- **Summary of solution**

Mem.	Design load	A_g	A_n	U	A_e	Yield strength	Fracture strength	Block-shear strength
*L*4x3x1/2	100 kips	3.25	2.69	0.9	2.41	105 kips	104.8 kips	119.13 kips
		Design strength = 104.8 kips (net section fracture governs) *L*4x3x1/2 is adequate for P_u = 100 kips and the given connection						

- Note: For this problem $A_e/A_g = 2.41/3.25 = 0.741$, which is < 0.745. As predicted by the AISC manual, when $A_e / A_g < 0.745$, net section fracture governs.

28/8/15

Design of compression members

. A vertical compression member in a building is called post (or) stanchion (or) column. compression member in a truss is strut.

, compression member in a crane is called boom.

. columns are classified into i) short ii) Intermediate iii) Long

when the loads are acting 100%. concentric, then the column fails due to crushing of the material (or) yielding of the material in compression. Generally the short columns will fail due to crushing of the material.

. All the times it is not possible to have 100% axial load Hence small (or) large eccentricity will form and a column will buckle (bend) when subjected to this eccentric loading. Due to this buckling the material will fail before it reaches crushing value ∴ the load carrying capacity for a eccentrically loaded column decreases.

Generally all the intermediate & long columns will buckle. As Per IS 800 : 2007, the buckling is classified into four categories class a, class b, class c and class d.

Theoretically the buckling is classified into

i) Local buckling ← In this the individual elements of the column like flange (or) web buckle due to external loading. This can be prevented by providing suitable width to thickness ratio

2) Flexural buckling (Euler's buckling) – Here the total column bends Generally the column section buckles along its length. buckles about the axis corresponding to the largest slenderness ratio. usually the minor principal axis will have least radius of gyration and hence buckles around that axis.

* Compression members of all types of cross-section configuration will fail in this way.

3) Torsional buckling – Thin wall members with open c/s shape are sometimes weak in torsion and hence may buckle by twisting rather than buckling. Torsional buckling occurs when

Torsional rigidity (CJ) < Flexural rigidity (EI)

⊛ All standard hot rolled sections are not susceptible to

Built-up members with thin plates will sometimes subjected Torsional buckling *

4) Flexural - Torsional buckling - This type of failure occurs due to combination of flexural buckling & torsional buckling ie the member bends and twists simultaneously.

* Unsymmetrical cross-sections with one axis of symmetry (or) no axis of symmetry are susceptible for such type of buckling *

* clauses 7.1 (section-7) / Page 34-44 *

(Flexural buckling)

* Calculate the design compressive load capacity of the column ISHB 300 @ 577 N/m if the length of the column is 3m.
i) If both ends are pinned ii) if column is restrained in direction and position at both ends.

Sol From steel tables, details of ISHB 300 @ 577 N/m

$h = 300mm$, $b_f = 250\ mm$, $t_f = 10.6\ mm$, $A = 7484\ mm^2$

$\left(\frac{h}{b_f} = \frac{300}{250} = 1.2;\ t_f = 10.6\ mm \le 100\ mm\right)$ From Table-10 / 44, For I-section the

$\frac{h}{b_f} \le 1.2$

column buckles about z-z axis with Buckling class-b
and y-y " " " " c

case

i) Given Length of the column (L) = 3 m

Effective length (KL) = 1.0L (∵ from table-11 / 45)

Both ends pinned (Hinged)

i.e (KL) = 3000 mm

a) column buckles about z-z axis with buckling class-b

slenderness ratio $\left(\frac{KL}{r}\right) = \frac{KL}{r_{zz}} = \frac{3000}{129.5} = 23.17$ $r_{zz} = 12.95cm$, $r_{yy} = 54.1\ mm$

As per cl 7.1.2.1 / 34, f_{cc} = Euler buckling stress $= \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

$= \frac{\pi^2 \times 2 \times 10^5}{(23.17)^2}$

$\Rightarrow f_{cc} = 3678.13\ N/mm^2$

$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{3678.13}} \Rightarrow \lambda = 0.26$

$\phi = 0.5\left[1+\alpha(\lambda-0.2)+\lambda^2\right]$ $\alpha = 0.34$ $\left(\because \frac{\text{table-7}}{35}\right)$ for class-b

$= 0.5\left[1+0.34(0.26-0.2)+0.26^2\right]$

$\Rightarrow \phi = 0.544$

Design compressive stress, $f_{cd} = \dfrac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda^2\right]^{0.5}}$ $\left(= \dfrac{\chi f_y}{\gamma_{mo}} \le \dfrac{f_y}{\gamma_{mo}}\right)$

$\Rightarrow f_{cd} = \dfrac{250/1.1}{0.544+\left[0.544^2-0.26^2\right]^{0.5}} = 222.41\ N/mm^2$

~~$\neq$ 384.716 N/mm²~~

$\left(\chi = \dfrac{1}{\left[\phi+(\phi^2-\lambda^2)^{0.5}\right]} = 0.9786\right)$

As per $\frac{7.1.2}{34}$, $P_d = A_e f_{cd}$

here $A_e = A_g$ (As per $\frac{7.3.2}{46}$)

$\Rightarrow P_d = A_g f_{cd}$

$= 7484 \times 222.41$

$\Rightarrow P_d = 1664.52\ KN$

(b) column buckles about y-y axis with buckling class-c

slenderness ratio $= \dfrac{KL}{r_{yy}} = \dfrac{3000}{54.1} = 55.45$

As per cl. $\frac{7.1.2.1}{34}$, $f_{cc} = \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ $\lambda = \sqrt{\dfrac{f_y}{f_{cc}}}$

$f_{cc} = 641.98\ N/mm^2$ $= \sqrt{\dfrac{250}{641.98}}$

$\alpha = 0.49$ for class-c $\left(\because \frac{\text{table-7}}{35}\right)$ $\Rightarrow \lambda = 0.624$

$\phi = 0.5\left[1+\alpha(\lambda-0.2)+\lambda^2\right] = 0.5\left[1+0.49(0.624-0.2)+0.624^2\right]$

$\Rightarrow \phi = 0.798$

Now, $f_{cd} = \dfrac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda^2\right]^{0.5}} = \dfrac{250/1.1}{0.798+\left[0.798^2-0.624^2\right]^{0.5}}$ $\Rightarrow f_{cd} = 175.36\ N/mm^2$

$P_d = A_g f_{cd}$

$= 7484 \times 175.36$

$P_d = 1312.43\ KN$

$\therefore$ Design Compressive strength capacity of column when both ends are pinned $= 1312.43\ KN$

Scanned by CamScanner

Case-(I) Restrained in direction & position at both ends

Now Effective length (KL) = 0.65L (∵ From table-11, 45)

= 0.65×3000

⇒ KL = 1950 mm

(a) Column buckles about z-z axis with buckling class 'b'.

Slenderness ratio = $\frac{KL}{r_{zz}} = \frac{1950}{129.5} = 15.05$

By inter polation,

$\frac{KL}{r}$	fcd
10	227
20	225

for $\frac{KL}{r}$ at 15.05, $f_{cd} = 227 - \frac{2}{10}(227-225)$

$f_{cd} = 226.6\ N/mm^2$

$P_d = A_g f_{cd}$

= 7484×~~250~~ 226.6

P_d = 1695.874 KN

(b) column buckles about y-y axis, with buckling class 'c'

$\frac{KL}{r_{yy}} = \frac{1950}{54.1} = 36.04$

$\frac{KL}{r}$	fcd
30	211
40	198

f_{cd} at $\frac{KL}{r} = 36.04 = 211 - \frac{6.04}{10}(211-198)$

$= 203.148\ N/mm^2$

$P_d = A_g f_{cd}$

= 7484×203.148

P_d = 1520.35 KN

Problem on Flexural torsional buckling

Q) An ISA 100 100, 6 mm is used as a strut in a truss. Length of strut b/n intersections at each end is 3m. Calculate the strength of strut if (a) ends are fixed (b) ends are hinged (i) connected by two bolts at each end (ii) connected by 1 bolt at each end (iii) Connected by welding at each end

Sol For ISA 100 100, 6 mm, steel tables $A = 1167\ mm^2$; r_{yy} (r_{min}) $= 19.5mm$

(a) Ends are fixed, Given $L = 3\ m$

(i) connected by two bolts at each end :-

As per $\frac{7.5.1.2}{48}$, $\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\varphi^2}$

From table-12 / 48,
$k_1 = 0.20$
$k_2 = 0.35$
$k_3 = 20$
(for fixed and two bolts)

$$\lambda_{vv} = \frac{\left(\frac{L}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{3000/19.5}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}} \; ; \; \varepsilon = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$$

$$\Rightarrow \lambda_{vv} = 1.73$$

$$\lambda_\varphi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(100+100)/2\times6}{1\sqrt{\frac{\pi^2\times2\times10^5}{250}}} \Rightarrow \lambda_\varphi = 0.187$$

$$\therefore \lambda_e = \sqrt{0.2 + (0.35\times1.73^2) + (20\times0.187^2)}$$

$$\Rightarrow \lambda_e = 1.395$$

1/3/15 As per $\frac{7.1.2.1}{34}$, $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$ $(\lambda = \lambda_e)$

$\alpha = 0.49$ (Table-7 / 35) ($\because$ L-section-class C — Table-10 / 44)

$$\Rightarrow \phi = 0.5\left[1 + 0.49(1.395-0.2) + 1.395^2\right]$$

$$\Rightarrow \phi = 1.765$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{1.765 + [1.765^2 - 1.395^2]^{0.5}} \Rightarrow f_{cd} = 79.84\ N/mm^2$$

cl. $\frac{7.1.2}{34}$, $P_d = A_e f_{cd}$ $\quad A_e = A_g$ ($\because \frac{7.3.2}{46}$)

$P_d = 1167 \times 79.84 \Rightarrow$ **$P_d = 93.17\ kN$**

Scanned by CamScanner

(ii) connected by 1 bolt at each end

From Table-12 (48), $K_1 = 0.75$; $K_2 = 0.35$; $K_3 = 20$

$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\varphi^2}$$

$$= \sqrt{0.75 + (0.35 \times 1.73^2) + (20 \times 0.187^2)}$$

$$\Rightarrow \lambda_e = 1.58$$

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] = 0.5\left[1 + 0.49(1.58 - 0.2) + 1.58^2\right]$$

$$\Rightarrow \phi = 2.086$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{2.086 + [2.086^2 - 1.58^2]^{0.5}} \Rightarrow f_{cd} = 65.9 \text{ N/mm}^2$$

$$P_d = A_e f_{cd} = 1167 \times 65.9 = 76.9 \times 10^3 \text{ N}$$

$$\Rightarrow \boxed{P_e = 76.9 \text{ kN}}$$

(iii) connected by welding at each end

The strength of welded strut will be same as that of single angle strut connected with two bolts at each end with the ends considered fixed.

(b) Ends are hinged

(i) connected by two bolts at each end

$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\varphi^2}$$

From table-12 (48), $K_1 = 0.70$, $\lambda_{vv} = 1.73$

$K_2 = 0.60$, $\lambda_\varphi = 0.187$

$K_3 = 5$

$$\Rightarrow \lambda_e = \sqrt{0.7 + (0.6 \times 1.73^2) + (5 \times 0.187^2)}$$

$$\Rightarrow \lambda_e = 1.634$$

$$\phi = 0.5 \left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] = 0.5 \left[1 + 0.49(1.634 - 0.2) + 1.634^2\right]$$

$$\Rightarrow \phi = 2.18$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} \Rightarrow f_{cd} = 62.73 \text{ N/mm}^2$$

$$P_d = A_e f_{cd} = 1167 \times 62.73 = 73.2 \times 10^3 \text{ N}$$

$$\Rightarrow \boxed{P_d = 73.2 \text{ kN}}$$

||ly (ii) & (iii) are calculated.

* Design of Compression Members:

Procedure:

step-1: Assume design Compressive stress (f_{cd})

f_{cd} = 90 N/mm² — For Angle struts having $\frac{KL}{r}$ = 110 to 130

f_{cd} = 135 N/mm² — For Beam Sections (I) $\frac{KL}{r}$ = 70 to 90

f_{cd} = 200 N/mm² — for Compression members carrying large loads

step-2: calculate 'A_g' using $A_g = \frac{P_u}{f_{cd}}$; P_u = factored Compressive load

step-3: Select a section having area more than A_g required

step-4: calculate r_{min}

step-5: Determine Effective length based on end conditions

Find slenderness ratio $\left(\frac{KL}{r_{min}}\right)$

step-6: Calculate $P_d = A_e f_{cd}$ must be > P_u

* Design a single angle discontinuous strut to carry a factored axial compressive load of 65 kN. Length of strut is 3 m b/n intersection. It is connected to a 12 mm thick gusset plate by 20 mm dia. 4.6 grade bolts. Use Fe410 steel.

Sol Given data, P_u = 65 kN, L = 3 m, d = 20 mm, d_h = 22 mm

Assuming f_{cd} = 90 N/mm² for Angle strut

$$A_g \text{ required} = \frac{P_u}{f_{cd}} \Rightarrow A_{g\,req} = \frac{65\times10^3}{90}$$

$$= 722.22 \text{ mm}^2$$

Try ISA 70 70, 8 mm having A_g = 1058 mm² (> 722.22 mm²)

r_{vv} = 13.5 mm

Calculation of No. of bolts required

Strength of 20 mm dia. 4.6 grade bolts

i, In single shear, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ (cl 10.3.3, 75)

$$= \frac{f_u}{\sqrt{3}}(n_n A_{nb} + A_{sb} n_s)\frac{1}{\gamma_{mb}}$$

$$= \frac{400}{\sqrt{3}}\left(0 + 0.78 \times \frac{\pi}{4}\times 20^2\right)\frac{1}{1.25}$$

$\Rightarrow V_{dsb}$ = 45.27 kN

(ii) In bearing, $V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$

(Cl. 10.3.4 / 75)

$k_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$e \not< 1.5 d_0 \not< 1.5 \times 22 \not< 33mm$

$p \not< 2.5 d \not< 2.5 \times 20 \not< 50mm$

$= \frac{40}{3\times22}, \frac{50}{3\times22} - 0.25, \frac{400}{410}, 1.0$

0.606, 0.507 (k_b), 0.975

$\Rightarrow V_{dpb} = 2.5 \times 0.507 \times 20 \times 8 \times \frac{410}{1.25}$

$\Rightarrow V_{dpb} = 66.58$ KN

∴ strength of bolt is 45.27 KN

No. of bolts required $= \frac{P_u}{V_{dsb}} = \frac{65}{45.27} = 1.4 \approx$ 2 bolts

Assume ~~Hinged~~ Fixed ends, from Table-12 / 48, $k_1 = 0.20$, $k_2 = ~~0.60~~\ 0.35$, $k_3 = 20$

As per 7.5.1.2 / 48, $\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\varphi^2}$

$\lambda_{vv} = \frac{(l/r_{vv})}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(3000/13.5)}{1\sqrt{\frac{\pi^2\times2\times10^5}{250}}} = 2.5$

$\varepsilon = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$

$\lambda_\varphi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(70+70)/2\times8}{1\sqrt{\frac{\pi^2\times2\times10^5}{250}}} = 0.0984$

$\Rightarrow \lambda_e = \sqrt{(0.20) + (0.35\times2.5^2) + (20\times0.0984^2)}$

$\Rightarrow \lambda_e = ~~2.12~~\ 1.606$ (1.637)

Table-7/35 ($\alpha = 0.49$ for class-C)

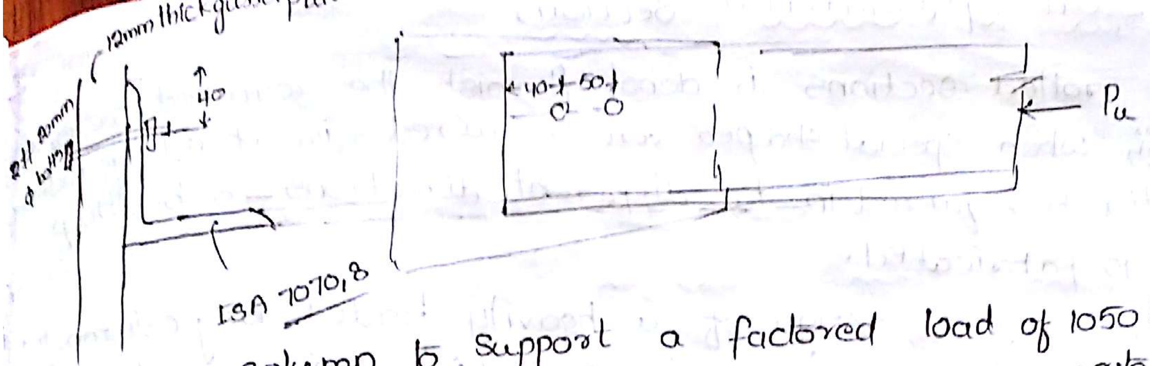
$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.49(1.606 - 0.2) + 1.606^2]$

$\Rightarrow \phi = ~~3.2176~~\ 2.134$

$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = ~~40.81~~\ 64.21\ N/mm^2$

(7.1.2/34) $P_d = A_e f_{cd} = A_g f_{cd} = 1058 \times 64.21 = 67.93$ KN ($>$ 65 KN)

Hence safe



Design a column to support a factored load of 1050 kN. Column has an Effective length of 7m w.r.to Z-axis and 5m w.r.to y-axis use Fe410 steel

Sol Given $P_u = 1050$ kN

Assuming $f_{cd} = 135$ N/mm² — for I-section

A_g required $= \frac{P_u}{f_{cd}} = \frac{1050 \times 10^3}{135} \Rightarrow A_g \text{ req} = 7777.77 \text{ mm}^2$

Try ISHB 350 @ 661.2 N/m having $A_g = 8591$ mm² (> 7777.77 mm²)

$I_{zz} = 19159.7 \times 10^4$ mm⁴ ; $I_{yy} = 2451.4 \times 10^4$ mm⁴ ; $h = 350$ mm ; $b_f = 250$ mm

$t_f = 11.6$ mm ; $r_{yy} = 53.4$ mm ; $r_{zz} = 149.3$ mm

$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2$ and $t_f = 11.6 \text{ mm} < 40$ mm

From Table-10 (44), the given column buckles about z-z axis with class (a)
y-y axis " " (b)

i) Buckling about z-z axis with class-a :

Given, Effective Length (KL) = 7m about zz axis

$r_{zz} = 149.3$ mm

Slenderness ratio $\left(\frac{KL}{r_{zz}}\right) = \frac{7000}{149.3} = 46.88$

$f_y = 250$ N/mm²

From Table-9(a) (40), $f_{cd} = 207.4$ N/mm²

$P_d = A_e f_{cd} = 8591 \times 207.4 = 1781.77$ kN (> 1050 kN)
($A_e \to A_g$)

Hence safe

ii) Buckling about y-y-axis with class-b :

KL = 5mm about y-y axis ; $r_{yy} = 53.4$ mm

$\frac{KL}{r_{yy}} = \frac{5000}{53.4} = 93.63$ and $f_y = 250$ N/mm²

$P_d = A_e f_{cd} = 8591 \times 128.2$

[illegible] N/mm² ; = 1101.36 kN (> 1050 kN)

Built-up (Columns) Sections

When rolled sections (i) donot furnish the required sectional area (ii) when special shapes are required (iii) when large radius of gyration is required in two different directions — a builtup section is fabricated.

For economical design of a heavily loaded long column the least radius of gyration of column section is increased to the maximum. For this the rolled sections are kept away from the centroidal axis of the column and connected by some connecting systems — called Lattice systems.

Commonly, Lattice systems are two types — Lacing / Batten

Primary function of Lacings (or) Battens is to hold and keep the main member of builtup column in their relative position. Hence they are not carried as load carrying elements.

• Flats ~~are~~ / angle sections are normally used as Lacings.

Generally Lacings are connected with single bolt at the end. Lacings are subjected to shear forces due to horizontal forces on the column.

Lacings are designed for compression members and checked for tension.

• Design of Laced Columns :

(i) Design of the column (Built-up) :

ⓐ For built-up columns assume $f_{cd} = 150$ to $190\ N/mm^2$

ⓑ While calculating Effective Slenderness ratio take 'r_{min}' as the maximum value of r_{yy} and r_{zz}

ⓒ As per cl. $\frac{7.6.1.5}{48}$, Increase the $\left(\frac{KL}{r}\right)$ value by 5%.

ⓓ For spacing of column elements equate I_{yy} to I_{zz} for a built-up column.

(2) Design of Lacings :

ⓐ Assume single or double Lacing

ⓑ Assume inclination of Lacings as per $\frac{7.6.4}{50}$

ⓒ Find compressive force in each Lacing part as per $\frac{7.6.6.1}{50}$

ⓓ Check $\frac{a_1}{r_1}$ as per $\frac{7.6.5.1}{50}$

Find dimensions of Lacing part as per 7.6.2 and 7.6.3

(4) Find slenderness ratio, f_{cd} and P_d for Lacing part.
(5) check the lacing bar for tension as per $\frac{6.2 \& 6.3}{32}$.
(6) Design the connection of Lacing bar with main column
(7) Design the tie bar at the end as per $\frac{7.7.2.2 \text{ and } 7.7.2.3}{51}$

Design a built-up column 10m long to carry a factored axial load of 1080 kN. column is restrained in position but not in direction at both ends. provide single Lacing system with bolted connection.
(a) Design the column with two channels placed back to back
(b) " " " " " toe to toe
(c) design of built-up column: " for built-up column

Sol Assuming $f_{cd} = 150\ N/mm^2$ for built-up column

Given $P_u = 1080$ KN

$$A_g \text{ required} = \frac{P_u}{f_{cd}} = \frac{1080 \times 10^3}{150}$$

$$\Rightarrow A_{g\,req} = 7200\ mm^2$$

Try two ISMC 300 @ 351.2 N/m having $A_g = 2 \times 4564 = 9128\ mm^2$ ($> 7200\ mm^2$)

From steel tables Details of ISMC 300,
$A = 4564\ mm^2$; $r_{yy} = 26.1$ mm; $r_{zz} = 118.1$ mm; $C_{yy} = 23.6$ mm
$I_{zz} = 6362.6 \times 10^4\ mm^4$; $I_{yy} = 310.8 \times 10^4\ mm^4$; $b_f = 90$ mm; t_f

In the design of built-up columns with two sections the sections are so spaced that the Least radius of gyration of the builtup section becomes as large as possible. This can be achieved by section in such a way that 'r_{min}' is max. value of spacing the section in r_{yy} (or) r_{zz}

$\therefore r_{min} = r_{zz} = 118.1$ mm

Effective length of column } $KL = 1.0L$ (∵ Table-11/45)

$$\frac{KL}{r_{min}} = \frac{10 \times 10^3}{118.1} = 84.67 = \left(\frac{KL}{r}\right)_o$$

As per $\frac{7.6.1.5}{48}$, $\left(\frac{KL}{r}\right)_e = 1.05\left(\frac{KL}{r}\right)_o$

$$= 1.05 \times 84.67$$

$$\Rightarrow \left(\frac{KL}{r}\right)_e = 88.9 \quad ; \quad f_y = 250\ N/mm^2$$

From Table-$\frac{10}{44}$, channel sections — Buckling class-C

From Table-9(c), $f_{cd} = 122.65\ N/mm^2$

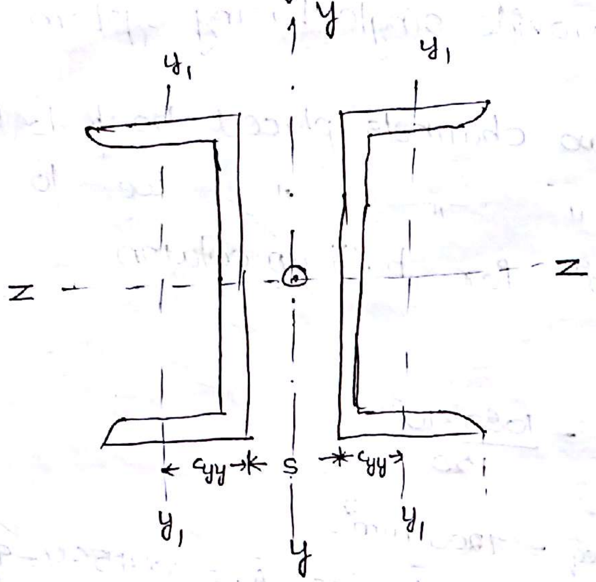
$\Rightarrow P_d = A_e f_{cd} = (2\times4564)\times122.65$
(A_g)

$\Rightarrow P_d = 1119.55$ KN $(> 1080$ KN$)$

Hence safe

@ Two channels placed back to back:

Calculation of Spacing of Channels:



Let 's' be spacing of channels. z-z and y-y axes are centroidal axis for built-up section. y_1-y_1 is centroidal axis for individual channel section.

For calculation of 's' (for built-up section) I_{zz} is equated to I_{yy}

i.e $I_{zz} = I_{yy}$

$2\times6362.6\times10^4 = [I_{y_1y_1} + ah^2]2$

$= 2[310.8\times10^4 + 4564\times(\frac{s}{2}+23.6)^2]$

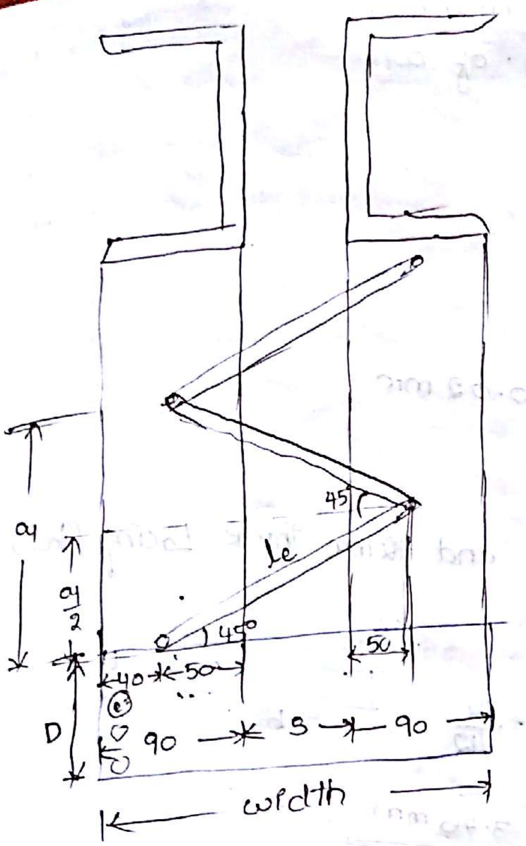
$6362.6\times10^4 = [310.8\times10^4 + (\frac{s^2}{4}+556.96+2(\frac{s}{2}\times23.6))4564]$

$\Rightarrow (s^2 + 2227.84 + 94.49)4564 = 6051.8\times10^4$

$\Rightarrow s = 183.1$ mm Say 183.5 mm

Lacing system: Providing Single Lacing system.

As per $\frac{7.6.4}{50}$, Lacing bars can be inclined at 40° to 70° with horizontal. Hence provide single Lacing system at inclination of 45°



$$\tan 45^\circ = \frac{(a_1/2)}{183.5+50+50}$$

$$\Rightarrow a_1 = 567\ mm$$

let 'l_e' be effective length of lacing $l_e = \sqrt{283.5^2 + 283.5^2}$

$\Rightarrow l_e = 400.93\ mm$

as per $\frac{7.6.5.1}{50}$, r_1 = min. radius of gyration of individual member

$\Rightarrow r_1 = 26.1\ mm$

$\frac{a_1}{r_1} = \frac{567}{26.1} = 21.72 \not> 50$ or $0.7\left(\frac{KL}{r}\right)_e$ whichever is less
62.23

Hence OK

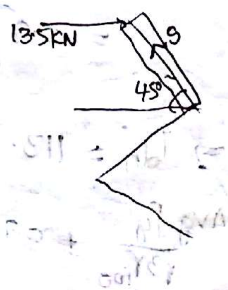
Design of Lacing bar:

as per $\frac{7.6.6.1}{50}$, Total transverse shear $(V_t) = 2.5\%$ of the axial force $= \frac{2.5}{100} \times 1080$

$= 27\ KN$

It shall be divided equally among all transverse-lacing systems in parallel planes.

Transverse shear in each panel $= \frac{27}{2} = 13.5\ KN$



$S\cos 45^\circ = 13.5$

$\Rightarrow S = 19.09\ KN$

Lacing is subjected to a compressive force of 19.09 KN. Hence the lacing has to be designed for this compressive load and should be checked for tension.

Size of Lacing as per $\frac{7.6.2}{50}$, Assuming 16 mm dia. bolts for th

Connection of Lacing with the main member.

width of Lacing = 3 × Nominal dia. of bolt

$= 3 \times 16$

$= 48\ mm$

Say width = 50 mm

Thickness (t) $\nless \frac{1}{40} \times l_e$

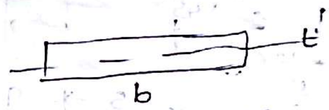
$\therefore \nless \frac{1}{40} \times 400.93 \nless 10.02\ mm$

provide 12 mm thick lacing

∴ Size of Lacing is 50 mm wide and 12 mm thick Lacing flats i.e

50 ISF 12.

$$r_{min} = \sqrt{\frac{I_n}{A}} = \sqrt{\frac{\frac{bt^3}{12}}{bt}} = \frac{t}{\sqrt{12}} = \frac{12}{\sqrt{12}}$$



$\Rightarrow r_{min} = 3.46\ mm$

slenderness ratio (λ) $= \frac{l_e}{r_{min}} = \frac{400.93}{3.46} = 115.74$

Table-10 / 44, Flat is under class-c buckling

from Table-9(c) / 42, $f_{cd} = 88.2\ N/mm^2$

$P_d = A_e \times f_{cd}$ (A_e → A_g) $= (50 \times 12)\,88.2 = 52.92 \times 10^3\ N$

$\Rightarrow P_d = 52.92\ kN > 19.09\ kN$

Hence Safe

Check for Tension for Lacing flats: cl. 6.2 and 6.3 / 32

(a) $T_{dg} = \frac{A_g \times f_y}{\gamma_{mo}} = \frac{(50\times12)(250)}{1.1} \Rightarrow T_{dg} = 136.36\ kN\ (> 19.09\ kN)$

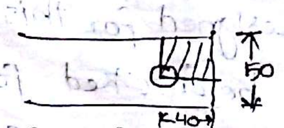
(b) T_{dn} for plate: $= 0.9 \frac{A_n f_u}{\gamma_{m1}}$

$A_n = (50 - 18)\,12 = 384\ mm^2$

$= 0.9 \left(\frac{384 \times 410}{1.25}\right)$

$\Rightarrow T_{dn} = 113.36\ kN\ (> 19.09\ kN)$

(c) $T_{db} = \left(\frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + 0.9 \frac{A_{tn} f_u}{\gamma_{m1}}\right)$ (or)



$T_{db} = 119.66\ kN$

$A_{vg} = 40 \times 12 = 480\ mm^2$

$A_{vn} = (40 - \frac{1}{2} \times 18)\,12 = 372\ mm^2$

$A_{tg} = (25 \times 12) = 300\ mm^2$

$A_{tn} = (25 - \frac{1}{2} \times 18)\,12 = 192\ mm^2$

$$T_{db} = \left(\frac{0.9 A_{nc} f_u}{\sqrt{3}\, \gamma_{m1}} + A_{tg} \frac{f_y}{\gamma_{m0}} \right) = 131.5 \text{ kN} > 19.09 \text{ kN}$$

Hence safe ($T_d > 19.09$ kN)

connection with 16 mm dia. bolt to the main column

strength of 16mm dia. bolt

i, In Double shear,
$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\, \gamma_{mb}} \left[n_n A_{nb} + n_s A_{sb} \right]$$

$$= \frac{400}{\sqrt{3} \times 1.25} \left[\frac{\pi}{4} \times 16^2 + 0.78 \times \frac{\pi}{4} \times 16^2 \right]$$

$$\Rightarrow V_{dsb} = 66.12 \text{ kN}$$

ii, In bearing,
$$V_{dpb} = 2.5\, k_b\, d t \frac{f_u}{\gamma_{mb}}$$

$k_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$$= 2.5 \times 0.491 \times 16 \times 12 \times \frac{410}{1.25} \left(\text{See } \frac{\text{cl } 10.3.4}{75} \right)$$

$e \nless 1.5 \times 18 \nless 27$mm

$p \nless 2.5 \times 16 \nless 40$mm

$$\Rightarrow V_{dpb} = 77.303 \text{ kN}$$

∴ strength of the bolt = 66.12 kN

SF coming on the bolt from the two lacing flats as shown } $SF = 2 \times 19.09 \cos 45°$
$= 27$ KN

∴ No. of bolts required $= \frac{27}{66.12} = 0.408$ Say 1 bolt

connect 50 ISF 12 with ① 16 mm dia. bolt to the main column.

Design of Tie members

as per cl. $\frac{7.7.2.3}{51}$, Tie plates are provided at the ends of Laced Columns

Effective depth $\nless$ ⊥r distance b/n Centroids of main members

width = 183.5+90+90
= 363.5 mm

$\nless S + (2 \times 23.6)$

$\nless 230.7$ mm $\nless$ 2x width of 1 member
2x 90 = 180 mm

overall depth $= 230.7 + (2 \times e) = 230.7 + (2 \times 30) = 290.7 \approx 300$ mm

Thickness (t) $\nless \frac{1}{50} \times$ Distance b/n innermost connecting lines of bolts ⊥r to main member

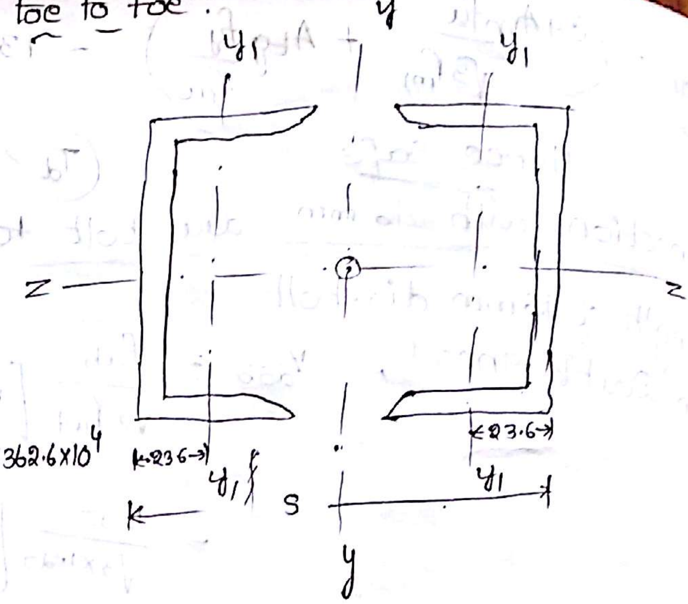
$\nless \frac{1}{50} \times (183.5 + 50 + 50) \nless 5.6$ mm

$\Rightarrow t = 6$ mm

Provide Tie plate of width 363.5 mm, 300mm overall depth and 6mm ... at the bottom

2/3/15 (b) Two channels placed toe to toe :

Spacing of channels (s):



$I_{yy} = I_{zz}$

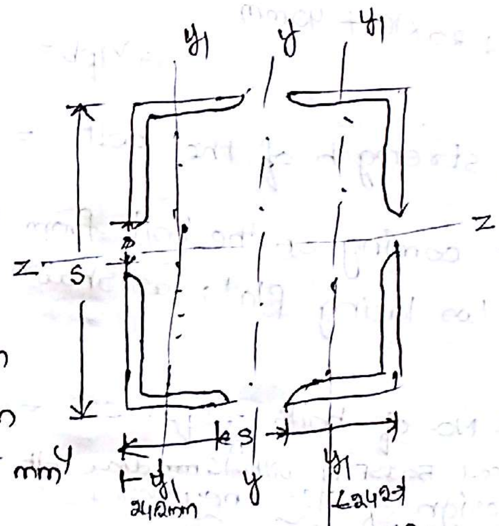
$2\left(I_{y_1y_1} + ah^2\right) = 2\, I_{zz}$

$2\left[310.8\times10^4 + (4564)\left(\frac{S}{2}-23.6\right)^2\right] = 2\times6362.6\times10^4$

$\Rightarrow S = 277.5\,mm$

Lacing system Same as (a)

* Design a built-up column with four angles. The column is 12m long and supports a factored axial compressive load of 700 KN The ends of the column are held in position and restrained against rotation & Design a double Lacing system use Fe410 steel.



Sol Given data, $P_u = 700\,KN$

Assuming $f_{cd} = 170\,N/mm^2$

A_g required $= \frac{P_u}{f_{cd}} = \frac{700\times10^3}{170}$

$\Rightarrow A_{g\,req} = 4117.65\,mm^2$

Try ISA 90.90, 6 mm 4 numbers each

area $A_g = 1047\,mm^2$; $c_{yy} = c_{zz} = 24.2\,mm$

$r_{yy} = r_{zz} = 27.7\,mm$; $I_{yy} = I_{zz} = 80.1\times10^4\,mm^4$

Area of 4 angle sections $= 4\times1047 = 4188\,mm^2$ ($>4117.65\,mm^2$)

Arrangement of the angles :

Length of column (L) = 12m

From Table-11 / 45 , $KL = 0.65L = 0.65\times12\times10^3 = 7800\,mm$

we have $f_y = 250\,N/mm^2$ and $f_{cd} = 170\,N/mm^2$

For Angle section, buckling class is c (∵ From Table-10 / 44)

From page 42 - Table-9(c) $\frac{KL}{r} = 58.67$

As per 7.6.1.5 / 48 , $\left(\frac{KL}{r}\right)_e = 1.05\left(\frac{KL}{r}\right)_0 = 1.05\times58.67$

$\Rightarrow r = \frac{7800}{61.6}$

$\Rightarrow r = 126.6$ mm

we know that $I = AK^2 = Ar^2$

$I = 4188 \times 126.6^2$

$\Rightarrow I_{req} = 6712.3 \times 10^4$ mm^4

Arranging 4 angle sections as shown. Let s be the spacing b/n the channels as shown.

$I_{yy} = I_{zz} = I_{required}$

$4(I_{y_1y_1} + ah^2) = I_{required}$

$\Rightarrow 4\left(80.1 \times 10^4 + 1047 \times \left(\frac{s}{2} - 24.2\right)^2\right) = 6712.3 \times 10^4$

$\Rightarrow s = 295.48$ mm

i.e. $s = 295.5$ mm

As $I_{yy} = I_{zz}$ for the builtup section, for the angle section $I_{yy} = I_{zz}$

∴ the spacing of the builtup column in other direction is also $S' = 295.5$mm

Hence the builtup column is a square column.

For $\left(\frac{KL}{r}\right)_e = 61.6$, $f_y = 250$ N/mm^2 and class-c buckling

as per Table-$\frac{9C}{42}$, $f_{cd} = 165.44$ N/mm^2

$\therefore P_d = A_e f_{cd} = (4 \times 1047)(165.44) = 692.8$ kN ($<$ 700 kN)

Hence Unsafe

Increase the spacing to 300 mm. $I_{provided}$ with 300mm spacing is

$4\left[80.1 \times 10^4 + 1047\left(300/2 - 24.2\right)^2\right] = 6948.17 \times 10^4$ mm^4

$I = Ar^2 \Rightarrow r = \sqrt{\frac{6948.17 \times 10^4}{4188}} = 128.8$ mm

$\left(\frac{KL}{r}\right)_e = \frac{7800}{128.8} = 60.5$

from Table-$\frac{9(C)}{42}$, $f_{cd} = 167.5$ N/mm^2

$\therefore P_d = (4 \times 1047)(167.5) = 701.5$ kN ($>$ 700 kN)

Hence Safe

Double Lacing System:

let the lacing flats be inclined at 45° ($\because \frac{7.64}{50}$)

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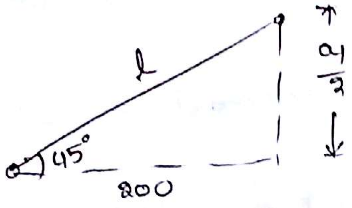

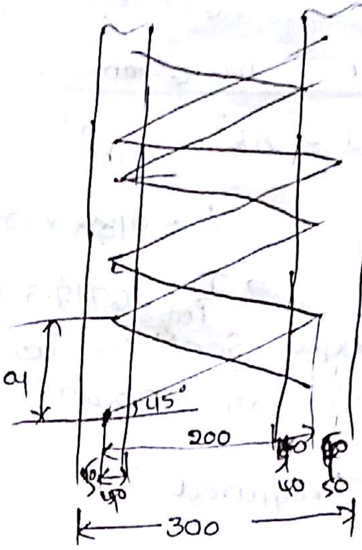

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$\tan 45° = \frac{a_1/2}{200} \Rightarrow a_1 = 400\,mm$

$l = \frac{200}{\cos 45°} \Rightarrow l = 282.84\,mm$

$\left(\frac{a_1}{r_1}\right) = \frac{400}{27.7} = 14.44 \not> 50$ (or) $0.7\left(\frac{KL}{r}\right)_e$ $= 0.7 \times 60.5 = 42.35$

r_1 = min. radius of gyration for individual mem. $= 27.7\,mm$ $\left(\because cl \frac{7.6.5.1}{50}\right)$

Hence ok

Design of Lacing bar

As per $\frac{7.6.6.1}{50}$, Transverse shear $(V_t) = 2.5\%$ of Axial force

$= \frac{2.5}{100} \times 700$

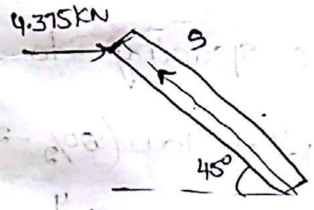
$\Rightarrow V_t = 17.5\,kN$

Force Transfered in each lacing system $= \frac{17.5}{2} = 8.75\,kN$

For double Lacing system each lacing flat will be subjected to

$\frac{8.75}{2} = 4.375\,kN$ load (transverse).

Compressive force on the lacing bar



$S\cos 45° = 4.375$

$\Rightarrow S = 6.187\,kN$

Lacing flat Design:

Each lacing flat should be designed for a compressive force of 6.187 KN and should be checked for tension.

Effective length of lacing bar is 'l_e'

As per $\frac{7.6.3}{50}$, $t \not< \frac{1}{60} \times l_e$. $\left[cl \frac{7.6.6.3}{50}, l_e = 0.7 l\right.$ $= 0.7 \times 282.84$ $\left.\Rightarrow l_e = 197.98\,mm\right]$

$t \not< \frac{1}{60} \times 197.98$

$\not< 3.299$ say 3.3mm

As per $\frac{7.6.2}{50}$, minimum width = 3 dia. of bolt (Assume 16mm dia. bolt)

$= 3 \times 16$

$= 48\,mm$ Say 50 mm

∴ provide 50 ISF 6

r_{min} for the flat $= \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{bt^3}{12}}{bt}} = \frac{t}{\sqrt{12}} = \frac{6}{\sqrt{12}}$

$\Rightarrow r_{min} = 1.732$ mm

$\frac{l_e}{r} = \frac{197.98}{1.732} = 114.3$

For slenderness ratio – 114.3, $f_y = 250 \ N/mm^2$ and class-C buckling from Table-9(c) 42, $f_{cd} = 89.7 \ N/mm^2$

$P_d = A_e \times f_{cd} = A_g \times f_{cd} = (50\times6) \times 89.7 = 26.91 \ kN > 6.187 \ kN$

∴ Hence Safe

check for stresses in Tension:

(a) $T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(50\times6)(250)}{1.1} = 68.18 \ kN > 6.187 \ kN$

(b) $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9\times(50-18)6\times410}{1.25}$

$\Rightarrow T_{dn} = 56.67 \ kN > 6.187 \ kN$

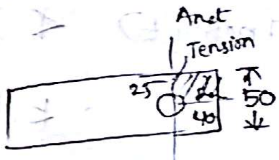
$A_n = (50-18)6$

(c) $T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$ (or) $\frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$

59.83 kN , (or)

$A_{vg} = 40\times6$ $A_{tg} = 25\times6$

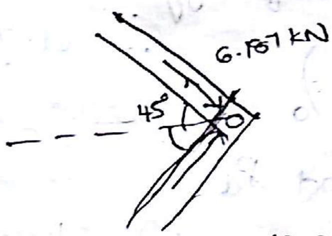
$A_{vn} = (40 - \frac{1}{2}\times18)6$ $A_{tn} = (25 - \frac{1}{2}\times18)6$



Hence safe in compression as well as in Tension.

Connections:

05/3/15



Force transmitted from the lacing bars on to the bolt $= 6.18 \cos 45° \times 2$

$= 8.74$ kN

Using 16 mm ϕ bolts, strength of bolt in

(a) Double shear (V_{dsb}) $\frac{f_u}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3}\times1.25}\left[1\times\frac{\pi}{4}\times16^2 + 1\times0.78\times\frac{\pi}{4}16^2\right]$

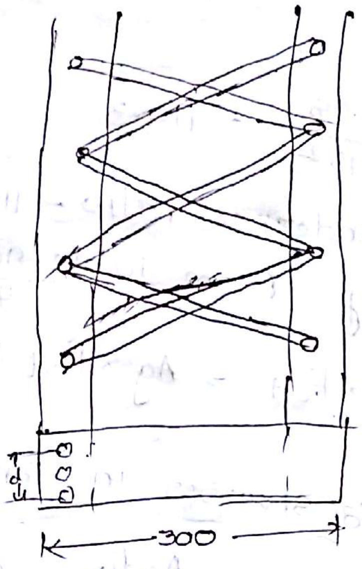
$\Rightarrow V_{dsb} = 66.12$ kN

(b) Bearing (V_{dpb}) $= 2.5 k_b d t \frac{f_u}{\gamma_{mb}} = 2.5\times0.49\times16\times6 \times \frac{410}{1.25}$

(k_b = d/3d0 ...)

∴ strength of bolt = 38.5 KN

no. of bolts required = $\frac{8.75}{38.5}$ = 0.227 ≃ 1 bolt



Tie plate

as per $\frac{cl.7.7.2.2}{51}$, width = 300 mm

Eff. depth = In distance b/n centroids of main members
(d)

= 300 − (2×24.2)

⇒ d = 251.6 mm

Overall depth = d + (2×e)   e ≮ 1.5 d_h

= 251 + (2×40)   p ≮ 2.5 d

= 331 mm say 340 mm

thickness (t) ≮ $\frac{1}{50}$ (300 − (2×50))

≮ 4 mm

say 6 mm thickness

Provide 340 mm overall depth and 6 mm thick tie plate one at the top end and another at the bottom end.

* Design of column with Battens :

① Design of built-up column

(a) for built-up column assume fcd = 150 to 190 N/mm²

(b) Take r_{min} as max. of r_{yy} and r_{zz}.

Find $\left(\frac{KL}{r}\right)_e = 1.1\left(\frac{KL}{r}\right)_0$ $\left(\because \frac{cl.7.7.1.4}{51}\right)$

(c) calculate fcd and P_d

(d) spacing of elements in the column

Equate I_{yy} to I_{zz} and increase the spacing to increase I_{yy} val

② Design of Battens : $\left(\frac{cl.7.7.1.9}{50}\right)$

(a) $\frac{cl.7.7.3}{51}$ Spacing of battens

(b) $\frac{7.7.2.3}{51}$ Size of end and intermediate battens.

(c) $\frac{7.7.2.1}{51}$ Design forces on battens

(d) check for shear stress and bending stresses for end battens and intermediate battens.

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(e) Connections – Should be designed to transmit both shear and bending Moment.

(f) No. of batten plates

(g) Real spacing of battens

* Design a builtup column 9m long to carry a factored axial compressive load of 1100 kN. column is restrained in position but not in direction at both ends. Design the column with connecting systems as battens.

Use two channels back-to-back.

Design of built-up column

Sol Given data, $P_u = 1100\text{ kN}$

$L = 9\text{ m}$

Effective length $(KL) = 1.0L$ ($\because$ from Table-11 / 45)

$= 9\text{ m}$

Assume $f_{cd} = 150\text{ N/mm}^2$

$$A_{g\,required} = \frac{P_u}{f_{cd}} = \frac{1100\times10^3}{150} = 7333.33\text{ mm}^2$$

Try 2 ISMC 300 @ 351.2 N/m having $A_g = 2\times4564$

$= 9128\text{ mm}^2 (> 7333.33\text{ mm}^2)$

Details

$r_{zz} = 118.1\text{ mm}$ $\quad I_{zz} = 6366\times10^4\text{ mm}^4$ $\quad c_{yy} = 23.6\text{ mm}$

$r_{yy} = 26.1\text{ mm}$ $\quad I_{yy} = 310.8\times10^4\text{ mm}^4$ $\quad b_f = 90\text{ mm}$

Slenderness ratio $\left(\frac{KL}{r_{min}}\right) = \frac{9\times10^3}{118.1} = 76.2$

As per 7.7.1.4 / 51, $\left(\frac{KL}{r}\right)_e = 1.1\left(\frac{KL}{r}\right)_0$

$= 1.1\times76.2$

$= 83.8$

For $\frac{KL}{r} = 83.8$, $f_y = 250\text{ N/mm}^2$ For channel class-C buckling (Table-10 / 44)

From Table-9(C) / 42, $f_{cd} = 130.3\text{ N/mm}^2$

$P_d = A_e f_{cd} = A_g f_{cd}$

$P_d = (2\times4564)\,130.3 \Rightarrow P_d = 1189\text{ kN} (> 1100\text{ kN})$

Hence safe

Arrangement and Spacing of channels:

Providing channels back to back to get the spacing b/n the channels Equate I_{yy} to I_{zz}.

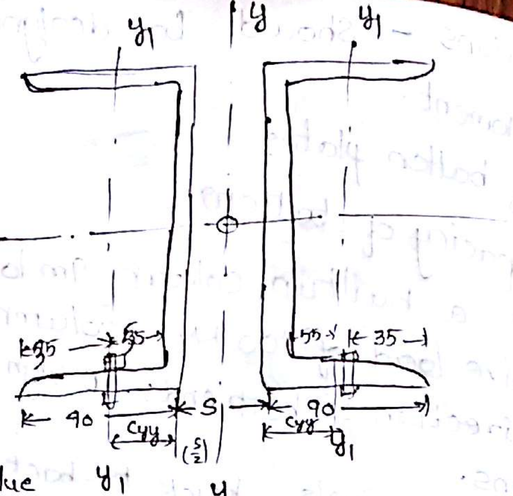
$I_{yy} = I_{zz}$

$2[I_{y_1y_1} + ah^2] = 2 \times I_{zz}$

$\Rightarrow 2[310.8 \times 10^4 + (4564 \times (\frac{S}{2} + 23.6)^2)]$

$= 2 \times 6366 \times 10^4$

$\Rightarrow S = 183.1$ mm

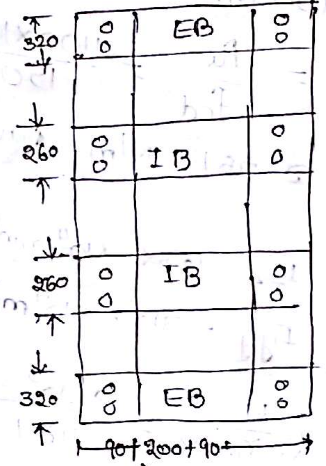


To make $I_{yy} > I_{zz}$ increase S value

Take S = 200 mm

Battens : (See cl. 7.7.1.3/50)

Battens (or) plates provided one at top end of column, other at bottom end of column & others uniformly at equal spacing.



Minimum no of battens = 2+1 = 3 (✱)

Spacing of Battens: (C)

$\frac{C}{r_y} <$ 0.7 × slenderness ratio of column as a whole

$< 0.7 \times 83.82$

< 58.67

$C < 26.1 \times 58.67 < 1531.4$ mm

$\frac{C}{r_y} \not> 50 \Rightarrow C \not> 1305$ mm (see page-51)

provide battens at a spacing of 1300 mm

Size of End battens:

Using flats for battens, using 20mm dia bolts for connection

d = 20mm $\quad e \not< 1.5 d_h \not< 1.5 \times 22 \not< 33$ mm Say 35 mm

d_h = 22 mm

As per $\frac{7.7.2.3}{51}$, d ≮ ⊥r distance b/n centroids of main members

$\not< 200 + (2 \times 23.6)$

$d \not< 247.2$ mm

width of batten = 200+90+90 = 380mm

Overall Depth = d + 2e

$= 247.2 + 2 \times 35$

= 317.2mm say 320mm

thickness (t) $\not< \frac{1}{50} \times (55+200+55) \not< 6.2$ mm Say 8mm

Hence provide 380 mm width, 320mm overall depth and 8mm thick end batten plate one at the top end and other at the bottom end of the column.

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Design of intermediate battens:

As per cl. 7.7.3 (IS 800), $d \nless \frac{3}{4} \times [200 + (2 \times 23.6)]$

$\Rightarrow d \nless 185.4$ mm

Overall depth = (d + 40)

= 185.4 + (2 × 45)

= 275.4 mm say 260 mm

thickness is same as end batten thickness i.e 8mm ✓

Hence provide 320mm wide, 260mm overall depth and 8mm thick intermediate battens.

As per cl. 7.7.2 (IS 800) Design of Battens:

Transverse SF (V_t) = 2.5% of the total axial force

$= \frac{2.5}{100} \times 1100$

$\Rightarrow V_t = 27.5$ kN

Shear $V_b = \frac{V_t C}{NS}$ $= \frac{27.5 \times 10^3 \times 1300}{2 \times (55 + 200 + 55)}$ $= 57.66 \times 10^3$ N (or) 57.66 kN

(see fig-11 IS 800)

Moment, $M = \frac{V_t C}{2N}$ $= \frac{27.5 \times 10^3 \times 1300}{2 \times 2}$ $= 8.9375 \times 10^6$ Nmm

Check for stresses:

i) For End battens, shear stress $= \frac{V_b}{c/s\ area}$

$= \frac{57.66 \times 10^3}{320 \times 8}$

$= 22.52$ N/mm² $< \frac{f_y/\sqrt{3}}{\gamma_{m0}}$ $(< 131.21$ N/mm²$)$

Hence safe

Bending stress $= \frac{M}{Z}$ $Z = \frac{I}{y} = \frac{bd^3/12}{d/2} = bd^2/6$

$= \frac{8.9375 \times 10^6}{\left(\frac{8 \times 320^2}{6}\right)}$

$= 65.46$ N/mm² $< \frac{f_y}{\gamma_{m0}} < 227.27$ N/mm²

ii) For Intermediate battens,

Shear stress $= \frac{V_b}{c/s\ area} = \frac{57.66 \times 10^3}{260 \times 8} = 27.72$ N/mm² $< \frac{f_y/\sqrt{3}}{\gamma_{m0}} < 131.2$ N/mm²

Bending stress $= \frac{M}{Z} = \frac{8.937 \times 10^6}{\left(\frac{8 \times 260^2}{6}\right)} = 99.1$ N/mm² $< \frac{f_y}{\gamma_{m0}} < 227.27$ N/mm²

Connections: The connections should be designed to transmit both shear and BM.

07/3/15 using 20 mm dia. bolts strength of single bolt in shear } $V_{dsb} = \frac{f_u}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3}\times 1.25}\left(0.78\times\frac{\pi}{4}\times 20^2\right)$

$= 45.27\ kN$

In bearing, $V_{npb} = 2.5\, k_b\, d\, t\, \frac{f_u}{\gamma_{mb}}$

$k_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$= 0.606, \ 0.507, \ 0.97, \ 1.0$

$e \nless 1.5 d_h$

$\nless 1.5\times 22$

$\nless 33\,mm$ say $40\,mm$

$p \nless 2.5d$

$\nless 50\,mm$ say

$= 2.5\times 0.507\times 20\times 8\times \frac{410}{1.25}$

$= 66.52\ kN$

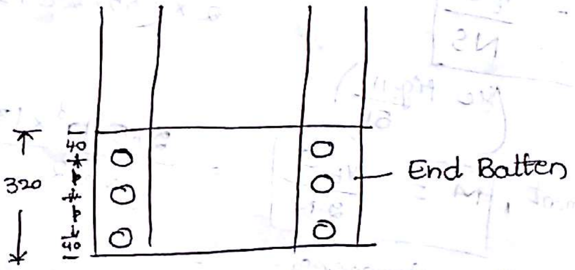
∴ Strength of bolt $= 45.27\ kN$

$V_b = 57.66\ kN$

⇒ no. of bolts required $= \frac{57.66}{45.27} = 1.274$ Say 3 bolts i.e. to accomodate for BM also.

Each pitch $= \frac{320 - 2\times 40}{2}$

⇒ $p = 120\,mm$



Shear to be resisted by each bolt } $= \frac{57.66}{3}$

$= 19.22\ kN$

~~BM to be~~ Force in each bolt i) due to shear $= 19.22\ kN$

ii) due to BM $= \frac{Mr}{\Sigma r^2}$ where r is pitch

$= \frac{8.9375\times 10^6 \times 120}{120^2 + 120^2}$

$= 37.24\ kN$

Resultant force on each bolt $= \sqrt{19.22^2 + 37.24^2}$

$= 41.9\ kN$ < strength of bolt (45.27 kN)

Hence safe

No. of batten plates required on each phase $= \frac{\text{Length of column}}{\text{batten spacing}} + 1$

$= \frac{9000}{1300} + 1$

$= 7.92$ Say 8 battens

Revise the centre to centre distance of battens $= \frac{9\times 10^3}{8-1} = 1285.7$ < 1305

Provide 2 End Battens & 6 Intermediate battens on each Face of the column and connect it with 3 # 20 dia. bolts with an edge distance of 40 mm & a pitch of 120 mm.