3. Compression Members

Types of compression members - Theory of columns - Basis of current codal provision for compression member design - Slenderness ratio - design of single section and compound section compression members - design of laced and battened type columns - design of column bases - gusseted base.

Dfn : compression member:

It is a structural member which is straight and subjected to two equal and opposite compressive forces applied at its ends.

Types of compression members :

* column, stanchion or post - It is a vertical compression members supporting floors, girders in a building. It is subjected to heavy loads.

* Strut - It is used for roof truss and bracing. It is a small span and lightly loaded compression member.

Types of structs :-

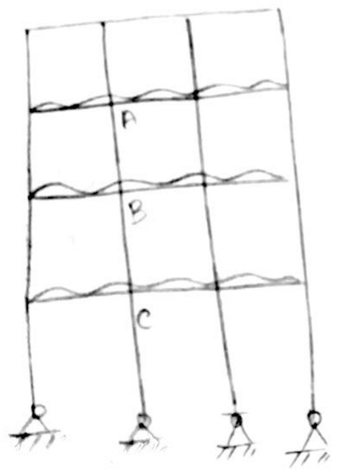
1) continuous strut - which is continuous over the no. of joints such as top chord member of a truss bridge girder, principle rafter of roof truss.

2) Discontinuous struct - which extends b/w two adjacent joints only.

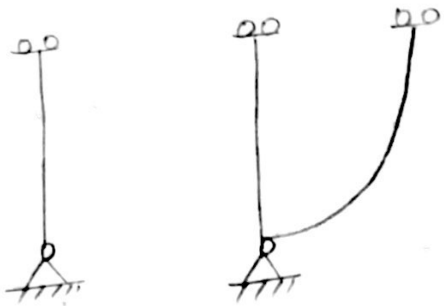
Eg:- vertical (or) Inclined compression member in a roof truss.

Effective length of the compression member:

Pg: 45 Table 11 IS 800:2007



End	Translation	Rotation
A	Free	Restrained
B	Restrained	Restrained



End	Translation	Rotation
C	free	Restrained
D	Restrained	free

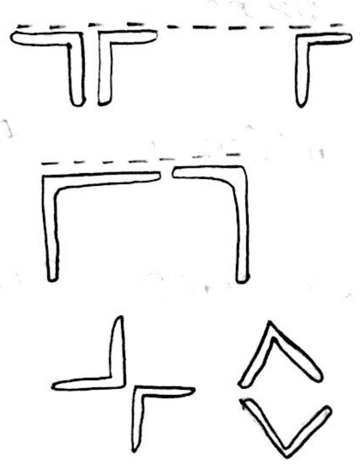
Slenderness Ratio (λ):

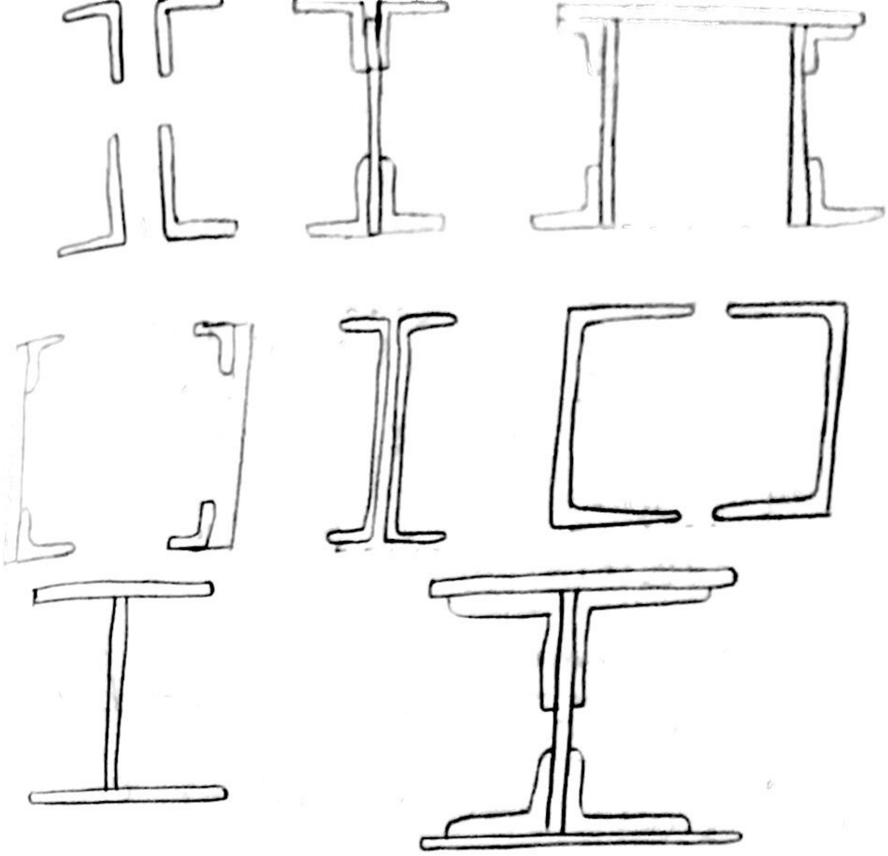
It is defined as the ratio of effective length to the radius of gyration

(i.e) $\lambda = \dfrac{KL}{r_{min}}$

$r = \sqrt{I/A}$

Types of sections (or) shape of compression member:





3.17 Failures in columns:

Long column → fails by elastic buckling

Intermediate column } → fails by inelastic buckling

Very short column } → fails by crushing or yielding.

In long column stress will not exceed proportional limit.

In Intermediate column, fails after extreme fibres reach yield point.

1. Determine the axial load capacity of the column section ISHB at 547 N/m if the length of the column is 3m and both ends of column are pinned.

Sol: ① Properties of section:

ISHB 250 @ 547 N/mm²

Area = 6971 mm²

b_f = 250 mm $D = h = 250$ mm

t_f = 9.7 mm

r_{min} (r_z & r_y) = 53.7 mm

(1)

Pg: 44 Table (10) IS 800:2007

To find buckling class of given column refer IS800 Pg: 44 Table (10). The given Problem is rolled I-section.

D: $h = 250$ (from steel table Pg: 4)

$$\frac{h}{b_f} = \frac{250}{250} = 1 \leq 1.2$$

$$t_f \leq 100mm$$

$$9.4 \leq 100mm$$

$\therefore$ Buckling class above z-z axis $= b$ class

Buckling class above y-y axis $= c$

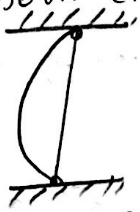
(2) Slenderness ratio:

$$\frac{K_L}{r}$$

$K_L \rightarrow$ effective length

$r \rightarrow$ radius of gyration.

give $\rightarrow$ columns both ends are pinned.



1.0 L

$$K_L = 1 \times 3000 = 3000mm$$

$$r = 53.7$$

$$\therefore \frac{K_L}{r} = \frac{3000}{53.7} = 55.86$$

(3) Design compressive strength (f_{cd}):

Pg: 41 IS800-2007 class b

For $K_L = 50$ & $f_y = 250$, $f_{cd} = 194$

$K_L = 60$ & $f_y = 250$. $f_{cd} = 181$

$$\boxed{f_{cd} = 186.38 \text{ MPa}}$$

Scanned by CamScanner

∴ The axial load on column

load = Stress × Area

$P_d = f_{cd} \times A$

$= 186.38 \times 6971$

$\boxed{P_d = 1299.25\,KN}$

∘ Design the compressive strength of a bolted steel angle section 90×90×12mm @ 158 N/m with the length of the member has 2.8m. Both the ends are hinged (pinned).

① Properties of sections :-

ISA 90×90×12mm @ 158 N/m

$A = 2019\,mm^2$

$b_f = 90mm$

$t_f = 12mm$

h (or) $D = 90mm$

$r_{min} = 17.4mm$

② To find Buckling ~~class for rolled~~ steel bolted angle section:

Table 10 pg:- 44 IS800-2007

Buckling class = 'c'

For the given prob. (Rolled ~~bolt~~ steel angle bolted section)

buckling class = c.

pg:- 8 Table 2 . IS800-2007

For single angle section with the components separated, axial compression all three criterias given in table 2 pg:- 18 should be satisfied.

$b/t < 15.7\varepsilon$

$d/t < 15.7\varepsilon$

$\left(\frac{b+d}{t}\right) < 25\varepsilon$

$$\varepsilon = (250/f_y)^{1/2}$$

$$= \sqrt{\frac{250}{250}} = 1$$

$$\boxed{\varepsilon = 1}$$

$$\frac{b}{t} = \frac{90}{12} = 7.5 < 15.7$$

$$d/t = \frac{90}{12} = 7.5 < 15.7$$

$$\frac{b+d}{t} = \left(\frac{90+90}{12}\right) = 15 < 25$$

∴ All 3 criterias are satisfied. Hence given section is <u>semi.compact</u>.

(2) <u>compressive strength</u>:

for buckling class 'c'

Pg: 42 IS 800-2007

$$\frac{KL}{r} = \frac{2800}{17.4} = 160.92$$

Pg: 42

160 → 53.3

170 → 48.1

160.92 → 52.82

$$\boxed{f_{cd} = 52.82 MPa}$$

$$P_d = f_{cd} \times A$$

$$= 52.82 \times 2019$$

$$\boxed{P_d = 106.64 KN}$$

8.3.17 Design a rolled steel section unequal ISA 100X65X10 at 188N/m as a compression member with effective length of 1400mm assume that the angle is <u>loaded through only one leg</u> and both ends are fixed.

Properties of section ISA 100X65X10.

Pg: 18 steel tables $A_g = 1551 mm^2$

$d(h) = 100mm$, $t = 10mm$

$r_{min} = 13.8mm$

② classification of buckling

Table ⑩, angle → 'c'

③ checking for limiting thickness

(Table 2): $b/t = \frac{65}{10} = 6.5 < 15\varepsilon_1$ $\left(\varepsilon_1 = \left(\frac{250}{f_y}\right)^{1/2}\right)$

$\frac{d}{t} = \frac{100}{10} = 10 < 15$

$\left(\frac{b+d}{t}\right) = \left(\frac{65+100}{10}\right) = 16.25 < 25\varepsilon_1$

All 3 criterias are satisfied. Therefore section is semi-compact

The given angle section is loaded through only one leg. For this section, equivalent slenderness ratio

Pg: 48 IS800-2007 cls: 7.5.1.2

④ Slenderness ratio: $\lambda_c = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\phi^2}$

K_1, K_2, K_3 → Table ⑫

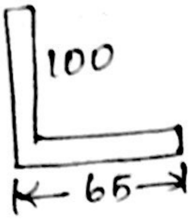
Given problem, both ends are fixed. Assume no. of bolt @ each end is 1.

$\therefore$ $\left.\begin{array}{l} K_1 = 0.45 \\ K_2 = 0.35 \\ K_3 = 20 \end{array}\right\}$ Table ⑫

$$\lambda_{vv} = \frac{\left(l/r_{vv}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{1400}{13.8}\right)}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}}$$

$\boxed{\lambda_{vv} = 1.142}$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon_1\sqrt{\frac{\pi^2 E}{250}}}$$



$$= \frac{(100+65)/2\times10}{1\sqrt{\frac{\pi^2\times2\times10^5}{250}}}$$

$$\boxed{\lambda_\phi = 0.093}$$

Sub λ_{vv} & λ_ϕ values.

$$\lambda_e = \sqrt{0.45 + (0.35\times(1.142)^2 + (20\times(0.093)^2}$$

$$\lambda_e = 1.174$$

(5) Design compressive strength:

Pg: 34 IS800 : 2007

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

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

$$\lambda = \lambda_e = 1.174.$$

Pg: 35 Table (7)

$\alpha = 0.49$ (for class c).

$$\phi = 0.5[1 + 0.49(1.174 - 0.2) + (1.174)^2]$$

$$\boxed{\phi = 1.428}$$

$$f_{cd} = \frac{250/1.1}{1.428 + (1.428^2 - 1.174^2)^{0.5}}$$

$$\boxed{f_{cd} = 101.42\ N/mm^2}$$

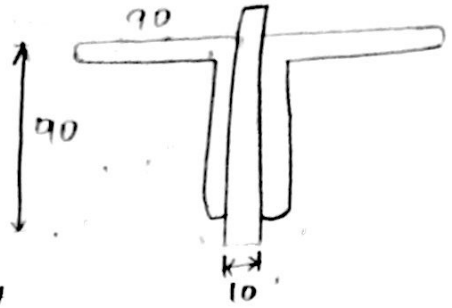
$$P_d = f_{cd} \times A_g$$

$$= 101.42 \times 1551$$

$$\boxed{P_d = 157.3\,kN}$$

2\. Find the compressive strength of the strut. It consists of 2 angles ISA 90x90x6mm & length 8m. The angles are connected on both sides of 10mm gusset plate. Find the compressive strength if the angles are connected by 1 bolt, 2 bolt (or) welding which is rigid?

Pg: 40 steel tables.



(1) Properties of section (ISA 90x90x6):

$A = 2094\ mm^2$

$I_{xx} = 160.2 \times 10^4\ mm^4$

$r_{min} = 27.7\ mm$

Case (i): one bolt is used

for connection both ends are fixed.

$\frac{KL}{r} = \frac{1 \times 8000}{27.7} = 108.4$ ~~Pg: 40.~~

buckling class (angle) = 'c'.

Table (10).

$\frac{KL}{r} = 110$. $f_{cd} = 94.6$ $P_d = 96.58 \times 2094 = \underline{202.24\ KN}$

$\frac{KL}{r} = 100$, $f_{cd} = 107$

by interpolation, $f_{cd} = 96.58\ N/mm^2$.

case (ii) Two bolts are used for connections:

Angles are connected by 7.5.2.1 with 2 no.s of bolts.

$eH = (0.7\ to\ 0.85)\ L$

$K = 0.85L$

$\frac{KL}{r_{min}} = \frac{0.85 \times 8000}{27.7}$

$= 92.05$

buckling class 'c'.

ref. Table (10)

$f_{cd} = 118.13\ N/mm^2$

$P_d = 118.13 \times 2094$

$= 247.34\,KN$

case (iii) connection using welding

7.5.2.1

(0.7 to 0.85)

$K_L = 0.7 \times 8000$

$\frac{K_L}{r_{min}} = \frac{0.7 \times 8000}{27.7} = 75.81$

Pg 42, $\frac{KL}{r} = 75.81$ & buckling 'c'.

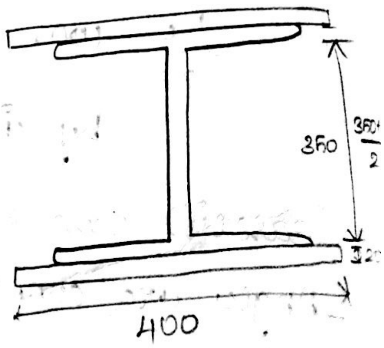
$f_{cd} = 142.7\,MPa$

$P_d = 2094 \times 142.7$

$\boxed{P_d = 298.82\,KN}$

calculate the safe compressive strength of a built up column consisting of ISHB@ 724 N/m with plates of 400x20 attached on one of each flange of I section, the length of the column is 4m. Assume 1/2 column is fixed and other is hinged

Sol:



Steel tables

① Pg:-4

$A = 9221\,mm^2$

$I_{yy} = 2510.5 \times 10^4\,mm^4$

$I_{xx} = 19802.8 \times 10^4\,mm^4$

$h = 350mm$,

$b_f = 250mm$

$t_f = 11.6$

② Properties of builtup sections:-

$I_{xx} =$ I_{xx} of ISHB350

$+ I_{xx}$ plate $\rightarrow \left(\frac{bd^3}{12} + Ah^2\right)$

neglect (because its so small)

$= 19802.8 \times 10^4 + (400 \times 20)\left(\frac{350+20}{2}\right)^2$

$= 19802.8 \times 10^4 + 2738 \times 10^4$

$= 4.71 \times 10^8\ mm^4$

$I_{yy} = I_{yy}$ of ISHB350 $+ 2\left(\frac{b^3 d}{12}\right)^s$

$= 2510.5 \times 10^4 + 2\left(\frac{20 \times 400^3}{12}\right)$

$= 2.38 \times 10^8\ mm^4$

$\therefore I_{min} = 2.38 \times 10^8\ mm^4$

$r_{min} = \sqrt{\frac{I_{min}}{A}}$

$= \sqrt{\frac{2.38 \times 10^8}{9221 + (2(400 \times 20))}}$

Area of ISHB

Area of 2 plate

$\boxed{r_{min} = 97.14\,mm}$

Buckling class:

Pg: 44 IS800-2007

building section = 'c'

Pg: 45 IS800-2007 Table (11) case (5).

Eff. length } = 0.8 L

Effective length } = 0.8 × 4000

= 3200 mm

$\frac{KL}{r_{min}} = \frac{3200}{97.14\,mm} = 32.94$

Pg: 42 IS800-2007

30 - 211

40 - 198

$f_{cd} = 207.18\ N/mm^2$

$\boxed{f_{cd} = 207.18\ MPa}$

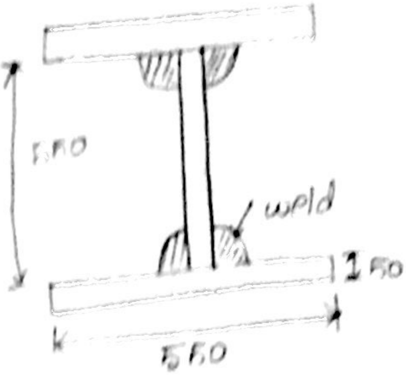
$P_d = f_{cd} \times A \Rightarrow 207.18 \times (9221 + 2(400 \times 20))$

$\boxed{P_d = 5228.28\ KN}$

Scanned by CamScanner

compute the design strength of the given welded section. the length of the column is 7m & both ends are pinned.

Sol



$$I_{xx} = 2\left[\frac{550\times50^3}{12} + \left(\overbrace{(550\times50)}^{A}\times\left(\frac{550+50}{2}\right)^2\right) + \frac{50\times550^3}{12}\right]$$

$= 5654\times10^8\,mm^4$

$$I_{yy} = 2\left(\frac{50\times550^3}{12}\right) + \left(\frac{550\times50^3}{12}\right)$$

$= 18.92\times10^8\,mm$

$\therefore\ I_{min} = 13.92\times10^8\,mm^4$

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$

$A = (2(550\times50)) + (50\times550)$

$= 82500\,mm^2$

$$r_{min} = \sqrt{\frac{13.92\times10^8}{82500}}$$

$= 129.89\,mm$

Both ends are pinned, $KL = 1.0\times L$

$$\frac{KL}{r_{min}} = \frac{1\times7000}{129.89}$$

$= 53.89$

$f_{cd} = 177.65\,N/mm^2$

$P_d = A\times f_{cd}$

$= 82500\times177.65$

$\boxed{P_d = 1461.6\,KN}$

Design of compression members:

Design steps:

Step 1: Design stress (f_{cd})

(i) Rolled steel Beam (ISHB & ISHC) - Slenderness ratio → (70 to 90)

design stress → 135 N/mm²

(ii) Angle struts

Slenderness ratio - (110 to 130mm)

Design stress - 90 N/mm²

(iii) large builtup section carry loads above 2000 KN.

design stress = 200 N/mm²

Step 2: Effective sectional area required

$$A_c = \frac{P_d}{f_{cd}}$$

Step 3: Select a suitable section to have required effective area and find r_{min}.

Step 4: Based on the end condition of the column determine the effective length (l_e). Refer Table (11)

Step 5: Find the slenderness ratio, design stress and load carrying capacity of column section.

Step 6: Revise the section if calculated P_d is less than design load.

Note:

Until the calculated P_d is more than given load, the section should be revised.

3.3.17

2. A 4m long column has to support a factored load of 6000kN. The column is effectively held at both ends (1 end fixed and other hinged). Design the column using the built up Beam section and the plate.

Step 1: Design stress

The given section is large built-up section (combination of beam & plate) & It's carry a load of 6000 kN which is more than 2000 kN.

So, Design stress is 200 N/mm²

$$\boxed{f_{cd} = 200\ N/mm^2}$$

Step 2: Eff. sectional Area (A_e):

$$A_e = \frac{P_d}{f_{cd}} = \frac{6000 \times 10^3}{200}$$

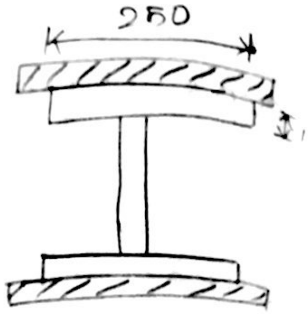
$$= 30{,}000\ mm^2$$

Step 3: Select the section

ISHB 450 @ 925 N/mm

$A = 11789\ mm^2$

$b_f = 250$, $t_f = 13.7\ mm$



Area of plate = 30,000 - 11789

= 18211 mm²

Assume 20mm thick plate

$2(20 \times b) = 18211$

$b_p = 455.8 \simeq 500\ m$

Therefore provide 500×20mm plate on each side of the flanges.

Step 4: Check for overhanging:

$$\text{Overhang portion} = \frac{\overset{b_p}{500} - \overset{b_f}{250}}{2} = 125\ mm \quad < 16 \times 13.7 \quad 16t$$

Over hanging of the plate, should not be more than 16 t of the plate

thickness of plate

$16 \times 20 = 320$

overhang < 16 t → Hence safe.

(5) calculation of eff. length:-

$$I_{xx} = I_{xx} \text{ of ISHB 450} \rightarrow I_{xx} \text{ of plate} \quad \left(\frac{bd^3}{12} + Ah^2\right)$$

$$= 40349.9 \times 10^4 + 2(500 \times 20)\left(\frac{450 \times 20}{2}\right)^2$$

$$= 15.08 \times 10^8\ mm^4.$$

I_{yy} = I_{yy} of ISHB 450 + I_{yy} of plate

$= (8045\times10^4) + 2\left(\frac{20\times500^3}{12}\right)$

$= 447.12\times10^6 mm^4$

$I_{min} = 447.12\times10^6 mm^4$

$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{447.12\times10^6}{11789+2(20\times500)}} = 118.6 mm$

→ one end fixed & other end hinged

eff. length, $KL = 0.8L$ Table (11)

$= 0.8\times4000 = 3200$

$\frac{KL}{r_{min}} = \frac{3200}{118.6} = 26.98$

Buckling class = 'c' (for all built up sections)

$\frac{KL}{r} = 26.98$, 'c'

$\frac{20}{30} \Rightarrow \begin{matrix}224\\211\end{matrix}$

$\boxed{f_{cd} = 214.9 N/mm^2}$

Step 5: check for design stress:

$P_d = A\times f_{cd}$

$= (11789 + (2\times20\times500))\times214.9$

$= 31789\times214.9$

$= 6831.4 KN > 6000 KN$

Hence safe.

8. Design $\boxed{\text{single angle strut}}$ connected to the gusset plate, to carry a factored load of 150KN. The length of the strut b/w c/c is 3m.

① Design stress (f_{cd}):

For angle strut, Assume

$f_{cd} = 90 N/mm^2$

② Eff. sectional area (A_e):

$A_e = \frac{P_d}{f_{cd}} = \frac{150\times10^3}{90} = 1666.7 mm^2$

⑧ Selection of section:-

Section - 100x100x8mm

$A_g = 1539mm^2$

$r_{min} = 19.5mm$

Assume 2 bolts used in connection

Pg:- no.48 cls 7.5.2.1

If 2 bolts are used @ one end, $K_L = 0.7 - 0.8$

$K_L = 0.8L = 0.8 \times 3000$

$= 2400mm$

$$\frac{K_L}{r_{min}} = \frac{2400}{19.5} = 123.08$$

⇒ Buckling class ⇒ 'C' (for angle strut)

⇒ f_{cd}

	f_{cd}
120	83.7
130	74.3

$f_{cd} = 80.80\ N/mm^2$

⇒ check:-

$P_d = A_e \times f_{cd}$

$= 1539 \times 80.80$

$= 124.3KN < 150KN$

Not safe, Revise the section.

⇒ For revising 100x100x10mm

$A_g = 1963mm^2$.

$r_{min} = 19.4mm$

Assume 2 bolts used in connection.

$$\frac{K_L}{r} = \frac{0.8 \times 3000}{19.4} = 127.7$$

⇒ f_{cd}

$f_{cd} = 80.08$

check ⇒ $P_d = A_e \times f_{cd}$

$= 1963 \times 80.08$

$= 157KN > 150KN$

Hence safe.

→ Laced columns → column with zig zag member.

The size and shape of rolled steel sections are limited because of the limitations of rolling mills.

When rolled steel sections donot furnish the required sectional area (or) special shape (or) large radius of gyration is required in two different directions.

A built up section is fabricated.

For economical design, the heavily loaded long columns, the least radius of gyration of column section is increased to maximum ($r_y \geq r_x$)

To achieve this condition the rolled sections are kept away from the centroidal axis of the column. These are connected by lattice system. Such columns are called latticed column (or) open column.

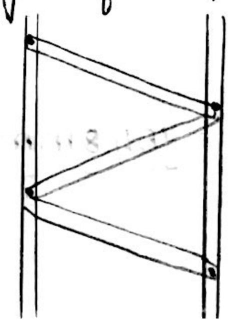
Some commonly used laced systems:

1) Lacing
2) Latticing
3) Battening

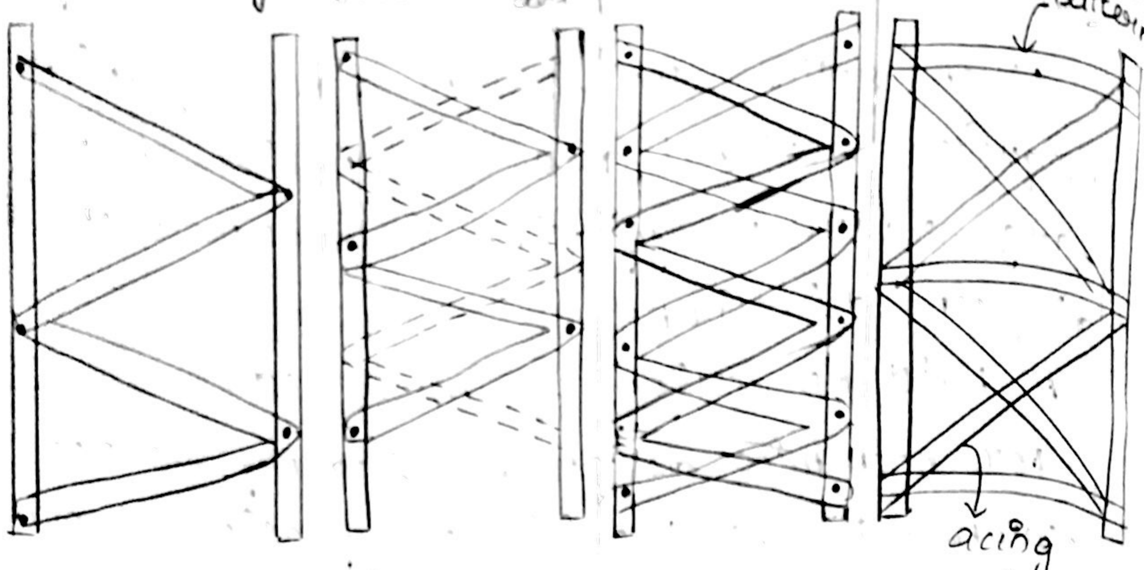
Flat & Angle sections are used for lacing.

Purpose of using "lacing":

To hold the various parts of column straight, parallel @ a correct distance apart & it to equalize the stress distribution b/w various parts. Eg:- single flat ~~lacing~~ lacing not intersecting.



Single flat lacing Intersecting | Single flat lacing non mirror image (not recommended) | Double flat lacing intersecting | Double flat lacing with battens (recommended)



Design a built-up column with 2 channel sections, column of 6.4m eff. length & supports a load of 1000kN.

⇒ Design compressive stress:

$f_y = 250\ N/mm^2$ vary

$f_{cd} = 125 - 175\,MPa$

Take, $f_{cd} = 135\,MPa = 135\ N/mm^2$

Factored load $= 1.5 \times 1000 = 1500\,N$

⇒ Area required:

$$A_e = \frac{P_d}{f_{cd}} = \frac{1500}{135} = 1111.1\,mm^2$$

⇒ Selection of channel sections:

In given problem,

Two channel sections

Refer steel table → Pg.no → 69

Provide ISMC 400

∴ Total Area of 2 channels $= 12586\,mm^2$.

$r_y > r_z$

∴ $r_{min} = r_{zn} = \underline{154.8\,mm}$

Step 4: Slenderness ratio:

Pg: 48 → cl: 7.6.1.5

Eff. slenderness ratio of laced column = 1.05 × Actual slenderness ratio

$$\left(\frac{KL}{r}\right)_{CH} = 1.05 \times \left(\frac{KL}{r}\right)_{actual}$$

$$= 1.05 \times \left(\frac{6400}{154.8}\right)$$

$$= 43.41$$

Step 5: Check for comp. stress:

for $\frac{KL}{r} = 43.41$ & buckling class 'c'.

$f_{cd} = 192.9\ N/mm^2$

$P_d = f_{cd} \times A$

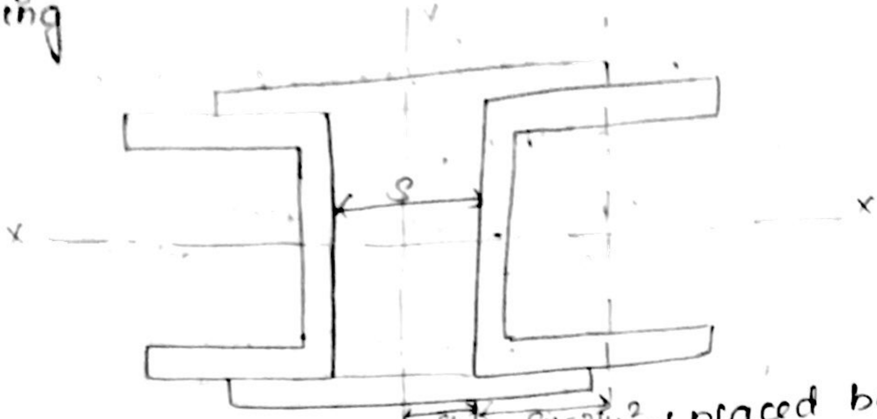
$= 12586 \times 192.9$

$= 2427.8 \times 10^3\ N$

$= 2427.8\ KN > 1500\ KN$

⑥ Spacing b/w two channels

Two channels are provided back to back with lasing



Pg. 69 (steel tables) for channel placed back to back

Pg: 6 (steel table) ISMC 400

$A = 6293\,mm^2$

$I_y = 504.8 \times 10^4\,mm^4$

$I_x = 15082.8 \times 10^4\,mm^4$

$r_y = 88.3$

$C_y = 24.2\,mm$

To find s, equate $\boxed{2I_x = 2\left[I_y + A\left(S/2 + C_y\right)^2\right]}$

$$2(15082.8 \times 10^4) = 2\left[504.8 \times 10^4 + 6293\left(\frac{S}{2} + 24.2\right)^2\right]$$

$$301656 \times 10^3 = 2\left[504.8 \times 10^4 + 6293(0.25S^2 + 585.64 + 24.2S)\right]$$

$$30165.6 \times 10^4 = 2(504.8 \times 10^4 + 1573.25S^2 + 3685432.5 + 152290.6S)$$

$$30165.6 \times 10^4 = 2(873.34 \times 10^4 + 1573.25S^2 + 152290.6S)$$

$$30165.6 \times 10^4 = 3146.5S^2 + 304581.2S + 1746.68 \times 10^4$$

$$3146.5S^2 + 304581.2S - 28418.92 \times 10^4 = 0$$

$\boxed{S = 256mm}$

Pg:- 69 Steel tables.

Approximate this value as 260mm (referring ST Pg: 69)

$\therefore$ The distance b/w two channels $\boxed{S = 260mm}$

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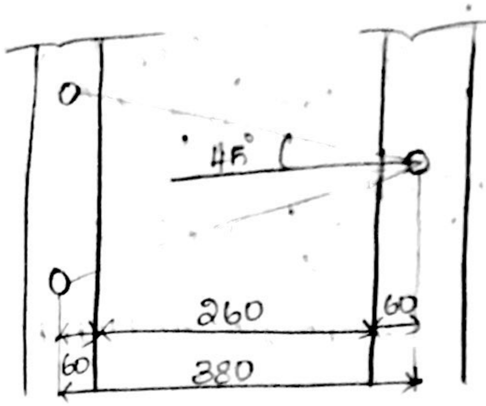
(7) Design lacing:

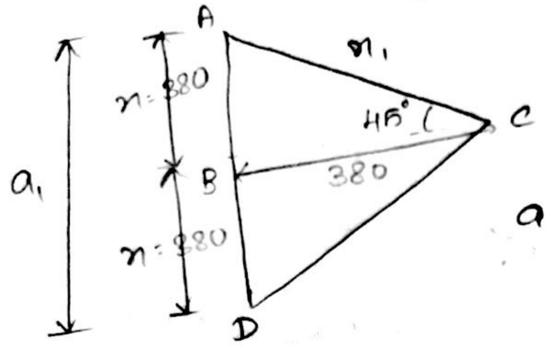
According to IS800:2007 cls 7.6.4

Angle of inclination for lacing = 40°- 70°

Take it as 45° bolts of gauge distance of 60mm



$\therefore$ Horizontal spacing is $260+60+60 = 380mm$



$\tan 45° = \frac{x}{380}$

$\boxed{x = 380}$

$a_1 = x + x = 380 + 380$

$= 760mm$

Inclined length of lacing (AC), $\cos 45° = \frac{380}{x_1}$

$\boxed{x_1 = 537.4mm}$

Pg: 50 (7a) check for spacing:

IS800-2007 Pg: 50 cl: 7.6.5.1 $(r_1 \Rightarrow r_y)$

Spacing $\not>$ 50mm (or) 0.7 x slenderness ratio

$\frac{a_1}{r_y} = \frac{760}{28.3} = 26.86$

ISMC 400 ← r_y pg: 6

$0.7 \times \left(\frac{KL}{r}\right)_e$

0.7×43.4

$= 30.38$

$26.86 < 50mm$ (or) $0.7 \frac{KL}{r}$

$\therefore$ Spacing is safe.

(7b) minimum width of lacing:

Pg: 50 IS800-2007 cls: 7.6.2

min. width of lacing = 3x nominal dia of bolt

Use 20mm ϕ bolt

Min. width of lacing $= 3 \times 20$

$= 60mm$.

(7c) Thickness of lacing:

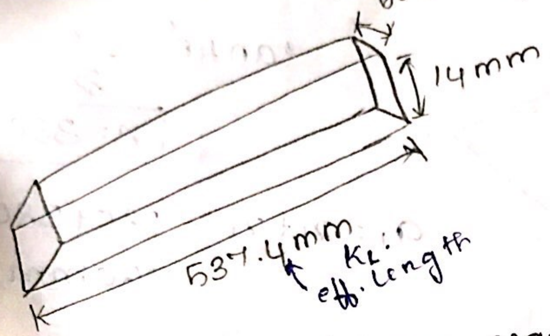
IS 800-2007 Pg: 50 cls: 7.6.3

Thickness of lacing for single lacing $\Big\} = \frac{1}{40} \times$ eff. length of lacing

($r_1 \Rightarrow$ eff. length) $= \frac{1}{40} \times 537.4$

$= 13.43 \simeq 14mm$

$\therefore$ Size of lacing bar $= (60 \times 14)$ mm



(8) Check for slenderness ratio of lacing:

IS 800-2007 cls: 7.6.6.3 Pg: 50

$\frac{KL}{r} \not> 145.$

$\left(\frac{KL}{r}\right)$ of lacing bars $\not> 145mm$

$r = \sqrt{I/A}$

$I = \frac{bd^3}{12} = \frac{60 \times 14^3}{12} = 13720\ mm^4$

$A = 60 \times 14 = 840mm^2$

$r = \sqrt{\frac{13720}{840}} = 4.04mm$

$\frac{KL}{r} = \frac{537.4}{4.04} = 133.01 < 145mm$

Hence safe.

⑨ Design of bolt for lacing

a) Shear strength of bolt in single single shear

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400 \times 10^3}{\sqrt{3} \times 1.25} (1 \times 0.78 \times \pi/4 \times 20^2)$$

$$\boxed{V_{dsb} = 45.27 \text{ KN}}$$

If time is not there assume bearing strength is greater than shear strength.

b) bearing strength of bolt

$$V_{dpb} = \frac{2.5 \times K_b \times d \times t \times f_u}{\gamma_{m0}}$$

$$= \frac{2.5 \times 0.5 \times 14 \times 20 \times 410 \times 10^{-3}}{1.25}$$

$$\boxed{V_{dpb} = 114.8 \text{ KN}}$$

cls: 7.6.6.1 pg: 50 IS 800-2007

Lacing should resist a total transverse shear (V_t) = 2.5% of axial force on member

$$V_t = \frac{2.5}{100} \times 1500 \times 10^3$$

$$\boxed{V_t = 37500 \text{N}}$$

Shear resisted by single section

$$\text{load} = \frac{37500}{2} = 18750 \text{ N} = 18.75 \text{ KN}$$

$$\therefore \text{No. of bolt} = \frac{\text{load on lacing}}{\text{strength of bolt}}$$

$$= \frac{18.75}{45.27}$$

$$= 0.41 \simeq 1 \text{ bolt.}$$

∴ Provide 1 no. of bolt @ each end of lacing

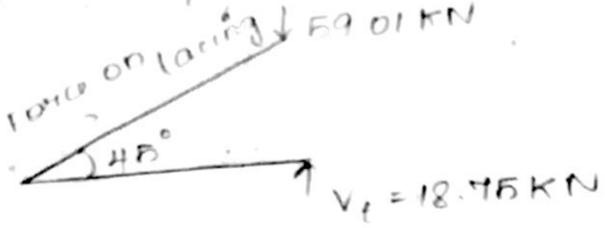
⑩ Check for force acting on lacing:

$\frac{KL}{r}$ for lacing = 134.01 → from step ⑧

buckling class 'c'.

f_{cd} = 70.25 ← avg

∴ Load carrying capacity of lacing in compression } = 70.25 × (60×14) / 1000
= 59.01 KN



Force on lacing $= \frac{18.75}{\sin 45^\circ}$

$= 26.52$ KN < 59.01 KN.

Safe

Batten: Pg: 50 7.7.1.3 → IS 800

Battens are plates on any other rolled section used to connect the main component of compression member.

It should be placed opposite to each other on two parallel forces of the compression member & should be spaced and proportion uniformly throughout.

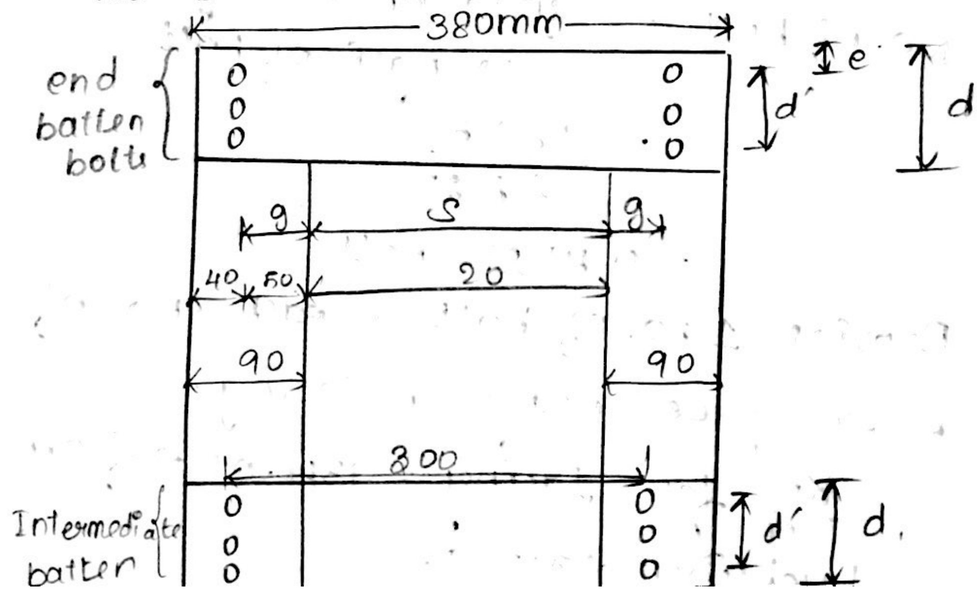
The no. of battens should be such that the no. is divided into atleast three base within its actual length.

End batten:-

Its provided at the end of columns.

Intermediate batten:-

The battens other than end battens.



Battens have same strength as laced columns but it is not recommended Because, It is uneconomical. battens are not recommended for eccentric loading.

1. Design a built-up column 9m long to carry a factored axial compressive load of 1100 kN. The column is restrained in position but not in direction @ both ends. Design the column with connecting system as battens with bolted connection. Use 2 channels back to back.

Sol:

$P = 1100 \times 10^3 N$

$L = 9m$

$KL = 1.0L$ (Both end fixed)

$= 1.0 \times 9 = 9m$

① Assume compressive stress, $f_{cd} = 150 N/mm^2$

② Area, $A = \frac{P_d}{f_{cd}} = \frac{1100 \times 10^3}{150}$

$A = 7333.33 mm^2$

③ Selection of section:-

Use Steel table Pg.no. 44

ISMC 300 @ 358 N/m

$A = 4564 mm^2$

$r_y = 26.1$

$C_{yy} = 23.6$

$r_z = r_{xx} = 118.1mm$

$b_f = 90mm$

$I_z = I_{xx} = 6362.6 \times 10^4 mm^4$

$I_{yy} = 310.8 \times 10^4 mm^4$

④ Effective slenderness ratio:-

Pg.no.48 cls 7.6.1.5 (refer the fig.)

$$\left(\frac{KL}{r}\right)_e = 1.05\left(\frac{KL}{r}\right)_o$$

$$= 1.05 \times \frac{9000}{118.1}$$

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$\left(\frac{KL}{r}\right)_e = 80.02$

⑤ check for compressive stress:

Buckling class ⇒ 'c'

$\frac{KL}{r} = 80.02$

$f_{cd} = 130.27\ N/mm^2$

→ $P_d = A \cdot f_{cd} = (2 \times 4564) \times 130.27$

$P_d = 1189.1 KN > 1100 KN$

Hence selection section is safe.

⑥ Spacing b/w the channels:-

$2I_x = 2\left(I_y + A\left(S/2 + C_y\right)^2\right)$

$2 \times 6362.6 \times 10^4 = 2\left[310.8 \times 10^4 + 4564\left(S/2 + 23.6\right)^2\right]$

$1.27 \times 10^8 = 6.22 \times 10^6 + 9128\left(\frac{S^2}{4} + 556.96 - 23.6S\right)$

$1.27 \times 10^8 = (6.22 \times 10^6) + 2282S^2 + 5.08 \times 10^6 - 2.15 \times 10^5 S$

$2282S^2 - (2.15 \times 10^5)S - 1.2 \times 10^8 = 0$

$S = 183.5$

$\boxed{S \simeq 206mm}$

⑦ Spacing b/w battens:-

Pg:-51 $\quad C/r_y < 50 \rightarrow ①$

(or) $\quad \frac{C}{r_y} < \left(0.7 \times \frac{KL}{r}\right) \rightarrow ②$

ⓐ ⇒ $\frac{C}{r_y} = 50 \Rightarrow C = 50 \times 26.1 = 1305$

$C = 1300mm$

ⓑ ⇒ $\frac{C}{r_y} = 0.7\frac{KL}{r} = 0.7 \times 80.02 \times 26.61$

$C = 1461.96mm$

Take least value of above,

$C = 1300mm$

⑧ Size of end battens:-

Assume ϕ of bolt 20mm,

$d_h = 20 + 2$

$= 22mm$.

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a) Min edge distance, $e = 1.5 d_h = 1.5 \times 22 = 33$

$e = 35mm$

eff. depth of end battern $\Big\}$, $d' = S + 2Cy$

$= 200 + (2 \times 23.6)$

$= 247.2mm > bf \ (90mm)$

b) overall depth of end battern:

$d = d' + 2e$

$= 247.2 + (2 \times 35)$

$d = 317.2$

$= 320mm$//

c) Tks of end battern $\nless \frac{1}{50}(S + 2g)$

↓

Pg.no. 51 (7.7.2.3)

$= \frac{1}{50}(200 + 2 \times 50) = 6mm$.

d) Length of end batten $= 200 + (2 \times 90)$

$= 380mm$

(a) Size of Intermediate batten:

eff. depth of intermediate batten $\Big\}$ $d' = \frac{3}{4} d'$

$= \frac{3}{4} \times 247.2$

$d' = 185.4mm$

overall depth of intermediate batten $\Big\}$ $d_1 = d_1' + 2e$

$= 185.4 + 2 \times 35$

$= 255.2mm$

$d_1 = 260mm$

Thk of Intermediate batten $\Big\} \nless \frac{1}{50}(S + 2g)$

$= \frac{1}{50}(200 + 2 \times 50)$

$= 6mm$//.

$\quad\quad\quad l \quad b \quad t$

Size of end batten : 380 x 320 x 6mm

Size of Intermediate batten : 380 x 860 x 6mm.

(10) Design forces

cls ~~10.6.6.1~~ cls 7.7.2.1

Longitudinal shear on batten $\Big\}$ $V_b = V_t C / NS$

Pg:- 51

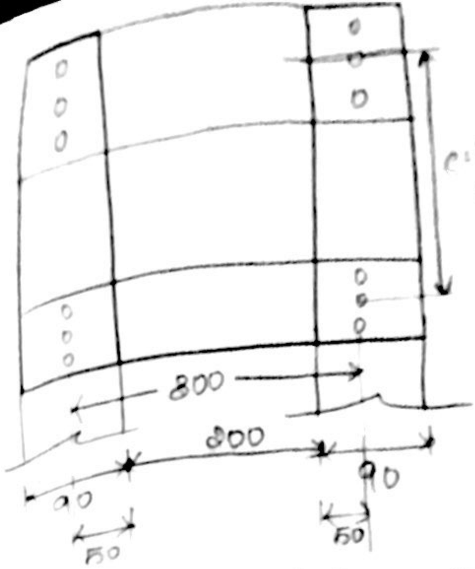
$M = V_t C / 2N$

C → c/c distance between intermediate and end columns

C = 1300mm (from step 7)

N → no. of parallel planes.

$\boxed{N = 2}$



$S = 800\,mm$

V_t = transverse shear (7.7.2.1) pg. 51

$= \frac{2.5}{100} \times$ axial load

$= \frac{2.5}{100} \times \underset{given}{1100\times10^3}$

$= \cancel{2.75\times10^3 N} = 27.5\times10^3 N$

$V_b = \frac{V_t C}{NS} = \frac{27.5\times10^3 \times 1300}{2\times300}$

$= 59.58\,kN$

$M = \frac{V_t C}{2N} = \frac{27.5\times10^3 \times 1300}{2\times2}$

$= 89.37\times10^5\,Nmm$

(11) Check for stress & moment on end batten.

(11a) check for stress:

$\frac{V_b}{bt} < \frac{f_y}{\sqrt{3}\gamma_{mo}}$

$\frac{V_b}{bt} = \frac{59.58\times10^3}{320\times6} = 31.03\ N/mm^2$

$\frac{f_y}{\sqrt{3}\gamma_{mo}} = \frac{250}{\sqrt{3}\times1.1} = 131.21\ N/mm^2$

$\frac{V_b}{bt} < \frac{f_y}{\sqrt{3}\gamma_{mo}}$ Hence safe.

(11b) check for moment on end batten

$\boxed{\frac{6M}{bd^2} < \frac{f_y}{\gamma_{mo}}}$

$\sigma_{cal} = \frac{M}{Z}$

$= \frac{M}{bd^2/6}$

$= \frac{6M}{bd^2}$ → $\boxed{b = t}$ $\boxed{d = b}$

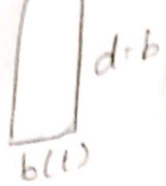
$\boxed{t = d}$

$\frac{6M}{\cancel{bd^2}} = \frac{6\times89.37\times10^5}{320^2\times6}$

$= 87.27\ N/mm^2$

$f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\ N/mm^2$

$$\boxed{\frac{6M}{tb^2} < \frac{f_y}{\gamma_{mo}}}$$



(12) check for stress & moment on intermediate batten:

(12a) check for stress:

$$\frac{V_b}{bt} < \frac{f_y}{\sqrt{3}\gamma_{m0}}$$

$$\frac{V_b}{bt} = \frac{59.58\times10^3}{260\times6} = 88.19\ N/mm^2$$

$$\frac{f_y}{\sqrt{3}\gamma_{mo}} = \frac{250}{\sqrt{3}\times1.1} = 131.21\ N/mm^2$$

$$\frac{V_b}{bt} < \frac{f_y}{\sqrt{3}\gamma_{mo}}\ ,\ \text{safe.}$$

(12b) check for moment:

$$\frac{6M}{tb^2} < \frac{f_y}{\gamma_{mo}}$$

$$\frac{6M}{tb^2} = \frac{6\times89.37\times10^5}{6\times260^2}$$

$$= 132.21\ N/mm^2$$

$$\frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27\ N/mm^2$$

$$\frac{6M}{tb^2} < \frac{f_y}{\gamma_{mo}}\ \text{Hence safe.}$$

(13) Design of connection:

use 20mm ϕ bolt Assume shear capacity is minimum.

$$\text{Shear capacity of bolt} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb})$$

$$= \frac{400\times10^{-3}}{\sqrt{3}\times1.25}(0.78\times\pi/4\times20^2)$$

$$= 45.27\ KN$$

$$\text{No. of bolt} = \frac{\text{Load}(V_b)}{45.27} = \frac{59.58}{45.27}$$

$$= 1.31 \simeq 3$$

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Pitch, $p = \frac{1}{2}(d - (2 \times e))$

$= \frac{1}{2}(380 - (2 \times 35))$

$= 155 mm$

No. of battens on one face of column

$= \frac{\text{length of column}}{\text{Spacing of batten}} + 1$

$= \frac{9000}{1300} + 1 = 7.92 \simeq 8$

$\therefore$ Revised c/c of battens $= \frac{9000}{8-1} = 1285.75 mm$

column bases:

Introduction: Refer IS800:2007 Pg: 46

Steel columns are supported over concrete block when loads supported by column is large. The bearing pressure of concrete is insufficient to resist load it may fail.

$\therefore$ To distribute column loads to steel base plate which are placed over the concrete block. columns are subjected to bearing pressure from below, B.M and S.F

Types of column bases:

* It is the type of base transmit direct loads only. (slab base)

* It can carry bending moment and direct load (Gusseted base)

cleat angle → connecting angle of column & slab.

Design a slab base for a column ISHB 350 @ 710.2 N/m subjected to a factored axial compression load of 1500KN for the following condition condition
(a) Load is transferred to the base plate by direct bearing of column flanges.
(b) Load is transfered to the base plate by welded connections. whether ancher bolts are required?
The base resists on concrete pedestrial of grade M_{20}.

Sol:

condition ①:

Step 1: Properties of ISHB 350 @ 710.2 N/m.

$t_f = 11.6mm$

$t_w = 10.1 mm$

$l = D = 350mm$.

$b_f = 250mm$

Given: load : 1500KN
Factored load

If factored load is not given multiply by 1.5

Grade of concrete ~~cement~~ = M_{20} (f_{ck})

IS 800:2007 Pg: 46 7.4.1

Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 20$

$= 9 N/mm^2$

Step 2: Area of slab base: (A)

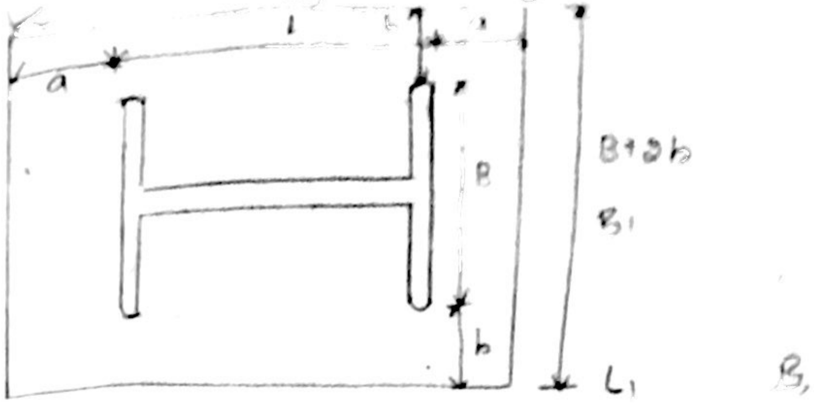
$$A = \left(\frac{1500 \times 10^3}{9}\right) \quad \left(\frac{\text{Load}}{\text{Bearing strength of concrete}}\right).$$

$= 166,666.6 mm^2$

$= 0.166 m^2$

Let us provide rectangular slab base of length 'L' and Breadth = 'B'.

Assume overhangings of base plate beyond column flanges as 'a' and 'b' and take a=b



Area of rectangular slab base $= (l+2a) \times (B+2b)$

$\underline{a=b}$ $\quad (350+2a) \times (250+2b) = 166,666.6\ mm^2$

$\boxed{a=b}$ $\quad (350+2a) \times (250+2a) = 166,666.6$

$87500 + 1200a + 4a^2 = 166,666.6$

$4a^2 + 1200a - 79166.6 = 0$

$a = 55.24 \simeq 60mm$

$a=b \therefore b = 60mm.$

$L_1 = l+2a = 350 + (2\times60) = \underline{470mm}$

$B_1 = B+2b = 250 + (2\times60) = \underline{370mm}$

$\therefore$ Bearing pressure of ~~corner~~ concrete $= \frac{P}{A_1} = \frac{1500\times10^3}{470\times370}$

$= 8.62 < 9\ N/mm^2$

Hence safe.

③ Thickness of base plate (t_s) :-

Pg:- 47, 7.4.3.1 IS 800: 2007

$t_s = \sqrt{2.5w(a^2-0.3b^2)\gamma_{mo}/f_y} > t_f$

$w =$ actual pressure $= 8.62\ N/mm^2$ From step②

$t_s = \sqrt{\frac{2.5(8.62(60^2-0.3(60)^2)1.1}{250}}$

$= 15.45 \simeq 16mm.$ $\quad 16 > 11.6$

$t_f = 11.6mm$

$\downarrow$ ISHB 350.

$\therefore$ Size of base plate $\underline{\overset{L_1}{470} \times \overset{B_1}{370} \times \overset{t}{16}}.$

~~Condition I~~

Load is transferred to the base plate by direct bearing to keep the column in position, two cleat angles are provided connecting column flanges with base plate

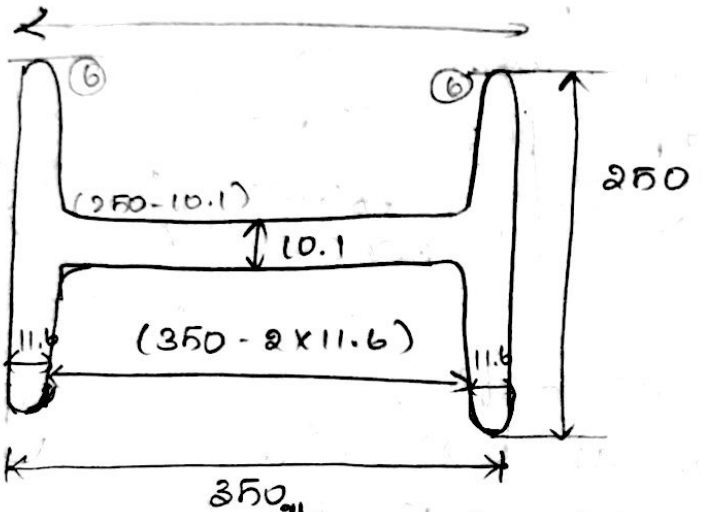
condition II

column is connected by welding to the slab base
∴ load from column is transferred to the base plate by means of welding
∴ length of available for welding around the column profile (l_a)

$= (2 \times 250) + (2(250 - 10.1)$

$+ 2 \times (350 - 2 \times 11.6)$

$= 1633.4$



Provide 8mm fillet weld.
welding is not possible @ toes and fillets of ISHB section end returns will have to be subtracted @ the end of each fillet weld length to get effective length that can be provided. no. of end return (curves, turnings) = 12 i.e (2×6)

Effective length of weld $= 1633.4 - 12(2 \times 8)$ $(2 \times S)$

$l - no. (2s)$

$= 1441.4$ mm

Size of weld = 8mm

Throat tks $(t_t) = 0.7 \times S$

$= 0.7 \times 8$

$= 5.6\,mm$

$\therefore$ Strength of weld/mm length

$$= 1 \times t_t \times \frac{f_u}{\gamma_{mw}\sqrt{3}}$$

taking as 1m — Area

$$= 1 \times 5.6 \times \frac{410}{\sqrt{3} \times 1.25}$$

$= 1060.5\ N/mm$

Req. length of weld $= \frac{1500 \times 10^3}{1060.5}$ $\frac{\text{Load}}{\text{str}}$

$= 1414.43 < 1441.4\,mm$

Hence safe.

Since the base plate is subjected to axial compressive load, no B.M is occured. Therefore, provide 2 no.s of 20mm dia bolt to keep the base plate in position.

Gusseted base:-

1. A column ISHB 350 @ 661.2 N/m carries an axial compressive factored load of 1700 KN. Design a suitable bolted gusset base. The base rests on M15 concrete pedestrial use 24mm grade 4.6 bolt making the connections.

Sol: Properties of ISHB 350 @ 661.2 N/m.

Pg: 4 Steel tables

$b_f = 250\,mm$

$t_f = 11.6\,mm$

$t_w = 8.3\,mm$

$D = 350\,mm$

$d_h = 24 + 2$

$= 26\,mm$

Given:- M_{15}, $\phi = 24\,mm$, $P = 1700\,KN$

Bearing strength of concrete (7.4.1) $= 0.45 f_{ck}$

$= 0.45 \times 15$

$= 6.75\ N/mm^2$

$e = 1.5 d_h$

$= 1.5 \times 26$

$\boxed{e = 39mm}$

Pitch $= 2.5 \times d$

$= 2.5 \times 24$

$= 60mm$

② Area of base plate (A):

$$A = \frac{1700 \times 10^3}{6.75}$$

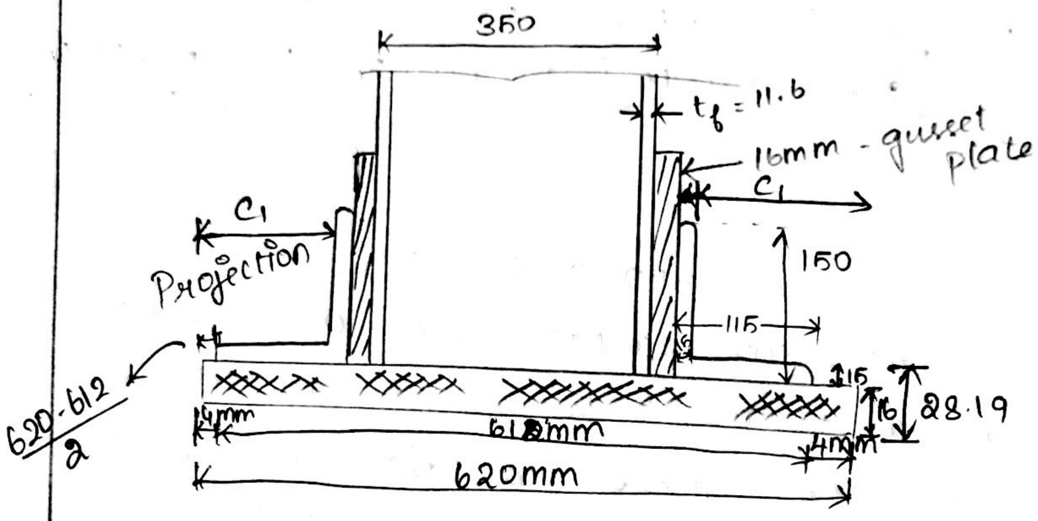
$$\boxed{A = 251.85 \times 10^3 mm^2}$$

The min width required

$= 350 \times (2 \times 16) + (2 \times 115)$

$= 612 \simeq 620mm$

Assume 16mm thick gusset plate on one on each on the two flanges of column and ISA ~~1504~~ 150 x 115 x 15mm by gusseted angle on each side of flange.



∴ Projection of base plate beyond flange angle toe

$$= \frac{620-612}{2}$$

$= 4mm$

Length of base plate $= \frac{A}{L} = \frac{251.85 \times 10^3}{620}$

$= 406.2 \simeq 410mm$

∴ Provide size of base plate $= L \times B$

$= (620 \times 410)\ mm$

(A1).

Bearing pressure on concrete $= \dfrac{[illegible]}{620 \times 410}$

$= 6.68\ N/mm^2 < 6.75\ N/mm^2$

③ B.M calculation:-

critical section of the base for bolted gusset plate base will be @ section X-X as shown in fig.

Let. take section X-X each @ a distance of c_1 from end of base plate.

length of base plate acting as cantilever

$\therefore\ c_1 = (\underbrace{4}_{\text{projection}} + \underbrace{115}_{\text{width of angle}}) - \underbrace{15}_{\text{tks of angle}}$

$= 104\ mm$

calculation of the combined thickness of base plate and gusseted angle @ critical section (t)

$$B.M = M_x = \frac{w c_1^2}{2} = \frac{6.68 \times 104^2}{2}$$

$$= 36125.44\ Nmm.$$

Moment capacity of base plate and angle leg combined.

$$\boxed{M_d = \frac{1.2 f_y Z_e}{\gamma_{mo}}}$$

$Z_e \Rightarrow$ elastic modulus $= \dfrac{bd^2}{6}$

$$= \frac{1 \times t^2}{6}$$

$$M_d = \frac{1.2 \times 250 \times \left(\frac{1 \times t^2}{6}\right)}{1.1}$$

$$M_d = 45.45 t^2$$

$$M_x = M_d$$

$$36125.44 = 45.45\ t^2$$

$$\boxed{t = 28.19\ mm}$$

Tks of base plate $= 28.19 - 15$

$= 13.19 \simeq 16\ mm$

$> 11.6\ mm$

(Tks of flange)

According to 7.4.3.1 $t > t_f$

∴ Provide size of base plate = (620 × 410 × 16) mm

(4). Design of bolted connection:

Assume shear capacity of bolt is minimum.

$$V_{dsb} = \frac{f_u}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s \overset{0}{A_{sb}})$$

Assume as single shear for safety (no. of bolts will be more hence safe)

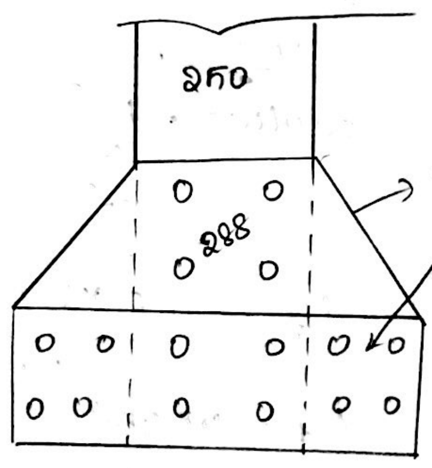
$$= \frac{400 \times 10^{-3}}{\sqrt{3} \times 1.25} (1 \times 0.78 \times \pi/4 \times 24^2)$$

$= 65.21\,KN$

Assume column end and gusset material to have complete bearing, 50% of load will be assumed to pass directly and remaining 50% of load will pass through connections.

∴ No. of plate to connect column flange with gusset plate

$$n_1 = \frac{\left(\frac{50}{100} \times 1700\right)}{65.21} = 13.03 \simeq 16 \text{ nos plate}$$



Provide 8 nos of 24 mm φ bolt.

Take no. of bolts required to connect gusset angle with gusset plate is also same

Size of gusset plate:

Ht. of gusset plate = $150 + (2\times39) + (1\times60)$

$= 288mm$.

Length of gusset plate = width of base plate

$= 410mm$.

Size of G.P : $(410\times288\times16)mm$.

⑥ check for buckling of compression edge of gusset plate:

$$\epsilon_1 = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

Gusset outstand from column

$80 \ngtr 13.66\, \epsilon_1\, t_g$

$= 13.6 \times 1 \times 16$

$= 217.6mm$

Gusseted outstand $\Big\} = \frac{410-250}{2} = 80$

Slant edge of outstand $\Big\} = S_0 = \sqrt{80^2 + (\underbrace{60+(2\times39)}_{288})^2}$

$= 159.51mm \simeq 160mm$

$< 217.6mm$

Hence safe.