DESIGN OF

STEEL STRUCTURES

Design of Compression Member

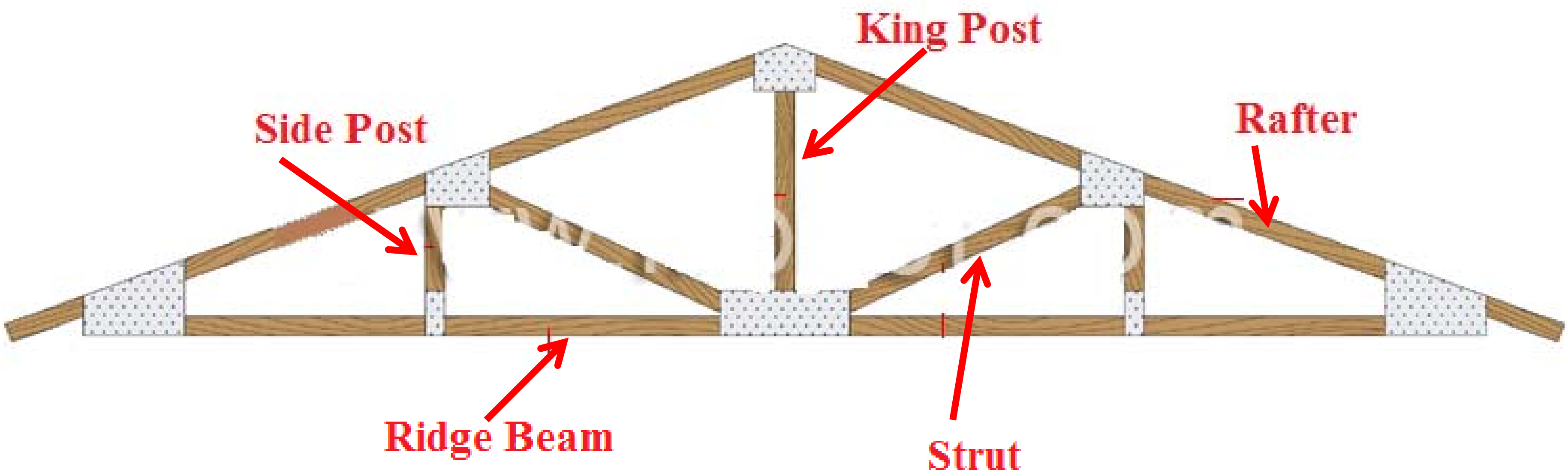
- A structural member carrying axial or eccentric compressive load is called compression member.

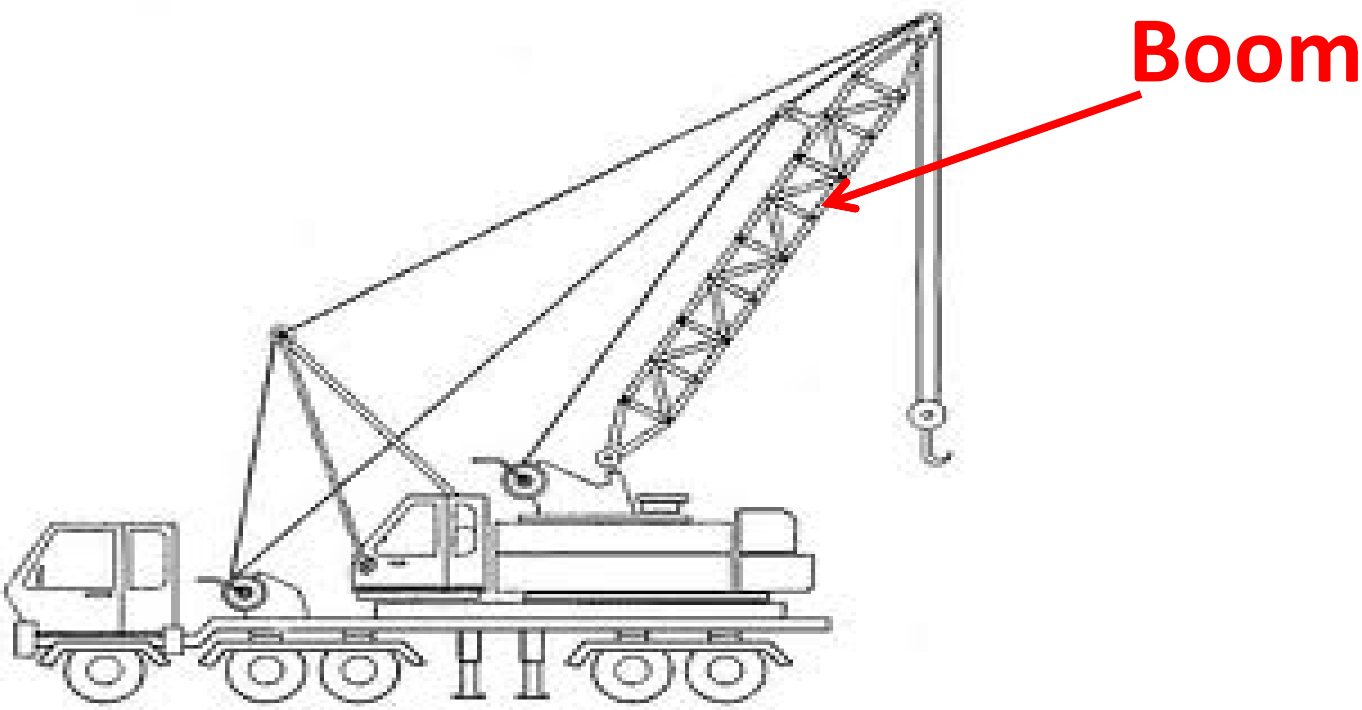
➢ Types of compression member :

- The vertical compression member in RCC building is termed as column, where as for steel structure it is called stanchion.

- The compression member in roof truss or bracing is called as strut.

- The principle compression member in a crane is called boom.


King Post
Side Post
Rafter
Ridge Beam
Strut


Boom

➢ Mode of failure of column :

- **Crushing failure:-** This type of failure occur in short column Such member has a critical load cause material failure.
- **Buckling failure :-** This type of failure occur in long column Such member has a critical load which cause elastic instability due to which the member fail.
- **Mixed mode of failure:-** The above two failure occur in the extreme cases. For all intermediate value of slenderness ratio the column fail due to combined effect. Most of the practical column fail in this mode.

Effective length (l_{eff}) :- It is the length between two adjacent Point of zero moments. Thus length depend upon end condition.

From, IS 800 : 1984, Clause 5.2.2

Where accurate frame analysis is not done, the effective length of a compression member in a given plane may be Determined by the procedure given in Appendix C. However, in most cases the effective length in the given plane assessed on the basis of Table 5.2, would be adequate. Effective length as given in Table 5.2 may also be adopted where columns directly form part of framed structures.

$$\beta_1 = \frac{\sum K_c}{\sum K_c + \sum K_b}$$

$$\beta_2 = \frac{\sum K_b}{\sum K_c + \sum K_b}$$

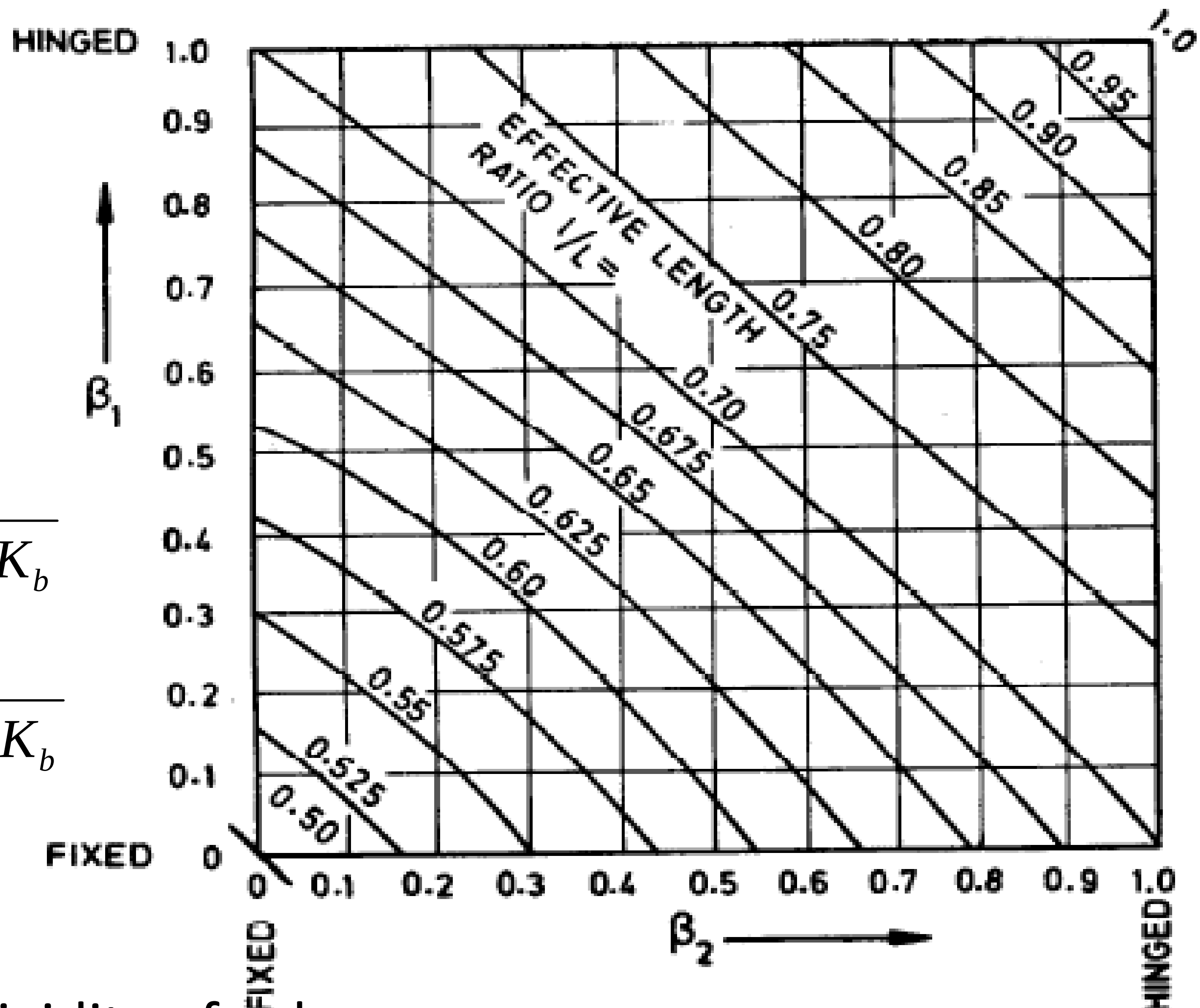


FIG. C-1 EFFECTIVE LENGTH RATIOS FOR A COLUMN IN A FRAME WITH NO SWAY

K_c – Flexural Rigidity of column

K_b – Flexural Rigidity of beam.

Effective Length of Columns

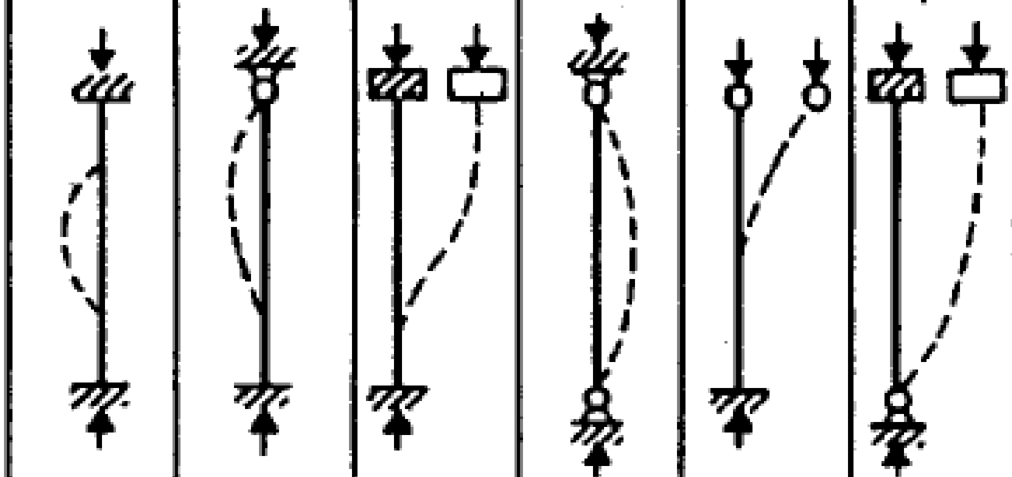
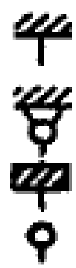
Effective length factors, K						
Buckled shape of column is shown by dashed line	(a) 	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code		Rotation fixed Translation fixed Rotation free Translation fixed Rotation fixed Translation free Rotation free Translation free				

TABLE 3.1 MAXIMUM SLENDERNESS RATIOS

SL No.	MEMBER	MAXIMUM SLENDER-NESS RATIO λ
(1)	(2)	(3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress due to loads other than wind or seismic forces occurs	180
iii)	A member subjected to compression forces resulting from wind/earthquake forces provided the deformation of such member does not adversely affect the stress in any part ofthe structure	250
iv)	Compression flange of a beam	300
v)	A member normally acting as a tie in a roof truss or a bracing system but subject to possible reverse of stress resulting from the action of wind or earthquake forces	350
vi)	Tension members (other than pretensioned members)	400

Buckling failure : Euler's Theory

Assumptions Made in Euler's Theory:

- The column is initially straight (it is not crooked)
- The load acting passes through centerline (axial compression)
- The slenderness of column is high (Long column)
- The cross-section of column is solid, with constant section throughout height of column
- The column material is homogeneous, isotropic, and elastic
- The boundary conditions are ideally met

- From moment curvature relation :

$$\frac{d^2 y}{dx^2} = -\frac{M}{EI}$$

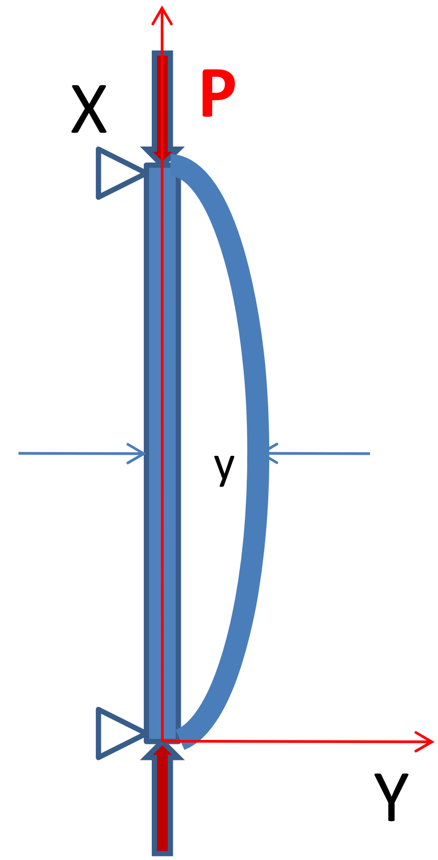
$$M = Py$$

Critical Load (P_c) $= \dfrac{\pi^2 EI}{l_{eff}^2}$

Critical stress (σ_c)

$$f_{cc}, or, \sigma_c = \frac{P_c}{A} = \frac{\pi^2 EI}{A l_{eff}^2} = \frac{\pi^2 E}{\lambda^2}$$

$$slenderness.Ratio(\lambda) = \frac{l_{eff}}{r}$$



Column design formula

Merchant Rankin formula

$$\frac{1}{f^{n}} = \frac{1}{f_{cc}^{n}} + \frac{1}{f_{y}^{n}}$$

$$f = \frac{f_{cc} \; f_{y}}{\left(f_{cc}^{n} + f_{y}^{n}\right)^{1/n}}$$

$$n = 1.4$$

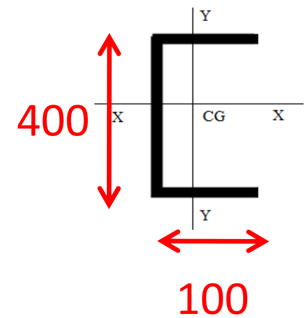
- In WSM method of design

Axial compression strength (σ_{ac}) = 0.6 f

For different grade of steel, σ_{ac} is given in Table – 5.1

A built-up column consists of two ISMC400@49.4Kg/m
And two plates of size 500x10 mm. the clear distance
Between back-to-back of channels is 200 mm. one plate
Is connected to each flange side. Determine the safe load
Carry capacity of the section if the effective length of the
Column is 5 m .

Sol:- Properties of ISMC 400:-



Thickness of web (t_w) = 8.6 mm

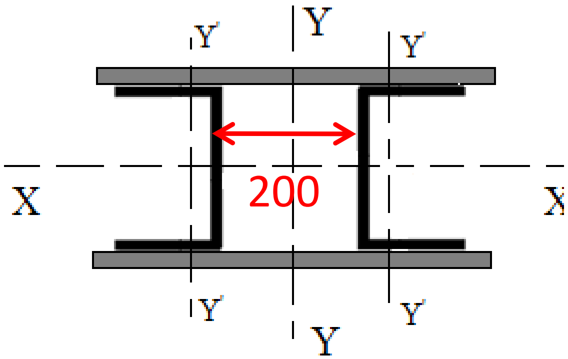
Thickness of flange (t_f) = 15.3 mm

Cross-section area of the section(A_{st}) = 6293 mm^2

Moment of inertia along X-X axis (I_{xx}) = 150828x10^3 mm^4

Moment of inertia along Y-Y axis (I_{yy}) = 5048x10^3 mm^4

For built-up section:



$$I_{xx} = 2\times150828\times10^3 + 2\left\{\frac{1}{12}\times500\times10^3 + 500\times10\times205^2\right\}$$

$$= 721.989\times10^6\,mm^4$$

$$I_{yy} = 2\times5048\times10^3 + 2\left\{6.293\times10^3\times124.2^2\right\} + 2\times\frac{1}{12}\times10\times500^3$$

$$= 421.576\times10^6\,mm^4 = I_{\min}$$

Minimum radius of gyration:

$$r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{421.576\times10^6}{2\times6.293\times10^3 + 2\times500\times10}}$$

$$= 136.62\,mm$$

Slenderness Ratio (λ) :-

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{5000}{136.62} = 36.59$$

From IS 800 : 1984, Table – 5.1, Pg - 39

$$\sigma_{ac} = 141.046 \ MPa$$

Safe load for the section (P)

$$P = A\sigma_{ac} = \left(2\times 6.293\times 10^{3} + 2\times 500\times 10\right)\times 141.046N$$
$$= 3185.665KN$$

➢ Bending stress :

- If compression flange is restrained laterally against buckling then permissible compressive or tensile bending stress is (Clause 6.2.1, Pg - 55)

$$\sigma_{bc} / \sigma_{bt} = 0.66 f_y$$

- For unrestrained compression flange (Clause 6.2.3)

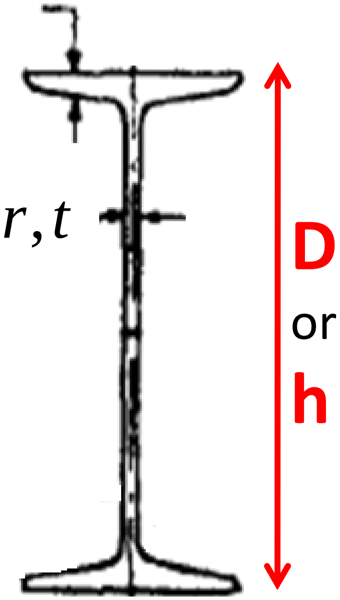
$$\sigma_{bc} = 0.66 \frac{f_{cb} \, f_y}{\left(f_{cb}^{\,n} + f_y^{\,n} \right)^{1/n}}$$

f_{cb} - Elastic critical stress in bending (Clause 6.2.4)

$$f_{bc} = K_1 \left(X - K_2 Y \right) \frac{C_2}{C_1}$$

$$\tau = K_1 t_f$$

$$t_w, or, t$$



Where,

$$Y = \frac{26.5 \times 10^5}{\left(l / r_y \right)^2} MPa$$

$$X = Y \sqrt{\left[1 + \frac{1}{20} \left(\frac{l \tau}{r_y D} \right)^2 \right]} MPa$$

$$Coefficient(K_1) = f(\psi)$$

$$\psi = \frac{A_1}{A_2}$$

C_1 , C_2 - respectively the lesser and greater distances from the section neutral axis to the extreme fibers.

A_1 - Total area of both flanges at the point of least bending moment.

A_2 - Total area of both flanges at the point of maximum bending moment.

$$Coefficien\ t(K_2) = f(\omega)$$

$$\omega = \frac{I_{yy,cf}}{I_{yy,f}}$$

$I_{yy,cf}$- *M*oment of inertia of the compression flange alone.
$I_{yy,f}$- *M*oment of inertia of the both flanges about its own axis parallel to the y-y axis of the girder,.

IS : 800 - 1984

TABLE 6.3 VALUES OF k_1 FOR BEAMS WITH CURTAILED FLANGES

(*Clause* 6.2.4)

ψ	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2	0·1	0·0
k_1	1·0	1·0	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2

NOTE — Flanges should not be reduced in breadth to give a value of ψ lower than 0·25.

TABLE 6.4 VALUES OF k_2 FOR BEAMS WITH UNEQUAL FLANGES

(*Clause* 6.2.4)

ω	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2	0·1	0·0
k_2	0·5	0·4	0·3	0·2	0·1	0	−0·2	−0·4	−0·6	−0·8	−1·0

Bending compression strength = σ_{bc}

For different grade of steel, σ_{bc} is given in Table – 6.1A to F

TABLE 6.1A MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL-FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with $f_y = 250$ MPa, $\frac{T}{t} > 2{\cdot}0$ or $\frac{d_1}{t} > 85$

$D/T \rightarrow$ $l/r_y \downarrow$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	160	160	159	159	158	158	158	158	158	157	157	157	157	157	157
45	159	158	157	157	156	156	156	155	155	155	155	155	154	154	154
50	158	157	156	155	154	154	153	153	152	152	152	151	151	151	151
55	157	155	154	153	152	151	150	149	149	148	148	148	148	147	147
60	156	153	152	150	149	148	148	146	145	145	144	144	144	143	143
65	154	152	150	148	147	145	144	143	142	141	140	140	139	139	139
70	153	150	148	146	144	142	141	139	138	137	136	135	135	135	134
75	152	148	145	143	141	139	138	136	134	133	132	131	130	130	129
80	150	147	143	141	138	136	135	132	130	128	128	126	126	125	125
85	149	145	141	138	136	133	132	128	126	124	123	122	121	120	120
90	147	143	139	136	133	130	128	124	122	120	119	117	116	115	115
95	146	141	137	133	130	128	125	121	118	116	114	112	111	110	110
100	145	140	135	131	128	125	122	117	114	112	110	108	107	105	105

➢ Combined Axial Compression and Bending :

From IS 800 :1984, Clause – 7.1.1, Pg - 90

- Members subjected to axial compression and bending shall be proportioned to satisfy the following requirements:

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left\{1-\frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right\}\sigma_{bcx}} + \frac{C_{my}\sigma_{bcy,cal}}{\left\{1-\frac{\sigma_{ac,cal}}{0.6 f_{ccy}}\right\}\sigma_{bcy}} \leq 1.0$$

$C_m = 0.85$ → Sway frame or Non-sway with ends are restrained against rotation.

$= 0.6 - 0.4\beta \geq 0.4$ → Non-sway and not subject to transverse loading

$= 1.0$ → Non-sway with ends are unrestrained against rotation

$$\beta = M_{min} / M_{max}$$

$$If, \frac{\sigma_{ac,cal}}{\sigma_{ac}} < 0.15, then$$

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma_{bcx,cal}}{\sigma_{bcx}} + \frac{\sigma_{bcy,cal}}{\sigma_{bcy}} \leq 1.0$$

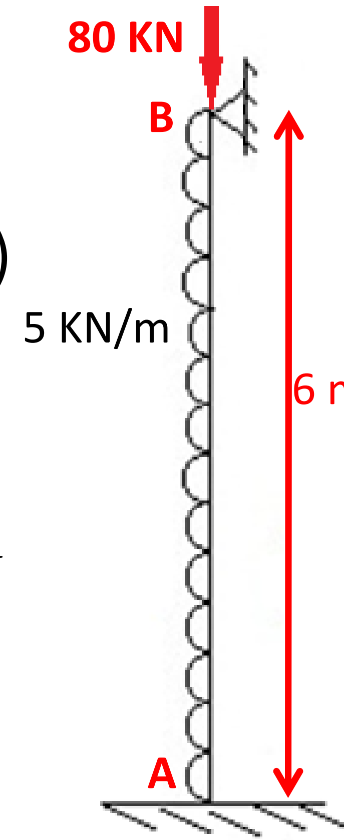
At support we can use following equation -

$$\frac{\sigma_{ac,cal}}{0.6 f_y} + \frac{\sigma_{bcx,cal}}{\sigma_{bcx}} + \frac{\sigma_{bcy,cal}}{\sigma_{bcy}} \leq 1.0$$

- Combined Axial Tension and Bending:

$$\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{btx,cal}}{0.66 f_y} + \frac{\sigma_{bty,cal}}{0.66 f_y} \leq 1.0$$

Design a steel column for the loading as shown in figure as per IS 800 :1984, steel is Fe250.



Sol :-

Effective length for the propped cantilever column (l_{eff})

$$l_{eff} = 0.8 \times l = 4.8m$$

Total Axial Load $= (selfWt. + 80) = (0.1 \times 80 + 80)KN = 88KN$

Maximum bending moment is at support "A" is :

$$M_A = \frac{wl^2}{8} = \frac{5 \times 6^2}{8} = 22.5KN - m$$

Let’s assume, $\sigma_{ac/bc}$ = 80 MPa

Thus,

Required sectional area and sectional modules of the column is:

$$A_{req} = \frac{A.L}{\sigma_{ac}} = \frac{88 \times 1000}{80} = 1100 \ mm^2$$

$$Z_{req} = \frac{M_A}{\sigma_{bc}} = \frac{22.5 \times 10^6}{80} = 281250 \ mm^3$$

Try with ISMB300@44.2Kg/m

- Check for axial compression:

$$\sigma_{ac,cal} = \frac{88 \times 10^3}{5626} = 15.64 MPa$$

$$\text{Slenderness ratio}\,(\lambda) = \frac{l_{eff}}{r_{\min}} = \frac{4.8 \times 1000}{28.4} = 169.01 < 180, OK$$

From IS 800 :1984, Table – 5.1, Pg - 39

Permissible axial compression stress is :

$$\sigma_{ac} = 36.6\,MPa > \sigma_{ac,cal}, OK$$

- Check for bending tension :

$$\sigma_{bt,cal} = \frac{M_A}{Z_{xx}} = \frac{22.5 \times 10^6}{573.6 \times 10^3} = 39.22\,MPa$$

$$\sigma_{bt} = 0.66\,f_y = 165\,MPa > \sigma_{bt,cal}, OK$$

- Check for bending compression :

$$\sigma_{bc,cal} = \frac{M_A}{Z_{xx}} = \frac{22.5\times10^6}{573.6\times10^3} = 39.22\,MPa$$

For, ISMB300@44.2Kg/m

$$\frac{t_f}{t_w} = \frac{12.4}{7.5} = 1.6533 < 2$$

$$\frac{d_1}{t_w} = \frac{241.5}{7.5} = 32.2 < 85$$

$$\frac{h}{t_f} = \frac{D}{\tau} = \frac{300}{12.4} = 24.193$$

$$\lambda = \frac{l_{eff}}{r_{yy}} = \frac{4.8\times1000}{28.4} = 169.01$$

From IS 800 1984, Table- 6.1B, Pg - 58

D/T → L_{eff}/r_y ↓	20	25	24.193
160	101	93	94.2912
170	98	89	90.4526

$$\sigma_{bc} = 90.8326 MPa > \sigma_{bc,cal}, OK$$

- Combined Axial Compression and Bending check:

$$\sigma_{ac,cal} = 15.64 MPa, \sigma_{ac} = 36.6 MPa$$

$$\sigma_{bc,cal} = 39.22 MPa, \sigma_{bc} = 90.83 MPa$$

For, ISMB300@44.2Kg/m

Radius of Gyration along X-X axis:

$$r_{xx} = 123.7mm \qquad \lambda_{xx} = \frac{l_{eff}}{r_{xx}} = \frac{4.8 \times 1000}{123.7} = 38.80$$

$$f_{cc,xx} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 2 \times 10^5}{38.8^2} = 1311.19MPa$$

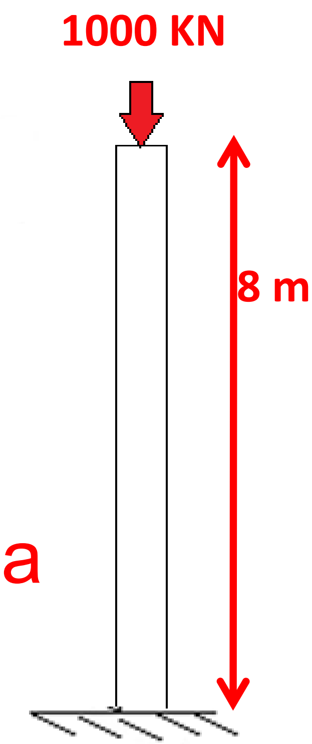
From interaction formula

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6 f_{cc,xx}}\right\}\sigma_{bcx}}$$

$$= \frac{15.64}{36.6} + \frac{0.85 \times 39.22}{\left\{1 - \frac{15.64}{0.6 \times 1311.19}\right\} \times 90.83}$$

$$= 0.801 < 1.0, OK$$

Design a steel compound column for the loading as shown in figure as per IS 800 :1984, steel is Fe250.



Sol -

Effective length for the column (l_{eff})

$$l_{eff} = 2 \times l = 16m$$

Assume slenderness ratio (λ) = 100 , σ_{ac} = 80 MPa

$$\lambda = \frac{l_{eff}}{r}$$

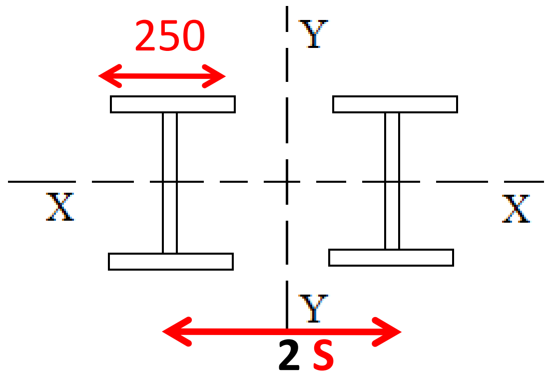
$$so\ ,\ r_{req} = \frac{l_{eff}}{\lambda} = \frac{16 \times 1000}{100} = 160\ mm$$

Required Radius of Gyration

$$A_{req} = \frac{P}{\sigma_{ac}} = \frac{1000 \times 10^3}{80} = 12500 mm^2$$

Required Sectional Area

Let's try with 2 X ISHB350@67.4Kg/m



$$r_{xx} = 149.3mm, A = 2 \times 8591mm^2$$

$$I_{xx} = 19159.7 \times 10^4 mm^4$$

$$I_{yy} = 2451.4 \times 10^4 mm^4$$

For $I_{xx} = I_{yy}$

$$2 \times 19159.7 \times 10^4 = 2\left\{2451.4 \times 10^4 + 8591 \times S^2\right\}$$

$$S = 139.46mm \approx 140mm$$

slenderness ratio (λ)

$$\lambda = \frac{l_{eff}}{r_{xx}} = \frac{16 \times 10^3}{149.3} = 107.6 < 180, OK$$

From IS 800 :1984, Table – 5.1, Pg - 39

$$\sigma_{ac} = 75MPa$$

$$\sigma_{ac,cal} = \frac{P}{A} = \frac{1000 \times 10^3}{17182} = 58.2MPa < \sigma_{ac}, OK$$

- **For compound column we have to provide either Batten or, Lacing :**

  - Design of Lacing:- It behave like truss member and will be under tension or compression.

    Single Lacing.

    Double Lacing.

IS 800 :1984, Clause 5.7.5, Pg - 51

$$\theta \in \{40^0, 70^0\}$$

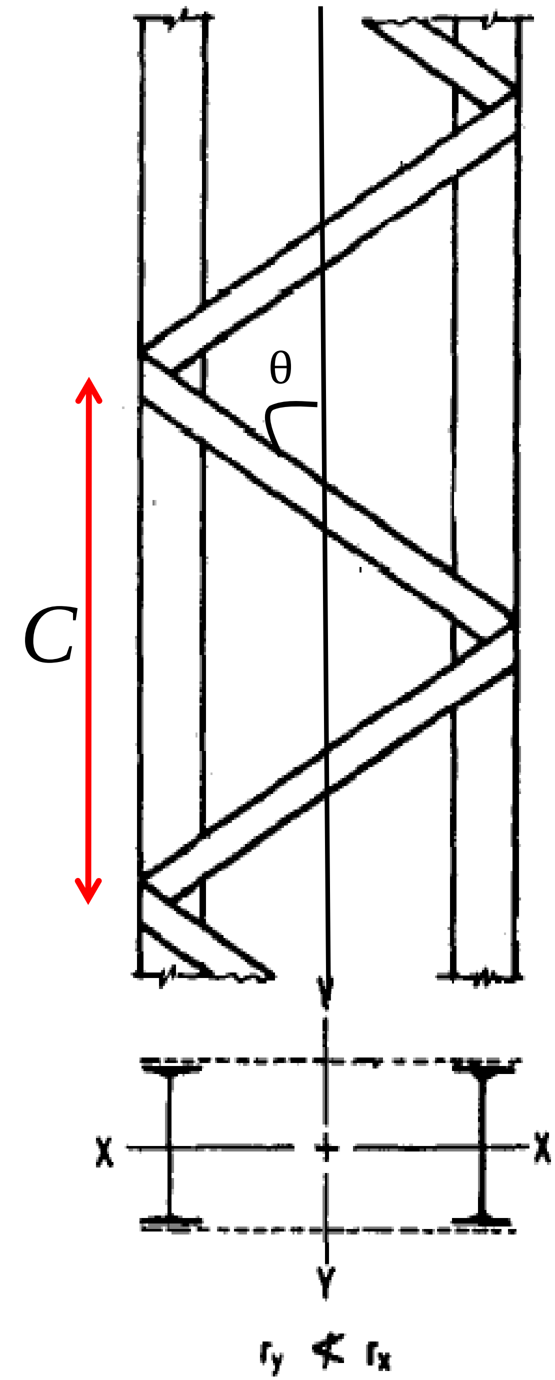
Let's

$$\theta = 45^0$$

$$\frac{C}{r_{yy}} < 50$$

$$or, < 0.7\lambda_{column}$$

r_{yy} – Radius of gyration of column element



Width of Lacing Bars.- (IS 800, Clause 5.7.3, Pg - 50)

- In riveted construction, the minimum width of lacing bars shall be as follows:

Nominal Rivet Dia	*Width of Lacing Bars*
mm	mm
22	65
20	60
18	55
16	50

Let's

Assume rivet diameter = **20 mm**

Lacing width = **75 mm** .

Thickness of Lacing Bars .- (IS 800, Clause 5.7.4, Pg - 50)

- Minimum thickness of Lacing :

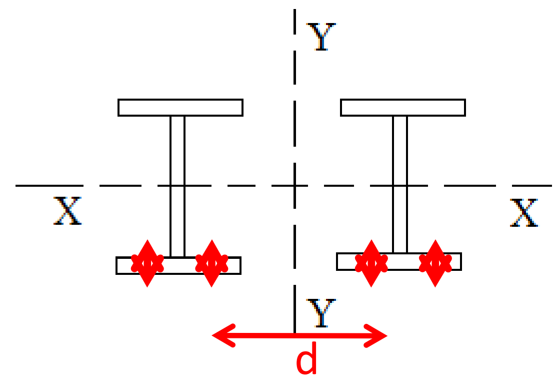
$$t \geq \frac{l}{40} \longrightarrow \text{For Single Lacing}$$

$$\geq \frac{l}{60} \longrightarrow \text{For Double Lacing}$$

l – length between inner ends of rivets.

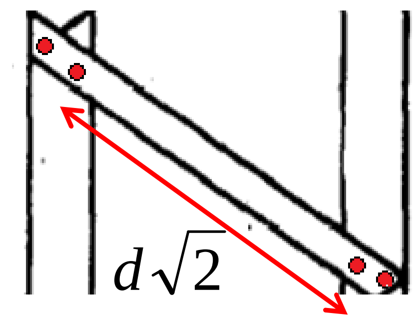
$$d = (2S - 125) = (2 \times 140 - 125) = 155mm$$

$$d\sqrt{2} = 219.2mm$$



Thus, Minimum required thickness

$$\frac{l}{40} = \frac{219.2}{40} = 5.48mm$$



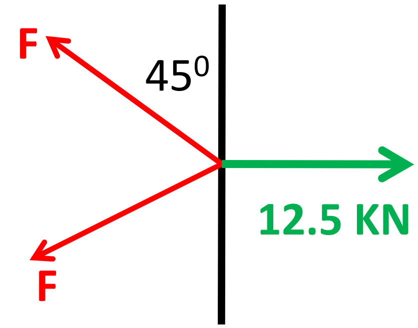
Let's provide 8 mm thick Lacing

Design force taken by Lacing (IS 800, Clause 5.7.2.1, Pg - 48)

$$V = 2.5\% of, P = \frac{2.5}{100} \times 1000 = 25KN$$

$$F \sin 45^0 \times 2 = 12.5\,KN$$

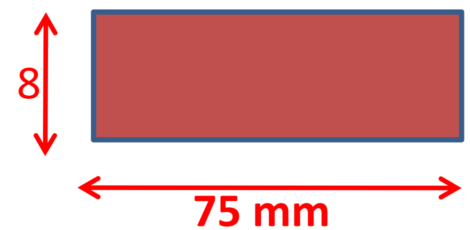
$$F = 8.84\ KN$$



Minimum radius of Gyration:

$$r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{\frac{1}{12} \times 75 \times 8^3}{75 \times 8}} = 2.31\,mm$$

$$\lambda = \frac{d\sqrt{2}}{r_{\min}} = \frac{219.2}{2.31} = 94.89 < 145\ , OK$$



From IS 800 :1984, Table – 5.1, Pg - 39

$$\sigma_{ac} = 85.21 MPa$$

$$\sigma_{ac,cal} = \frac{F}{A} = \frac{8.84 \times 10^3}{75 \times 8} = 14.73\,MPa < \sigma_{ac}\,, OK$$

- Check in axial Tension:-

Gross diameter of rivet (ϕ)

$$\phi = 20 + 1.5 = 21.5mm$$

Net sectional area of the lacing:

$$A_{net} = (75 - 21.5) \times 8 = 428mm^2$$

Thus, Tensile stress in the lacing:

$$\sigma_{at,cal} = \frac{F}{A_{net}} = \frac{8.84 \times 10^3}{428} = 20.65N / mm^2 < \sigma_{ac}\left(150N / mm^2\right), OK$$

For ISHB350@67.4Kg/m

Thickness of flange (t_f) = 11.6 mm

Thickness of Lacing (t) = 8 mm

Rivet value of the 20 mm diameter rivet:

- Single shear strength value

$$V = \tau_{av} \frac{\pi}{4} \phi^2 = \frac{100}{1000} \times \frac{\pi}{4} \times 21.5^2 = 36.3 KN$$

- Bearing strength value

$$P = \sigma_{pt} \phi t = \frac{300}{1000} \times 21.5 \times 8 = 51.6 KN$$

Thus Rivet value (R) = 36.3 KN

Number of rivet $= \frac{F}{R} = \frac{8.84}{36.3} = 0.24$

Minimum No. of rivet to be provided is 2, in each side

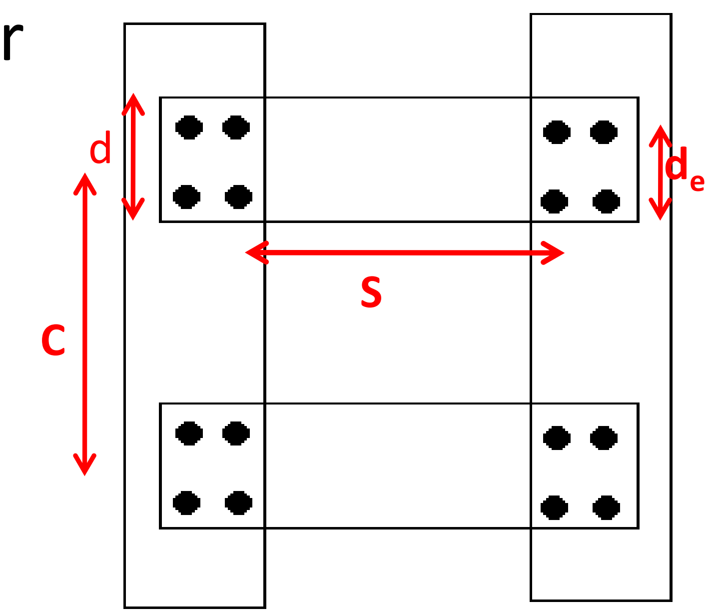
Batten:- It behave like very small beam member and Subjected to bending moment.

- The effective length of battened column should be increased by 10%.
- Minimum 4 No. of batten should be provided.
- Provided batten on opposite faces such that one should be the mirror image of other.
- Thickness of batten (t) is same order of thickness of flange.

$$t \geq \frac{S}{50}$$

$$\frac{C}{r_{yy}} < 50$$

$$or \;, < 0.7\,\lambda_{column}$$



- **Longitudinal shear (V_1):-**

$$V_1 = \frac{VC}{NS}$$

V - Transverse shear force (2.5% of P)

N - The number of parallel planes of battens

$$\tau_{av,cal} = \frac{V_1}{A_{batten}} \le \tau_{av}\left(0.4 f_y\right)$$

- **Longitudinal Moment (M):-**

$$M = \frac{VC}{2N}$$

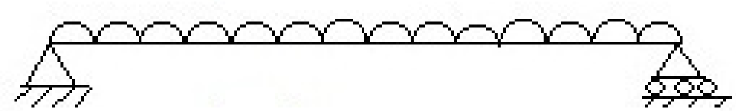
$$\sigma_{bt,cal} = \sigma_{bc,cal} = \frac{M}{Z_{batten}} \le \sigma_{bc/bt}\left(0.66 f_y\right)$$

Design of Flexural Member

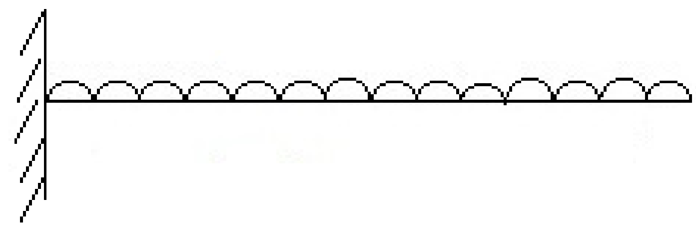
- A beam is a structural member subjected to transverse load, i.e load perpendicular to the longitudinal axis.

➢Types of beam (Based on end condition):-

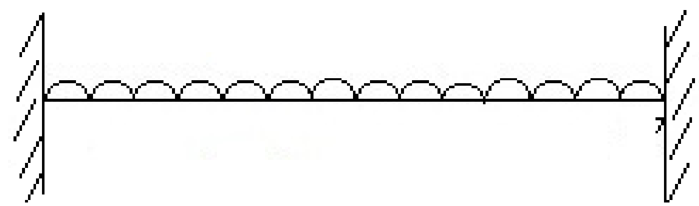
- Simply supported



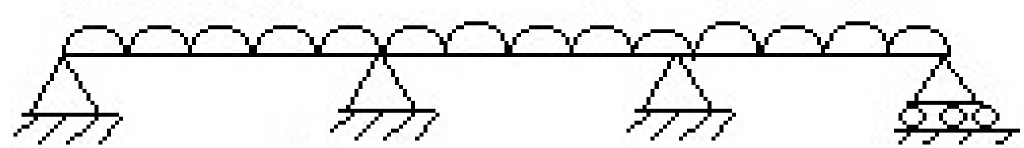
- Overhanging Cantilever



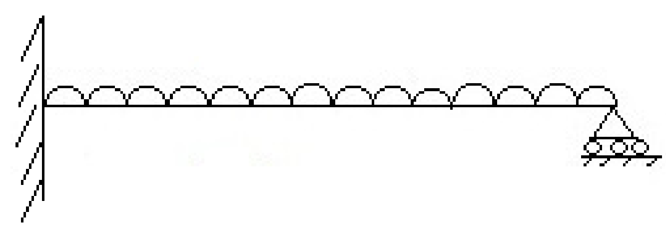
- Fixed beam



- continuous



- Propped cantilever



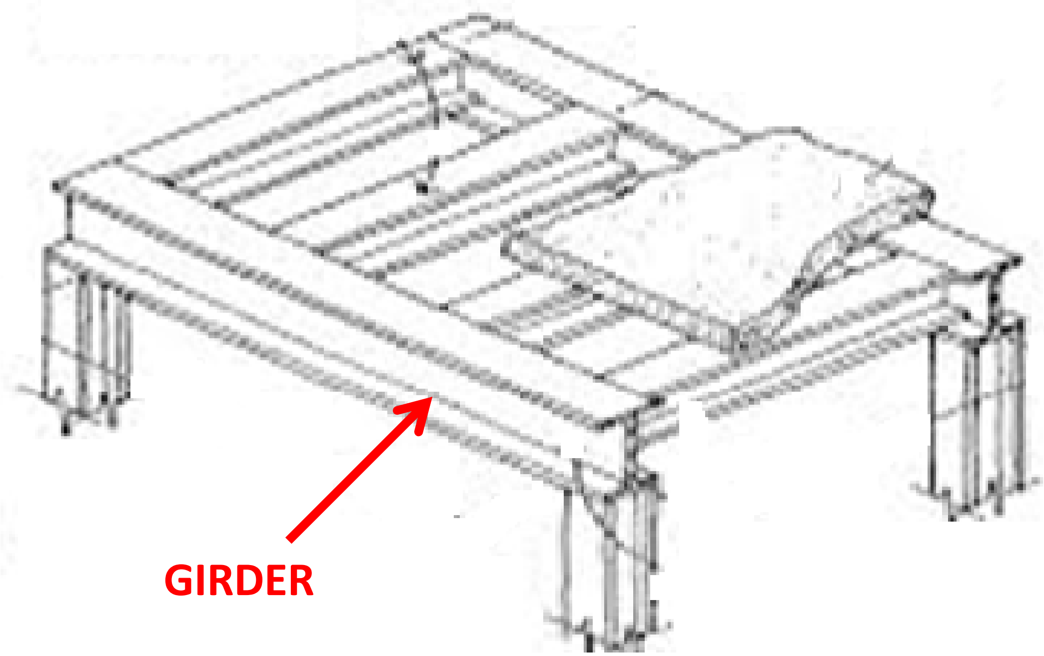
➢ Typical name of beam :

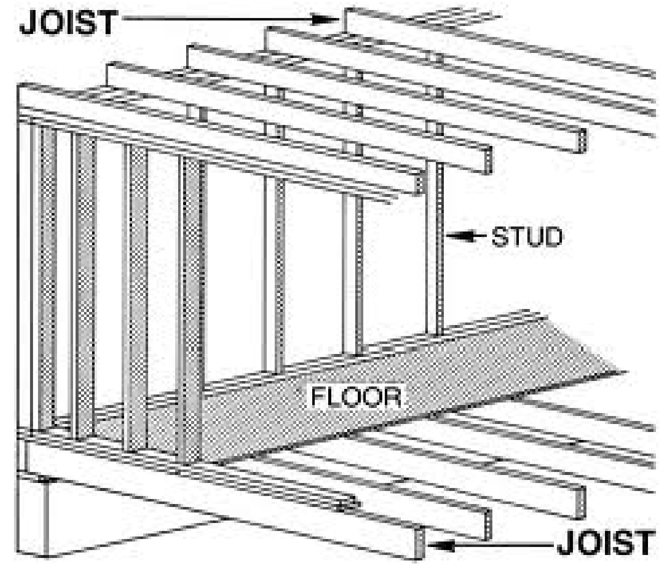
- Joist :- A closely spaced beam supporting floors or, roof of Building but not supporting the other beam .

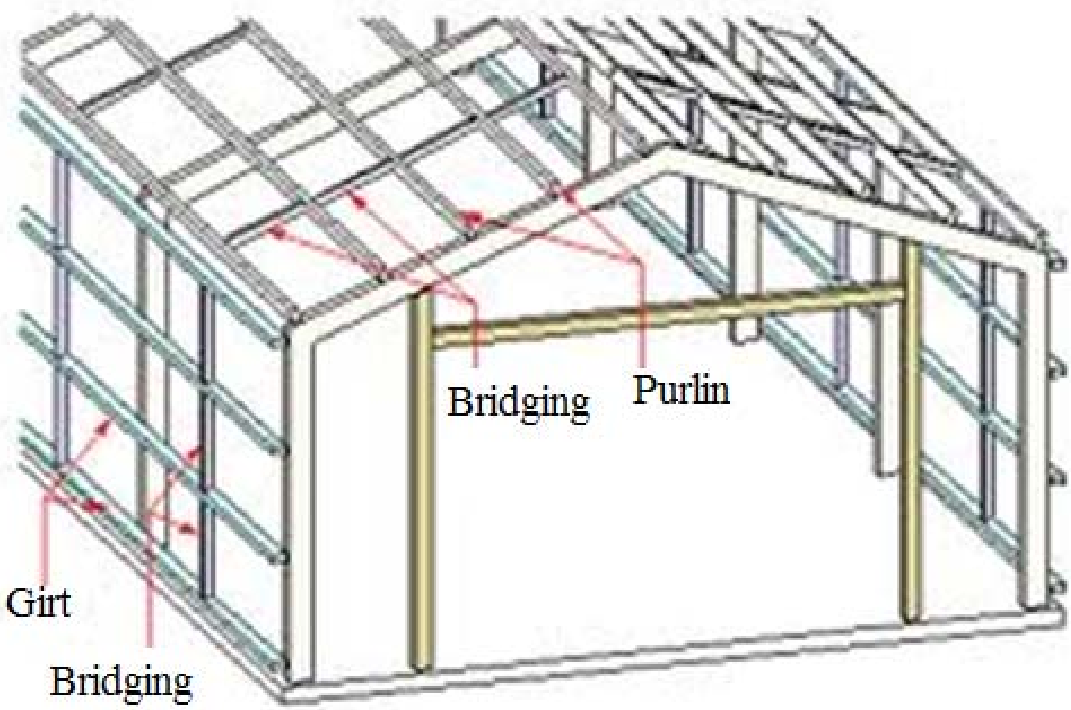
- Larger beam are used for supporting a number of joists. They are called Girders.

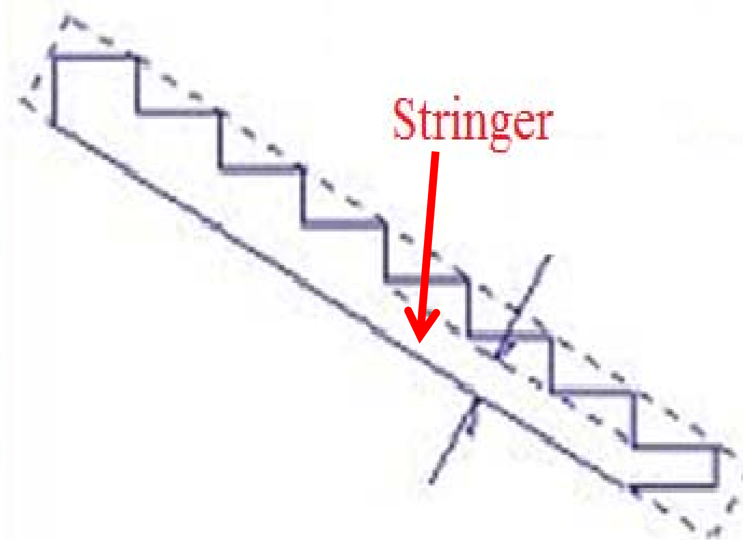
- Beam are also used to carry roof loads in trusses. These Beam are called Purlins.

- Stringer :- In building , beams supporting stair steps, in bridges a longitudinal beam supporting deck floor and Supported by floor beam.


GIRDER


JOIST
STUD
FLOOR
JOIST


Bridging
Purlin
Girt
Bridging


Stringer

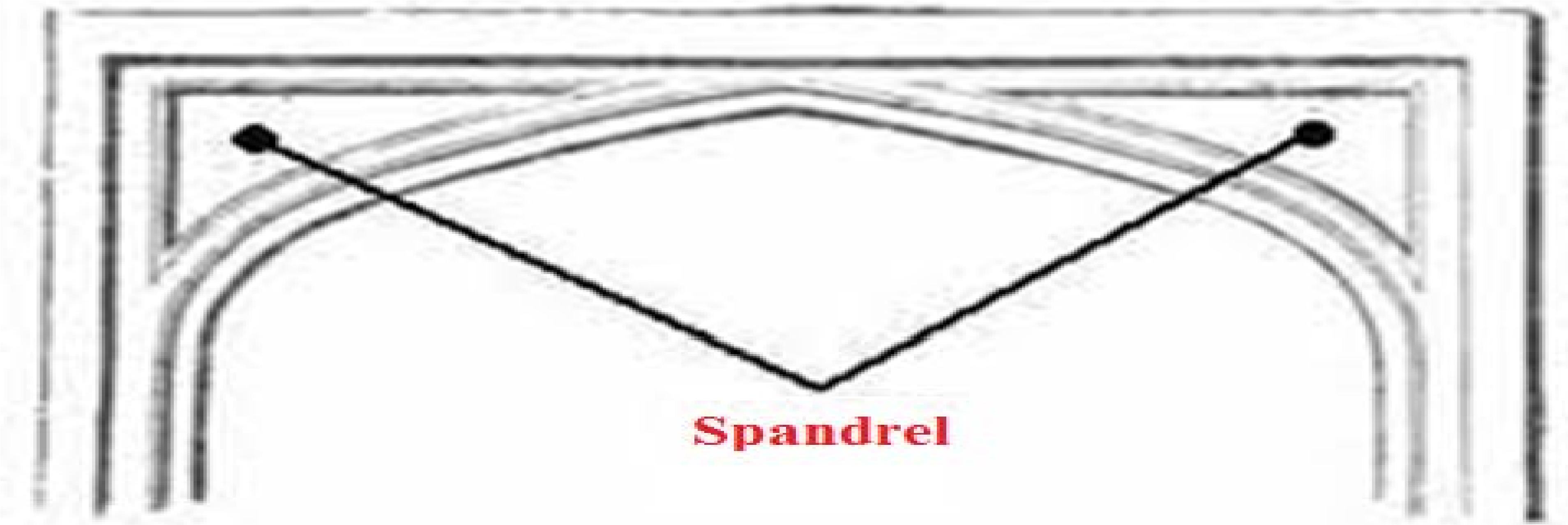
- Spandrel beam :- In a building a beam on the outside Perimeter of a floor, supporting the exterior walls and Outside edge of the floors.

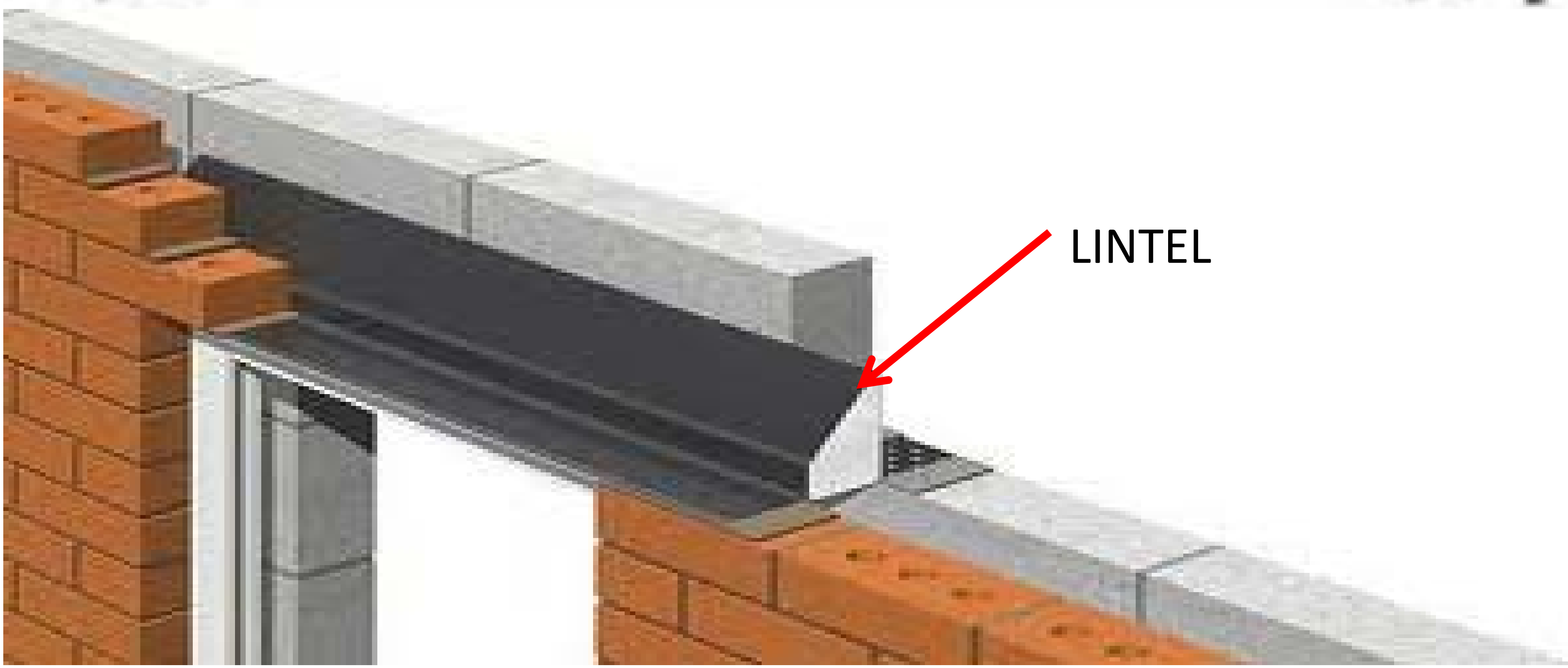
- A horizontal beam spanning the wall columns of industrial Building used to support wall coverings is called a GIRT.

- A roof beam usually supported by purlins is called a Rafter.

Beam are also used to support the loads from the masonry Over the openings. Such types of beam are called Lintels.

- Floor beam:- A major beam supporting other beams or Joints in a building; also the transverse beam in bridge.


Spandrel


LINTEL

➢ Mode of failure of beam :-

Primary mode of failure of beams are as follows :

- Bending failure :- Due to crushing of compression flange or fracture of tension flange.

- Shear failure :- Due to buckling of web near location of high shear force.

- Deflection failure :- Structure is assumed to be fail or unsuitable if excessive deflection occur.

3.13 Limiting Deflection

3.13.1 *Limiting Vertical Deflection*

3.13.1.1 The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads causing deflection.

3.13.1.2 The deflection of member shall not be such as to impair the strength or efficiency of the structure and lead to damage to finishings. Generally, the maximum deflection should not exceed 1/325 of the span, but this limit may be exceeded in cases where greater deflection would not impair the strength or efficiency of the structure or lead to damage to finishings.

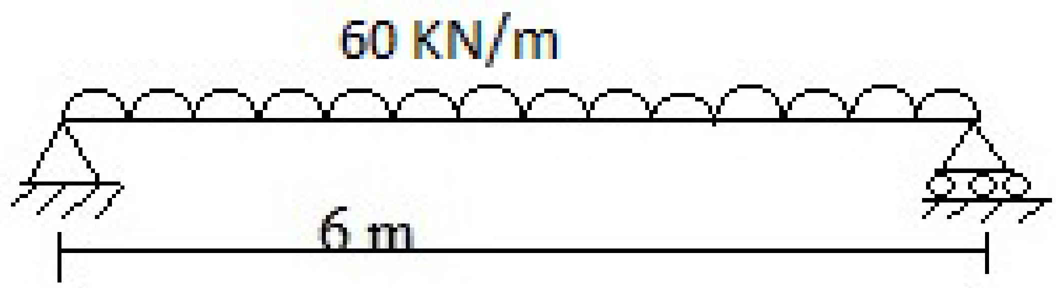
3.13.1.3 In the case of crane runway girder the maximum vertical deflection under dead and imposed loads shall not exceed the following values:

a) Where manually operated cranes are operated and for similar loads	$\frac{L}{500}$
b) Where electric overhead travelling cranes operate, up to 50*t*	$\frac{L}{750}$
c) Where electric overhead travelling cranes operate, over 50*t*	$\frac{L}{1\,000}$
d) Other moving loads such as charging cars, etc	$\frac{L}{600}$

where,

L = span of crane runway girder.

Design a steel beam for the loading as shown in figure as per IS 800 :1984



Maximum bending moment

$$M_{\max} = \frac{wl^2}{8} = \frac{60 \times 6^2}{8} = 270 KN - m$$

From IS 800 1984, Clause 6.2, Pg-55

Permissible stress in Bending Tension/ Compression($\sigma_{bt/bc}$) = 0.66 f_y

For Mild steel f_y = 250 MPa

$$\sigma_{bt} = \sigma_{bc} = 165 MPa$$

Let’s start with σ_{bc} = 100 MPa

Thus,

$$\text{Required section modulus } (Z) = \frac{M}{\sigma_{bc}} = \frac{270 \times 10^6}{100}$$

$$= 27 \times 10^5 \, mm^3$$

Let’s try with ISMB 600 @ 122.6 Kg/m

$$Z_{xx} = 3060.4 \times 10^3 mm^3$$

Self wt. of section = 1.226 KN/m

Total load (DL+LL) = 61.226 KN/m

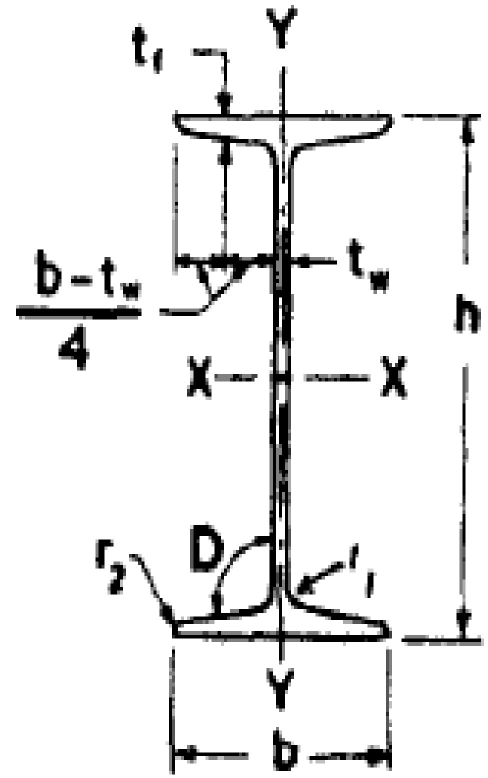
Maximum bending moment:

$$M_{\max} = \frac{wl^2}{8} = \frac{61.226 \times 6^2}{8} = 275.517 KN - m$$

Check for bending tension :-

$$\sigma_{bt,cal} = \frac{275.517 \times 10^6}{3060.6 \times 10^3} = 90.026 MPa < 165 MPa \quad OK$$

Check for bending compression :-



For ISMB 600

Thickness of flange (t_f) = 20.8 mm

Thickness of web (t_w) = 12.0 mm

$$\frac{t_f}{t_w} = \frac{20.8}{12.0} = 1.733 < 2$$

$$\frac{d_1}{t_w} = \frac{509.7}{12.0} = 42.475 < 85$$

From IS 800 1984, Table- 6.1B, Pg - 58

$$\frac{h}{t_f} = \frac{D}{\tau} = \frac{600}{20.8} = 28.846$$

$$\lambda = \frac{l_{eff}}{r_y} = \frac{6000}{41.2} = 145.63 < 180, OK$$

D/Γ → L_{eff}/r_y ↓	**25**	**30**	**28.846**
140	103	97	98.3632
150	98	92	93.3632

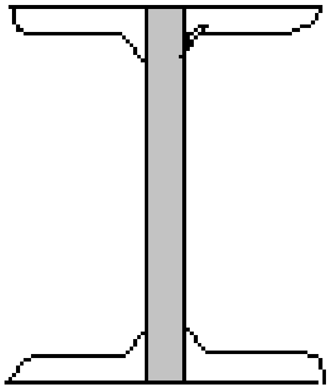
$$\sigma_{bc} = 95.5482 MPa$$

Bending stress due to load :-

$$\sigma_{bc,cal} = \frac{275.517 \times 10^{6}}{3060.6 \times 10^{3}} = 90.026 MPa < \sigma_{bc} (95.5482)$$

So, section is safe

Check for shear stress :-



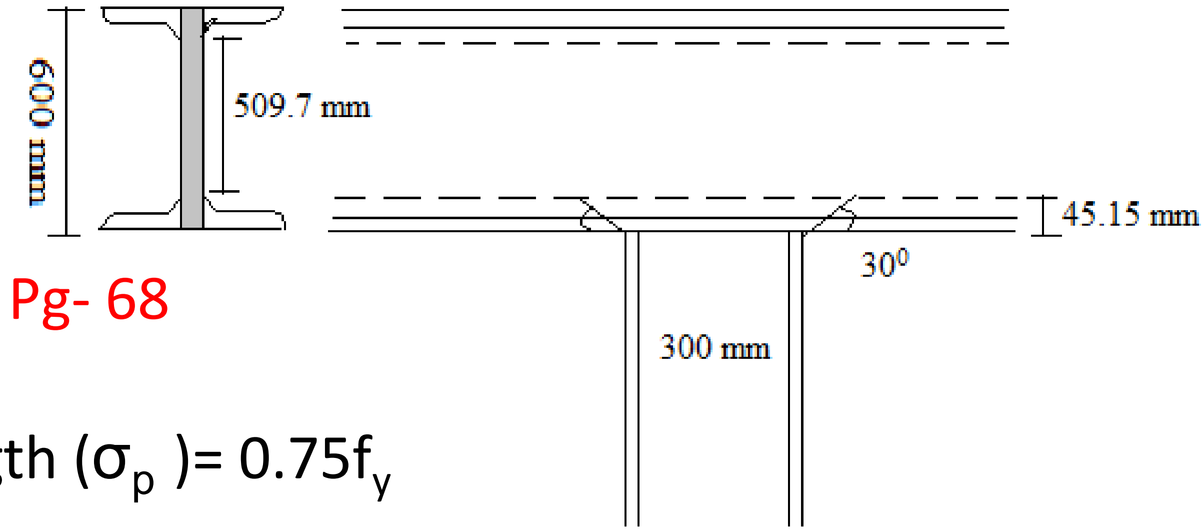
$$\tau_{av} = 0.4 f_y = 100 MPa$$

$$V_{\max} = \frac{61.226 \times 6}{2} = 183.678 KN$$

$$\tau_{av.cal} = \frac{V_{\max}}{h \times t_w} = \frac{183.678 \times 10^{3}}{600 \times 12} = 25.51 MPa < \tau_{av} (100 MPa)$$

So, section is safe

Check for web crippling :- Crippling is to be checked under point load or support only .



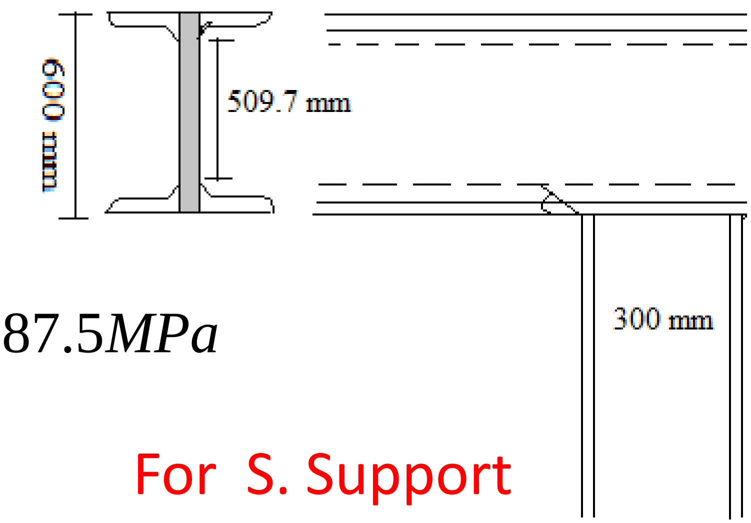
From IS 800, clause 6.3, Pg- 68

Maximum Bearing strength (σ_p)= $0.75f_y$

= 187.5 MPa

For continues support

Calculated bearing stress :



$$\sigma_{p,cal} = \frac{183.678\times10^3}{\left(300+45.15\sqrt{3}\right)\times12} = 40.47MPa < 187.5MPa$$

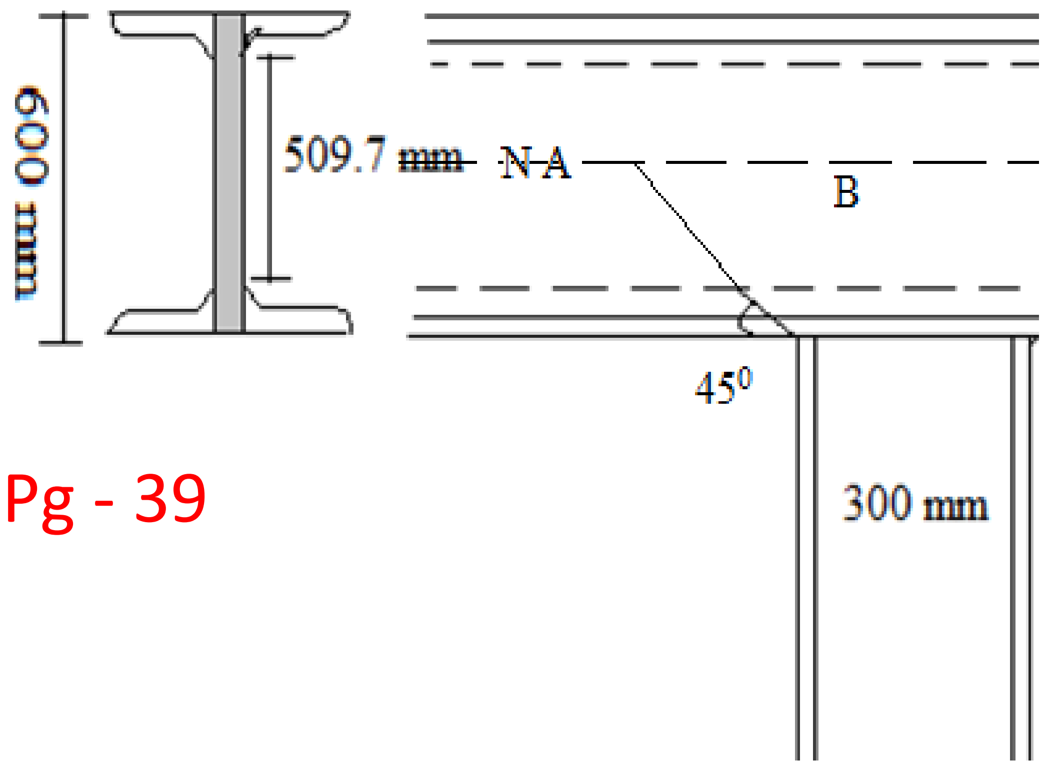
For S. Support

Check for web buckling:- Crippling is to be checked under point load or support only .

Web buckling is critical at N.A. and crippling is critical where the Web start to be straight.

From, IS 800 1984, Clause – 6.7.5.1, Pg - 87

Slenderness ratio of web:



$$\lambda = \frac{d_1\sqrt{3}}{t_w} = \frac{509.7 \times \sqrt{3}}{12}$$

$$= 73.57$$

From IS 800, Table – 5.1, Pg - 39

$$\sigma_{ac} \approx 110 MPa$$

Calculated axial compression stress :

$$\sigma_{ac.cal} = \frac{V_{\max}}{Bt_w} = \frac{183.678\times10^3}{\left(300+600/2\right)\times12} = 25.51MPa < \sigma_{ac}\left(110MPa\right)$$

So section is safe

Check for deflection :

Deflection due to load $(\delta_{cal}) = \dfrac{5wl^4}{384EI}$

$$= \frac{5\times61.226\times6^4\times10^{12}}{384\times2\times10^5\times91813\times10^4} = 5.63mm$$

From Clause 3.13.1.2, Pg- 34

Permissible value of deflection $= \dfrac{Span}{325} = \dfrac{6000}{325} = 18.46mm > \delta_{cal}$