lacing system, *le = 0.7l*
5. *Find out thickness of lacing system.*
   t>l/40 for single lacing
   t>l/60 for double lacing

6. Check slenderness ratio of lacing

for flat plate $$\lambda_{lacing} = \frac{l_e \sqrt{12}}{t} < 145$$

7. Calculate Compressive strength of member

8. Calculate transverse shear force V=0.025 P

9. Find out the compressive stress and tensile stress of the lacing system and we have to check that this compressive stress and tensile stress is less than the permissible compressive and tensile stress.

10. Find the diameter of bolt and number of bolts.

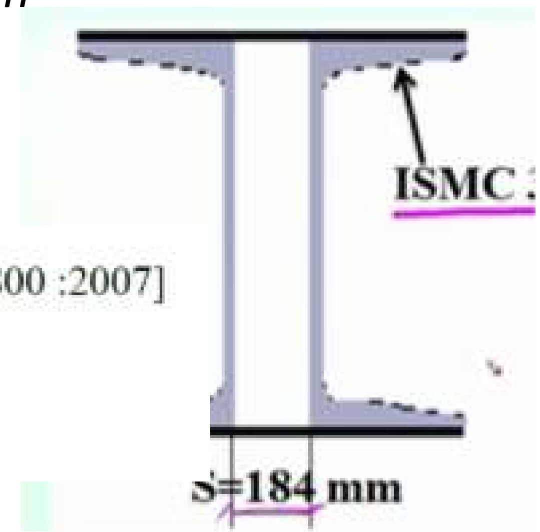
11. Design end connections for lacing system.

Design a laced column 10.5 m long to carry factored axial load of 1000 kN. The column is restrained in position but not in direction at both the ends. Provide single lacing system. Use 2 channel section placed as back to back. Assume steel of grade Fe 410 and bolts of grade 4.6.

- a) Design the lacing system with bolted connections
- b) Design the lacing system with site welded connections.
- From Previous problem,
  $(KL/r)_e$=1.05×*88.91*=93.35<180

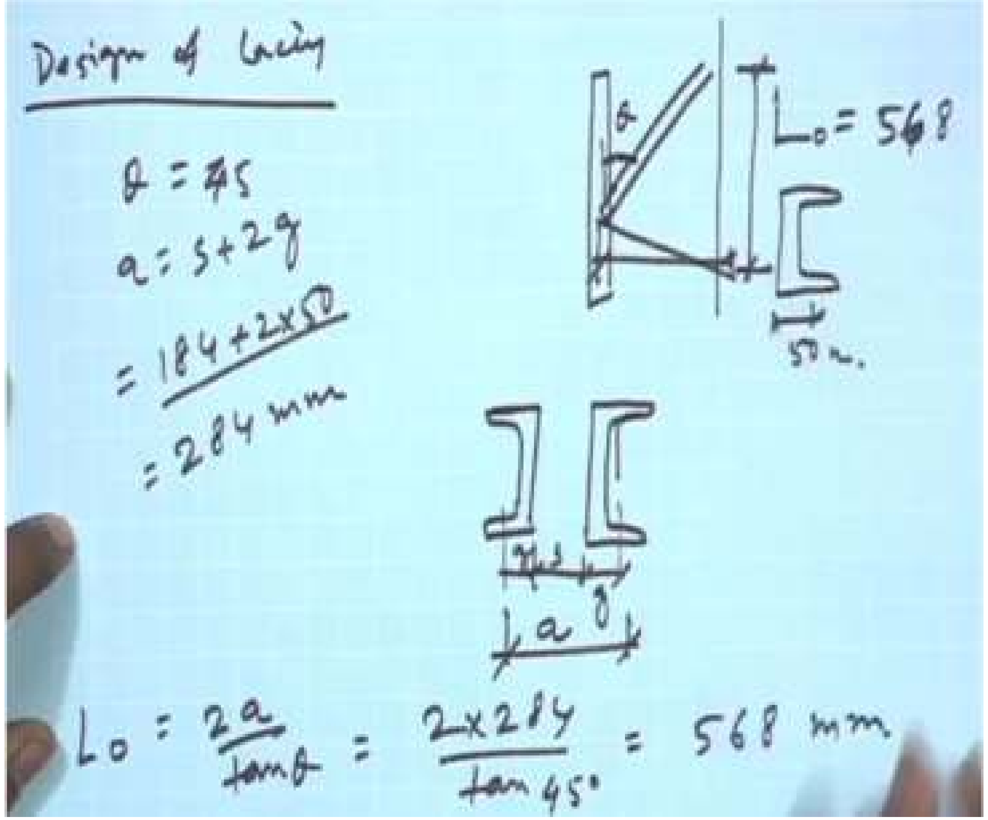
Design of lacing system, Let use a single lacing system with inclination of lacing bar .45°

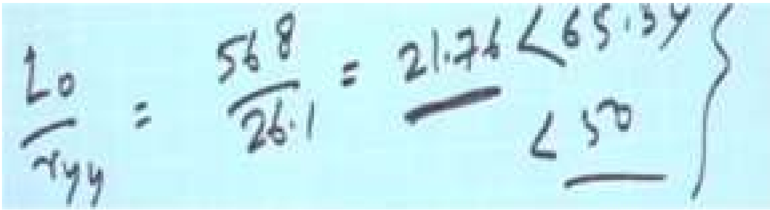
- Assume gauge length = 50 mm
- Spacing of lacing bar, L_0*=2×(184+50+50) cot 45°=568 mm*
- *R will be taken as minimum of* r_{zz} *and* r_{yy} *of channel*
- r_{zz}*=118.1 mm,*r_{yy}*=26.1 mm(*



L_0/r_{yy} should be < $0.7 \times (L/r)$ of whole column [cl. 7.6.5.1 of IS 800 :2007]

$$\frac{L_0}{r_{yy}} = \frac{568}{26.1} = 21.76 < 0.7 \times 93.35 = 65.34$$





Maximum shear, V =(2.5/100)*$\times 1000 \times 10^3$ ***(cl. 7.6.6.1 IS 800: 2007)=***25000 N

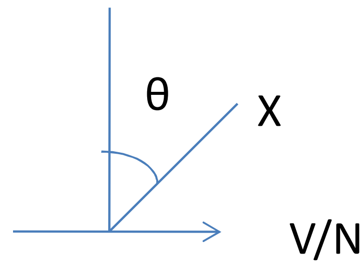
Transverse shear in each panel . *V/N*=25000/2=12500 N

- Transverse shear in each panel . *V/N*=25000/2=12500 N

Compressive force in lacing bars . *(V / N) cosec 45°*=12500×*1.414*=17675 N

X Sinθ =V/N

X=V/N Cosec θ



Lacing flats:

Let us provide 16 mm diameter bolts

Minimum width of lacing flat **(cl. 7.6.2 IS 800: 2007)**3×*16=48 mm*

Let us provide 50 mm wide flats.

Length of lacing flat = (184+50+50)*cosec 45° = 401.6=402 mm*

Minimum thickness of lacing flat **(cl. 7.6.3 IS 800: 2007)** 1/40 × *length of flat between inner end bolts.=* 1/40*×*402*=10.05 mm

Provide 12 mm thick plate with a width of 50 mm.

Minimum radius of gyration, $r=\frac{t}{\sqrt{12}}=\frac{12}{\sqrt{12}}=3.464$ mm

r=SQRT(I/A)=sqrt [(bt3/12)/bt]=t/[sqrt(12)] *l/r of lacing bar = 402/*3.464=116<145

- Hence, ok
- For *l/r* =116 , *fy=250 MPa and buckling class c, the design compressive stress from* Table 9c of IS 800 :2007

$$f_{cd}=94.6-\frac{94.6-83.7}{10}\times 6=88.06 \quad \text{MPa}$$

Design compressive strength, *Pd=Ae* f cd*
. (12×*50)×88. 06×10*$^{-3}$
52.8 4 kN > 17.67 kN

The tensile strength of flat is minimum of **(cl. 6.2 and 6.3.1 of IS 800: 2007)**

i) $0.9\times\frac{(B-d_h)t f_u}{\gamma_{m1}}=0.9\times\frac{(50-18)\times 12\times 410}{1.25}\times 10^{-3}=113.36 \quad \text{kN}$

and

ii) $\frac{A_g f_y}{\gamma_{m0}}=\frac{(50\times 12)\times 250}{1.1}\times 10^{-3}=136.363 \quad \text{kN}$

Hence, the tensile strength of the flat is (minimum of 113.36 kN and 136.36 kN) 113.36 kN > 17.67 kN

Hence, safe.

a) Bolted connection (If lacings are not over lapped each other)

Assuming that the 16 mm bolts of grade 4.6 are connecting both the lacing flats with the channel at one point and that the shear plane will not pass through the threaded portion of bolt.

Strength of bolt in single shear $= A_{sb}(f_u/\sqrt{3})/\gamma_{mb}$

$$= \left(\frac{\pi\times16^2}{4}\right)\times\left(\frac{400}{\sqrt{3}}\right)/1.25$$

$$= 37147 \text{ N}$$

Minimum pitch, $p = 2.5d = 2.5\times16 = 40$

Minimum end distance, $e = 1.5\, d_0 = 1.5\times18 = 27$ mm

Provide p=50 mm and e= 31 mm

k_b is smaller of 31/(3×18), 50/(3×18)-0.25, 400/410, 1

$K_b = 0.57$

a) Bolted connection (If lacings are not over lapped each other)

Assuming that the 16 mm bolts of grade 4.6 are connecting both the lacing flats with the channel at one point and that the shear plane will not pass through the threaded portion of bolt.

Strength of bolt in single shear ¿ $A_{sb}(f_u/\sqrt{3})/\gamma_{mb}$

¿ $\left(\frac{\pi\times16^2}{4}\right)\times\left(\frac{400}{\sqrt{3}}\right)/1.25$

¿37147 N

Minimum end distance, e = 1.5 d_0 =1.5×18 = 27 mm

Provide p=50 mm and e= 30 mm

k_b is smaller of 30/(3×18), 50/(3×18)-0.25, 400/410, 1

K_b = 0.56

Strength in bearing $¿ 2.5 k_b dt f_u / \gamma_{mb}$

$$¿ 2.5 \times 0.56 \times 16 \times 12 \times \frac{410}{1.25} = 88167 \quad \text{N}$$

Hence, strength of bolt = 37147 N = 37 kN

No of bolts required = 17.67/37 = 0.5 ≈ 1

Hence, provide one bolt at each end.

b) Bolted connection (If lacings are over lapped each other)

Assuming that the 16 mm bolts of grade 4.6 are connecting both the lacing flats with the channel at one point and that the shear plane will not pass through the threaded portion of bolt.

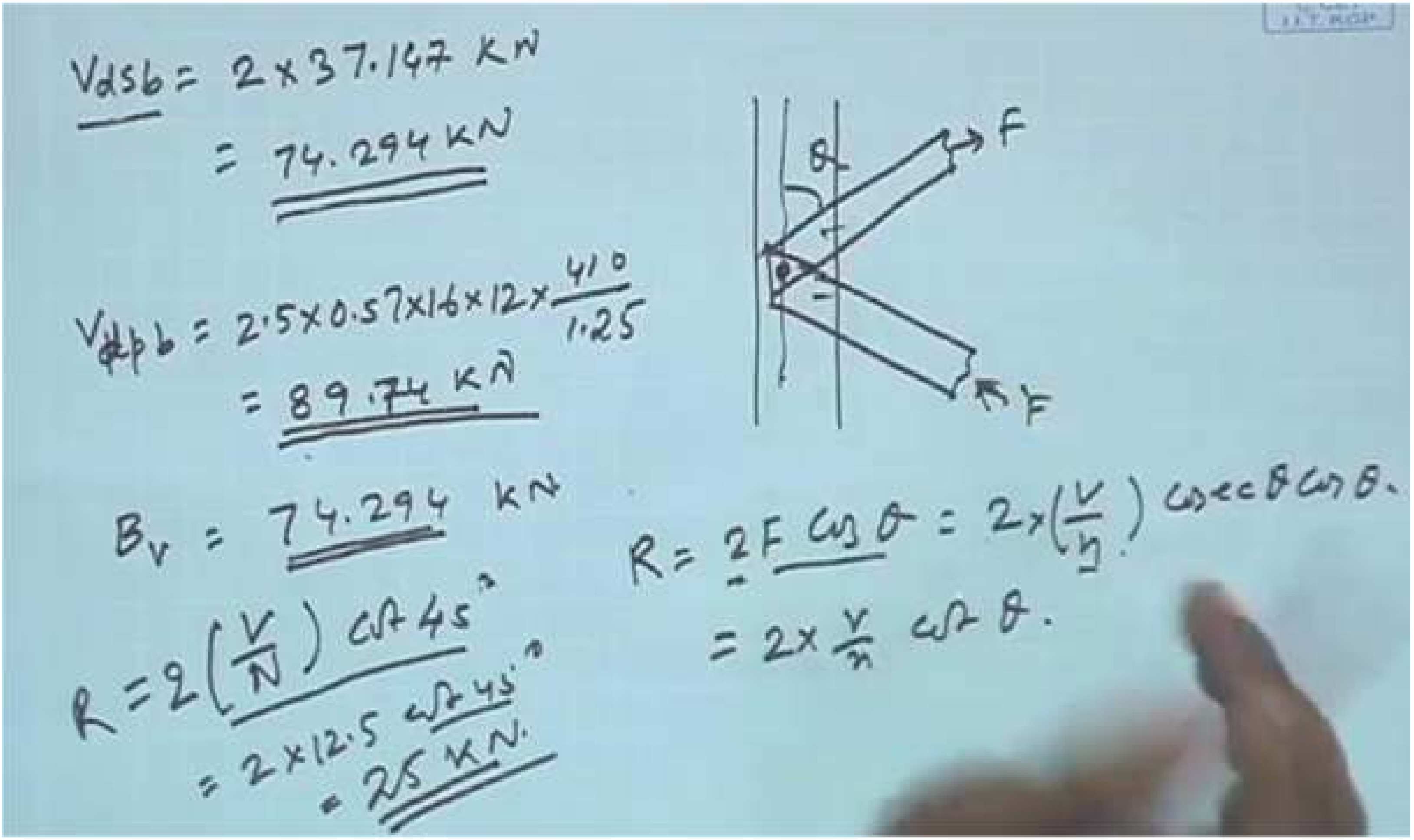
Strength of bolt in **double** shear $= 2 \times A_{sb}(f_u/\sqrt{3})/\gamma_{mb}$

$$= 74294\text{N}$$

Strength in bearing $= 2.5 k_b d t f_u/\gamma_{mb}$ (let k_b as 0.49)

$$= 2.5 \times 0.57 \times 16 \times 12 \times \frac{410}{1.25} = 89741\text{N}$$

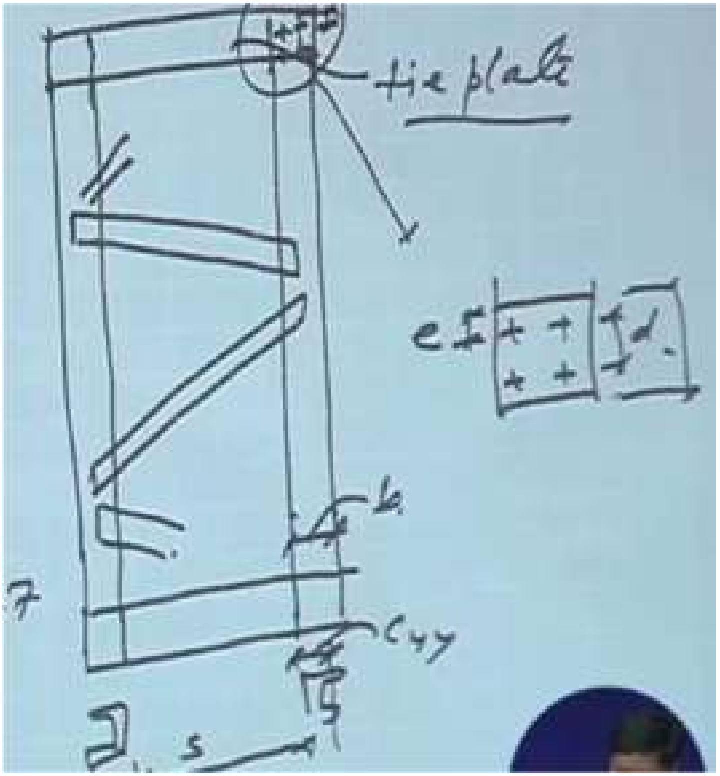
Hence, strength of bolt = 74294 N = 74 kN



F=V/N Cosec θ   R=2*F cos θ=2*(V/n)Cosec θ*cos θ=2*(25/2*cot θ=25

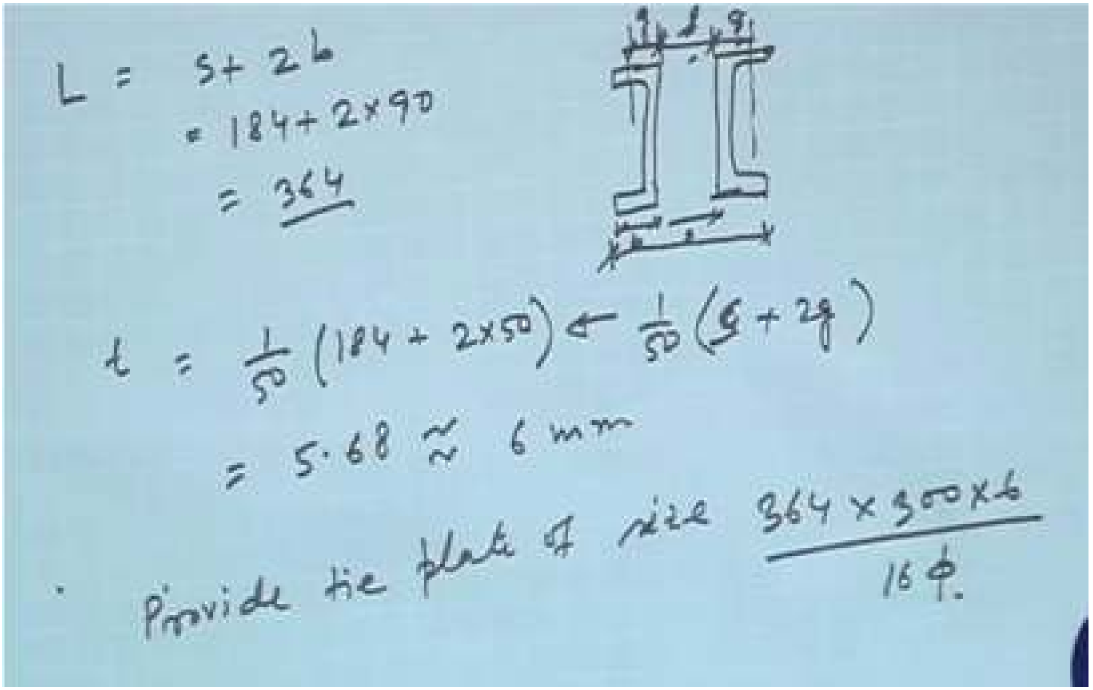
No. of bolts required=25/74.29= one bolt

Tie plate



7.7.2.2 *Tie plates*

Tie plates are members provided at the ends of battened and laced members, and shall be designed by the same method as battens. In no case shall a tie plate and its fastenings be incapable of carrying the forces for which the lacing or batten has been designed.



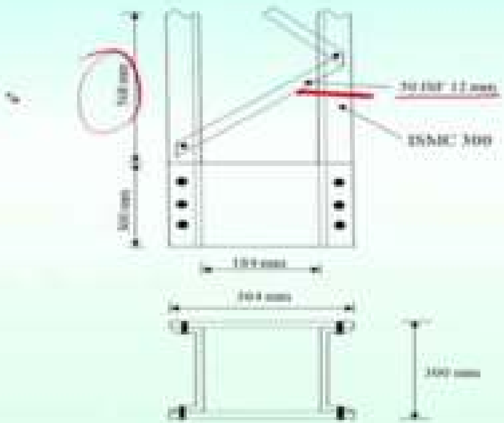
Tie plates are provided at the ends of the laced column.

Effective depth $d=(184+2\times C_{yy})>2\ b_f$

$184+2\times 23.6=231.2\ mm>2\times 90=180$ Which is all right.

- Minimum edge distance for 16 mm diameter bolt
- . $1.5\times(16+2)=27\ mm$, say 30 mm
- Overall depth of tie plate D=d+2 edge ditsance=$231.2+2\times 30=291.2\ mm$
- Provide a tie plate of 300 mm depth.
- The plate is having depth 300 mm, Length=364 mm and thickness 6mm

Provide a tie plate of size 364×300×6 mm at both ends with 16 mm diameter bolts as shown in the figure.



Lacing flats and tie plates are connected by bolts (overlapped each other)

b) Welded connection:

Flange thickness of ISMC 300 = 13.6 mm

Minimum size of weld for 13.6 mm thick member = 5 mm **[Table 21 IS 800 :2007]**

Strength of weld/unit length $= 0.7 \times 5 \times \frac{410}{\sqrt{3} \times 1.5} = 552.33$ N/mm

Required length of weld $= \frac{17670}{552.33} = 32$ mm

Adding extra length for ends, the weld length to be provided

$$= 32 + 2 \times (2 \times 5) = 52 \text{ mm}$$

Provide 100 mm weld length at both ends.

Tie plate:

Overall depth $= 184 + 2 \times 23.6 = 231.2$ mm

Depth provided = 240 mm

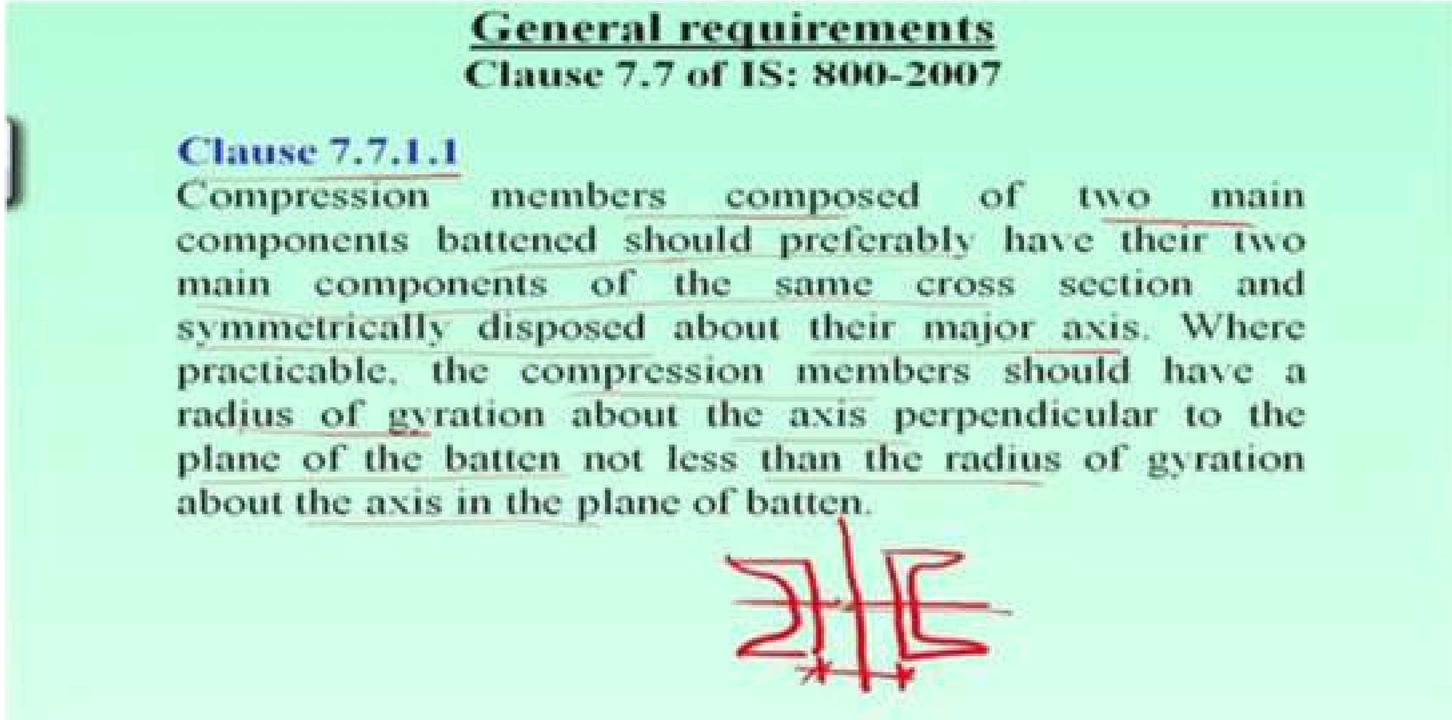
Let length of the tie plate $= 184 + 2 \times 50 = 284$ mm

Thickness of tie plate $= \frac{1}{50} \times 284 = 5.68$ mm

Provide a 8 mm plate to accommodate a 5 mm weld.

Provide a tie plate of size $\boxed{283 \times 240 \times 8}$ mm size and connect it with 5 mm welds as shown in the figure.

BATTENS



General requirements
Clause 7.7 of IS: 800-2007

Clause 7.7.1.1
Compression members composed of two main components battened should preferably have their two main components of the same cross section and symmetrically disposed about their major axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis in the plane of batten.

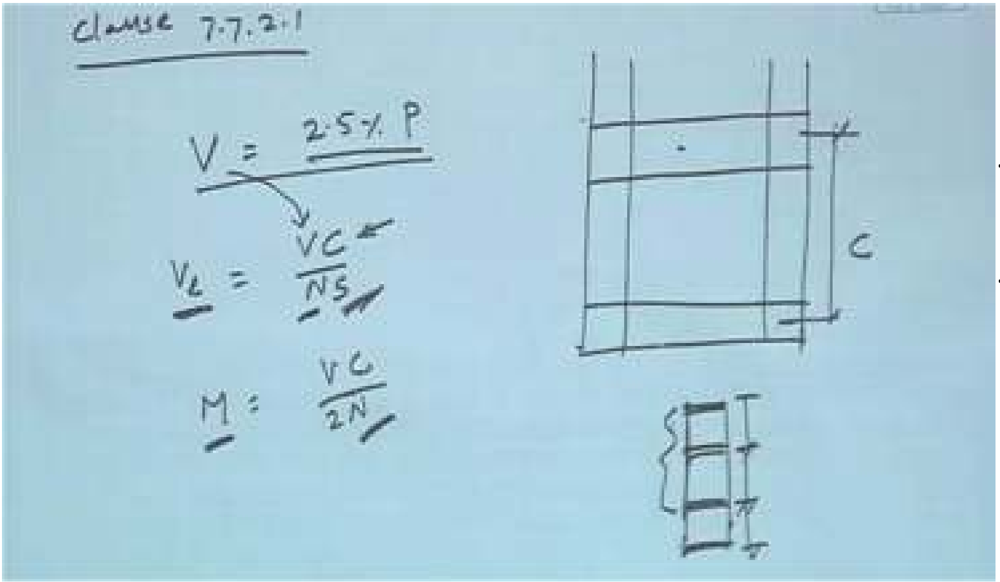
Where ever practicable the compression member should have a radius of gyration about the axis perpendicular to the plane of batten not less than the radius of gyration about the axis in the plane of batten.
Let us provide two channel section back to back. Now this sections has to be provided in such a way that r_{yy} and r_{xx} will be becoming mostly same or ryy should be little higher. So because we cannot change the value of r_{xx}, r_{xx} will be same for all the cases but if we increase the spacing between these two r_{yy} value will be increased because I_{yy} value is going to increase.

Clause 7.7.1.3

The battens shall be placed opposite each other at each end of the member and points where the member is stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout. Number of battens shall be such that the member is divided into not less than three bays within its actual length from center to center of connection.

Effective Slenderness ratio (7.7.1.4)

The effective slenderness ratio of the column is increased by 10% of the actual one.



transverse shear to the batten is considered as 2.5 percent of the axial force,

V = The transverse shear force

Longitudinal shear $V_l = \frac{VC}{NS}$ along the column axis

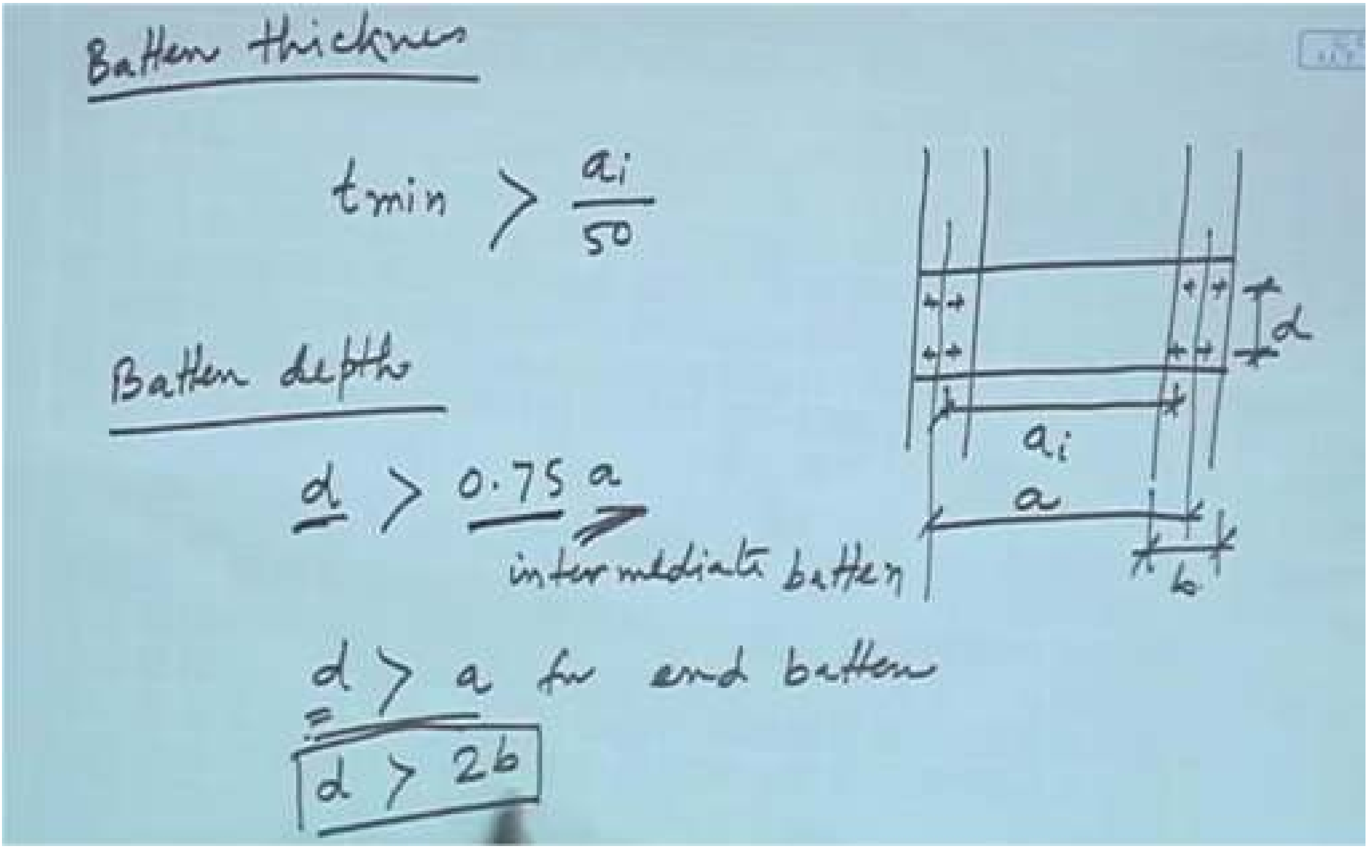
And moment $M = \frac{VC}{2N}$ at each connection,

Where,

C = Center to center distance of battens longitudinally

N = No of parallel planes of battens

S = minimum transverse distance between the centroid of the bolt group/welding connecting the batten to the main member



clause 7.7.2.3 batten minimum thickness t_{min} can be considered as

$T_{min} > a_i/50$

Where, a_i is the distance between the inner most connecting lines of rivet or bolt or weld perpendicular to the main member.

The effective depth should be

- d > 0.75*a for intermediate battens*
- d > *a for end battens*
- d > 2*b for any battens*

a = centroid distance of members

b = width of member in the plane of batten

- 7.7.2.3 Size When plates are used for battens, the end battens and those at points where the member is stayed in its length shall have an effective depth, longitudinally, not less than the perpendicular distance between the centroids of the main members. The intermediate battens shall have an effective depth of not less than three quarters of this distance, but in no case shall the effective depth of any batten be less than twice the width of one member, in the plane of the battens. The effective depth of a batten shall be taken as the longitudinal distance between outermost bolts, rivets or welds at the ends. The thickness of batten or the tie plates shall be not less than one-fiftieth of the distance between the innermost connecting lines of rivets, bolts or welds, perpendicular to the main member clause 7.7.3, The spacing of the battens (C) be such that the slenderness ratio of the lesser main component over the distance is not greater than 50 or 0.7 times the slenderness ratio of the main member as a whole, about the axis parallel to the batten.

$$\frac{C}{r^{c}_{min}} < 50 \vee 0.7\,\lambda$$

End connections (Clause 7.7.4):

Design the end connections to resist the longitudinal shear force V_l and the moment M as calculated in earlier step.

- For welded connection: Lap > $4t$
- Total length of weld at edge of batten >$D/2$
- Length of weld at each edge of batten < 1/3 total length of weld required.
- Return weld along transverse axis of column < $4t$.

Total length of weld at edge of batten >*D/2*

Length of weld at each edge of batten < 1/3 total length of weld required.

Return weld along transverse axis of column < 4*t.*

DESIGN STEPS

Step 1:

Find the transverse shear $V_t = \frac{2.5}{100} \times P$

Calculate longitudinal shear along the column axis as $V_l = \frac{V_t C}{NS}$ and

Calculate moment at each connection $as\, M = \frac{V_t C}{2N}$,

Step 2:

Calculate effective slenderness ratio (λe) as 1.1 × λ

Step 3: -

For a given shape, find out gauge distance g on each side & find the distance a between the bolt

Step 4: -

Calculate spacing of the batten plates (C) from the following conditions:

$$\frac{C}{r^{c}_{min}} < 50 \vee 0.7\lambda$$

Minimum 3 nos of batten should be provided along column length.

Step 5: - Size of end battens:

Effective depth $(d) = s + 2 \times C_{yy}$ and $d > 2b$,

[b is width of member in the plane of batten]

Overall depth (D) = d+2e, e being edge distance

Length of batten *S+2b*

Thickness of batten = *ai/50 where ai is the distance between inner bolt/rivet/weld*

Step 6: - Size of intermediate battens:

Effective depth

(*d*)=*[3/4]* [s+2×Cyy]* and *d >2b,*

[b is width of member in the plane of batten]

Overall depth (D) = d+2e, e being edge distance

Length of batten *S+2b*

Thickness of batten = *a/50 where a is the distance between inner bolt/rivet/weld*

Step 7: -

Design the end connections for batten system to resist calculated *Vl and M.*

Example: A batten column of 10-m long is carrying a factored load of 1150 kN. The column is restrained in position but not in direction at both ends. Design a built up column using channel sections placed back to back. Design batten plates using bolt connection.

Solution:

Design of column:

$P=1150\ kN$

$L=1.0\times10\times10^3=10000\ mm$

Let design axial compressive stress for the column be 125 MPa

Required area .$1150\times10^{3}/125=9200\ mm^2$

Let us try two ISMC 350 @ 413 N/m

Relevant properties of ISMC 350 [Table II SP 6 (1): 1964]

$A=5366\ mm^2$, $r_{zz}=136.6\ mm$, $r_{yy}=28.3\ mm$ $t_f=13.5\ mm$

$I_{zz}=10008\times10^4\ mm^4$ $I_{yy}=430.6\times10^{4}\ mm^{4}$ $c_{yy}=24.4\ mm$

$b=100\ mm$

Area provided $2 \times 5366 = 10732\ mm^2$

L/r_{zz}=10000/136.6=73.21

The effective slenderness ratio, $(KL/r)e = 1.1 \times 73.21 = 80.53 < 180$; *ok*

For $(KL/r)e = 80.53$, $f_y = 250$ *MPa and buckling class c, the design compressive stress* from Table 9c of IS 800: 2007

$f_{cd} = 136 - [(136 - 121)/10] \times 0.53 = 135.2\ MPa$

Therefore load carrying capacity . $A_e * f_{cd} = 10732 \times 135.2 \times 10^{-3} = 1451$ kN > 1150 kN, OK

Spacing of channels:

$2\ I_{zz} = 2[I_{yy} + A(S/2 + C_{yy})^2]$ or $= 2 \times [430.6 \times 10^4 + 5366(S/2 + 24.4)^2] = 2 \times 10008 \times 10^4$

$\Rightarrow S = 218.4\ mm$

Let us keep the channels at a spacing of 220 mm

Spacing of battens:

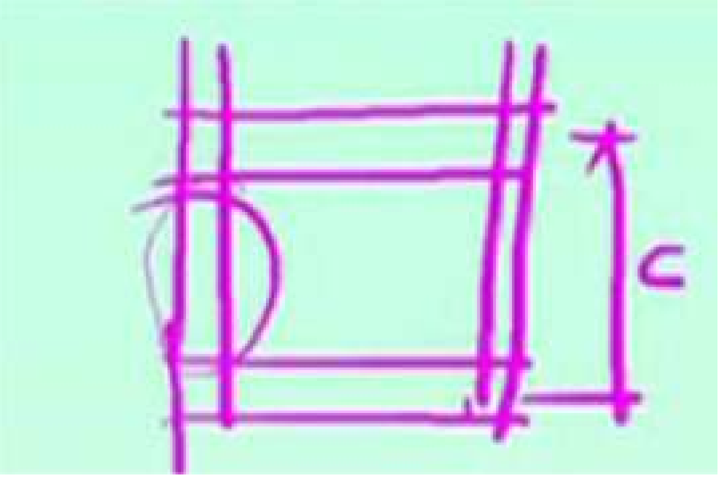
As per clause **7.7.3 of IS 800: 2007,**

$C/r_{yy} < 0.7\ \lambda$

$C < 0.7 \times \lambda \times r_{yy} = 0.7 \times 80.53 \times 28.3 = 1595.3\ mm$

Also C/r_{yy} <. 50 or $C < 50 \times 28.3 = 1415\ mm$

Hence, provide battens at a spacing of 1400 mm.



Size of end battens (cl. 7.7.2.3 of IS 800: 2007):

Provide 20 mm bolts.

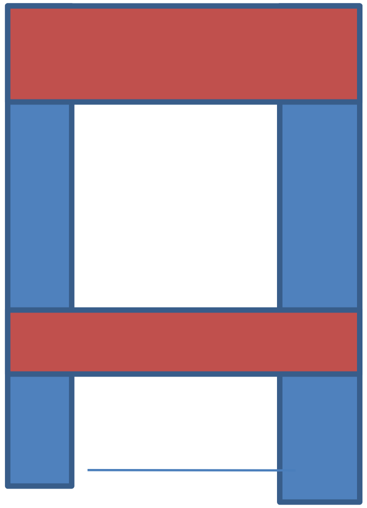
Edge distance . 1.5×*hole diameter* ***[Cl. 10.2.4.2 IS 800:2007]*** . 1.5×*(20+2)=33 mm*

Effective depth S+$2*C_{yy}$=220+2×*24.4=268.8 mm > 2×100 mm*

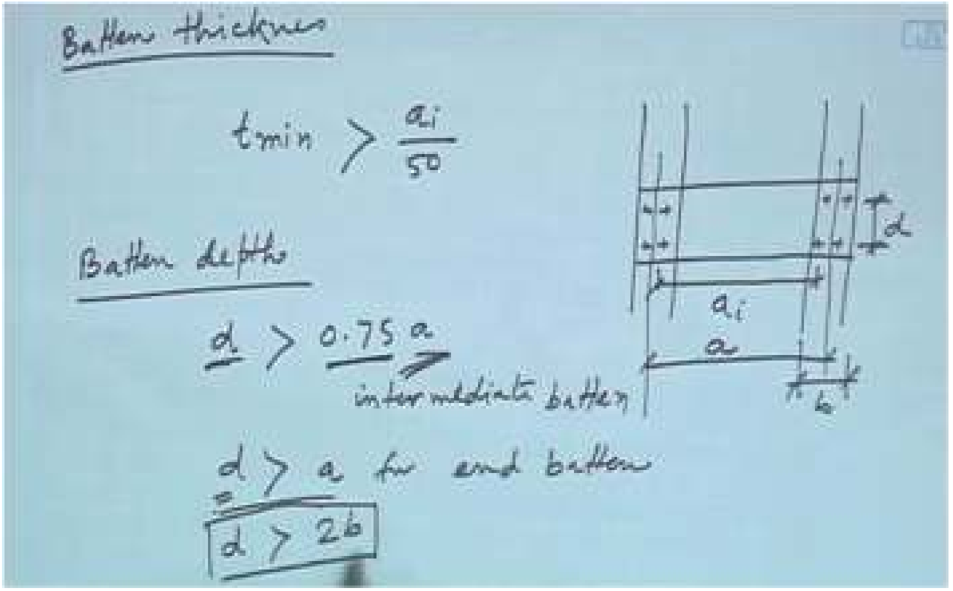
Hence, chosen effective depth is safe.

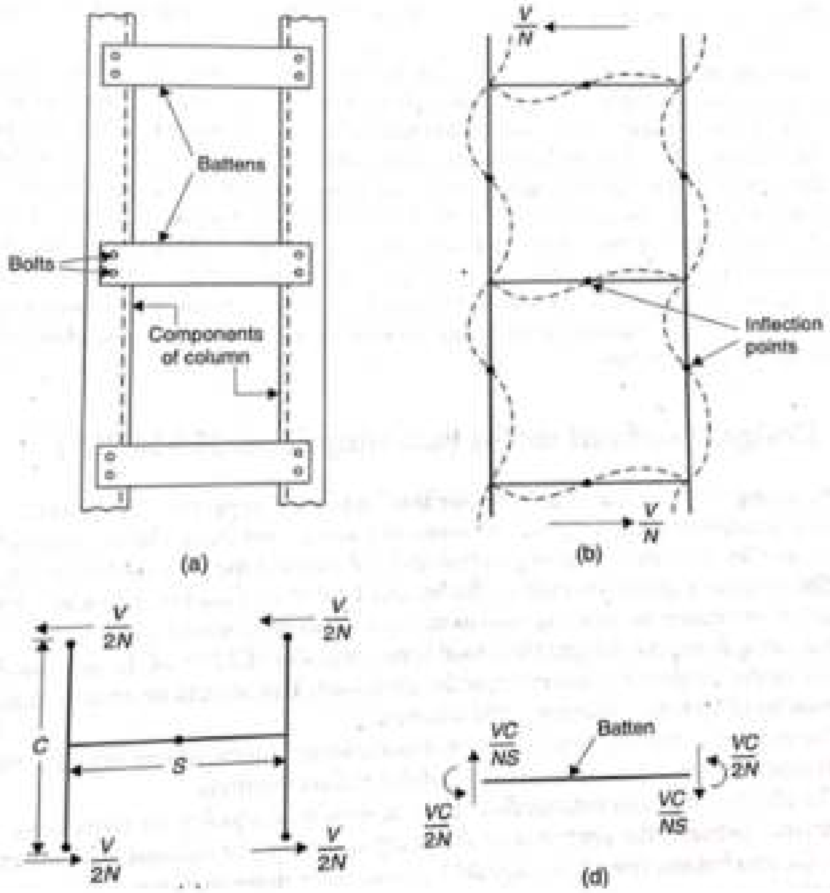
Overall depth 268.8+2×*33=334.8 mm*

Required thickness of batten (1/50)×*(220+2×50)=6.4 mm*



- Length of batten 220+2×*100=420 mm*
- Provide 420×340×8 mm end batten plates.





Battens
Bolts
Components
of column
(a)
V/N
Inflection
points
V/N
(b)
V/2N
V/2N
C
S
V/2N
V/2N
VC/NS
Batten
VC/2N
VC/2N
VC/NS
(d)

- **Size of intermediate battens (cl. 7.7.2.3 of IS 800: 2007):**

Effective depth ¾ $\times(220+2\times C_{yy})=3/4\times(220+2\times24.4)=201.6$ *mm*

$> 2\times100 = 200$ *mm*

Hence adopt an effective depth of 210 mm

Overall depth .210+2×*33=276 mm*

Therefore, provide a 420×300×8 mm batten plates @1400 mm c/c.

Design forces:

Transverse shear, $V_t= (2.5/100)\times P= (2.5/100)*(1150\times10^3)=28750$ N

Longitudinal shear $V_l=V_tC/NS$

Spacing of battens, C = 1400 mm

N = No of parallel planes of battens = 2

S = minimum transverse distance between the centroid of the bolt/weld group

(220+2×*50)=320 mm*

$V_l = 28750 \times 1400/(2 \times 320) = 62891$ N

Moment, $M = V_t C/2N = 28750 \times 1400/(2 \times 2) = 10.06 \times 10^6$ N-mm

Check

i) **For end battens**

Shear stress . $62891/340 \times 8 = 23.12$ MPa $< [250/(\sqrt{3} \times 1.1)] = 131.22$ Mpa

Bending stress = $6M/t\,d^2 = 6 \times 10.06 \times 106/8 \times 340^2 = 65.27$ MPa $< 250/1.1 = 227.27$ MPa

- Hence safe.

ii) **For intermediate battens**

Shear stress . $62891/300 \times 8 = 26.2$ MPa < 131.22 MPa

Bending stress . $6 \times 10.06 \times 10^6/(8 \times 300^2) = 83.83$ MPa < 227.27 MPa

Hence safe.

Connection:

The connection should be designed to transmit both shear and bending moment.

Assuming 20 mm diameter bolts.

Strength of bolt in single shear

Assuming 20 mm diameter bolts.

Strength of bolt in single shear= $A_{nb} \times f_{ub} / \sqrt{3} \times \gamma_{mb}$=

$0.78 \times (\pi \times 20^2/4) \times 400/(\sqrt{3} \times 1.25) \times 10^{-3} = 45.27$ kN

Minimum pitch, p = 2.5d=2.5×20=50 mm

Minimum end distance, e = 1.5 d0 =1.5×22=33 mm

Provide p = 60 mm and e = 35 mm

k_b is smaller of 35/(3×22), 60/(3×22)-0.25, 400/410, 1

K_b = 0.53

Strength of bolt in bearing .2.5 $k_b\, d t\, f_u / \gamma_{mb}$

$=(2.5 \times 0.53 \times 20 \times 8 \times 410/1.25)^* \times 10^{-3} = 69.5$ kN

Hence, strength of bolt = 45.27 kN

Number of bolts required = $62891/ 45.27 \times 10^3 = 1.39$

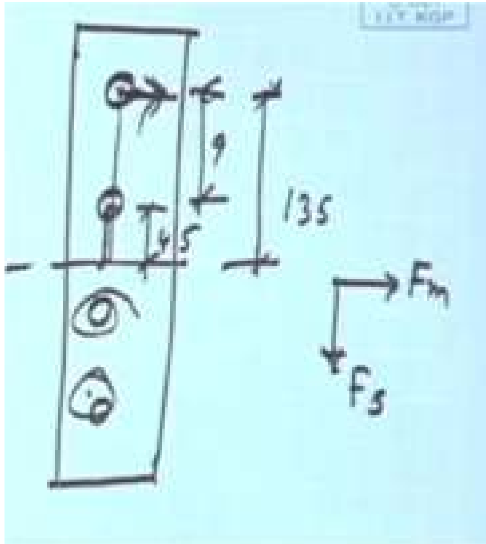
Let us provide four bolts to take account the stresses due to bending moments as well.

- ***Check for combined action: For end battens***

Force in each bolt due to shear . 62891/4

=15723 N

Pitch provided = (D-2e)/3= (340-2 × *35)/3 = 90 mm.*



Now $\Sigma r^2 = 2[(90/2)^2+(90+90/2)^2) = 2[452+1352] = 40500\ mm^2$

Force due to moment .

$Mr/\Sigma r^2=10.06\times10^6\times135/40500$

=33533 N

Resultant force =sqrt(157 23^2+33533 ^{2})=37036 N

.37 kN <45.26 kN

Hence safe.

Check for combined action: For intermediate battens

Force in each bolt due to shear = 62891/4=15723 N

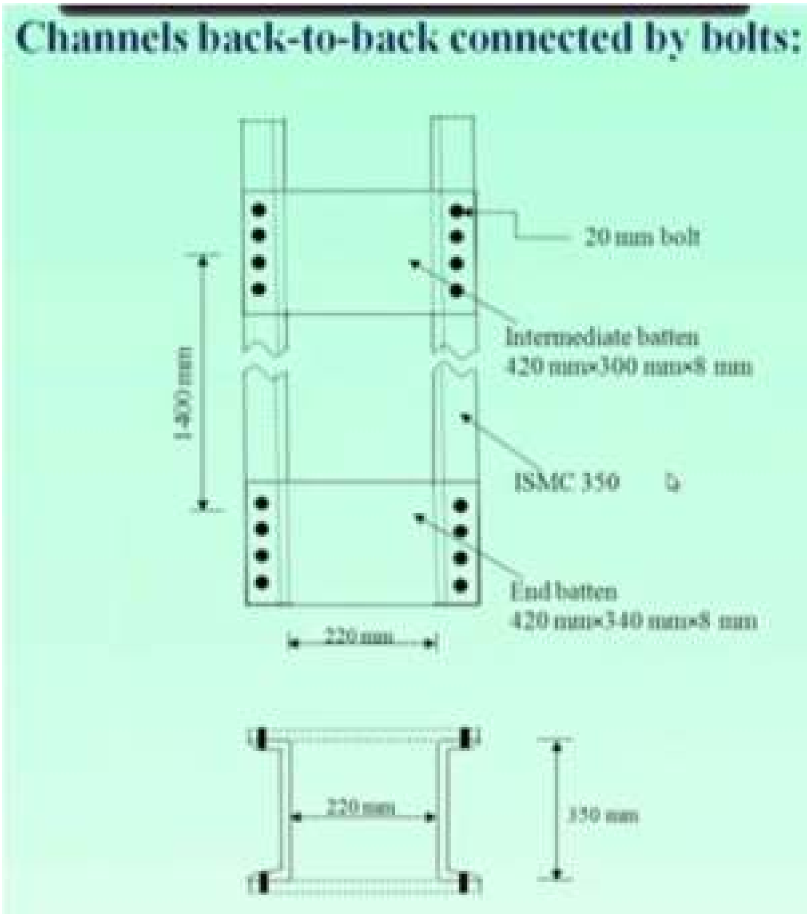
Pitch provided = (D-2e)/3= $(300-2 \times 35)/3 = 77$ *mm.*

$\Sigma r^2 = 2[(77/2)^2+(77+77/2)^2) = 2[38.5^2+115.5^2] =29645\ mm^2$

Force due to moment . $Mr/\Sigma r^2=10.06\times10^6\times115.5/29645=39195$ N

Resultant force =R=sqrt(157 $23^2+39195^2$)=42231N=42.23 kN < 45.26 kN

- Hence safe.



BATTEN WITH WELD CONNECTION

Example: A batten column of 10-m long is carrying a factored load of 1150 kN. The column is restrained in position but not in direction at both ends. Design a built up column using channel sections placed back to back. Design batten plates using weld connection.

Solution:

Design of column:

$P=1150\ kN\ .1150\times10^{3}\ N$

$L=1.0\times10\times10^{3}=10000\ mm$

Let design axial compressive stress for the column be 125 MPa

Required area . $1150\times10^{3}/125=9200\ mm^{2}$

Let us try two ISMC 350 @ 413 N/m

Relevant properties of ISMC 350 [Table II SP 6 (1): 1964]

$A=5366\ mm^{2}$, $r_{zz}=136.6\ mm$,

$r_{yy}=28.3\ mm$ $t_{f}=13.5\ mm$

$I_{zz}=10008\times104\ mm^{4}$ $I_{yy}=430.6\times10^{4}\ mm^{4}$

$c_{yy}=24.4\ mm$ $b=100\ mm$

Area provided .$2\times5366=10732\ mm2$

L/rzz=10000/136.6=73.21

The effective slenderness ratio, $(KL/r\ e$=1.1×73.21

=80.53<180 *; ok*

For $(KL/r\ )e$,=80.53 , f_y=250 MPa and buckling class c, the design compressive stress, from Table 9c of IS 800: 2007 f cd=135.2 Mpa

Therefore load carrying capacity=P= $A_e\ *f_{cd}$=10732×$135.2\times10-3$=1451 kN > 1150 kN, OK

Spacing of channels:

$2\ I_{zz}=2[I_{yy}+A(S/2+C_{yy})^2]$or $2\times10008\times10^4=2\times[430.6\times10\ 4+5366(S/2+24.4)^2]$

$\Rightarrow S=218.4\ mm$

Let us keep the channels at a spacing of 220 mm

Spacing of battens:

As per clause **7.7.3 of IS 800: 2007,**

$C/r_{yy}<0.7\ \lambda$

$C<0.7\times\lambda\times r_{yy}=0.7\times80.53\times28.3=1595.3\ mm$

$C/r_{yy}<$ 50 or $C<50\times28.3=1415\ mm$

Hence, provide battens at a spacing of 1400 mm.

Size of end battens (cl. 7.7.2.3 of IS 800: 2007):

Overall depth of batten $=220+2\times C_{yy}=220+2\times 24.4=268.8 \approx 270\ mm$

Required thickness of batten $=1/50\times 220=4.4\ mm$

Adopt battens with the thickness of 6-mm

Let provide a 70 mm overlap of battens on channel flange for welding.

[Overlap > 4 $t = 4 \times 6 = 24\ mm$] OK

Length of batten $=220+2\times 70=360\ mm$

Provide 360×270×6 mm end batten plates.

Size of intermediate battens (cl. 7.7.2.3 of IS 800: 2007):

Overall depth $3/4\times(220+2\times C_{yy})=3/4\times(220+2\times 24.4)=201.6\ mm$

$> 2\times 100 = 200\ mm$

Hence adopt overall depth of 220 mm

Therefore, provide a 360×220×6 mm batten plates.

Design forces:

Transverse shear, $Vt= [2.5/100]\times P= 2.5/100]\times 1150\times 10^3=28750$ N

Longitudinal shear $V_l=V_t*C/NS$

Spacing of battens, C = 1400 mm

N = No of parallel planes of battens = 2

S = minimum transverse distance between the centroid of the bolt/weld group

= $(220+2\times 50)=320$ *mm*

$\therefore V_l=28750\times 1400/[2\times 320]=62891$ N

- Moment, $M=V_tC/2N =28750\times 1400/[2\times 2]=10.06\times 10^6$ N-mm

Check

i) **For end battens.**

Shear stress . $62891/[270\times 6] =38.82$ MPa < $(250/(\surd 3\times 1.1)=131.22$ MPa

Bending stress . $6M/t\ d^2 =6\times 10.06\times 106/[6\times 270^2]=138$ MPa $<250/1.1=227.27$ MPa

Hence safe.

b) **For intermediate battens.**

Shear stress = $62891/ 20\times 6$

$=4\ 7\ .\ 64$ *MPa < 131.22 MPa*

Bending stress = $6\times 10.06\times 106/[6\times 202\ ^2] =2\ 07.85$ *MPa < 227.27 MPa*

Hence safe.

Design of weld:

Welding is done on all the four sides as shown in the figure.

Let *t=. throat thickness of weld***.**

$$I_{zz}=2\times\left[\frac{70\times t^3}{12}+(70\times t)\times\left(\frac{220}{2}\right)^2\right]+\frac{2\times t\times 220^3}{12}$$

Neglecting the term $2\times\frac{70\times t^3}{12}$ being insignificant.

Therefore, $I_{zz}=346.87\times 10^4 t$ mm^4

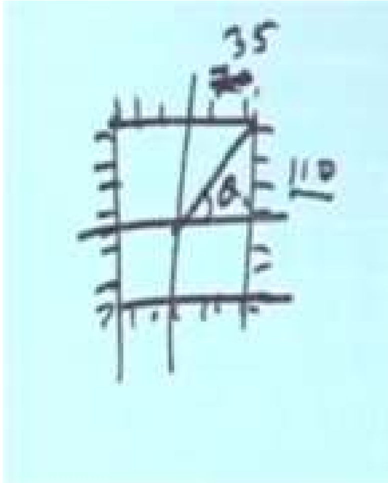
$$I_{yy}=2\times\left[\frac{t\times 703}{12}\right]+2\times\frac{220\times t^3}{12}+2\times 220\times t\times\left(\frac{70}{2}\right)^2$$

Neglecting the term $2\times\frac{220\times t^3}{12}$ being insignificant.

Therefore, $I_{yy}=59.62\times 10^4 t$ mm4

$I_p = I_{zz} + I_{yy} = 346.87 \times 10^4\, t + 59.62 \times 10^4\, t = 406.4 \times 10^4 t$ mm

$r = sqrt[(220/2)^2 + (70/2)^2] = 115.43$ mm

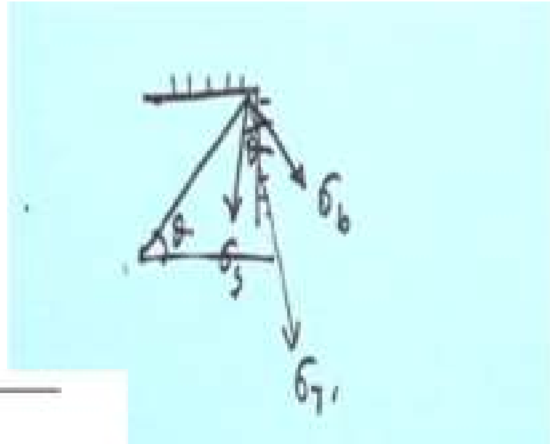


$\cos\theta = 35/115.43 = 0.30$

Direct shear stress (cl. 10.5.9 of IS 800:2007)

$62891/(2 \times 70 + 2 \times 220)\, t = 108.43t$ N/mm^2

Shear stress due to bending moment = $10.06 \times 10^6 \times 115.43/406.49 \times 10^4\, t = .\ 285.67\, t$ N/mm^2



Combined stress due to shear and bending

$= SQRT[(108.43t)^2 + (285.67t)^2 + 2 \times (108.43t) \times (285.67t) \times 0.3 = 334.59t < 410\sqrt{3} \times 1.25 = 189.4$

$$\sqrt{\left(\frac{108.43}{t}\right)^2 + \left(\frac{285.67}{t}\right)^2 + 2 \times \left(\frac{108.43}{t}\right) \times \left(\frac{285.67}{t}\right) \times 0.3}$$

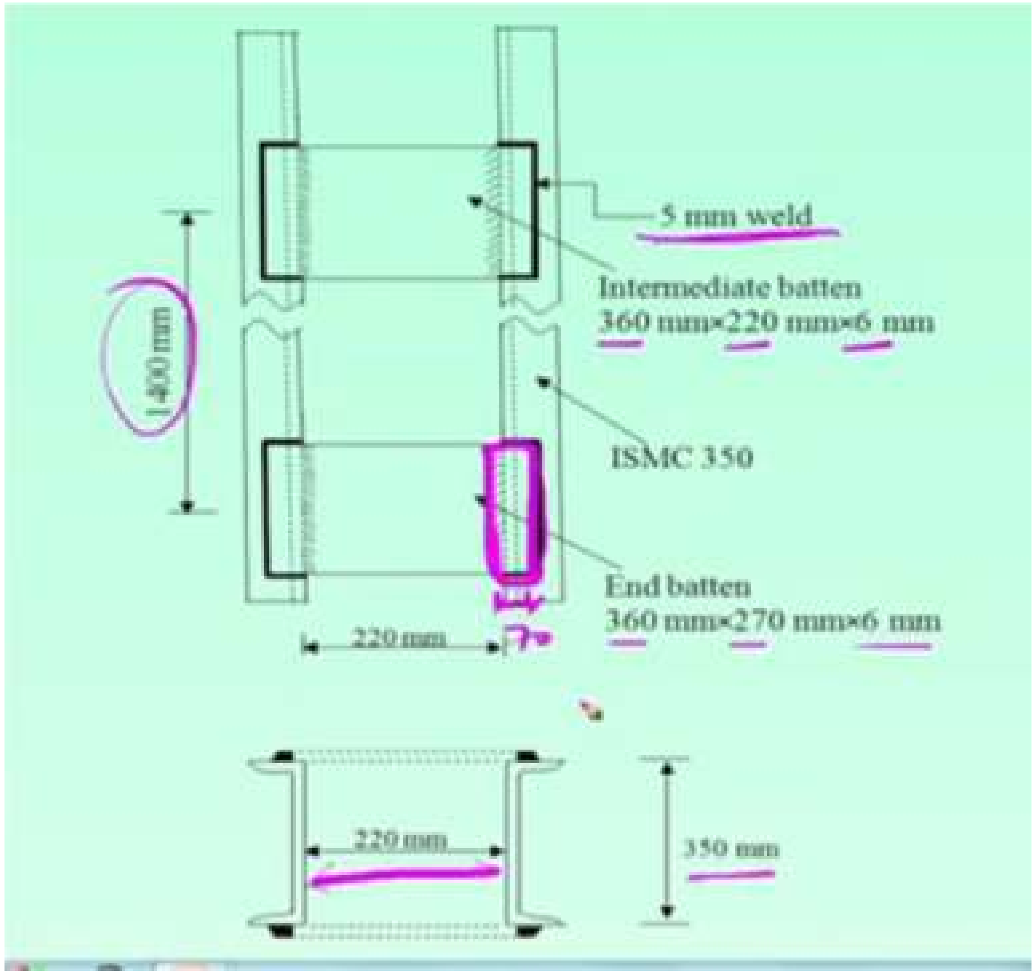
$$\frac{334.59}{t} < \frac{410}{\sqrt{3} \times 1.25} = 189.4$$

t=1.77 mm

Size of weld . 1.77/0.7=2.5 mm

The size of weld should not be less than 5 mm for 13.5 mm flange.

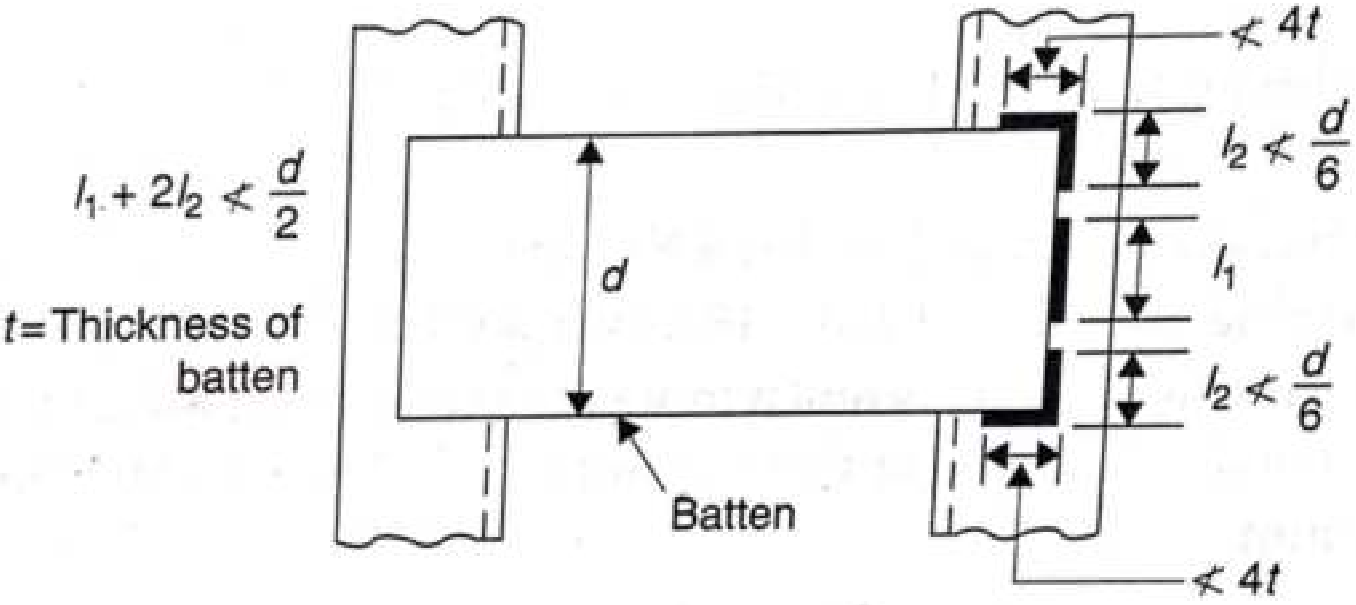
Hence provide a 5 mm weld to make the connection



End connections (Clause 7.7.4):

Design the end connections to resist the longitudinal shear force V_l and the moment M as calculated in earlier step.

- For welded connection: Lap > $4t$
- Total length of weld at edge of batten >$D/2$
- Length of weld at each edge of batten < 1/3 total length of weld required.
- Return weld along transverse axis of column < $4t$.



DESIGN OF COLUMN SPLICE

Dr.G.C. BEHERA

Basically when a joint is provided in the length of member is called splice. When the length of column is more than the available length of steel section, in such cases we use splice joint. So in many cases we have seen the available length of rolled steel section in the market is less than the required length of the column, so in that case we need to joint those together concentrically so that the load is transferred from one section to another section.

In case of multi storey building where the columns are provided along its height we have seen the columns section, size is required less because the load coming to the column across the height is gradually increasing towards the ground. Therefore we need to accommodate the column section size larger towards the ground level. And as a result we need to change the section size across the height and so that the economic design can be done, in such cases we have to provide splices between two floors to join two unequal sections.

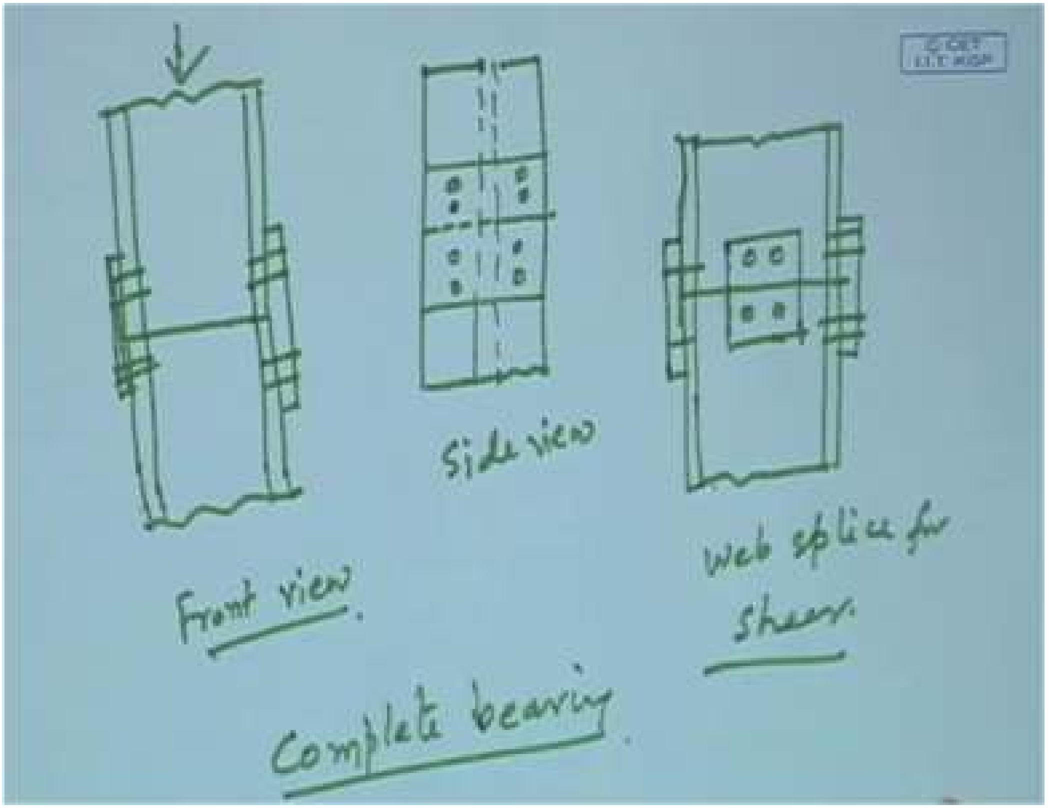
So basically if a compressive member is loaded concentrically we should not provide any splice, means theoretically we do not need to provide any splice but load is never axial and truly it is not axial and real column has to resist the bending due to the eccentricity of the load, therefore we have to provide the splice.

SPECIFICATION

For design of splices we can see that when the ends of the compression members are faced for complete bearing over the whole area, these should be spliced to hold the connected members accurately in position and to resist any tension when bending is present. Say for example these two columns are joined by the spliced, so basically to hold the two members properly we need to connect these members through splice.

When such members are not faced for complete bearing, splices should be designed to transmit all forces to which these are subjected, means sometimes it may be faced complete bearing or it may not be faced complete bearing. In case of complete bearing the whole area, then it should be spliced just for to transfer the load from upper storey to lower storey right.

And splices are basically designed as a short column.


Front view
Side view
Web splice for shear.
Complete bearing.

DESIGN STEPS

Step-1:

For axial compressive load the splice plates are provided on the flanges of the two column sections to be spliced.

If the column has machined ends, the splice is designed only to keep the columns in position and to carry tension due to the bending moment to which it may be subjected. The splice plate and the connection should be design to carry 50% of the axial load and tension.

If the ends are not machined, the splice and connections are design to resist the total axial load and any tension, if present due to the bending moment.

The load for the design of splice and connection due to axial load,

$Pu1=Pu/4$ (For machined ends)

$Pu1=Pu/2$ (For non machined ends)

Where, *Pu is the axial factored load.*

The load for the design of splice and connection due bending moment,

$Pu2=Mu/\text{lever arm}$

Where, *lever arm is the c/c distance of the two splice plates and*

Mu is the factored bending moment.

So then we will calculate total P_u on the splice plate, which will be $P_{u1} + P_{u2}$.

Step-2:

Splice plates are assumed to act as short columns (with zero slenderness ratio). So these plates will be subjected to yield stress (*f y).*

Step-3:

The cross-sectional area of the splice plate is calculated by dividing the appropriate portion of the factored load coming over the splice by the yield stress.

c/s area required=*Pu1+Pu2/f y*

Step-4:

The width of splice plate is usually kept equal to the width of the column flange.

Width of splice *bf (width of flange)*

The thickness of the splice plate can be found by dividing the c/s area of the plates with its width. Therefore the thickness of the splice plate can be found by dividing the cross sectional area with its width.

- **Step-5:**

Nominal diameter of bolts for connection is assumed and the strength of the bolt is computed.

Step-6:

In case of bearing plate is to be designed between two column sections, the length and width of the plate are kept equal to the size of lower-storey column and the thickness is computed by equating the ultimate moment due to the factored load to the moment of resistance of plate section.

Example 5.12: A column ISHB 300 @ 576.8 N/m is to support a factored axial load of 500 kN, shear force of 120 kN and bending moment of 40 kNm. Design the splice plate and connection using 4.6 grade bolts. Use steel of grade Fe 410.

Solution:

For steel of grade Fe 410: f_u=410 MPa, f_y=250 MPa

For bolts of grade 4.6: f_{ub}=400 MPa

Partial safety factors for material: **(Table 5 IS 800:2007)**

$\gamma_{m0}=1.10$ $\gamma_{mb}=1.25$

The relevant properties of ISHB 300 @ 576.8 N/m are **(Table I, SP 6-1)**

$A=7485\ mm^2$ $b_f=250\ mm$,

$t_f=10.6\ mm$ $t_w=7.6\ mm$

Assume the ends of the column sections to be machined for complete bearing. As the column ends are flush, it is assumed that 50% of the load is transferred directly and 50% is transferred through the splice and fastenings. Therefore, The direct load on each splice plate .50*% of 500 /*2 =125 kN

Load on splice due to moment =*Mu/lever arm*==40**10*^{3}*/(*300+6)=130.72 kN

(Assuming 6 mm thick splice plate, the lever arm=300+6 mm)

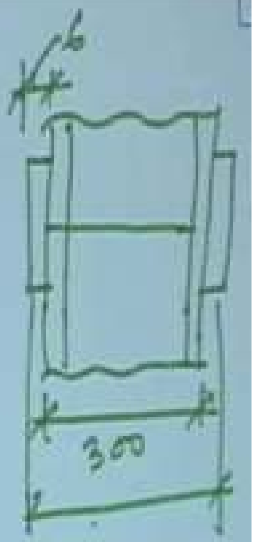
Total design load for splice, *Ps=125+130.72=255.72 kN*

Sectional area of splice plate required=*Ps/f y*=255.72**10*^{3}*/*250=1022.9 mm^2

Width of the splice plate should be kept equal to the width of the flange.
Here, the width of the splice plate =250 mm
Hence, thickness of splice plate= 1022.9/250==4.09 mm $\nless$6 mm
Provide a 250×6 mm splice plate.
The length of the splice plate depends upon the number of bolts in vertical row.
Let us provide 20 mm diameter bolts of grade 4.6.
Strength of 20 mm diameter bolt in single shear **(cl. 10.3.3, IS 800:2007)**
Anb(f ub/√3)/*γmb*=245**(400/*√3)1.25=45260 N

$$P_{u2} = \frac{M_u}{\text{lever arm}} = \frac{40 \times 10^3}{300 + 6}$$

$$= 130.72 \text{ kN}$$



$$P_s = 125 + 130.72$$

$$= 255.72 \text{ kN}$$

$$A = \frac{P_s}{f_y} = \frac{255.72 \times 10^3}{250}$$

$$= 1022.9 \text{ mm}^2$$

$$b_f = 250$$

$$t = \frac{1022.9}{250} = 4.09 \not< 6 \text{ mm}$$

$$t = 6 \text{ mm}$$

$$250 \times 6$$

$$\phi = 20 \text{ mm}$$

$$V_{dsb} = \frac{A_{nb} \times \frac{f_{ub}}{\sqrt{3}}}{\gamma_{mb}} = \frac{245 \times \frac{400}{\sqrt{3}}}{1.25} = 45.26 \text{ kN}$$

Strength of bolt in bearing 2.5 $k_b\, d\, t\, f_u / \gamma_{mb}$ ***(cl. 10.3.4, IS 800:2007)***
For 20 mm diameter bolts the minimum edge distance,
$e = 1.5 . d_0 = 1.5 . (20+2) = 33$ *mm*
The minimum pitch, $p = 2.5 * 20 = 50$ *mm*
Let us provide an edge distance (e) of 35 mm and pitch (p) of 60 mm.
kb is smaller of ($e/3d_0$)=35/(3*22)=0.53),($p/3d_0$)−0.25=[60/(3*22)]−0.25=0.66) ,
(f_{ub}/f_u=400/410=0.98) and 1.0
Hence *kb=0.53*

Strength in bearing=.2.5**0.53*20*6* 410/*1.25=52150N

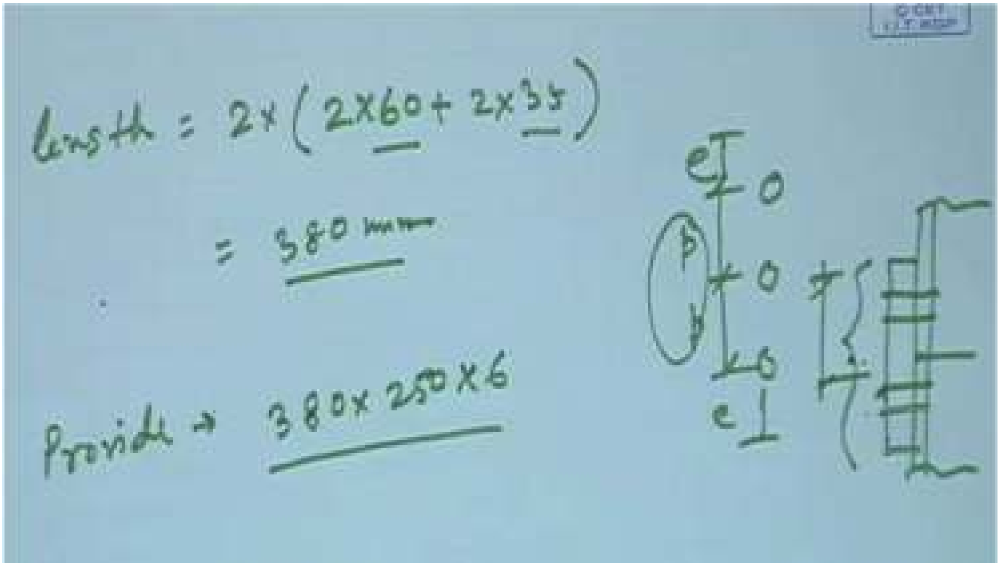
Hence, the strength of bolt (*Bv) =45.26 kN*

Number of bolts required, *n=Ps/Bv*=255.72/45.26=5.65≈ *6*

Provide 6 bolts for each splice.

Length of the splice plate .2*.(2.60+2.35)=380 mm*

Provide a splice plate 380×250×6 mm on column flanges as shown in the figure.



WEBSPLICE FOR SHEAR

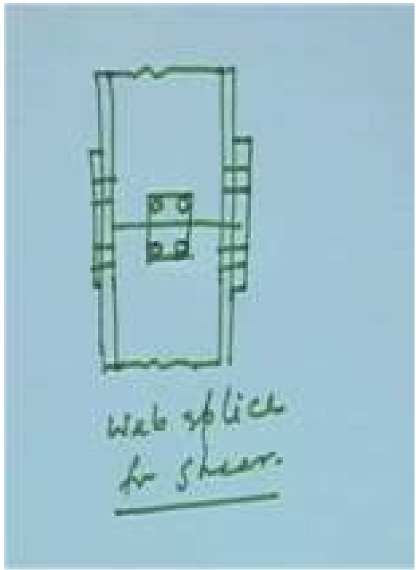
A column ISHB 300 @ 576.8 N/m is to support a factored axial load of 500 kN, shear force of 120 kN and bending moment of 40 kNm. Design the splice plate and connection using 4.6 grade bolts. Use steel of grade Fe 410.

Solution:

For steel of grade Fe 410: f_u=410 MPa, f_y=250 Mpa

For bolts of grade 4.6: f_{ub}=400 MPa

Partial safety factors for material: **(Table 5 IS 800:2007)**



γ_{m0}=1.10 γ_{mb}=1.25

The relevant properties of ISHB 300 @ 576.8 N/m are **(Table I, SP 6-1)**

A=7485 mm^2 b_f=250 mm,

t_f=10.6 mm t_w=7.6 mm

Assume the ends of the column sections to be machined for complete bearing. As the column ends are flush, it is assumed that 50% of the load is transferred directly and 50% is transferred through the splice and fastenings. Therefore, The direct load on each splice plate .50% of 500 /2 =125 Kn

Load on splice due to moment =*Mu/lever arm*==40*10^3/(300+6)=130.72 kN

(Assuming 6 mm thick splice plate, the lever arm=300+6 mm)

Total design load for splice, *Ps=125+130.72=255.72 kN*

Sectional area of splice plate required=*Ps/f y*=255.72*10^3/250=1022.9 mm^2

Width of the splice plate should be kept equal to the width of the flange.

Here, the width of the splice plate =250 mm

Hence, thickness of splice plate= 1022.9/250==4.09 mm $\nless$6 mm

Provide a 250×6 mm splice plate.

The length of the splice plate depends upon the number of bolts in vertical row.

Let us provide 20 mm diameter bolts of grade 4.6.

Strength of 20 mm diameter bolt in single shear **(cl. 10.3.3, IS 800:2007)**

$A_{nb}(f_{ub}/\sqrt{3})/\gamma_{mb}$=245*(400/√3)1.25=45260 N

Strength of bolt in bearing 2.5 $k_b\, d t f_u/\gamma_{mb}$ ***(cl. 10.3.4, IS 800:2007)***

For 20 mm diameter bolts the minimum edge distance,

e=1.5.d0=1.5.(20+2)=33 mm

The minimum pitch, *p=2.5.20=50 mm*

Let us provide an edge distance (e) of 35 mm and pitch (p) of 60 mm.

kb is smaller of(*e/*3*d0)*=35/(3**22)=0.53),(p/*3*d0)*−0.25=[60/(3**22)]−0.25=0.66) ,*

(*f ub/f u*=400/410=0.98) and 1.0

Hence *kb=0.53*

Strength in bearing=.2.5*$0.53*20*6*410/$1.25=52150N

Hence, the strength of bolt (Bv) $=45.26$ *kN*

Number of bolts required, $n=Ps/Bv=255.72/45.26=5.65 \approx 6$

Provide 6 bolts for each splice.

Length of the splice plate .2.*(2.60+2.35)=380 mm*
Provide a splice plate 380×250×6 mm on column flanges as shown in the figure.

Splice plates for shear:
The splice plate for the shear force is provided on the web. A pair of splice plate (one on each side of web) are provided.
Let us provide 20 mm diameter bolts of grade 4.6.
Strength of bolt in double shear .45.26*.2=90.52 kN*
Strength in bearing= 2.5 $k_b\, d t f_u/\gamma_{mb}$
Where, $k_b=0.53$ *(taking e=35 mm and p=60 mm),*
t=7.6 mm (web thickness)
Strength in bearing = $[2.5*0.53*20*7.6*410/1.25].10^{-3}=66.06$ *kN*
Hence, strength of 20 mm bolt .66.06 kN
∴

Shear force in the web, *V=120 kN*

Number of bolts required= 120/66.06=1.8≈ *2*

Provide 2, 20 mm diameter bolts on each side of the splice.

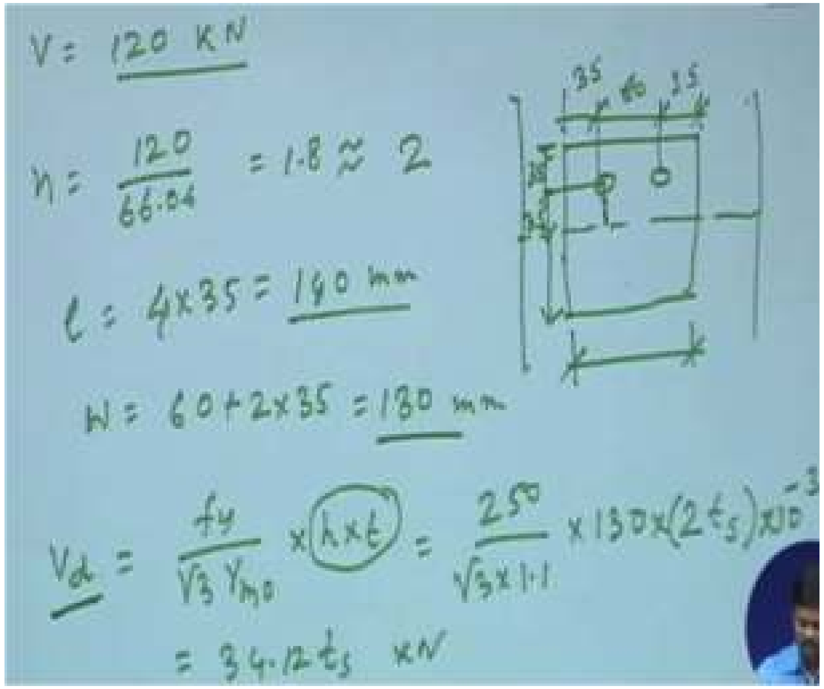
Length of the splice plate .4*.35=140 mm*

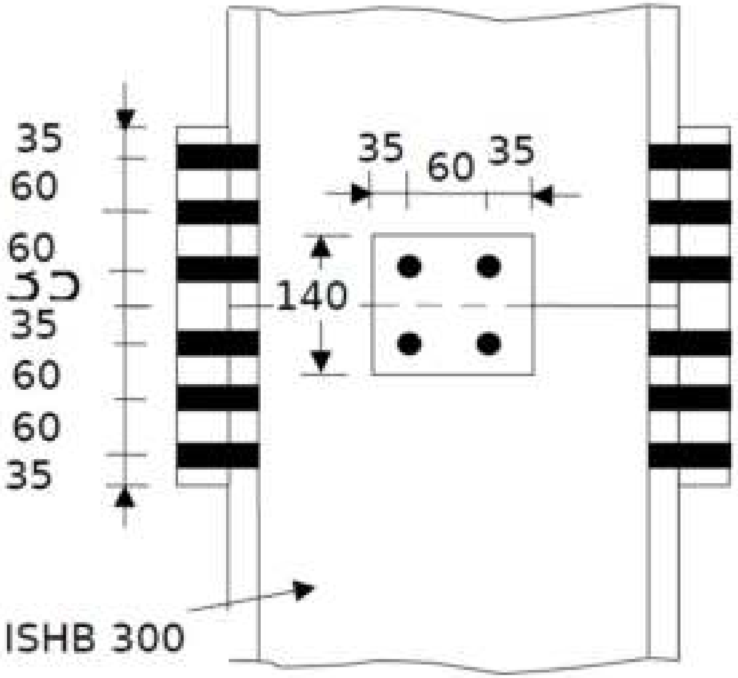
Width of the splice plate .60+2*.35=130 mm*

Design shear strength of splice plate (cl. 8.4, IS 800:2007),

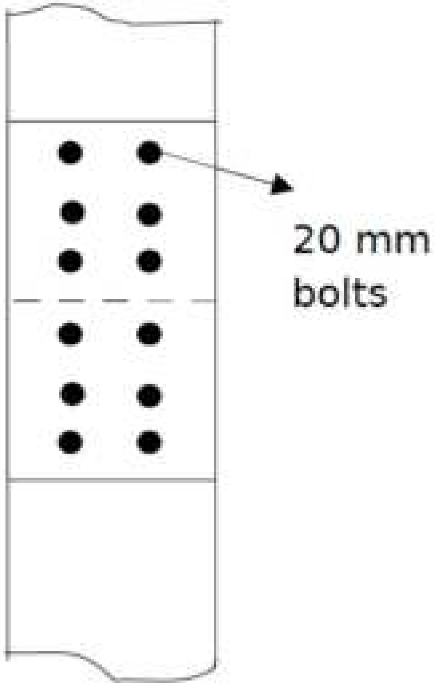
*Vd=[f y/(√3*γm0)]*h.t=[* 250/(√3**1.1)]*130.(2 ts).10−3=34.2ts*

Now, *Vd>V*
or 34.12*ts>120*
Thickness of the splice plate required,
*ts= 120/*34.12=3.52 mm ≮6 mm





Front view



Side view

COLUMN BASES

Dr. G.C.BEHERA

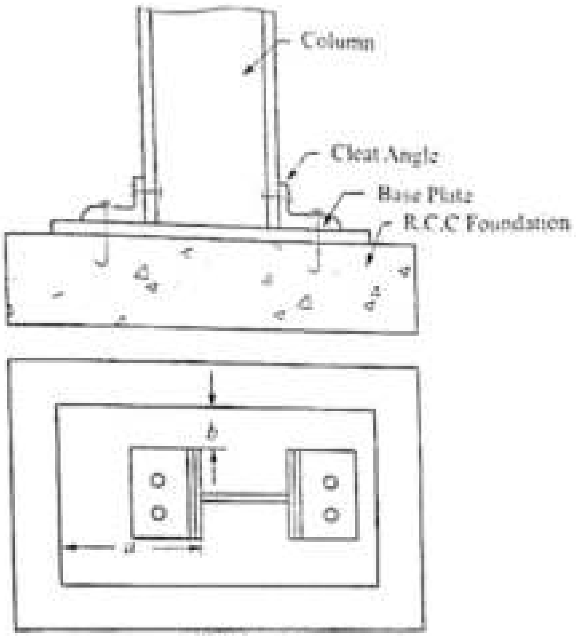
COLUMN BASES

Column bases transmit load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that intensity of bearing pressure on the foundation block is within the bearing strength. There are two type of column bases

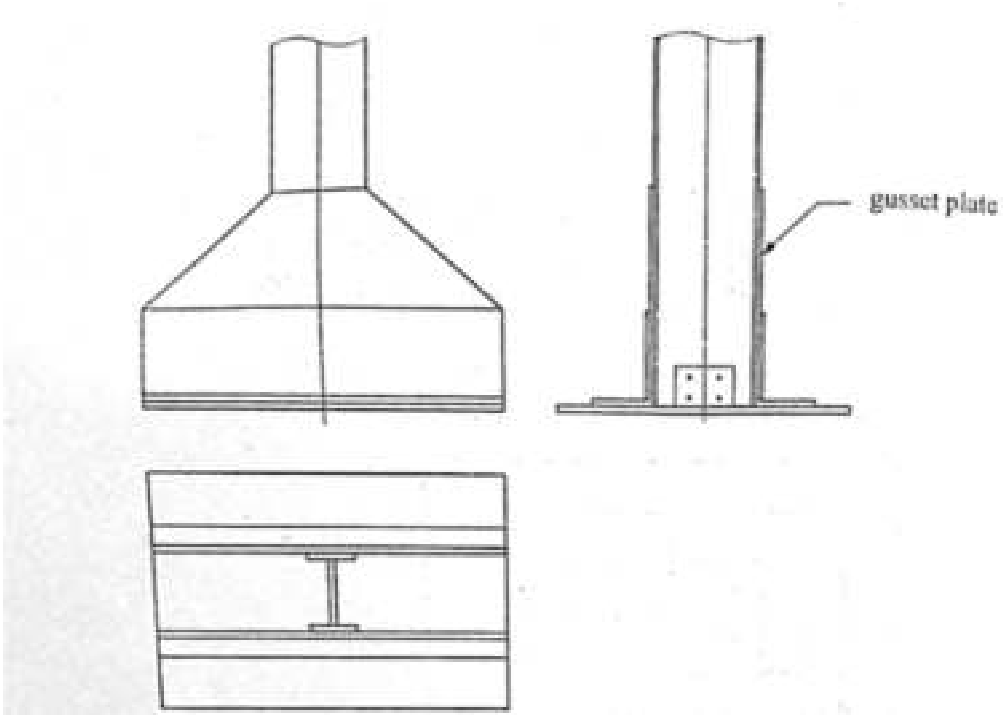
1. Slab Base
2. 2. Gusseted Base

Slab Base is used when the load carried by column is less. In this the column is directly connected to the base plate. The load transferred to the base plate through bearing.

When the load on column is more gusseted base is used. The load transferredto the base partly through bearing and partly through gusset.



Slab Base



Design of Slab Base:

1. Find the bearing strength of concrete=0.6fck
2. Area of base plate required:Pu/0.6f_{ck} Pu= Factored Load in Column
3. Select the size of the base plate, keep projections a and b equal if possible
4. Find the intensity of pressure w=Pu/Area of base plate

1. Thickness of base plate= $t_s = \left[\frac{2.5w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}\right]^{0.5} > t_f$

This formulae is derived from plate theory taking μ=0.3

Here t_f is the thickness of flange and t_s= thickness of base plate

Connect the base plate to the foundation with 20 mm diameter bolt and 300 mm anchor bolts. If bolted connection is to used for connecting column to base plate , use 2 ISA 65X65, 6 mm.

If weld is used , use fillet weld.

Design a slab base for a column ISHB 300@ 577 N/m carrying an axial factored load 1000 kN. Use M20 grade concrete provide welded connection between column and base plate.

SOLUTION:

Bearing strength of concrete=0.6fck=0.6*20=12N/mm^2

Factored Load=1000 kN

Area of base plate required=1000*1000/12=83333.33 mm^2

Provide 360 mmX 310 mm plate Area of base plate=111600 mm^2

Pressure=p=w=1000*1000/111600=8.96 N/mm^2

Projection a =(360-300)/2=30 mm

Projection b =(310-250)/2=30 mm

Thickness of bearing plate=t_s= $t_s = \left[\frac{2.5w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}\right]^{0.5} > t_f$

$$t_s = \left[\frac{2.5 * 8.96(30^2 - 0.3 * 30^2)1.1}{250}\right]^{0.5} \quad 7.88\,mm > t_f = 10.6\,mm$$

As t_s is less than t_f , take t_s=12 mm

Use 360*310*12 mm plate

Use 20 mm dia. 300 mm long bolt to anchor the plate.

Total length available=2(250+250-7.6+300-2*10.6)=1542.4 mm

Strength of weld= $1000^3 = \frac{400}{\sqrt{3}} * \frac{1}{1.25} * 0.7s * (l_e), \qquad s * l_e = 7543.8$

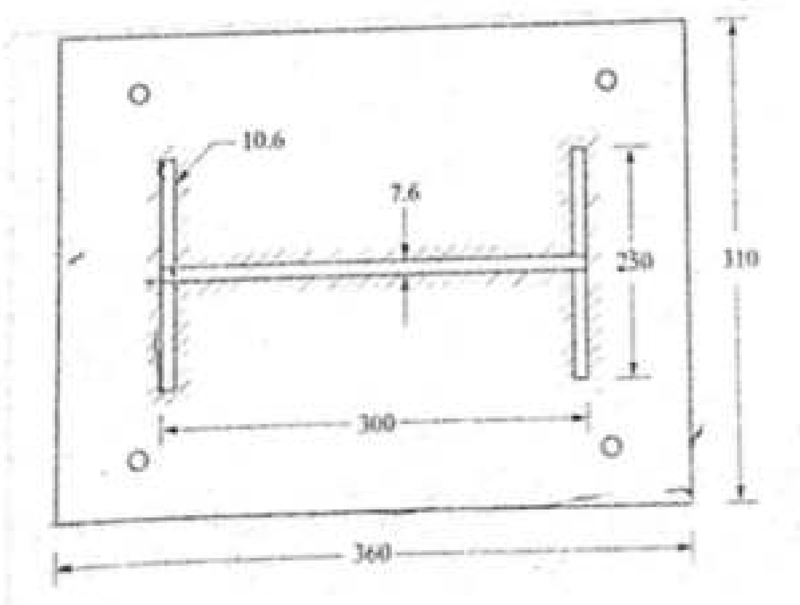
Taking weld size= 6mm le=1257.3 mm

Total length available=1542.4 mm

Effective length= Total length-2*s* no. of end returns=1542.4-2*6*12=1398.4 mm> 1257.3 mm.

So, 6 mm weld is sufficient.

Total length available=2(250+250-7.6+300-2*10.6)=1542.4 mm



GUSSETED BASE

- DESIGN PROCEDURE:

1. Area of base plate: Pu/(0.6*fck)
2. Assume thickness of gusset plate 16 mm.
3. Size of the gusset angle is assumed so that its vertical leg can accommodate two bolts in one vertical line. Other leg can accommodate one bolt. Thickness of the angle is taken as equal to the thickness of gusset plate.
4. Width of the gusset base is kept such that it will project outside the gusset angle. Length=Area of plate/width
5. As ends of column are machined, 50% load will be transferred by fastenings.
6. Thickness of the base plate is computed by flexural strength of the critical section.

Problem: Design a gusseted base for a column ISHB 350 @710 N/m with two plates 450*20 mm carrying a factored load 3600 kN. The column is to be supported on concrete pedestal to be built on M20 concrete.

Solution:

f_{ck}= 20 N/mm^2

A=P_u/(0.6*fck)=3600*1000/(0.6*20)=300000 mm^2

Selecting ISA 150*115,15 mm thick angle and 16 mm thick gusset plate

Minimum width required=350+2*20+2*16+2*115=652 mm

Use 700 width plate

Length=300000/700=428.57 mm

Use 700*600 mm plate.

Pressure under plate=3600*1000/(700*600)=8.57 N/mm^2

As cover plate, gusset plate and angle section are joined, they are acting as one member

So projection beyond this point XX,

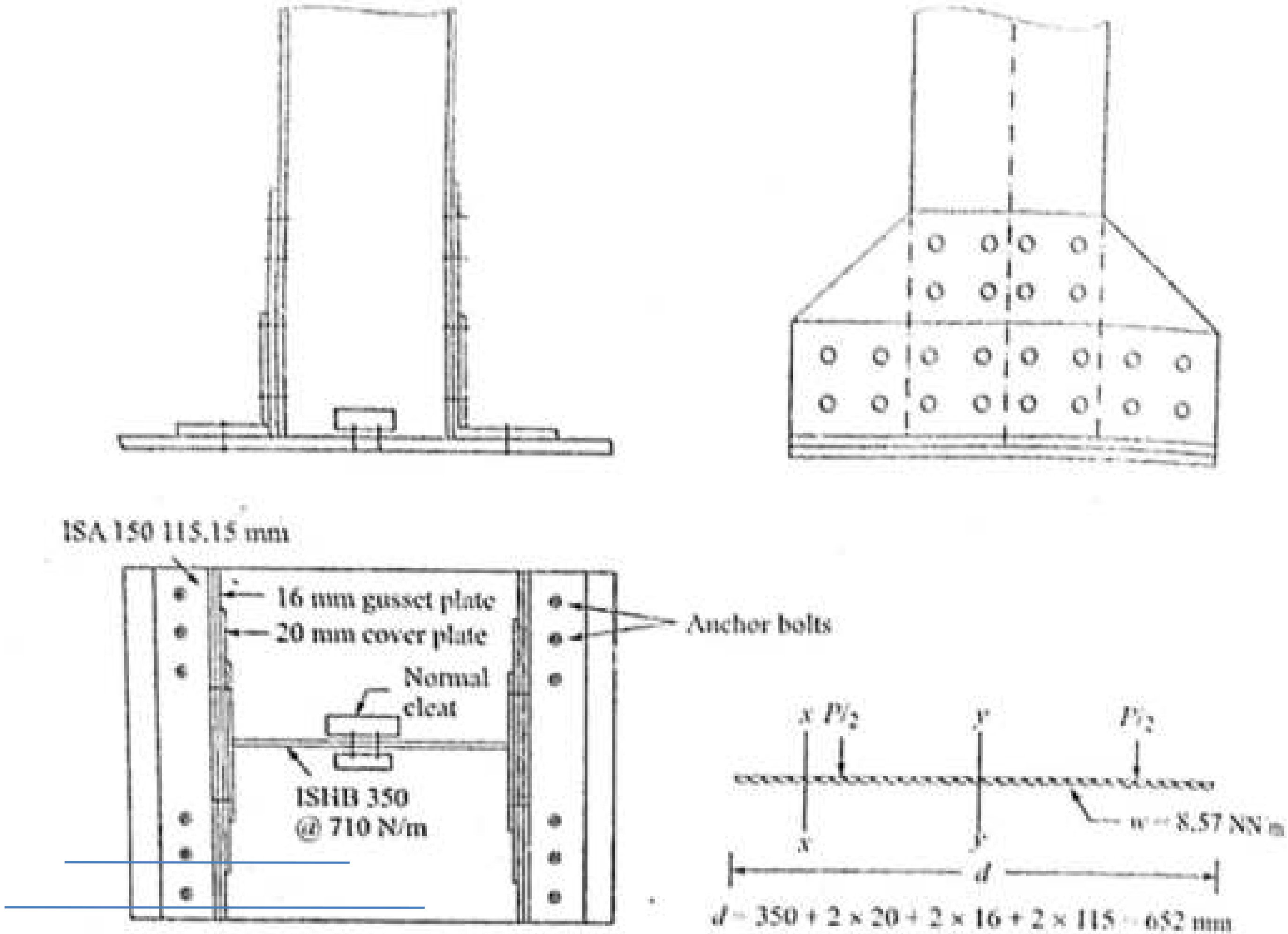
a=[700-(350+20*2+2*16+2*15)]/2=124 mm

BM at X-X per mm width=8.57*124*124/2=65886 Nmm

Moment at Y-Y, assuming the column load transferred between gusset and angle

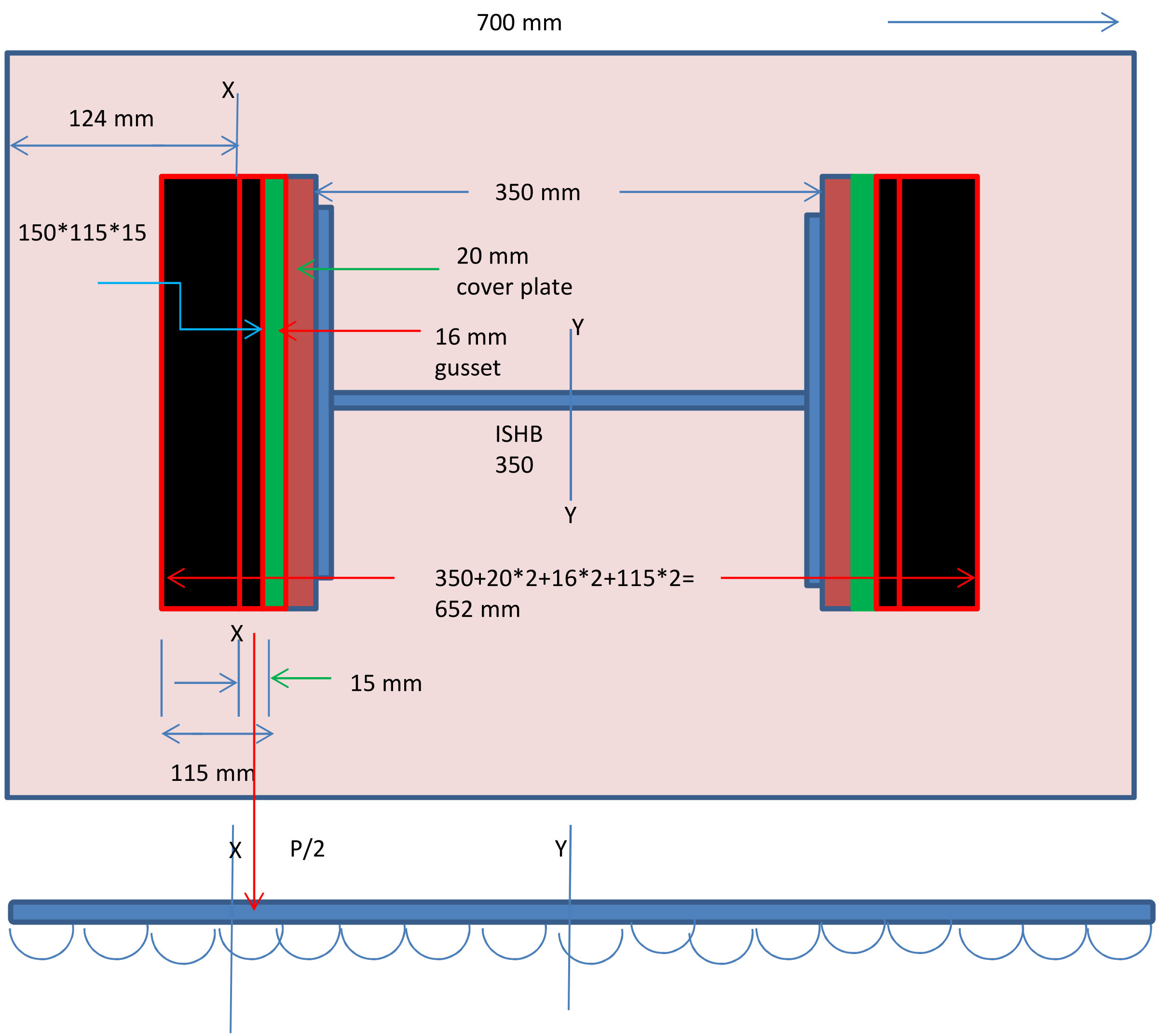
$M_{yy} = 8.57 * \frac{350^2}{2} - \frac{700}{2} * 8.57 * (\frac{350}{2} + 20 + \frac{16+15}{2})$=106482Nmm

(3600*1000/2)/600=700*8.57/2

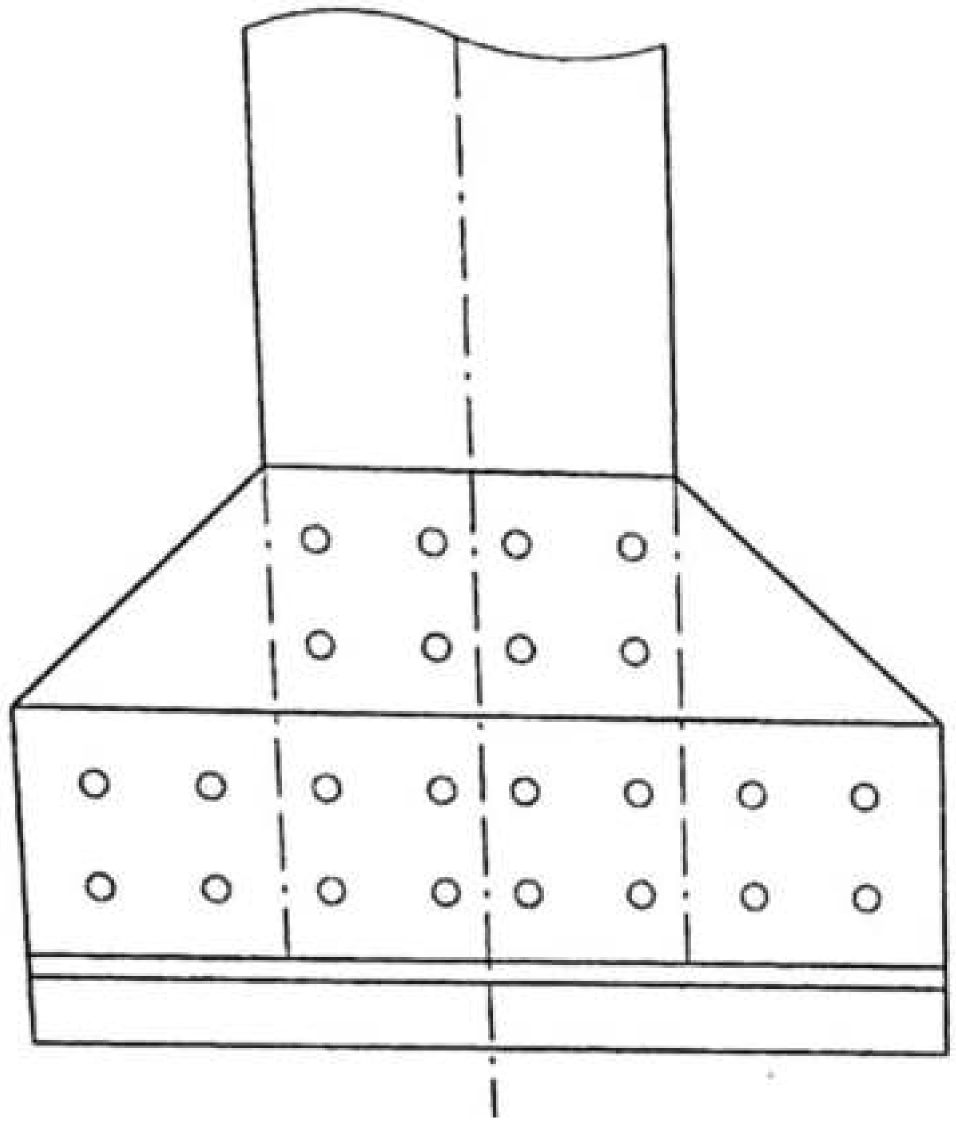


Minimum width required=350+2*20+2*16+2*115=652 mm

- DRAWING



700 mm
X
124 mm
350 mm
150*115*15
20 mm
cover plate
16 mm
gusset
Y
ISHB
350
Y
350+20*2+16*2+115*2=
652 mm
X
15 mm
115 mm
X
P/2
Y



Design Moment=106482Nmm

Moment of resistance $=1.2*(Ze)*f_y/\gamma_{m0}=$

$1.2*(1/6)t^2*1*(f_y/1.1)=106482$, t=48.4 mm

Use 700*600*56 mm thick plate.

Assuming column are faced for complete bearing, Design load on gusset plate for one face=1/2 of (50% 0f 3600 kN)=900 kN

Using 24 mm shop bolt=For shearing Value= $\frac{0.78*\pi*24^2}{4}*\frac{400}{\sqrt{3}}*\frac{1}{1.25}=65.192\ kN$

Bearing value is always higher

No of bolts=900 kN/65.192 kN=13.8 nos

Provide 16 no. of Bolts

Use another 8 bolts to cleat angle to connect gusset plate

BASE SLAB WITH BENDING

Design a slab base for a beam-column SC 250 to transfer a factored axial compression of 750 kN and a factored bending moment of 75 kNm. The grade of the steel is E250 and the grade of the concrete pedestal is M_{30}.

Eccentricity, $e = 75/750 = 0.1\,\text{m}$

The length of the base plate is kept equal to or more than $6e$ so that the entire plate is subjected to downward pressure and no tension develops in the anchor bolts.

$\therefore$ The length of the base plate, $L = 6 \times 0.1 = 0.6\,\text{m}$

The bearing strength of the concrete $= 0.6\, f_{ck} = 0.6 \times 30 = 18\,\text{MPa}$

It is assumed that the bearing pressure varies linearly below the base plate.

The maximum bearing pressure, $p_{max} = \dfrac{P}{BL}\left(1 + \dfrac{6e}{L}\right)$

$$\therefore \quad 18 = \frac{750 \times 10^3}{B \times 600}\left(1 + \frac{600}{600}\right)$$

or $$B = 139\,\text{mm}$$

The width of the base plate to be provided

= the width of the flange of SC 250 + the projections on either side

= 250 + 2 × 100 = 450 mm

Therefore, a rectangular base plate of 600 mm × 450 mm as shown in Figure 9.9 may be provided.

For this base plate, $p_{\max} = \dfrac{750 \times 10^3}{450 \times 600}\left(1 + \dfrac{600}{600}\right) = 5.5\,\text{MPa}$

The variation bearing pressure is shown in Figure 9.9.

The maximum bending moment in the base plate at Sec. X–X

$$= \frac{3.9 \times 175^2}{2} + \frac{1}{2} \times 175 \times (5.5 - 3.9) \times \frac{2}{3} \times 175 = 76{,}052\,\text{Nmm/mm width}$$

$$1.2 Z_e f_y / \gamma_{m0} = 1.2 \times \left(\frac{1 \times t^2}{6}\right) \times \frac{250}{1.1} = 76{,}052$$

or $\quad t = 41\,\text{mm}$

A base plate of 50 mm thick may be provided. Column section may be welded.