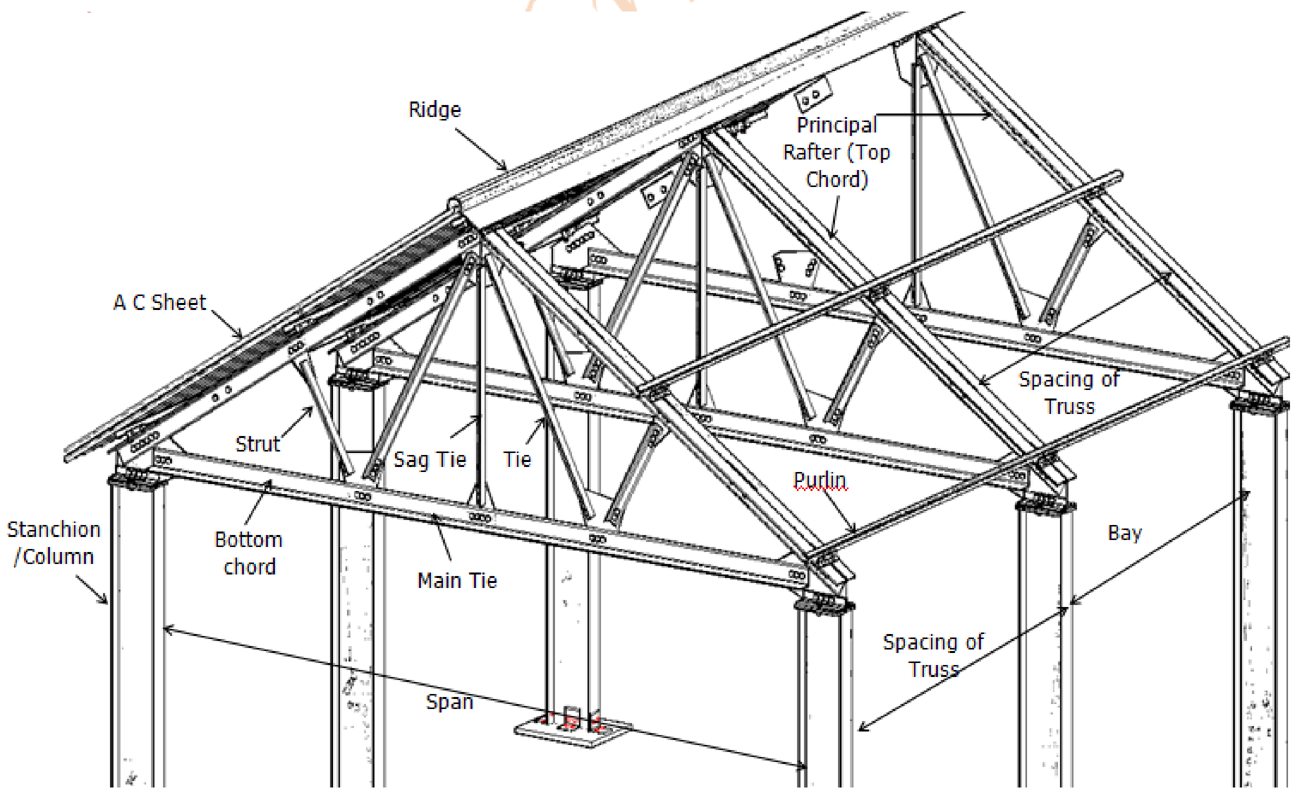
Thursday, January 18, 2018 21:16:04

Design of Compression Members: Introduction, Failure modes, Behaviour of compression members, Sections used for compression members, Effective length of compression members, Design of compression members and built up Compression members, Design of Laced and Battened Systems.

Introduction :

The structural members carrying compressive load in truss are called struts. The vertical members carrying axial loads in a building are called columns or stanchions. The compression member of a crane is called a boom the main compression members of a roof truss are called rafters (Principal rafter and common rafter).

Common hot rolled and built – up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial low and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by class a, b, c, or d as given in Table 7, P - 35.

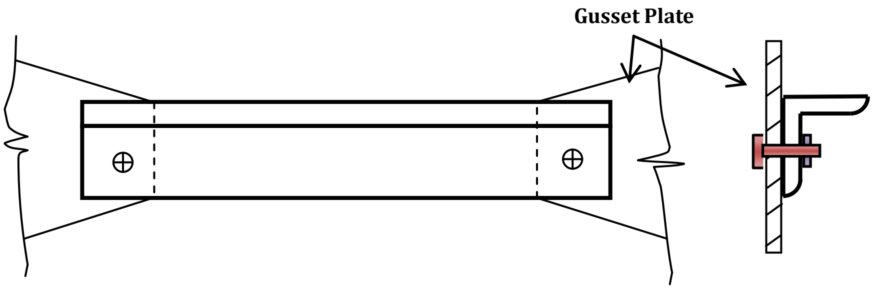


Analysis of single angle struts:

Data given: IS angle section, length of member.

Its required to determine the compressive strength of member

Procedure:



P-34, cl:7.1.2

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

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

f_{cd} can be obtained (P- 34)

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

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

λ = Non - dimensional elective slenderness ratio

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

Where, f_{cc} = Euler Buckling stress = $\dfrac{\pi^2 E}{\left(KL/r\right)^2}$

Where, KL/r = Effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration 'r'.

α = Imperfection factor given in Table 7 (Page 35).

Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

Table 7: Imperfection factor, α

Buckling Class	A	B	c	D
α	0.21	0.34	0.49	0.76

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

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

γ_{mo} = Partial safety factor for material strength.

E = Young's modulus of the member = 2 x 10^5 N/mm^2

Effective slenderness ratio: **P-48, Cl 7.5.1.2:**

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

k_1, k_2, k_3 = Constants depending upon the end condition as given in Table 12.

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \qquad \varepsilon = \text{Yield stress ratio} = \left(\frac{250}{f_y}\right)^{0.5}$$

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

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

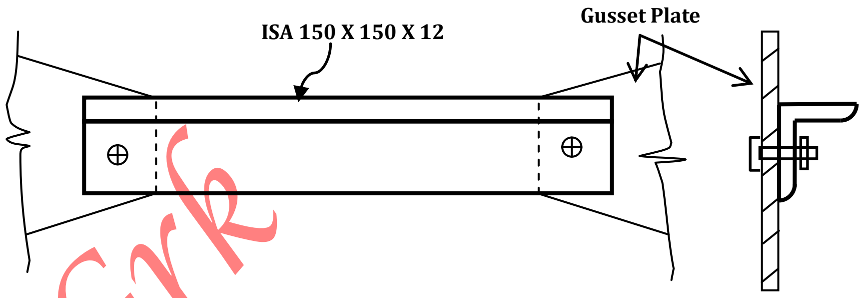
$$\mathbf{P_d = f_{cd} \times A_e}$$

Problem:®

A single angle discontinuous strut ISA 150 x 150 x 12 Th. @ 0.272KN/m with single Bolted connection is 3.5 m long. Calculate flexural buckling strength of section. Assume the fixidity as hinged.

Solution:

Properties of ISA 150 x 150 x 12 @ 0.272 KN / m.



$$a = 34.59\text{cm}^2 = 3459\text{mm}^2$$

$$r_{vv} = 2.93\text{cm} = 29.3\text{mm}$$

$$r_{vv} = 29.3\text{m}$$

$$\text{Effective length}(KL) = 3.5\text{m} = 3500\text{mm}$$

P-34, cl:7.1.2

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

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

P- 34 f_{cd} can be obtained

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} \le f_y/\gamma_{mo} \le \frac{250}{1.1} = 227.27\ \text{N/mm}^2$$

Effective slenderness ratio: **P-48, Cl 7.5.1.2:**

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

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$k_1 = 1.25, \quad k_2 = 0.5, \quad k_3 = 60$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{3500}{29.3}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 1.34$$

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

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(150+150)/2\times 12}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.14$$

$$\lambda_e = \sqrt{1.25 + 0.5\times 1.34^2 + 60\times 0.14^2} = 1.82$$

Where, $\phi = 0.5\times\left[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2\right]$

$\alpha =$ Imperfection factor given in Table 7. Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

$\alpha = 0.49$

$$\phi = 0.5\times\left[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2\right] = 0.5\left[1 + 0.49(1.82 - 0.2) + 1.82^2\right] = 2.55$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{250/1.1}{2.55 + \left[2.55^2 - 1.82\right]^{0.5}}$$

$$f_{cd} = 52.42\text{N/mm}^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\text{N/mm}^2 \qquad \text{Safe}$$

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

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

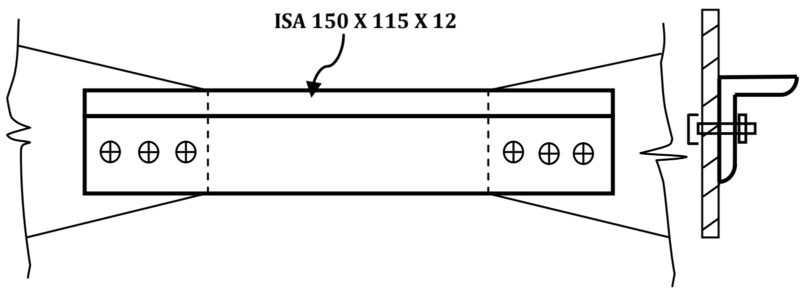
$$P_d = \frac{52.42\times 3459}{1000} = 181.30\text{ KN}$$

Problem:®

A single angle discontinuous strut ISA 150 x 150 x 12 Th. @ 0.272KN/m is 3.5 m long is fixed with more than 2 bolts. Calculate flexural buckling strength of section. Assume the end as fixed.

Solution:

Properties of ISA 150 x 150 x 12 @ 0.272 KN / m.



$a = 34.59\text{cm}^2 = 3459\text{mm}^2$

$r_{zz} = r_{yy} = 4.61\text{cm} = 46.1\text{mm}$

$r_{uu} = 5.83\text{cm} = 58.3\text{mm}$

$r_{vv} = 2.93\text{cm} = 29.3\text{mm}$

$r_{min} = r_{vv} = 29.3\text{m}$

$\text{Effective length}(KL) = 3.5\text{m} = 3500\text{mm}$

Effective slenderness ratio (P-48, Cl 7.5.1.2)

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

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$k_1 = 0.2, \qquad k_2 = 0.35, \qquad k_3 = 20$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{3500}{29.3}}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 1.34$$

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

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(150+150)/2\times 12}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.14$$

$$\lambda_e = \sqrt{0.2+0.35\times 1.34^2+20\times 0.14^2} = 1.1$$

From P- 34 f_{cd} can be obtained

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

Where,

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

α = Imperfection factor given in Table 7. Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

α = 0.49

$$= 0.5\left[1+0.49(1.1-0.2)+1.1^2\right] = 1.33$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.33+\left[1.33^2-1.1\right]^{0.5}}$$

$$f_{cd} = 109.27 N/mm^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\ N/mm^2 \qquad \text{Safe}$$

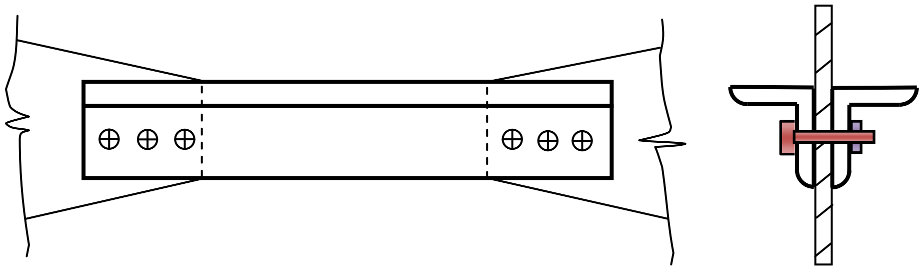
Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

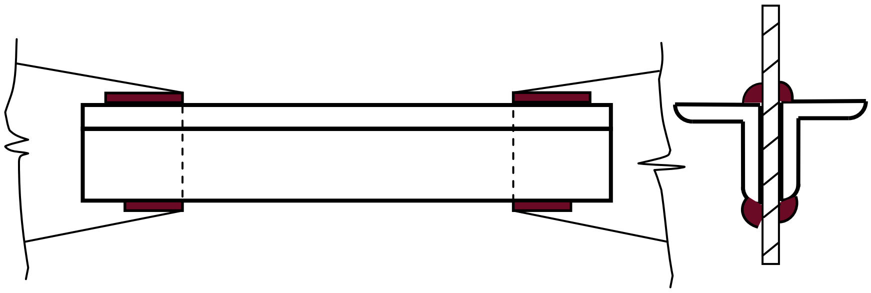
$$P_d = f_{cd} \times A$$

$$P_d = \frac{109.27\times 3459}{1000} = 377.95\ KN$$

<u>Clause 7.5.2.1, P- 48, Double Angle Struts</u>

A) <u>Double angle discontinuous struts back to back connected on both sides of the gusseted by not less than 2 rivets(Bolts) in a line or welding</u>





Data given: Double angle section, length of member.

It's required to determine the compressive strength of member

Procedure:

Effective length :

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

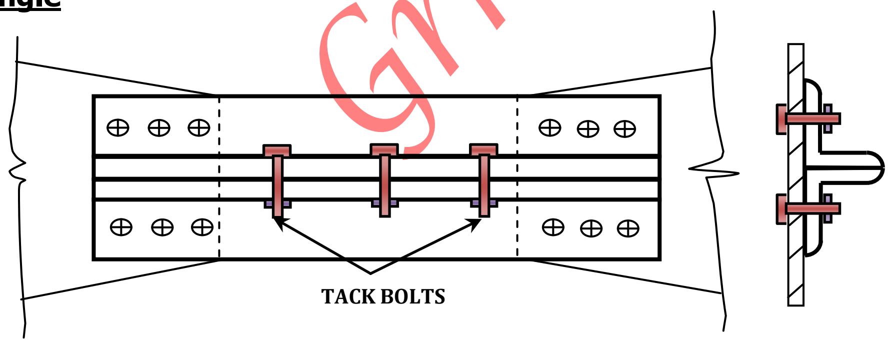
Effective slenderness ratio

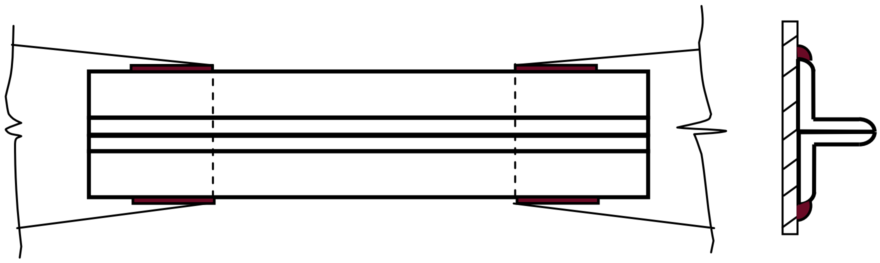
$$\lambda_e = \frac{KL}{r_{min}} \not> 180 \qquad \text{P - 20, Table 3.}$$

Ref Table 9(c) and find f_{cd}

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

B) Clause 7.5.2.2, P-48,Double angle discontinuous struts back to back connected to one side of a gusset by one or more rivets (Bolts) or welding in each angle

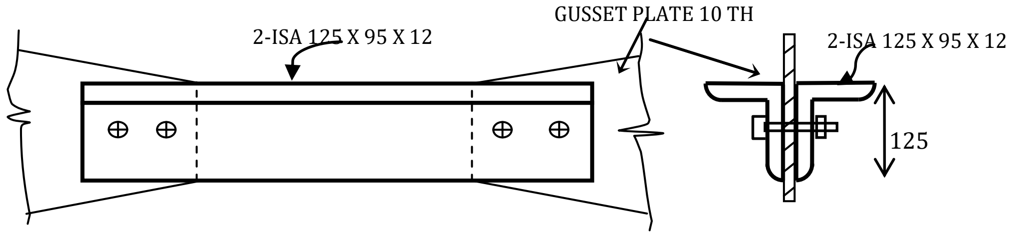




Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***long legs back to back*** is connected to both the sides of gusset plate 10 mm thick with 2 bolts. The length of strut b/w c/c of intersection is 4 m. determine the flexural torsional strength of the section.

Solution:



Properties of 2-ISA 125 x 95 x 12

$a = 49.96\ cm^2 = 4996\ mm^2$

$r_{zz} = 3.91\ cm = 39.1\ mm$

$r_{yy} = 4.05\ cm = 40.5\ mm$ (10mm th. gusset plate)

$r_{min} = r_{zz} = 39.1mm$

$KL = 0.7 \times L$ to $0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4m = 3400$ mm

Effective slenderness ratio $= \frac{KL}{r_{min}} = \frac{3400}{39.1} = 86.96 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250 N/mm^2$

λ	f_{cd}
80	136
86.96	?
90	121
10	15
6.96	? (x)

f_{cd} for $86.96 = 136 - \frac{6.96 \times 15}{10} = 125.56\ N/mm^2$

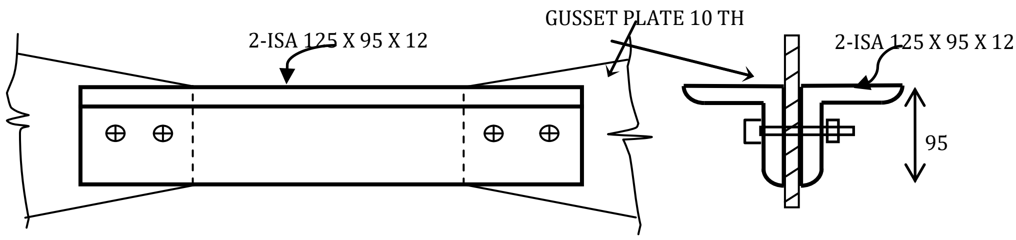
Buckling Strength of the member= Safe stress x area provided

$= f_{cd} \times A = \frac{125.56 \times 4996}{1000} = 627.30$ KN.

Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***short legs back to back*** is connected to both the sides of gusset plate 10 mm thick with 2 bolts. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

Solution:



Properties of 2-ISA 125 x 95 x 12

$a = 49.96cm^2 = 4996mm^2$ $r_{yy} = 5.93cm = 59.3mm$ (10mm th. gusset plate)

$r_{zz} = 2.76cm = 27.6mm$ $r_{min} = r_{zz} = 27.6mm$

$KL = 0.7 \times L$ to $0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4m = 3400$ mm

Effective slenderness ratio $= \frac{KL}{r_{min}} = \frac{3400}{27.6} = 123.20 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

$$f_{cd} \text{ for } 123.20 = 83.7 - \frac{3.2 \times 9.4}{10} = 80.69\ N/mm^2$$

λ	f_{cd}
120	83.7
123.20	?
130	74.3
10	9.4
3.20	? (x)

Buckling Strength of the member= Safe stress x area provided

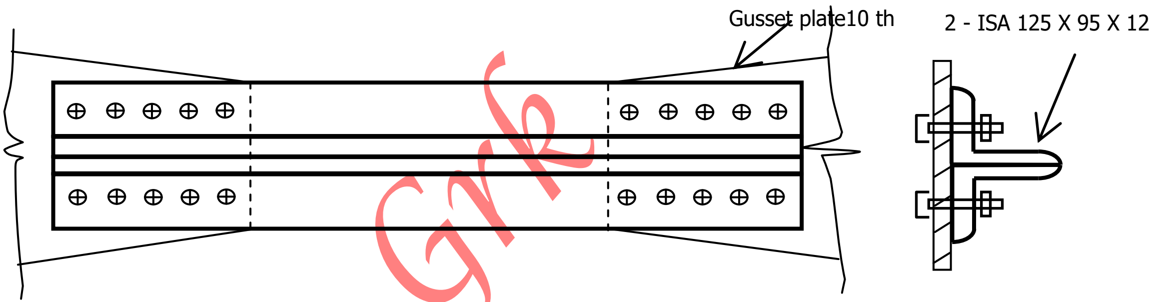
$$P_d = f_{cd} \times A = \frac{80.69 \times 4996}{1000} = 403.13KN.$$

Double angles connected to the same side of gusset plate

Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***long legs back to back*** is connected to same side of gusset plate 10 mm thick with 10 bolts on each end. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

Solution:



Properties of 2-ISA 125 x 95 x 12

$a = 49.96cm^2 = 4996mm^2$ $r_{yy} = 3.7cm = 37mm$('0' mm th. gusset plate)

$r_{ZZ} = 3.91cm = 39.1mm$ $r_{min} = r_{YY} = 37mm$

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

$$\text{Assuming } KL = 0.85 \times L = 0.85 \times 4m = 3400 \text{ mm}$$

$$\text{Effective slenderness ratio} = \frac{KL}{r_{min}} = \frac{3400}{37} = 91.89 < 180 \qquad \text{Safe}$$

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

$$f_{cd} \text{ for } 91.89 = 121 - \frac{1.89 \times 14}{10}$$

$$f_{cd} = 118.35\ N/mm^2 \prec \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27\ N/mm^2$$

λ	f_{cd}
90	121
91.89	?
90	107
10	14
1.89	? (x)

Buckling Strength of the member= Safe stress x area provided

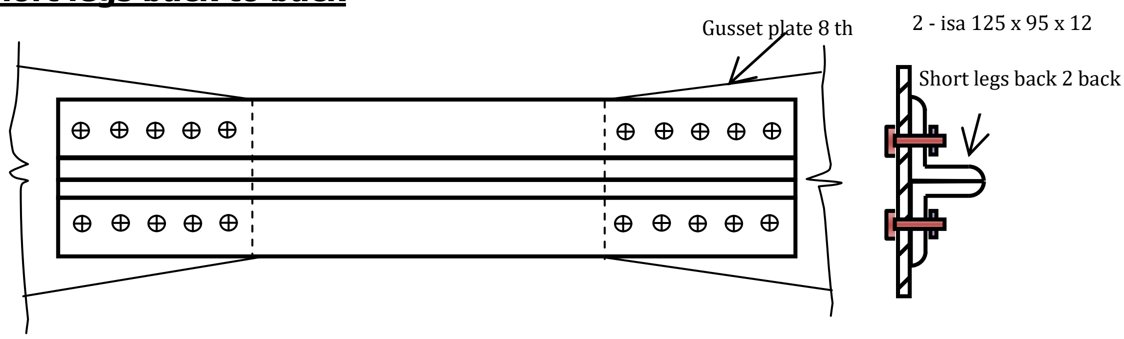
$$= f_{cd} \times A = \frac{118.35 \times 4996}{1000} = 591.30\ KN.$$

Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***short legs back to back*** is connected to same side of gusset plate 10 mm thick with 2 bolts or more bolts. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

Solution:

Short legs back to back



Properties of 2-ISA 125 x 95 x 12

$a = 49.96\text{cm}^2 = 4996\text{ mm}^2$ $\qquad r_{yy} = 5.57\text{cm} = 55.7\text{mm}(\text{'0' mm th. gap})$

$r_{zz} = 2.76\text{cm} = 27.6\text{ mm}$ $\qquad r_{min} = r_{zz} = 27.6\text{ mm}$

$KL = 0.7 \times L \text{ to } 0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4\text{m} = 3400\text{mm}$

Effective slenderness ratio $= \dfrac{KL}{r_{min}} = \dfrac{3400}{27.6} = 123.20 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250\text{ N/mm}^2$

f_{cd} for $123.20 = 83.7 - \dfrac{3.2 \times 9.4}{10} = 80.69\text{ N/mm}^2$

λ	f_{cd}
120	83.7
123.20	?
130	74.3
10	9.4
3.20	? (x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{80.69 \times 4996}{1000} = 403.13\text{ KN.}$$

Saturday, September 01, 2001 6:18:03 PM

DESIGN PROBLEMS

a) Design Procedure for single angle Struts:

1. Assume Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ \text{N/mm}^2$
2. Calculate Area of section required

$$\text{Area} = \frac{\text{Load}}{\text{Compressive stress}}$$

3. Choose a suitable section from the steel table by assuming 15 % to 25% more than Area required.
4. Calculate Effective slenderness Ratio

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

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$$\lambda_{vv} = \frac{\left(\dfrac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda_{vv} = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}}$$

Where,

l = C/c length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1 , b_2 = Width of the two legs of the angle

t = Thickness of the leg, and

ε = Yield stress ratio = $\left(\dfrac{250}{f_y}\right)^{0.5}$

Ref P- 42,

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

Where,

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

α = Imperfection factor given in Table 7.

α = 0.49

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$$\mathbf{P_d = f_{cd} \times A > P}$$

End Connection:

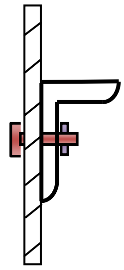
1) Bolted connection:

Bolt Value (BV):

The strength of a bolt in **shearing** and in **bearing** is computed and the **lesser** is called the **Bolt value (BV)** (i.e., Least of V_{nsb} and V_{npb})

1) Strength of one bolt in single shear

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$



2) Strength of bolt in bearing

$$V_{dpb} = \left(\frac{2.5k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

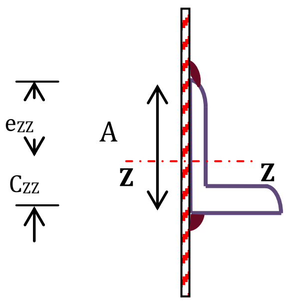
$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}}$$

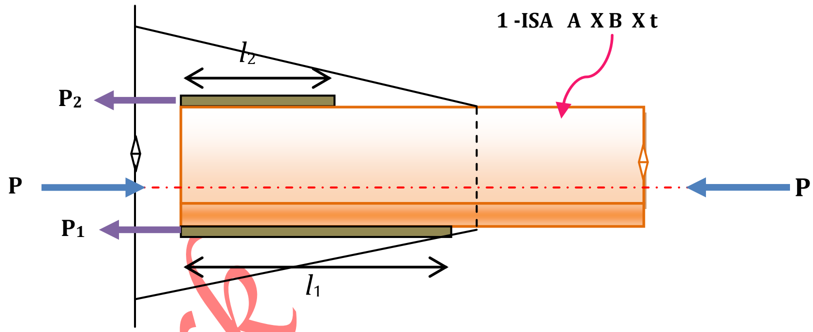
2) Welded connection:

Size of weld:

a) Min size as given table based on thickness of connecting material

b) Max size $\ngtr \frac{3}{4} \times t$





Taking $f_u = 410\ \text{N/mm}^2$.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}\ \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}\ \text{N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times A = P \times e_{xx}$$

$$l_1 = \ ? \quad \text{and} \quad l_2 = \ ?$$

Prob:®

Design a single angle strut for a roof truss carrying a compressive load of 100 KN. The length of strut between c/c intersections is 210 cm. Also design

a) Bolted End Connection, b) Welded End Connection.

Solution:

Load = 100 KN, Factored load = 1.5 x 100 = 150 KN

L= 210 cm = 2100 mm

Assuming 2 or more bolts for connections

Assuming Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ \text{N/mm}^2$

Permissible stress = 0.4 x f_y = 0.4 x 250 = 100 N/mm^2

Area of section required

$$\text{Area} = \frac{\text{Factored Load } (P_u)}{\text{Compressive stress } (f_{cd})} = \frac{150 \times 10^3}{100} = 1500 \text{ mm}^2$$

Try 1-ISA 100 x 100 x 10 mm @146.2 N/m

Properties of ISA 100 x 100 x 10 mm @ 0.272 KN / m.

$$a = 19.03 \text{cm}^2 = 1903 \text{mm}^2$$

$$r_{zz} = r_{yy} = 3.05 \text{cm} = 30.5 \text{mm}$$

$$r_{uu} = 3.85 \text{cm} = 38.5 \text{mm}$$

$$r_{vv} = 1.94 \text{cm} = 19.4 \text{mm}$$

$$r_{min} = r_{vv} = 19.4 \text{m}$$

$$\text{Effective length } (KL) = 210 \text{ cm} = 2100 \text{ mm}$$

P-48, Cl 7.5.1.2:

Effective slenderness ratio

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

k_1, k_2, k_3 = Constants depending upon the end condition as given in Table 12, P - 48.

$k_1 = 0.2$, $k_2 = 0.35$, $k_3 = 20$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{2100}{19.4}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 1.22$$

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

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(100 + 100)/2 \times 10}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 0.11$$

$$\lambda_e = \sqrt{0.2 + 0.35 \times 1.22^2 + 20 \times 0.11^2} = 0.98$$

Ref P-34, f_{cd} can be obtained

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

Where,

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

α = Imperfection factor given in Table 7 for class 'c'.

α = 0.49

$$\phi = 0.5[1 + 0.49(0.98 - 0.2) + 0.98^2] = 1.17$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda_e^2]^{0.5}} = \frac{250/1.1}{1.17 + [1.17^2 - 0.98]^{0.5}}$$

$$= 125.63 \text{N/mm}^2 \leq f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27 \text{N/mm}^2$$ Safe

Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$P_d = f_{cd} \times A$

$$P_d = \frac{125.63 \times 1903}{1000} = 239.10KN > 150KN$$

Safe

Provide 1-ISA100 x 100 x 10 mm .

Connection Details:

Assuming 20 mm bolts of grade 4.6

Dia of hole (d_0) = 20+2 = 22 mm

P-75, Cl: 10.3.3

1) For Single shear of bolts

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming Thread is interfering the shear plane

$$n_n = 1 \qquad n_s = 0\,, \qquad \gamma_{mb} = 1.25$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04\ mm^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04}{1.25 \times 1000}\right) = 45.27\ KN$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.61$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66$

P = 2.5 x 20 = 50 mm, Say 60mm

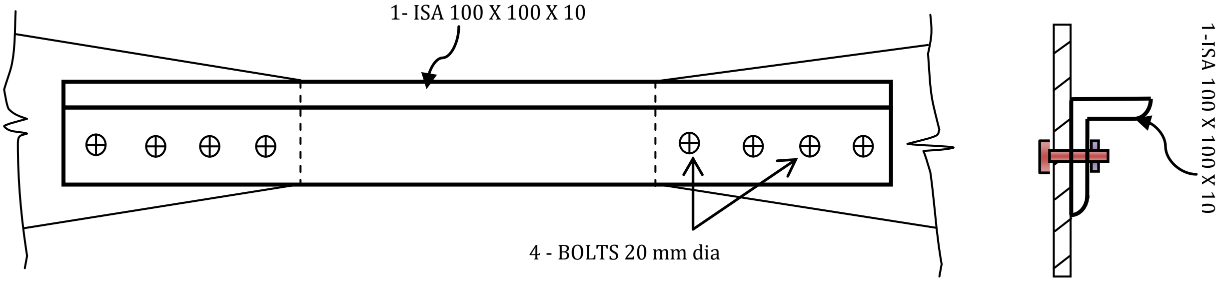
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

$$V_{dpb} = \frac{2.5 \times 0.61 \times 20 \times 10 \times 400}{1.25 \times 1000} = 97.6\ KN$$

Bolt value (BV) = 45.27 KN.

No of bolts = $\dfrac{150}{45.27} = 3.31$ Say 4 No's

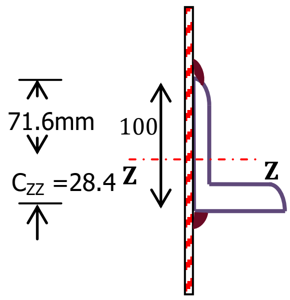


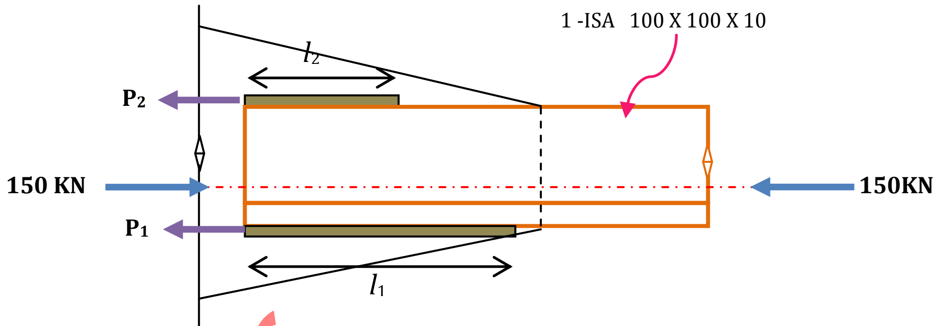
Welded connection:

C_{zz} = 2.84 cm =28.4mm, e_{zz} = 7.16 cm =71.6 mm,

Size of weld:

a) Min size = 3 mm





b) Max size $\ngtr \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5\text{mm}$ Say D = 6 mm.

Assuming field weld, $\gamma_{mw} = 1.50$

Taking $f_u = 410\ \text{N/mm}^2$.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670\, l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section. Taking moment about P_2

$$P_2 \times 100 = P \times 71.6$$

$$670 \times l_1 \times 100 = 150 \times 10^3 \times 71.6$$

$$l_1 = \frac{150 \times 10^3 \times 71.6}{670 \times 100} = 160.30 \text{ mm} \qquad \text{Say } 165 \text{ mm}$$

$$P_1 = 670 \times 165 = 110.55 \times 10^3 \text{N}$$

$$P_2 = P - P_1 = 150 \times 10^3 - 110.55 \times 10^3 = 39.45 \times 10^3 \text{N}$$

$$P_2 = 670 l_2 = 39.45 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{39.45 \times 10^3}{670} = 58.88 \text{ mm} \qquad \text{Say } 65 \text{ mm}$$

FEB 1997 –15 MARKS

3) b) Design a compression member of a roof truss to carry an axial load of 150 KN. Design the member using a single ***unequal angle*** and the corresponding connections to a gusset plate using 20mm dia bolts of 4.6 grade, ***connecting*** the ***longer legs*** to the gusset plate of 8mm thick. Take length of the member = 2.5 m

Solution:

Load = 150 KN, Factored load = 1.5 x 150 = 225 KN

L= 2.5 m = 2500 mm

Assuming Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ \text{N/mm}^2$

Permissible stress = 0.4 x f_y = 0.4 x 250 = 100 N/mm^2

Area of section required

$$\text{Area} = \frac{\text{Factored Load}\ (P_u)}{\text{Compressive stress}\ (f_{cd})} = \frac{225 \times 10^3}{100} = 2250\ \text{mm}^2$$

Try 1-ISA 150 x 75 x 12 mm

Properties of ISA 150 x 75 x 12 mm

$a = 25.62\text{cm}^2 = 2562\text{mm}^2$ $\qquad r_{uu} = 4.93\text{cm} = 49.3\text{mm}$

$r_{zz} = 4.79\text{cm} = 39.6\text{mm}$ $\qquad r_{vv} = 1.58\text{cm} = 15.8\text{mm}$

$r_{yy} = 1.97\text{cm} = 19.7\text{mm}$ $\qquad r_{min} = r_{vv} = 15.8\text{mm}$

$$\text{Effective length}(l) = 2.5\text{m} = 2500\text{mm}$$

Ref P-34, f_{cd} can be obtained

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

Effective slenderness ratio **(P-48, Cl 7.5.1.2):**

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

Assuming 2 or more bolts for connections and end is fixed

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12, P - 48.

$k_1 = 0.2,\qquad k_2 = 0.35,\qquad k_3 = 20$

$$\lambda_{vv} = \frac{\left(\dfrac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} = \frac{\dfrac{2500}{15.8}}{1 \times \sqrt{\dfrac{\pi^2 \times 2 \times 10^5}{250}}} = 1.78$$

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

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} = \frac{(150 + 75)/2 \times 10}{1 \times \sqrt{\dfrac{\pi^2 \times 2 \times 10^5}{250}}} = 0.105$$

$$\lambda_e = \sqrt{0.2 + 0.35 \times 1.78^2 + 20 \times 0.105^2} = 1.236$$

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

α = Imperfection factor given in Table 7 for class 'c'.

α = 0.49

$$= 0.5\left[1 + 0.49(1.78 - 0.2) + 1.78^2\right] = 1.517$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.76 + \left[1.76^2 - 1.39^2\right]^{0.5}}$$

$$f_{cd} = 94.83\,N/mm^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\,N/mm^2 \qquad \text{Safe}$$

Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$P_d = f_{cd}$ x A

$$P_d = \frac{94.83 \times 2562}{1000} = 242.95\,KN > 225\,KN \qquad \text{Safe}$$

Provide 1- ISA 150 x 75 x 12 mm.

Connection Details:

Taking 20 mm bolts of grade 4.6

Dia of hole (d_0) = 20 +2 =22 mm

P-75, Cl: 10.3.3

1) Strength of one bolt in Single shear:

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming thread is interfering the shear plane

$n_n = 1 \qquad n_s = 0 \quad , \gamma_{mb} = 1.25$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04\,mm^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04}{1.25 \times 1000}\right) = 45.27\,KN$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.61$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$ P = 2.5 x 20 = 50 mm

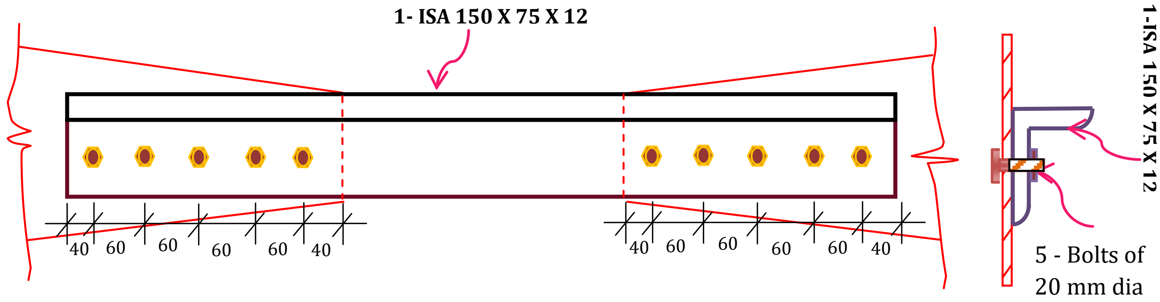
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

$$V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 8 \times 400}{1.25 \times 1000} = 65.28\,KN$$

Bolt value (BV) = 45.27 KN.

No of bolts = $\dfrac{225}{45.27} = 4.97$ $\boxed{\text{Say 5 No's}}$



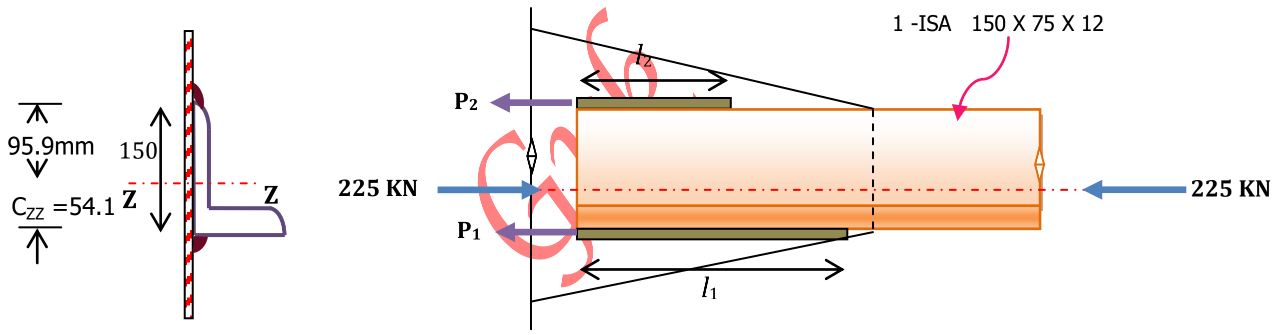
Welded connection:

C_{zz} = 5.41 cm =54.1 mm, e_{zz} = 9.59 cm =95.9 mm,

Size of weld:

a) Min size = 3 mm

b) Max size $\not> \dfrac{3}{4} \times t = \dfrac{3}{4} \times 12 = 9\,\text{mm}$ Say D = 6 mm.



Assuming field weld, $\gamma_{mw} = 1.50$

Taking $f_u = 410\ \text{N/mm}^2$.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670\, l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section. Taking moment about P_2

$$P_1 \times 150 = P \times 95.9$$

$$670 \times l_1 \times 150 = 225 \times 10^3 \times 95.9$$

$$l_1 = \frac{225 \times 10^3 \times 95.9}{670 \times 150} = 214.7 \text{ mm} \qquad \text{Say } 220 \text{ mm}$$

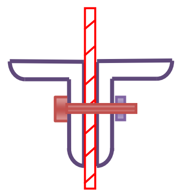
$$P_1 = 670 \times 220 = 147.4 \times 10^3 N$$

$$P_2 = P - P_1 = 225 \times 10^3 - 147.4 \times 10^3 = 77.6 \times 10^3 N$$

$$P_2 = 670 l_2 = 77.6 \times 10^3 N$$

$$\therefore l_2 = \frac{77.6 \times 10^3}{670} = 115.82 \text{ mm} \qquad \text{Say } 120 \text{ mm}$$

b) Design Procedure for double angle Struts:



1. Assume Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250 \text{ N/mm}^2$
2. Calculate Area of section required
   $$\text{Area} = \frac{\text{Factored Load}(P_u)}{\text{Compressive stress}(f_{cd})}$$
3. Choose a suitable section from the steel table by assuming 15 % to 25% more than Area required.

P-48, Cl 7.5.2.1, Effective length:

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

Effective slenderness ratio

$$\lambda_e = \frac{KL}{r_{min}} \not> 180$$

Ref P-42, Table 9(c) and find f_{cd}

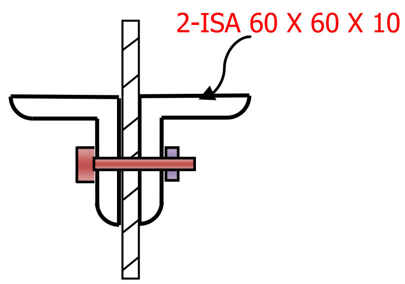
Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

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

1995 Aug - 06 marks

prob:

4(b) Design a compression member using double angles to carry 200 KN load. The length of the member between intersection is 1.5 m. The thickness of gusset plate is 10mm.



Solution:

Load = 200KN, Factored load = 1.5 x 200 = 300 KN

Assuming, stress $f_{ad} = 0.7f_y = 0.7 \times 250 = 175 \text{ N/mm}^2$

$$\text{Area required} = \frac{300 \times 10^3}{175} = 1714.30 \text{ mm}^2 = 17.14 \text{ cm}^2$$

Case 1: Equal angles on either side of gusset plate

Try 2-ISA 60 x 60 x 10

$a = 22\text{cm}^2 = 2200\text{mm}^2 \quad r_{yy} = 2.95\text{cm} = 29.5\text{mm}$(10mm th. gusset plate)

$r_{ZZ} = 1.78\text{cm} = 17.8\text{mm} \quad r_{min} = r_{ZZ} = 17.8\text{mm}$

$KL = 0.7 \times L \text{ to } 0.85 \times L$

Assuming $KL = 0.8 \times L = 0.8 \times 1500\text{mm} = 1200\text{mm}$

$$\text{Effective slenderness ratio} = \frac{KL}{r_{min}} = \frac{1200}{17.8} = 67.42 < 180 \qquad \text{Safe}$$

Ref Table 9(c) , P – 42 for $f_y = 250\ \text{N/mm}^2$

$$f_{cd}\ \text{for}\ 67.42 = 168 - \frac{7.42 \times 16}{10} = 156.13\ \text{N/mm}^2$$

λ	f_{cd}
60	168
67.42	?
70	152
10	16
7.42	?(x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{156.13 \times 2200}{1000} = 343.5\ \text{KN} > 300\ \text{KN.}$$

Safe

Provide 2-ISA 60 x 60 x 10

Connection Details:

1) Bolted Connection:

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Sum of the thickness of angles =10+10 =20mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t^*} = 6.04\sqrt{10} = 19.10mm$$

say 18mm

Dia of hole (d_0) = 18 + 2 = 20 mm

1) Strength of bolts in double shear :

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming shank and thread both interfere the shear plane

$n_n = 1$ $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16\ \text{mm}^2$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04\ \text{mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04 + 1 \times 314.16}{1.25 \times 1000}\right) = 103.31\ \text{KN}$$

2) **Strength of bolt in Bearing** $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$ Edge distance e = 1.5 x 20 =30 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 18 = 45 mm, Say 50mm

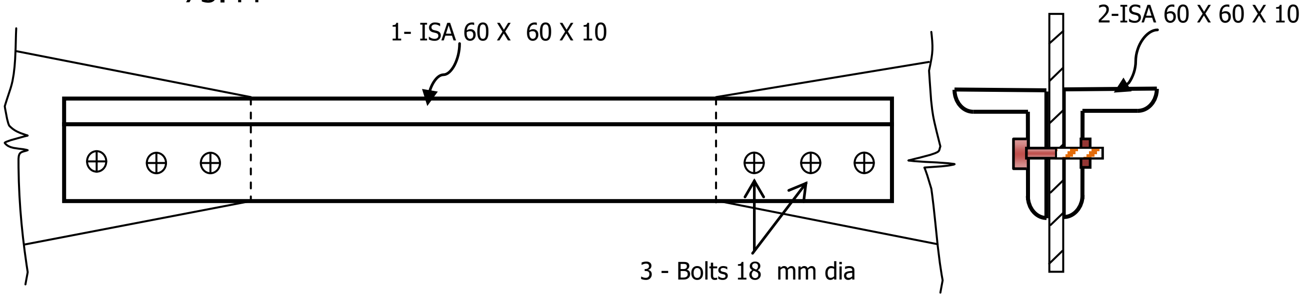
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

$k_b = 0.51$

$$V_{dpb} = \frac{2.5 \times 0.51 \times 18 \times 10 \times 400}{1.25 \times 1000} = 73.44\ \text{KN}$$

Bolt value (BV) = 73.44 KN.

No of bolts = $\frac{300}{73.44} = 4.08$ $\boxed{\text{Say 5 No's}}$



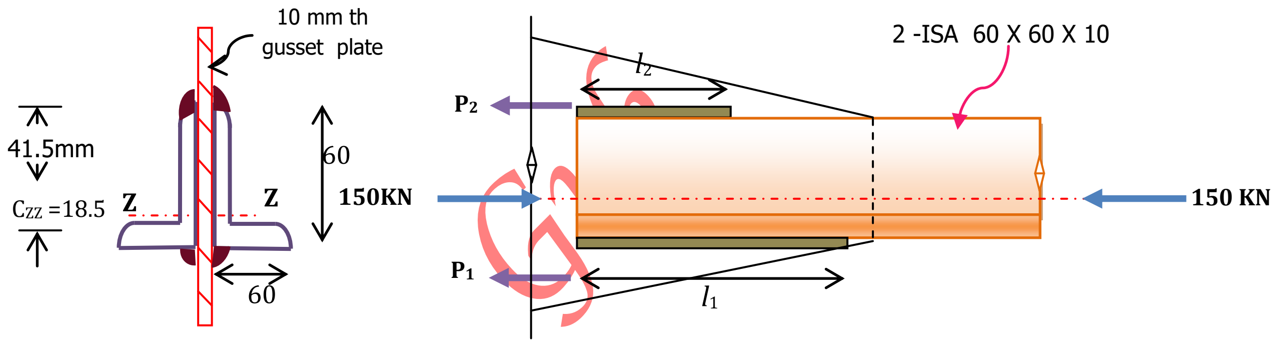
Welded connection:

C_{zz} = 1.85 cm =18.5 mm, e_{zz} = 4.15 cm = 41.5 mm,

Size of weld:

a) Min size = 3 mm

b) Max size $\not> \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5\,\text{mm}$ Say D = 6 mm.



Assuming field weld, $\gamma_{mw} = 1.50$

Taking $f_u = 410\,\text{N/mm}^2$.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670\, l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

Welding is done to both the angles on either side of the gusset plate, let us design one side of the gusset plate for a load of P' = P/2 = 300/2 =150 KN

$$P_1 + P_2 = P'$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times 60 = P' \times 71.6$$

$$670 \times l_1 \times 60 = 150 \times 10^3 \times 41.5$$

$$l_1 = \frac{150 \times 10^3 \times 41.5}{670 \times 60} = 154.85 \text{ mm} \qquad \text{Say } 160 \text{ mm}$$

$$P_1 = 670 \times 160 = 107.20 \times 10^3 \text{N}$$

$$P_2 = P - P_1 = 150 \times 10^3 - 107.20 \times 10^3 = 42.80 \times 10^3 \text{N}$$

$$P_2 = 670 l_2 = 42.80 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{42.80 \times 10^3}{670} = 63.88 \text{mm} \qquad \text{Say } 70 \text{ mm}$$

Case 2: Equal angles on same side of gusset plate

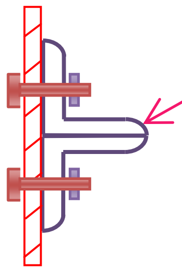
Try 2-ISA 60 x 60 x 10

$a = 22\text{cm}^2 = 2200 \text{ mm}^2$ $r_{yy} = 2.95\text{cm} = 29.5\text{mm}$ (10mm th. gusset plate)

$r_{zz} = 1.78\text{cm} = 17.8 \text{ mm}$ $r_{min} = r_{zz} = 17.8 \text{ mm}$

$KL = 0.7 \times L$ to $0.85 \times L$

Assuming $KL = 0.8 \times L = 0.8 \times 1500\text{mm} = 1200 \text{ mm}$



$$\text{Effective slenderness ratio} = \frac{KL}{r_{min}} = \frac{1200}{17.8} = 67.42 < 180 \qquad \text{Safe}$$

Ref Table 9(c) , P – 42 for $f_y = 250\text{N/mm}^2$

$$f_{cd} \text{ for } 67.42 = 168 - \frac{7.42 \times 16}{10} = 156.13\text{N/mm}^2$$

λ	f_{cd}
60	168
67.42	?
70	152
10	16
7.42	?(x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{156.13 \times 2200}{1000} = 343.5 \text{ KN} > 300 \text{ KN.}$$

Safe

Provide 2-ISA 60 x 60 x 10

<u>Connection Details:</u>

A) Bolted Connection:

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Thickness of angle =10 mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t^*} = 6.04\sqrt{10} = 19.10mm$$

say 18mm

Dia of hole (d_0) = 18 + 2 = 20 mm

1) Strength of bolts in single shear :

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming shank is interfering the shear plane

$$n_s = 1, \qquad \gamma_{mb} = 1.25 \qquad A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 314.16}{1.25 \times 1000}\right) = 58.04\text{KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$ Edge distance e = 1.5 x 20 =30 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 18 = 45 mm, Say 50mm

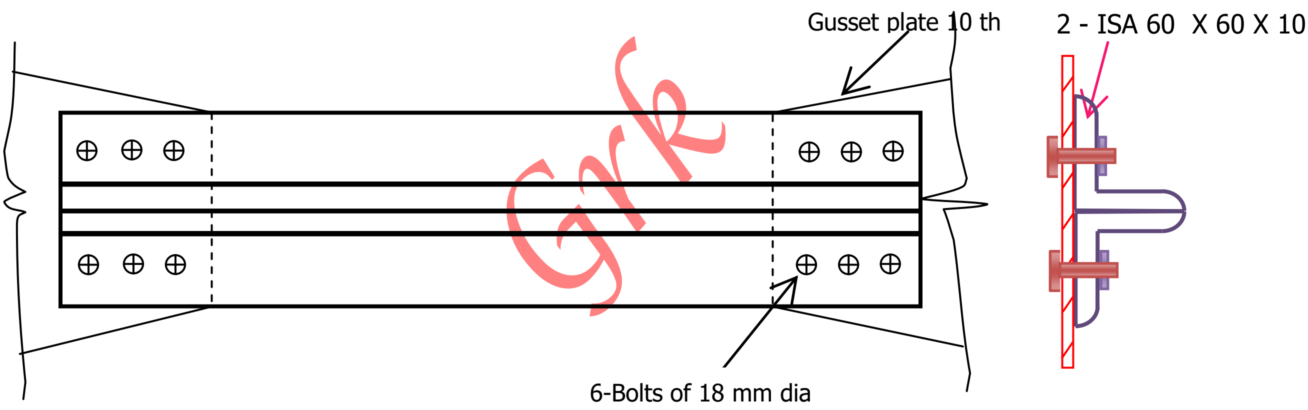
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

$k_b = 0.51$

$$V_{dpb} = \frac{2.5 \times 0.51 \times 18 \times 10 \times 400}{1.25 \times 1000} = 73.44 \text{ KN}$$

Bolt value (BV) = 58.04 KN.

No of bolts = $\dfrac{300}{58.04} = 5.17$ $\boxed{\text{Say 6 No's}}$

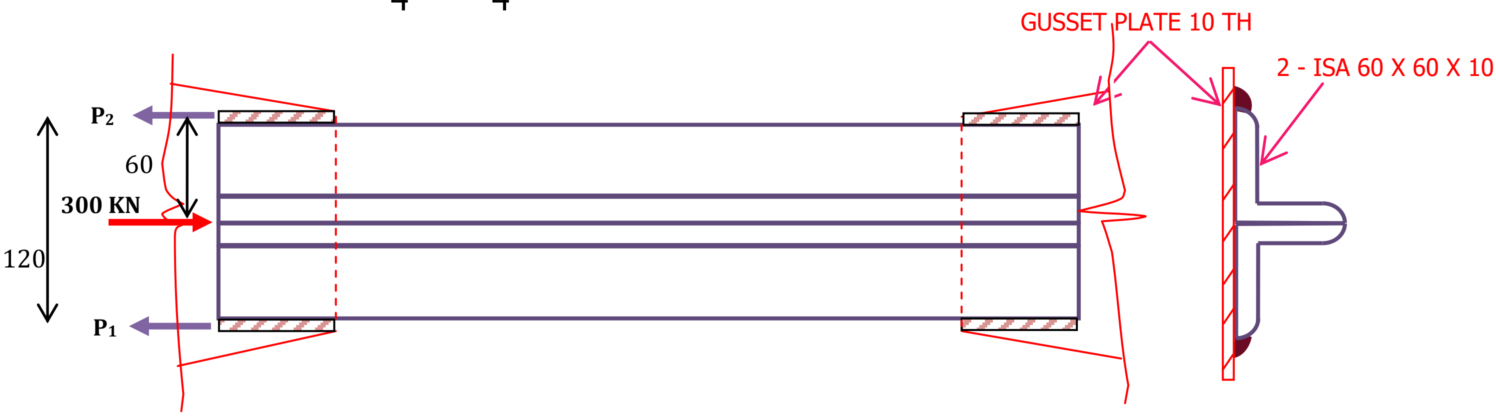


Welded connection:

Size of weld:

a) Min size = 3 mm

b) Max size $\not> \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5\text{mm}$ Say D = 6 mm.



Assuming field weld, $\gamma_{mw} = 1.50$

Taking $f_u = 410\ \text{N/mm}^2$.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670\, l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times 120 = P \times 60$$

$$670 \times l_1 \times 120 = 300 \times 10^3 \times 60 \qquad l_1 = \frac{300 \times 10^3 \times 60}{670 \times 120} = 223.88 \text{ mm Say } 230 \text{ mm}$$

Since the load is acting exactly at the centre of its connection, therefore $l_1 = l_2 = 230$ mm.

Case 3: Unequal angles on either side of gusset plate (Short legs back to back)

Case 4: Unequal angles on either side of gusset plate (Long legs back to back)

Case 5: Unequal angles on same side of gusset plate (Short legs back to back)

Case 6: Unequal angles on same side of gusset plate (Long legs back to back)

Feb-1996 –10 marks

Prob:

4(b) A strut in a roof truss carries an axial load of 200 KN. Design a suitable double angle section for the strut. The effective length of the strut is 2 m and yield stress for the steel is 260 MPa.The thickness of the gusset plate is 20mm.

Prob: Negi

A strut in a roof truss carries an axial compressive load of 180 KN. Design a suitable double angle section for the compression member. The length of strut between center to center of intersection is 2.3 m and yield stress of steel is 250 Mpa.

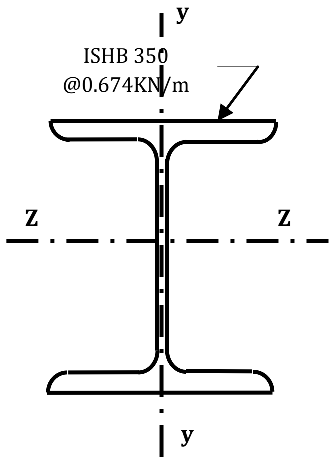
COLUMNS (STANCHION):

ANALYSIS PROBLEMS:

1) Depending on the boundary condition
   L_{eff} is calculated using table 11, P – 45
2) Determine buckling class of cross section from Table 10, Page 44
   $\frac{h}{b_f}$ & t_f values
3) Effective Slenderness ratio $\lambda_{ZZ} = \frac{KL}{r_{ZZ}}$ & $\lambda_{YY} = \frac{KL}{r_{YY}}$
4) Based on Slenderness ratio obtain the f_{cd} value from corresponding table from page No's 40 to 44.
5) Design stress f_{cd} = Min of the $f_{cd)ZZ}$ & $f_{cd)YY}$
6) Safe load = Design stress (f_{ad}) x Area provided

Problem:

A rolled steel beam section ISHB 350 @ 0.674 KN/m is used as stanchion. If the unsupported length of stanchion is 4 m, determine the safe load carrying capacity of stanchion.



Solution:

Properties of ISHB 350 @ 0.674 KN/m

$a = 85.91\text{cm}^2 = 85.91 \times 100\text{mm}^2$

$h = 350\text{mm}, \quad b_f = 250 \text{ mm}, \; t_f = 11.6 \text{ mm}, \; t_w = 8.3 \text{ mm}.$

$r_{ZZ} = 14.93\text{cm} = 149.3\text{mm}, \quad r_{yy} = 5.34\text{cm} = 53.4\text{mm}$

$l_{eff} = 4\text{m} = 4000\text{mm}$

Determination of buckling class of cross section

Since

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

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

1) About Z-Z axis :

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

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

λ = non dimensional effective slenderness ratio

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

Where,

KL/r = Effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration 'r'.

α = Imperfection factor given in Table 7, P -35

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

Effective length based on end condition

L= 4000 mm

$$\lambda_{ZZ} = \sqrt{\frac{f_y\left(\frac{KL}{r_{zz}}\right)^2}{\pi^2 E}} = \sqrt{\frac{250\left(\frac{4000}{149.3}\right)^2}{\pi^2 \times 2 \times 10^5}} = 0.30$$

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

$$\phi = 0.5\left[1 + 0.21 \times (0.3 - 0.2) + 0.3^2\right] = 0.56$$

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

$$\chi = \frac{1}{0.56 + \left[0.56^2 - 0.3^2\right]^{0.5}} = 0.97$$

$$f_{cd} = \frac{0.97 \times 250}{1.1} = 220.04 \text{ N/mm}^2$$

2) About Y-Y axis :

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

Table 7, P -35

α = Imperfection factor =0.34

Effective length based on end condition

L= 4000 mm

$$\lambda_{YY} = \sqrt{\frac{f_y\left(\frac{KL}{r_{YY}}\right)^2}{\pi^2 E}} = \sqrt{\frac{250\left(\frac{4000}{53.4}\right)^2}{\pi^2 \times 2 \times 10^5}} = 0.84$$

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

$$\phi = 0.5\left[1 + 0.34 \times (0.84 - 0.2) + 0.84^2\right] = 0.96$$

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

$$\chi = \frac{1}{0.96 + \left[0.96^2 - 0.84^2\right]^{0.5}} = 0.70$$

$$f_{cd} = \frac{0.70 \times 250}{1.1} = 159.52 \text{N/mm}^2$$

Compressive stress min of the above two values

$$f_{cd} = 75 \text{ N/mm}^2$$

Load carrying capacity = Safe stress x area provided

$$= \frac{159.52 \times 85.91 \times 10^2}{1000} = 1370.50 \text{ KN.}$$

OR

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

Compressive Stress (f_{cd}): P- 34, Cl 7.1.2.1

About Z-Z axis :

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{4000}{149.3} = 26.80$$

REF TO TABLE 9(a) P-40

λ_{zz}	f_{cd}
20	226
26.80	?
30	220
10	06
6.80	?

$$f_{cd} \text{ for } 26.80 = 226 - \frac{6.8 \times 06}{10} = 221.92 \text{ N/mm}^2$$

Compressive Stress (f_{cd}):

About Y-Y axis :

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{4000}{53.4} = 74.91$$

REF TO TABLE 9(b) P-41

λ_{zz}	f_{cd}
70	166
74.91	?
80	150
10	16
4.91	?

$$f_{cd} \text{ for } 74.91 = 166 - \frac{4.91 \times 16}{10} = 158.14 \text{N/mm}^2$$

f_{cd} is the min of 164.95 N/mm^2, and 158.14 N/mm^2.

i.e., f_{cd} = 158.14N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{158.14 \times 8591}{1000} = 1358.60 \text{ KN.}$$

Problem:

Determine the design strength of the rolled steel beam section ISHB 300 @ 0.588 kN/m to be used as stanchion. Effective length of stanchion is 3 m.

Solution:

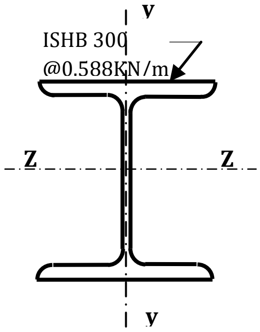
Properties of ISHB 300 @ 0.588 KN/m

$a = 74.85\text{cm}^2 = 7485\text{mm}^2$

$h = 300\text{mm}, \quad b_f = 250 \text{ mm}, \; t_f = 10.6 \text{ mm.}$

$r_{ZZ} = 12.95\text{cm} = 129.5\text{mm}, \quad r_{yy} = 5.41\text{cm} = 54.1\text{mm}$

$KL = 3000\text{mm}$



Determination of buckling curve classification

Since

$$\frac{h}{b_f} = \frac{300}{250} = 1.2 \quad \text{and} \quad t_f = 10.6 \le 100 \text{ mm}$$

We should use buckling class 'b' about Z-Z axis and 'c' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis : use buckling class 'b'

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{3000}{129.5} = 23.17$$

REF TO TABLE 9(b) P-41

λ_{zz}	f_{cd}
20	225
23.17	?
30	216
10	09
3.17	?

$$f_{cd} \text{ for } 23.17 = 225 - \frac{3.17 \times 09}{10} = 222.15 \text{ N/mm}^2$$

Compressive Stress (f_{cd}):

About Y-Y axis : use buckling class 'c'

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{3000}{54.1} = 55.45$$

REF TO TABLE 9(c) P-42

λ_{zz}	f_{cd}
50	183
55.45	?
60	168
10	15
5.45	?

$$f_{cd} \text{ for } 55.45 = 183 - \frac{5.45 \times 15}{10} = 174.83 \text{N/mm}^2$$

f_{cd} is the min of 222.15 N/mm^2, and 174.83 N/mm^2.

i.e., f_{cd} = 174.83 N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{174.83 \times 7485}{1000} = 1308.60 \text{ KN.}$$

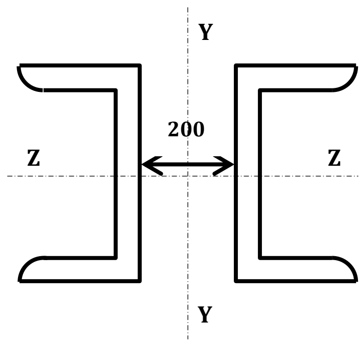
Problem: 1996-Feb (B.U) 20 marks

A steel stanchion is formed of two channels of ISMC 350 placed back to back with a clear spacing of 200 mm. If the effective length of channel is 6m, find safe axial load that the column can carry.

Calculate the extra load the column can carry if 2 plates of 400mm x 10 mm are welded to the channel flanges one on each side.

Solution:

Case-I **Properties of ISMC 350**



$$a = 107.32\text{cm}^2 = 107.32 \times 100\text{mm}^2$$

$$r_{ZZ} = 13.66\text{cm} = 1366\text{mm}, \qquad r_{yy} = 12.76\text{cm} = 1276\text{mm}$$

$$r_{min} = r_{yy} = 1276\text{mm}$$

$$l_{eff} = KL = 6\text{m} = 6000\text{mm}$$

Effective slenderness ratio

$$\lambda = \frac{KL}{r_{min}} = \frac{6000}{1276} = 47.02$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42 Table 9 (c) for $f_y = 250\text{N/mm}^2$

λ	σ_{ac}
40	198
47.02	?
50	183
10	15
7.02	?

$$f_{cd} \text{ for } 47.02 = 198 - \frac{7.02 \times 15}{10} = 187.47\text{N/mm}^2$$

flexural buckling strength = Safe stress x Area provided

$$= \frac{187.47 \times 107.08 \times 10^2}{1000} = 2007.50 \text{ KN.}$$

$$\text{Safe load} = \frac{2007.50 \times 10^3}{1.5} = 1338.33 \text{ KN.}$$

Case-II **Properties of 2- ISMC 350**

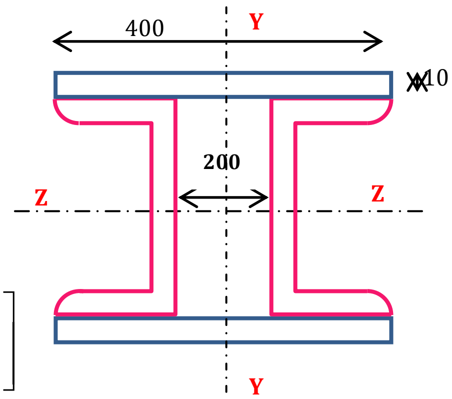
$$a = 107.32\text{cm}^2 = 107.32 \times 100\text{mm}^2$$

$$I_{zz} = 20016\text{cm}^4 = 20016 \times 10^4\text{mm}^4$$

$$I_{yy} = 17469.4\text{cm}^4 = 17469.4 \times 10^4\text{mm}^4$$

$$l_{eff} = KL = L = 6\text{m} = 6000\text{mm}$$

$$\text{Area}(A) = 10732 + 2 \times 400 \times 10 = 18732\text{mm}^2$$



I_{ZZ} of the built up section

$$I_{zz} = 20016 \times 10^4 + 2\left[\frac{400 \times 10^3}{12} + 400 \times 10\left(\frac{350}{2} + \frac{10}{2}\right)^2\right]$$

$$= 459.43 \times 10^6\text{mm}^4$$

I_{yy} of the built up section

$$I_{yy} = 17469.4 \times 10^4 + 2\left[\frac{10 \times 400^3}{12}\right]$$

$$= 281.36 \times 10^6\text{mm}^4$$

$$\therefore I_{min} = I_{yy} = 281.36 \times 10^6\text{mm}^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{281.36 \times 10^6}{18732}} = 122.55\text{mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{6000}{122.55} = 48.95$$

λ	σ_{ac}
40	198
48.95	?
50	183
10	15
8.95	?

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250\text{N/mm}^2$

$$f_{cd} \text{ for } 48.95 = 198 - \frac{8.95 \times 15}{10} = 184.58\text{N/mm}^2$$

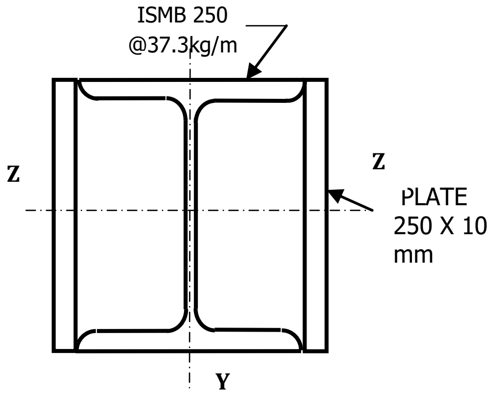
flexural buckling strength = = Safe stress x area provided

$$= \frac{184.58 \times 18732}{1000} = 3457.55\text{KN.}$$

Safe load= $\frac{3457.55\times10^3}{1.5}=2305$ KN.

P-187 , DSS BY B.C. PUNMIA:

An I- joist ISMB 250 @ 37.3 kg/m has an effective length of 5 m. It is used as a stanchion with two plates 250 x 10 mm welded to its sides, as shown in fig. compute the load carrying capacity. What will be its load carrying capacity if one plate is attached to each flange.



Solution: Properties of ISMB 250 37.3 kg/m.

a = 47.55 cm^2; I_{ZZ} = 5131.6 x 10^4 mm^4;

I_{yy} = 334.5 x 10^4 mm^4.

A = 4755 + 2 x (250 x 10) = 9755 mm^2.

a) Plates attached to sides:

I_{ZZ} of the built up section

$$I_{ZZ}=5131.6\times10^4+2\left[\frac{10\times250^3}{12}\right]=77.35\times10^6\,mm^4$$

I_{yy} of the built up section

$$I_{yy}=334.5\times10^4+2\left[\frac{250\times10^3}{12}+250\times10\left(\frac{125}{2}+\frac{10}{2}\right)^2\right]=26.17\times10^6\,mm^4$$

$$\therefore I_{min}=I_{yy}=26.17\times10^6\ mm^4$$

$$r_{min}=\sqrt{\frac{I_{min}}{A}}=\sqrt{\frac{26.17\times10^6}{9755}}=51.8\ mm$$

λ	σ_{ac}
90	121
96.53	?
100	107
10	14
6.53	?

Effective slenderness ratio

$$S.R(\lambda)=\frac{KL}{r_{min}}=\frac{5000}{51.8}=96.53$$

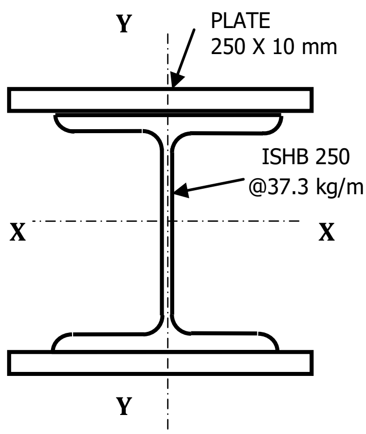
Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y=250N/mm^2$

$$f_{cd}\ \text{for}\ 96.53=121-\frac{6.53\times14}{10}=111.86N/mm^2$$

Load carrying capacity = Safe stress x area provided

$$=\frac{111.86\times9755}{1000}=1091.20\ KN.$$



b) Plates attached to the flange:

I_{ZZ} of the built up section

$$I_{ZZ}=5131.6\times10^4+2\left[\frac{250\times10^3}{12}+250\times10\left(\frac{250}{2}+\frac{10}{2}\right)^2\right]$$

$$=135.86\times10^6\,mm^4$$

I_{yy} of the built up section

$$I_{yy} = 334.5 \times 10^4 + 2\left[\frac{10 \times 250^3}{12}\right]$$

$$= 29.387 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{yy} = 29.387 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{29.387 \times 10^6}{9755}} = 54.89\ mm$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{5000}{54.89} = 91.1$$

λ	σ_{ac}
90	121
91.1	?
100	107
10	14
1.1	?

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 N/mm^2$

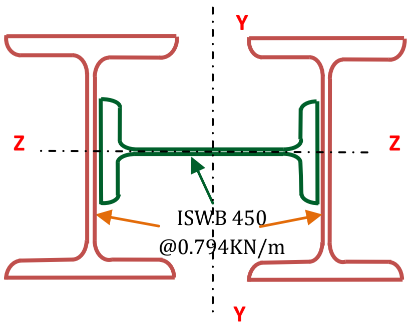
$$f_{cd} \text{ for } 96.53 = 121 - \frac{1.1 \times 14}{10} = 119.46 N/mm^2$$

Load carrying capacity = Safe stress x area provided

$$= \frac{119.46 \times 9755}{1000} = 1155.33\ KN.$$

PROBLEM:

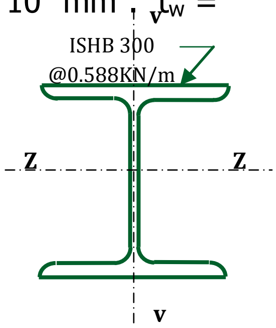
A built – up column consists of three rolled steel beam sections WB 450 @ 0.794 KN/m, connected effectively to act as one column as shown in fig. determine the safe load carrying capacity of built – up section. Unsupported length of column is 4.25m.



Solution:

Properties of 1- ISWB 450 0.794KN/m.

a =101.15 cm^2; I_{ZZ} =35057.6 x 10^4 mm^4; I_{yy} = 1706.7 x 10^4 mm^4, t_w = 9.2 mm.



Total area of built up section

A = 10115 + 2 x (10115) = 30345mm^2.

I_{ZZ} of the built up section:

$$I_{ZZ} = 2 \times 35057.6 \times 10^4 + 1706.7 \times 10^4 = 718.22 \times 10^6 mm^4$$

I_{yy} of the built up section:

$$I_{yy} = 35057.6 \times 10^4 + 2 \times \left[1706.7 \times 10^4 + 10115\left(\frac{450}{2} + \frac{9.2}{2}\right)^2\right] = 1451.15 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{ZZ} = 718.22 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{718.22 \times 10^6}{30345}} = 153.84\ mm$$

Effective slenderness ratio

$$S.R(\lambda)=\frac{4250}{153.84}=27.62$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250\ N/mm^2$

$$f_{cd}\ \text{for}\ 27.62 = 224-\frac{7.62\times 13}{10}=214.09\ N/mm^2$$

λ	f_{cd}
20	224
27.62	?
30	211
10	13
7.62	?

Load carrying capacity = Safe stress x Area provided

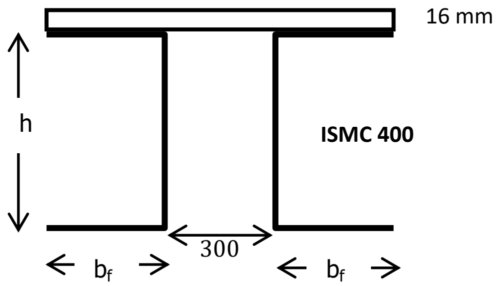
$$=\frac{214.09\times 30345}{1000}=6496.56\ kN.$$

$$\text{Safe load}=\frac{6496.56}{1.5}=4331\ kN.$$

Jan / Feb 2006 – 10 marks

Determine the allowable load which the member shown in fig can support, if the member is of 5.5 m effective length. Assume f_y = 250 N/mm^2.

Solution:



Properties of 1- ISMC 400

a =62.93 cm^2; b_f = 100mm, h = 400mm,

I_{ZZ} =15082.8 x 10^4 mm^4; I_{yy} = 504.8 x 10^4 mm^4.

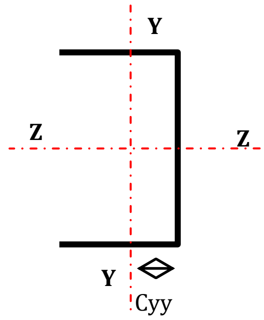
t_w = 9.2 mm. c_{yy} = 2.42 cm = 24.2 mm.

Width of plate at top = b_f + gap + b_f

= 100 + 300 + 100 = 500 mm

A = 2 x 6293 + 500 x 16 = 20586 mm^2.

Centroidal axis distance from Bottom AA reference axis $\bar{Y}$



$$\bar{Y}=\frac{a_1y_1+a_2y_2+a_3y_3}{a_1+a_2+a_3}=$$

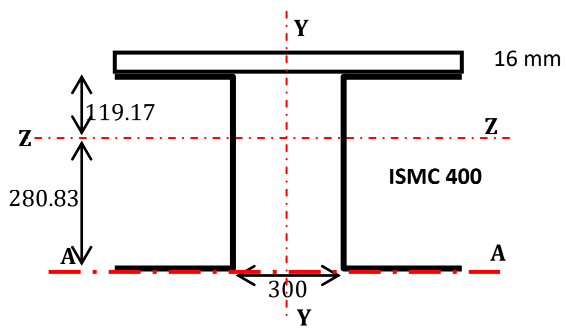
$$\frac{\left[2\times 6293\times\left\{\frac{400}{2}\right\}\right]+\left[500\times 10\times\left(400+\frac{16}{2}\right)\right]}{[2\times 6293]+[500\times 16]}=280.83\ mm$$

I_{ZZ} of the built up section:

$$I_{ZZ}=2\times\left[15082.8\times 10^4+6293\times\left(280.83-\frac{400}{2}\right)\right]+\left[\frac{500\times 16^3}{12}+500\times 16\times\left(119.17+\frac{16}{2}\right)^2\right]$$

$$I_{ZZ}=512.40\times 10^6 mm^4$$

I_{yy} of the built up section:



$$I_{yy}=2\times\left[504.8\times 10^4+6293\left(\frac{300}{2}+24.2\right)^2\right]$$

$$+\frac{16\times 500^3}{12}=558.69\times 10^6 mm^4$$

$$\therefore I_{min} = I_{ZZ} = 512.4 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{512.4 \times 10^6}{20586}} = 157.76 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r_{min}} = \frac{5500}{157.76} = 34.86$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 N/mm^2$

λ	f_{cd}
30	211
34.86	?
40	198
10	13
4.86	?

$$f_{cd} \text{ for } 34.86 = 211 - \frac{4.86 \times 13}{10} = 204.68 \text{ N/mm}^2$$

Flexural Strength of member = Safe stress x Area provided

$$= \frac{204.68 \times 20586}{1000} = 4213.58 \text{ kN.}$$

$$\text{Allowable load} = \frac{4213.58}{1.5} = 2809.06 \text{ kN.}$$

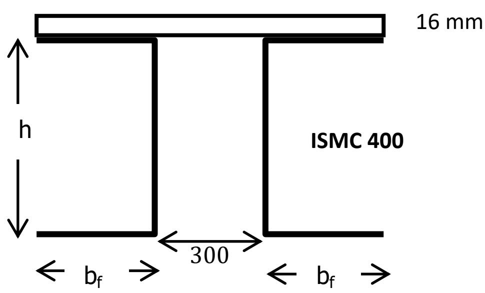
OR

Solution:

Properties of 2- ISMC 400

a = 125.86 cm^2= 12586 mm^2;

I_{ZZ} =30165.6 x 10^4 mm^4; I_{yy} = 39202.6x 10^4 mm^4.



Breadth and depth of single channel section

b_f = 100mm, h = 400mm,

Width of plate at top = b_f + gap + b_f

100 + 300 + 100 = 500 mm

A = 12586 + 500 x 16 = 20586 mm^2.

Centroidal axis distance from Bottom AA reference axis $\bar{Y}$

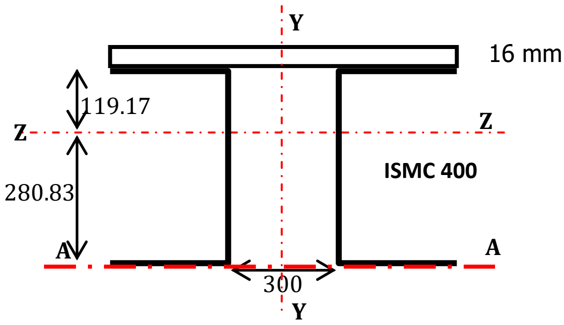
$$\bar{Y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2} = \frac{\left[12586 \times \left\{\frac{400}{2}\right\}\right] + \left[500 \times 10 \times \left(400 + \frac{16}{2}\right)\right]}{[12586] + [500 \times 16]} = 280.83 \text{ mm}$$

<u>I_{ZZ} of the built up section:</u>

$$I_{ZZ} = \left[30165.6 \text{ x } 10^4 \times 10^4 + 12586 \times \left(280.83 - \frac{400}{2}\right)^2\right] +$$

$$\left[\frac{500 \times 16^3}{12} + 500 \times 16 \times \left(119.17 + \frac{16}{2}\right)^2\right] = 512.40 \times 10^6 \text{ mm}^4$$

<u>I_{yy} of the built up section:</u>



$$I_{yy} = \left[39202.6 \times 10^4\right] + \frac{16 \times 500^3}{12} = 558.69 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{ZZ} = 512.4 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{512.4 \times 10^6}{20586}} = 157.76 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r_{min}} = \frac{5500}{157.76} = 34.86$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 \text{ N/mm}^2$

λ	f_{cd}
30	211
34.86	?
40	198
10	13
4.86	?

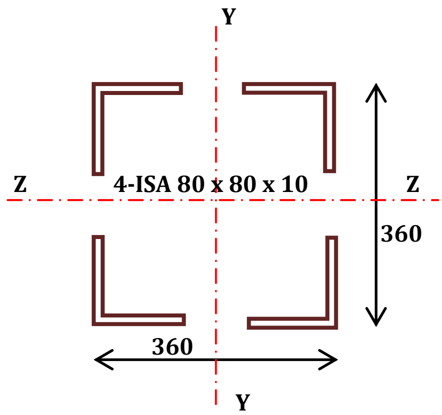
$$f_{cd} \text{ for } 34.86 = 211 - \frac{4.86 \times 13}{10} = 204.68 \text{ N/mm}^2$$

Flexural Strength of member = Safe stress x Area provided

$$= \frac{204.68 \times 20586}{1000} = 4213.58 \text{ kN.}$$

$$\text{Allowable load} = \frac{4213.58}{1.5} = 2809.06 \text{ kN.}$$

Problem:



A column height 5m is hinged at the ends. It is square in cross section (plan) of side 360 mm and consists of 4 angles of ISA 80 x 80 x 10 mm at each corner suitably laced. Find the minimum load on the column.

Solution:

Properties of 1- ISA 80 x 80 x 10 mm

$a = 15.05 cm^2 = 1505 mm^2, C_{ZZ} = 2.34 cm = 23.4 mm,$

$I_{ZZ} = 87.7 cm^4 = 87.7 \times 10^4 \text{ mm}^4, L = 5m$

End condition: Both ends hinged

$l_{eff} = KL = L = 5m = 5000 \text{ mm}$

I_{ZZ}= I_{YY} of the built up section:

$$I_{ZZ} = I_{YY} = 4\left[87.7 \times 10^4 + 1505 \times \left(\frac{360}{2} - 23.4\right)^2\right] = 151.14 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{151.14 \times 10^6}{4 \times 1505}} = 158.45 mm$$

Effective slenderness ratio

$$\lambda = \frac{KL}{r_{min}} = \frac{5000}{158.45} = 31.56$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42 Table 9 (c) for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 31.56 = 211 - \frac{1.56 \times 13}{10} = 208.97 \text{ N/mm}^2$$

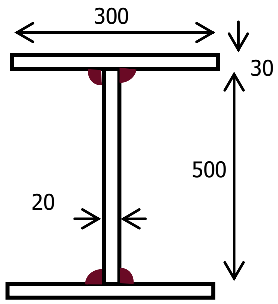
λ	σ_{ac}
30	211
31.56	?
40	198
10	13
1.56	?

Load carrying capacity = Safe stress x Area provided

$$= \frac{208.97 \times 4 \times 1505}{1000} = 1258 \text{ kN.}$$

Prob: P-334, Ex : 7.7, LSD of Steel Structure . S.K. Duggal

For a column section built up of shape as shown in fig, determine the axial load capacity of compression for the data indicated against the fig. f_y = 250 MPa, L = 6 m, t_w = 20 mm, t_f = 30 mm. End condition:- Both ends restrained in direction & position ,



Solution:

A = 2 x 300 x 30 + 500 x 20 = 28000 mm^2.

I_{ZZ} of the built up section:

$$I_{ZZ} = 2\left[\frac{300 \times 30^3}{12} + 300 \times 30 \times \left(\frac{500}{2} + \frac{30}{2}\right)^2\right] + \left[\frac{20 \times 500^3}{12}\right]$$

$$= 1473.73 \times 10^6 mm^4$$

I_{YY} of the built up section:

$$I_{yy} = 2 \times \frac{30 \times 300^3}{12} + \frac{500 \times 20^3}{12} = 135.33 \times 10^6 \text{ mm}^4$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{1473.73 \times 10^6}{28000}} = 229.41 \text{ mm}$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{135.33 \times 10^6}{28000}} = 69.52 \text{ mm}$$

Determination of Buckling curve classification according to Table 10 – P-44: $t_f = 30 \text{ mm} < 40 \text{ mm}$

We should use buckling class 'b' about Z-Z axis and 'c' about y-y axis.

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r}$$

End Condition: Both ends restrained in direction & position

L = 6 m, KL = 0.65 x 6000 = 3900 mm

$$S.R(\lambda_{zz}) = \frac{KL}{r_{zz}} = \frac{3900}{229.41} = 17 \qquad S.R(\lambda_{yy}) = \frac{KL}{r_{yy}} = \frac{3900}{69.52} = 56.09$$

Compressive Stress (f_{cd}): About Z-Z axis : use buckling class 'b'

Ref page 41, Table9 (b) for $f_y = 250\ \text{N/mm}^2$

λ	f_{cd}
10	227
17	?
20	225
10	02
07	?

$$f_{cd}\ \text{for}\ 17 = 227 - \frac{7\times 02}{10} = 225.6\ \text{N/mm}^2$$

About Y – Y axis : use buckling class 'c'

Ref page 42, Table - 9 (c) for $f_y = 250\ \text{N/mm}^2$

λ	f_{cd}
50	183
56.09	?
60	168
10	15
6.09	?

$$f_{cd}\ \text{for}\ 56.09 = 183 - \frac{6.09\times 15}{10} = 173.87\ \text{N/mm}^2$$

Design f_{cd} = 173.87 N/mm^2

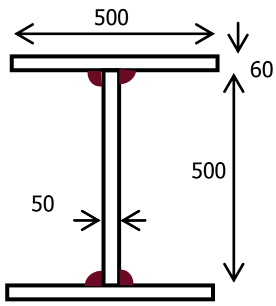
Load carrying capacity of member = Safe stress x Area provided

$$= \frac{173.87\times 28000}{1000} = 4868.36\ \text{kN.}$$

$$\text{Allowable load} = \frac{4868.36}{1.5} = 3245.57\ \text{kN.}$$

P- 759, Design of steel structures by N. Subramaniam:

A heavy column is required to support a gantry Girder and a special H – Section is to be fabricated. The trial section is shown in fig. check its suitability to support a fabricated load of 11,000 KN, assuming both ends are pinned and a length of 8m. Steel of design strength 250 N/mm^2 is to be used.



Solution:

A = 2 x 500 x 60 + 500 x 50 = 85000 mm^2.

I_{ZZ} of the built up section:

$$I_{ZZ} = 2\left[\frac{500\times 60^3}{12} + 500\times 60\times\left(\frac{500}{2}+\frac{60}{2}\right)^2\right] + \left[\frac{50\times 500^3}{12}\right] = 5.243\times 10^9\,\text{mm}^4$$

I_{yy} of the built up section:

$$I_{yy} = 2\times\frac{60\times 500^3}{12} + \frac{500\times 50^3}{12} = 1.255\times 10^9\,\text{mm}^4$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{5.243\times 10^9}{85000}} = 248.36\ \text{mm} \qquad r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{1.255\times 10^9}{85000}} = 121.51\ \text{mm}$$

Determination of Buckling curve classification according to Table 10 – P- 44:

$$t_f = 60 \text{ mm} > 40 \text{ mm}$$

We should use buckling class 'c' about Z-Z axis and 'd' about y-y axis.

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r}$$

End Condition: Both ends pinned.

L = 8 m, KL = L = 8000 mm

$$S.R(\lambda_{zz}) = \frac{KL}{r_{zz}} = \frac{8000}{248.36} = 32.21 \qquad S.R(\lambda_{yy}) = \frac{KL}{r_{yy}} = \frac{8000}{121.51} = 65.84$$

Compressive Stress (f_{cd}):

About Z-Z axis : use buckling class 'c'

Ref page 42, Table9 (c) for $f_y = 250\text{N/mm}^2$

λ	f_{cd}
30	211
32.21	?
40	198
10	13
2.21	?

$$f_{cd} \text{ for } 32.21 = 211 - \frac{13 \times 2.21}{10} = 208.13 \text{ N/mm}^2$$

About Y – Y axis : use buckling class 'd'

Ref page 43, Table - 9 (d) for $f_y = 250 \text{ N/mm}^2$

λ	f_{cd}
60	168
65.84	?
70	152
10	16
5.84	?

$$f_{cd} \text{ for } 65.84 = 168 - \frac{5.84 \times 16}{10} = 158.65 \text{ N/mm}^2$$

Design f_{cd} = 158.65 N/mm^2

Load carrying capacity of member = Safe stress x Area provided

$$= \frac{158.65 \times 85000}{1000} = 13487.8 \text{ kN.}$$

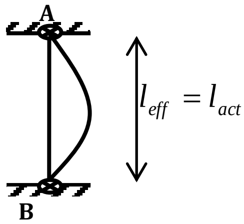
$$\text{Allowable load} = \frac{13487.8}{1.5} = 8992 \text{ kN.}$$

Design Problems:

Problem:

Design a rolled steel beam section column to carry an axial load of 1100 KN. The column is 4 m long and adequately restrained in position, but not in direction at both ends.

Solution:



Axial load = 1100 KN

Factored load = 1.5 x 1100 = 1650 KN

$l_{act} = 4m.$

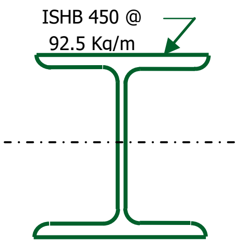
End condition: Adequately restrained in position but not in direction at both ends.

Ref page 45 Table 11 $l_{eff} = l_{act} = 4m$

Assuming permissible stress $(f_{cd}) = 0.6\ f_y = 0.6 \times 250 = 150\ N/mm^2\ (0.4\ f_y\ \text{to}\ 0.6\ f_y)$

Area required = $\dfrac{1650 \times 10^3}{150} = 11000\ mm^2 = 110\ cm^2$

Try ISHB 450 @ 92.5 kg/m.



$a = 117.89 cm^2 = 11789 mm^2, h = 450mm, b_f = 250mm, t_f = 13.7mm$

$r_{xx} = 18.5cm = 185mm, r_{yy} = 5.08cm = 50.8mm.$

$r_{min} = r_{yy} = 50.8mm$

Determination of buckling curve classification

$$\frac{h}{b_f} = \frac{450}{250} = 1.8 > 1.2$$

$$t_f = 13.7 \le 40\ mm$$

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis :use buckling class 'b'

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{4000}{185} = 21.62$$

λ_{zz}	f_{cd}
20	225
21.62	?
30	216
10	09
1.62	?

REF TO TABLE 9(a) P-40

$$f_{cd}\ \text{for}\ 21.62 = 225 - \frac{1.62 \times 09}{10} = 225.03\ N/mm^2$$

Compressive Stress (f_{cd}):

About Y-Y axis :use buckling class 'c'

$$\lambda_{yy} = \frac{KL}{r_{yy}} = \frac{4000}{50.8} = 78.74$$

λ_{yy}	f_{cd}
70	166
78.74	?
80	150
10	16
8.74	?

REF TO TABLE 9(b) P-41

$$f_{cd}\ \text{for}\ 78.74 = 166 - \frac{8.74 \times 16}{10} = 152.02 N/mm^2$$

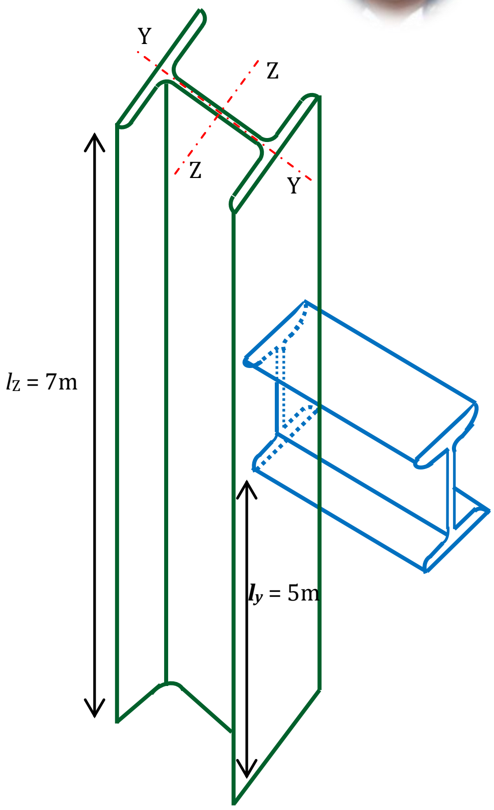
f_{cd} is the min of 222.15 N/mm^2, and 174.83 N/mm^2.

i.e., $f_{cd} = 174.83\ N/mm^2$.

Load carrying capacity = Safe stress x Area provided

$$= \frac{152.02 \times 11789}{1000} = 1792.16KN > 1650KN$$

Safe **Provide ISHB 450 @ 92.5 kg / m.**



P- 332, LSD by steel structures, S.K.Duggal

Design a column to support a factored load of 1050 KN. The column has an effective length of 7 m with respect to Z- axis and 5 m with respect to Y- axis. Use steel of grade Fe 410

Solution:

Factored load = 1050 KN

$l_{ZZ} = 7m, \quad l_{YY} = 5m,$

Assuming permissible stress $(f_{cd}) = 0.6\ f_y \quad (0.4\ f_y \text{ to } 0.6\ f_y)$

$= 0.6 \times 250 = 150\ N/mm^2$

Area required = $\frac{1050 \times 10^3}{150} = 7000mm^2 = 70cm^2$

Try ISHB 350 @ 67.4 kg/m.

$a = 85.91cm^2 = 8591mm^2, h = 350mm, b_f = 250mm, t_f = 11.6mm$

$r_{xx} = 14.95cm = 149.5mm, r_{yy} = 5.34cm = 53.4mm.$

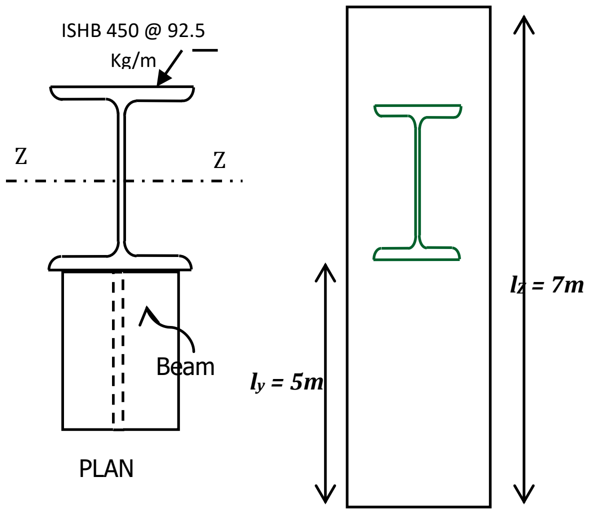
$r_{min} = r_{yy} = 53.4mm$

Determination of buckling curve classification

$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2 \qquad t_f = 11.6 \le 100\ mm$

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to

Table 10, P- 44, IS 800 – 2007.



P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis :use buckling class 'a'

$$\lambda_{ZZ} = \frac{KL}{r_{zz}} = \frac{7000}{149.5} = 46.82$$

λ_{zz}	f_{cd}
40	213
46.82	?
50	205
10	08
6.82	?

REF TO TABLE 9(a) P-40

$$f_{cd} \text{ for } 46.82 = 213 - \frac{6.82 \times 08}{10} = 207.5\ N/mm^2$$

Compressive Stress (f_{cd}):

About Y-Y axis :use buckling class 'b'

λ_{yy}	f_{cd}
90	134
93.63	?
100	118
10	16
3.63	?

$$\lambda_{yy} = \frac{KL}{r_{yy}} = \frac{5000}{53.4} = 93.63$$

REF TO TABLE 9(b) P-41

$$f_{cd} \text{ for } 93.63 = 134 - \frac{16 \times 3.63}{10} = 144.192 \text{ N/mm}^2$$

f_{cd} is the min of 207.5 N/mm^2, and 144.19 N/mm^2.

i.e., f_{cd} = 144.19 N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{144.19 \times 8591}{1000} = 1238.75 \text{ kN} > 1050 \text{ kN}$$

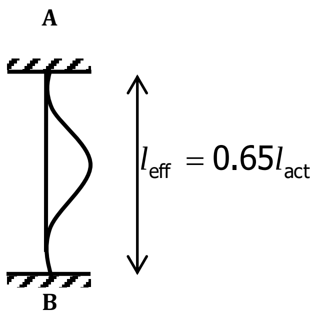
Safe **Provide ISHB 350 @ 67.4 kg / m**

P- 332, LSD by steel structures, S.K.Duggal

Design a column to support a factored load of 800 KN. The column has an effective length of 7 m with respect to Y - axis and 5 m with respect to Z - axis. Use steel of grade Fe 410

Problem:

Design a rolled steel beam section column to carry an axial load of 2500 KN. The column is 5 m long effectively held in position and restrained against rotation at both ends.



Solution:

Axial load = 2500 KN

Factored Load = 1.5 x 2500 = 3750 kN

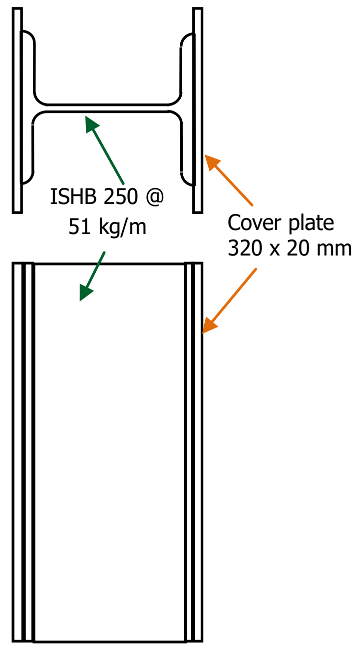
$l_{act} = 5$m.

End condition: Effectively held in position and restrained against rotation at both ends.

$l_{eff} = 0.65 l_{act} = 0.65 \times 5 = 3.25$m (P 45, T - 11)

Assuming permissible stress $(f_{cd}) = 0.8\, f_y = 0.8 \times 250 = 200$ N / mm^2

Area required = $\dfrac{3750 \times 10^3}{200} = 18750 \text{ mm}^2 = 187.50 \text{ cm}^2$



Try ISHB 250 @ 51 kg/m. with additional plates on both flanges of size 320 x 20 mm.

Area (a) = 192.96 cm^2, r_{min} = 8.17 cm = 81.7 mm

$$\text{Slenderness ratio } \lambda_{zz} = \frac{KL}{r_{zz}} = \frac{3250}{81.7} = 39.78 \approx 40$$

Ref page 42 Table 9(c) for $f_y = 250\text{N/mm}^2$

For buckling class 'c' for built up section

f_{cd} for $40 = 198\ N/mm^2$

Load carrying capacity = stress x area provided

$$= \frac{198 \times 19296}{1000} = 3820.61 \text{ kN} > 3750 \text{ kN}$$

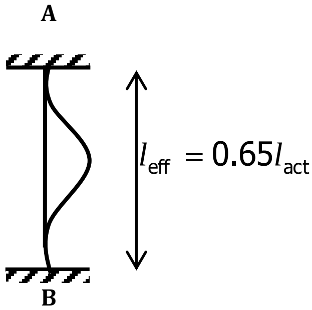
Safe

Provide ISHB 250 @ 51 kg/m. with additional plates on both flanges of size 320 x 25 mm.

Problem:

A column 5 m long is to support a load of 4500 KN. The ends of the columns are effectively held in position and directions. Design the column if rolled steel beams and 18 mm plates are only available.

Solution:



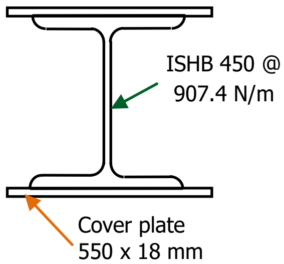
Axial load = 4500 KN, $l_{act} = 5m.$

End condition: Effectively held in position and direction.

Ref page 41 Table 5.2

$l_{eff} = 0.65 l_{act} = 0.65 \times 5 = 3.25$ m

Assuming, stress $\sigma_{ac} = 150 \text{ N/mm}^2$



Area required = $\frac{\text{Load}}{\sigma_{ac}} = \frac{4500 \times 10^3}{150} = 30000 \text{ mm}^2 = 300 \text{ cm}^2$

Ref steel table, Try ISHB 450 @ 907.4 N/m

$a = 117.89 \text{cm}^2 = 11789 \text{mm}^2, I_{xx} = 40349.9 \text{cm}^4 = 40349.9 \times 10^4 \text{mm}^4$

$I_{yy} = 3045 \text{cm}^4 = 3045 \times 10^4 \text{mm}^4$

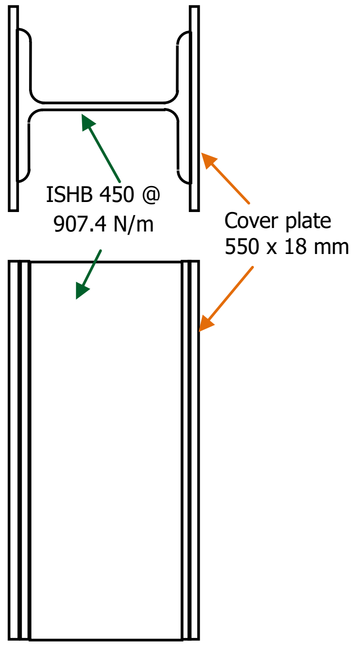
Area required for the two cover plates = $300 - 117.89 = 182.11 \text{ cm}^2$.

Area of one cover plate = $\frac{182.11}{2} = 91.06 \text{ cm}^2 = 9106 \text{ mm}^2$.

Given Th. of cover plate = 18 mm.

Width of cover plate = $\frac{9106}{18} = 505.9$ mm, say 550mm

Try a cover plate of size = 550 x 18 mm.



Check for local buckling

$$\frac{\text{Outstanding width}}{\text{thickness}} \not> 16$$

$$\frac{\left(\frac{550 - 140}{2}\right)}{18} = 11.38 < 16 \qquad \text{Satisfactory}$$

$$\text{Area}(A) = 11789 + 2(550 \times 18) = 31589 \text{ mm}^2$$

Ixx of the built up section

$$I_{xx} = 40349.9 \times 10^4 + 2\left[\frac{550 \times 18^3}{12} + 550 \times 18\left(\frac{450}{2} + \frac{18}{2}\right)^2\right] = 1.49 \times 10^9 \text{mm}^4$$

Iyy of the built up section

$$I_{yy} = 3045 \times 10^4 + 2\left[\frac{18 \times 550^3}{12}\right] = 529 \times 10^6 \text{ mm}^4$$

$$\therefore I_{min} = I_{yy} = 529 \times 10^6 \text{mm}^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{529 \times 10^6}{31589}} = 129.4 \text{ mm}$$

$$S.R(\lambda) = \frac{3250}{129.4} = 25.11$$

Ref page 42, Table 9(c) ,for $f_y = 250 \text{ N/mm}^2$

$$f_{cd} \text{ for } 25.11 = 224 - \frac{13 \times 5.11}{10} = 217.36 \text{ N / mm}^2$$

λ	f_{cd}
20	224
25.11	?
30	211
10	13
5.11	?

Load carrying capacity = Safe stress x area provided

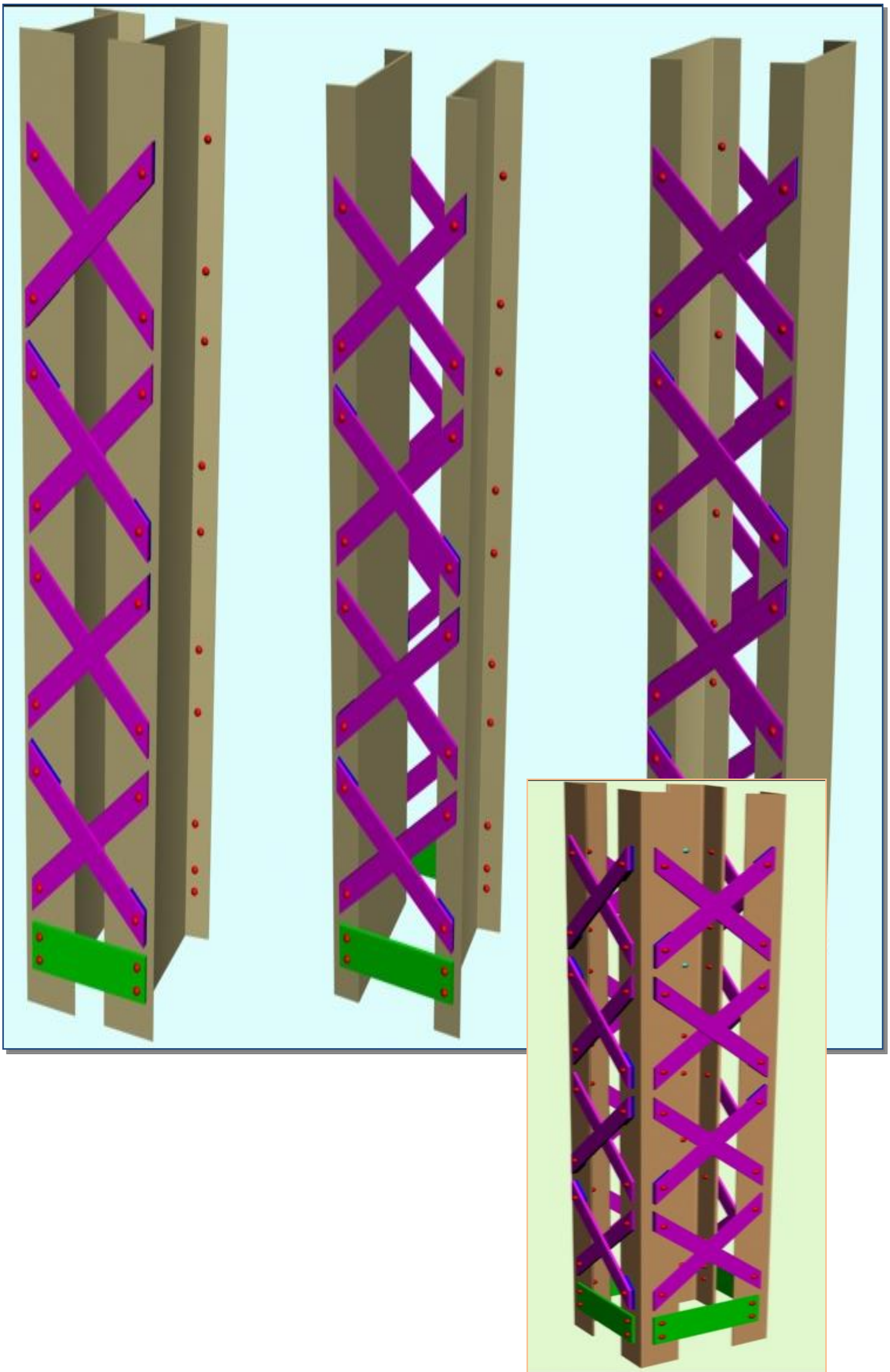
=

$$\frac{217.36 \times 31589}{1000} = 6866 \text{ kN} > 6750 \text{ kN}$$

SAFE

<u>Provide ISHB 450 @ 907.4N/m with additional cover plates of size 550 x 18mm one on each side.</u>

Lacing system



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LACING FOR BUILT-UP COMPRESSION MEMBERS (P - 48, 49, 50 Cl : 7.6)

The different components of the built-up section should be placed uniformly at a maximum possible distance from the axis of the column for greater strength of the column. The different components of the built up section are connected together so that they act as single column. Lacing is generally preferred in case of eccentric loads. Battening is normally used for axially loaded columns and where the components are not for apart. Flat bars are generally used for lacing. Angles, channels, and tubular sections are also used for the lacing of very heavy columns. Plates are used for battens.

Design procedure

<u>Cl 7.6.4 page 50</u>

1. The angle of inclination of the lacing with the longitudinal axis of the column should be between $40°$ to $70°$.

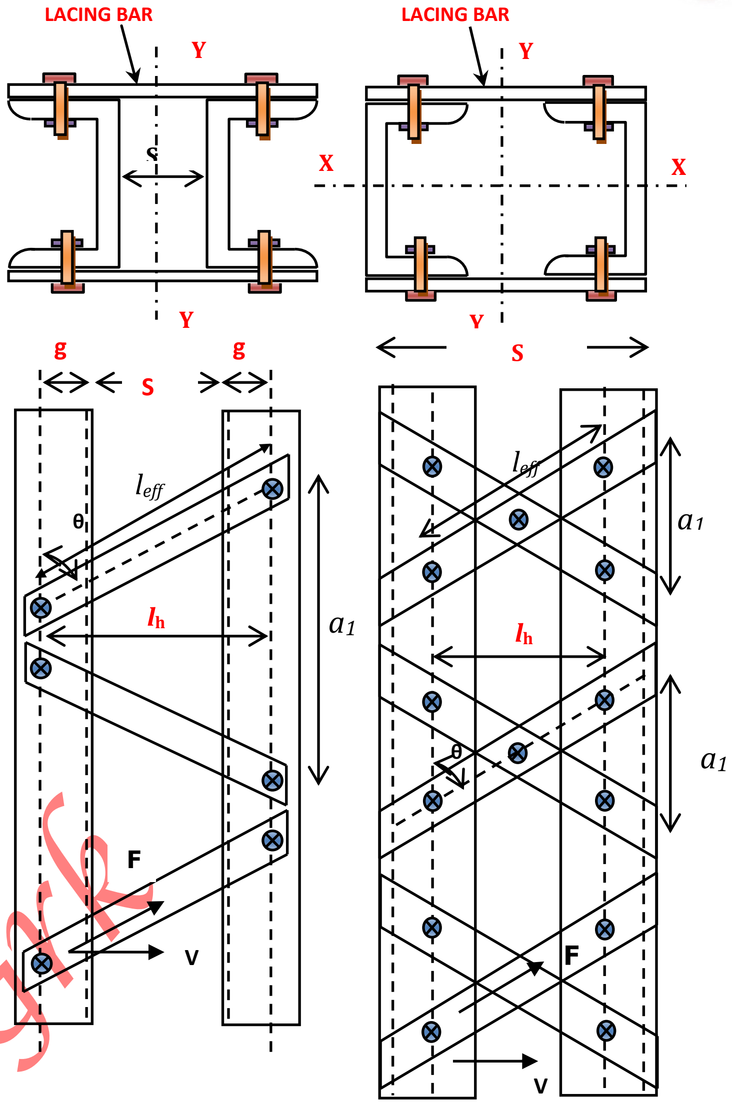
Cl 7.6.6.3 page 50

2. The slenderness ratio $\frac{l_{eff}}{r_{\min}}$ of the lacing bars should not exceed 145. The effective length 'l_e' of the lacing bars should be taken as follows.

Type of lacing	**Effective length (le)**
Single lacing, bolted at ends	Length b/w inner ends of bolts on lacing bar
Double lacing, bolted at ends at Intersection	0.7 times b/w inner ends of bolts on lacing bars (0.7x L)
Welded lacing	0.7 times distance b/w inner ends of effective lengths of welds at ends (0.7 x L)

If flat bars of width 'b' and thickness 't' are used for lacing, the maximum slenderness Ratio is given by

$$\text{Max. S.R}(\lambda) = \frac{l_e}{r_{\min}} = \frac{l_e}{\sqrt{\frac{I}{A}}} = \frac{l_e}{\sqrt{\frac{bt^3}{12} \times \frac{1}{bt}}} = \frac{l_e\sqrt{12}}{t} \ngtr 145$$



Cl 5.7.6.1 page 51

1. For bolted or welded lacing system- ----spacing

$\frac{a_1}{r_1} \not> 50$ or 0.7 times max SR of the compression member as a whole, whichever is less.

Where,

a_1 = Distance b/w the centers of connections of the lattice bars to each components as shown in fig.

r_1= Min radius of gyration of the components of compression members.

Cl 7.6.2 page 50

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

Cl 7.6.3 page 50

Thickness of lacing bars

$t \not< \frac{l_{eff}}{40}$ for Single lacing

$t \not< \frac{l_{eff}}{60}$ for Double lacing bolted or welded at intersection.

Where,

l = length b/w inner end bolts.

Design of Lacings: Cl : 7.6.6

The lacing of compression members should be designed to resist to transverse shear V = 2.5% of the axial force in member.

This shear is divided equally among all transverse lacing system in parallel planes. The lacing system should be designed to resist additional shear due to bending if the compression member carries bending due to eccentric load, applied end moments, and lateral loading.

For single lacing system on two parallel faces, the force (compressive or tensile) in each bar.

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

For double lacing system on two parallel planes, the force (compressive or tensile) in each bar.

$$\boxed{F = \frac{V}{4\sin\theta}}\ \text{vt}$$

If the flat lacing bars of width 'b' and thickness 't' have bolts of diameter 'd' then

$$\text{Compressive stress in bar } \frac{\text{Force}}{\text{Gross area}} = \frac{F}{b \times t} \not> \sigma_{ac}$$

$$\text{Tensile stress in each bar} = \frac{\text{Force}}{\text{Net area}} = \frac{F}{(b - d_o)t} \not> \sigma_{at}$$

$$\sigma_{at} = 0.6f_y$$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \not< F$$

<u>Tensile Force (P - 32)</u>

$$T_{dn} = \frac{0.9A_n f_u}{\gamma_{m1}} \not< F \qquad \text{OR}$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}}$$

<u>Connection Details:</u> $\text{No. of bolts} = \frac{F}{BV}$

P-75, Cl: 10.3.3

1) Strength of bolt in Single shear:

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0}$ Edge distance e = 1.5 x d_o

2) $\dfrac{p}{3d_0} - 0.25$

P = 2.5 x d

3) $\dfrac{f_{ub}}{f_u}$ 4) 1

t* → Min of 1) Thickness of flange of column section and
2) Thickness of lacing bar

Bolt value (BV) = Min of above two values.

Welded connection;

Lap joint: Overlap $\nless 4$ times thickness of bar or member, whichever is less.

Butt joint: Full penetration butt weld or fillet weld on each side. Lacing bar should be placed opposite to flange or stiffening member of main member.

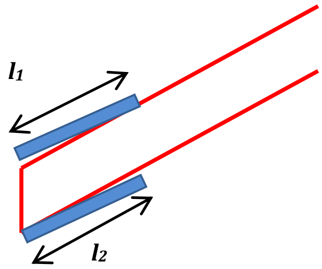
Welded connection:

Max Size of weld S = thickness of flat - 1.5

Force in lacing bar = Strength of the weld

$f_u = 410 \text{ N/mm}^2$.

$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$



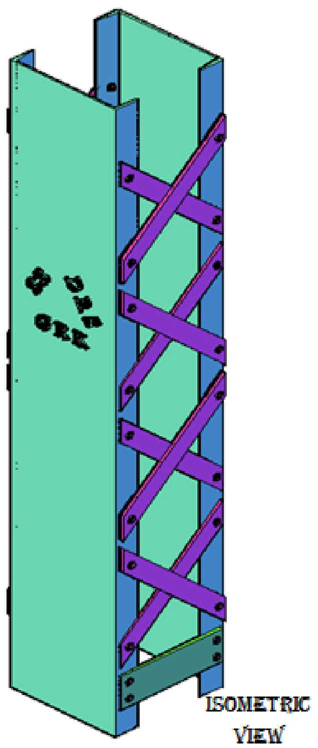
$$\text{Effective length of weld} = \frac{F}{0.707 \times D \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}}$$

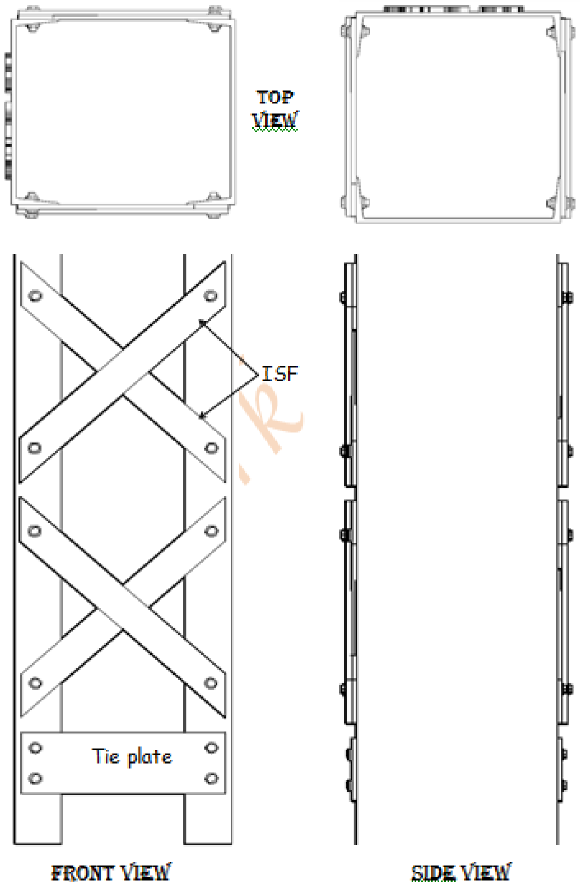
Provide Length of weld on each side of flat

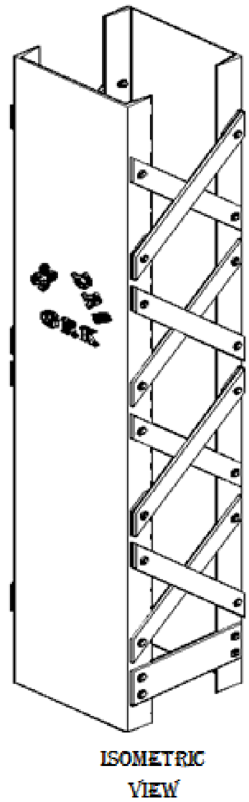
JAN / FEB 2004 – 8 marks

Why are lacing / battens provided in steel columns consisting of more than one section ? Explain with neat sketches the details of different types of lacing and battening system.

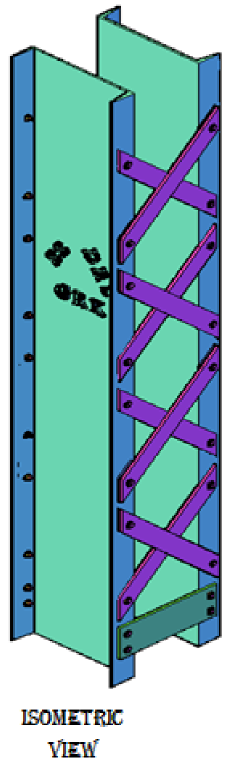
2 = CHANNELS TOE TO TOE LACED TOGETHER BY
Double lacing system

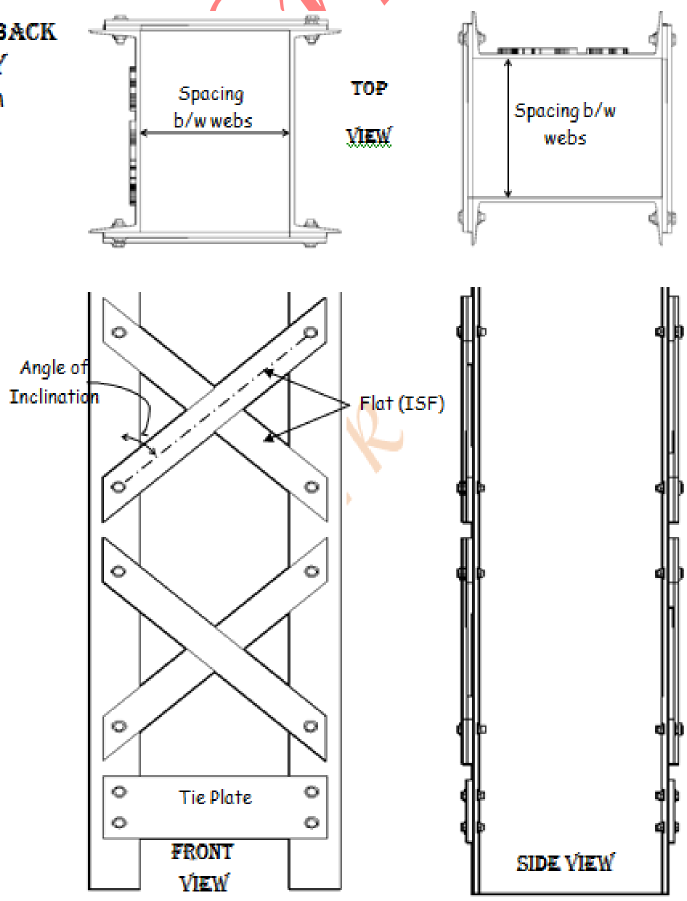


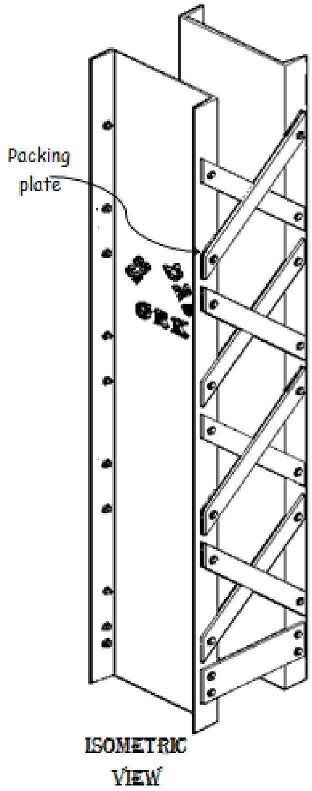




2 = CHANNELS BACK TO BACK LACED TOGETHER BY
Double lacing system







Problem:

Design a built- up column with 2 channel sections back to back to carry an axial factored load of 1300 KN. The height of the column is 7 m and effectively held in position at both ends, but not restrained against rotation. Take $f_y = 250\,\text{MPa}$. Design single lacing system with 16 mm dia bolt of 4.6 grade.

Solution:

Design of compression member

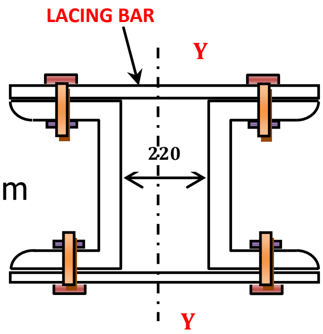
Factored Load= 1300 KN

Assuming permissible stress $= 0.5\,f_y = 0.5 \times 250 = 125\ \text{N/mm}^2 \quad (0.4\,f_y \text{ to } 0.6\,f_y)$

$$\text{Area of 2 channels} = \frac{\text{Load}}{f_{cd}} = \frac{1300 \times 10^3}{125} = 10400\ \text{mm}^2 = 104\ \text{cm}^2$$

Try 2 - ISMC 350 @ 84.2 Kg /m with spacing b/w webs S = 220 mm

[P - 48, Cl: 7.6.1.1
Note: - The spacing is choosen in such a way that
$I_{ZZ} \approx I_{YY}$ & $r_{yy} > r_{zz}$]



$a = 107.32\ \text{cm}^2 = 10732\ \text{mm}^2$

$r_{zz} = 13.66\ \text{cm} = 136.6\ \text{mm}, \quad r_{yy} = 13.74\ \text{cm} = 137.4\ \text{mm}$

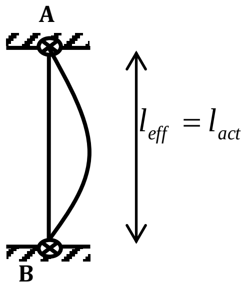
P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section } (\lambda) = 1.05 \times \frac{KL}{r}$$

Effective length (Table11, Cl: 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,
but not restained against rotation. (Both ends Hinged)

$KL = L = 7\ \text{m} = 7000\ \text{mm}$



P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05 \times \frac{7000}{136.6} = 53.81$$

Compressive stress f_{cd} = 177.30 N/mm^2.

λλ	f_{cd}
50	183
53.81	?
60	168
10	15
3.81	?

$$f_{cd} = 183 - \frac{3.81 \times 15}{10} = 177.30\ \text{N/mm}^2$$

Load carrying capacity $= f_{cd} \times \text{Area}$

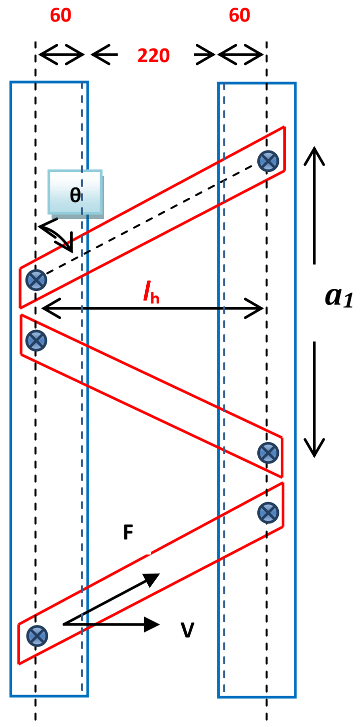
$$= \frac{177.30 \times 10732}{1000} = 1902.80\ \text{KN} > 1300\ \text{KN}$$

Safe

Provide 2 - ISMC 350 @ 84.2 Kg / m.

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$$\frac{a_1}{r_1} \ngtr 50 \text{ or } 0.7\lambda.7 \text{ builtup section, whichever is less.}$$

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $\left(40^0 \prec \theta \prec 70^0\right)$

The gauge distance 'g' for ISMC 350 is 60 mm.

$r_1 = 2.83\text{cm}$

$\therefore$ Horizontal length of lacing $l_h = 60 + 220 + 60 = 340$ mm

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \times 340 = 680$ mm

$$\frac{a_1}{r_1} = \frac{680}{28.3} = 24.03 < 50 \text{ and} < 0.7 \times 53.81 \Rightarrow 37.45$$

The local buckling of the column does not occur,

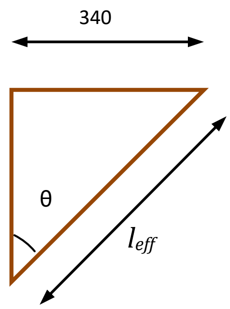
Hence single lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming dia of bolt = 16 mm

Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 50 mm



Thickness of bar (t) (P- 50, Cl 7.6.3)

$$t \nless \frac{1}{40} \text{ of distance of inner bolts}$$

$$\sin\theta = \frac{340}{l_{eff}}$$

$$\frac{340}{l_{eff}} = \frac{340}{\sin 45} = 480.83\text{mm}$$

$l_{eff} = 480.83\text{mm}$ For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 480.83 = 12.04 \text{ mm} \qquad \text{Say 16 mm}$$

Try a lacing bar of 50 mm width and 16 mm thick

i.e., 50 ISF 16

Note: (P- 50, Cl 7.6.3) Double lacing system $t = \frac{1}{60} \times l_{eff}$

$$r_{\min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{l_{eff}\,t^3}{12}}{l_{eff}\,t}}$$

$$r_{\min} = \frac{t}{\sqrt{12}}$$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{l_{eff}}{r_{\min}} \ngtr 145$$

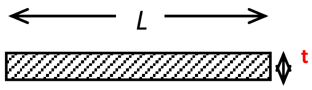
$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{480.83 \times \sqrt{12}}{16} = 104.10 < 145 \qquad \text{Safe}$$

Note: - (P-50, Cl 7.6.3)

For double lacing system l_{eff} = 0.7 x Length of lacing bar b/w inner bolts.

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \not> 145$$

Check for Compressive Force and Tensile Force:



Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1300 = 32.5 \text{ KN}$$

$$\text{Force } (F) = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{32.5}{2 \times \sin 45} = 22.98 \text{ KN}$$

OR

$$\text{Force } (F) = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Compressive Stress

for $\lambda = 104.10$

Ref Page 42 , Table 9(c)

$\lambda\lambda$	f_{cd}
100	107
104.10	?
110	94.6
10	12.4
4.10	?

$$f_{cd} = 107 - \frac{4.1 \times 12.4}{10} = 101.92 \text{ N / mm}^2$$

Compressive Force = Compressive Stress × Area of lacing bar $\not<$ F

$$\text{Compressive Force} = \frac{101.92 \times 50 \times 16}{1000} = 81.54 > 22.98\text{KN} \quad \underline{\text{Safe}}$$

Tensile force (P-32):

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9(50 - 18) \times 16 \times 410}{1.25 \times 1000} = 151.14\text{KN} > 22.98 \text{ KN} \quad \underline{\text{Safe}}$$

Provide 50 ISF 16 as lacing bar

Connection Details:

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 16 mm.

Dia of hole (d_0) = 16 + 2 =18 mm

Strength of one bolt in Single shear: P-75, Cl: 10.3.3

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming thread is interfering the shear plane

$$n_n = 1 \quad n_s = 0 \quad , \gamma_{mb} = 1.25 \quad , \quad A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83\text{mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83}{1.25 \times 1000}\right) = 28.97KN$$

2) Strength of one bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 = 27 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.68$ P = 2.5 x 16 = 40 mm Say 50 mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t → Min of 1) Thickness of flange of channel (13.5) and*
2) thickness of lacing bar (16 mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 13.5^* \times 400}{1.25 \times 1000} = 112.32KN$$

Bolt value (BV) = 28.97 KN.

No of bolts = $\dfrac{22.98}{28.97} = 0.79$ Say 2 No's (Min) One on each side

Problem:

Design a built- up column with 2 channel sections back to back to carry an axial factored load of 1300 KN. The height of the column is 7 m and effectively held in position at both ends, but not restrained against rotation. Take $f_y = 250\,\text{MPa}$. Design single lacing system with field weld.

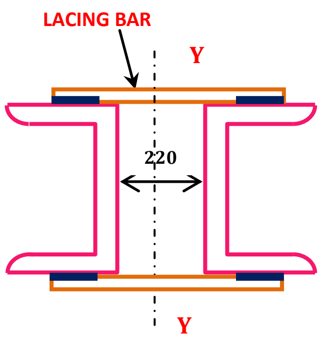
Solution:

Design of compression member

Factored Load= 1300 KN

Assuming permissible stress $= 0.5\,f_y = 0.5 \times 250 = 125\,N/mm^2$ $(0.4\,f_y \text{ to } 0.6\,f_y)$

Area of 2 channels $= \dfrac{Load}{\sigma_{ac}} = \dfrac{1300 \times 10^3}{125} = 10400\,mm^2 = 104\,cm^2$



Try 2 - ISMC 350 @ 84.2 Kg/m with spacing b/w webs S = 220 mm

$$\left[\begin{array}{l} \text{P - 48, Cl : 7.6.1.1} \\ \text{Note : - The spacing is choosen in such a way that} \\ \qquad I_{xx} \approx I_{yy} \quad \& \; r_{yy} > r_{xx} \end{array}\right]$$

$a = 107.32\,cm^2 = 10732\,mm^2$, $r_{zz} = 13.66\,cm = 136.6\,mm$,
$r_{yy} = 13.74\,cm = 137.4\,mm$

P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \frac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$KL = L = 7\ m = 7000\ mm$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

$\lambda\lambda$	f_{cd}
50	183
53.81	?
60	168
10	15
3.81	?

$$f_{cd} = 183 - \frac{3.81 \times 15}{10} = 177.30\ N/mm^2$$

Compressive stress about ZZ- axis, (f_{cd-zz})

$$\lambda_{ZZ} = 1.05 \times \frac{7000}{136.6} = 53.81$$

Compressive stress f_{cd} = 177.30 N/mm^2.

Load carrying capacity $= f_{cd} \times$ Area

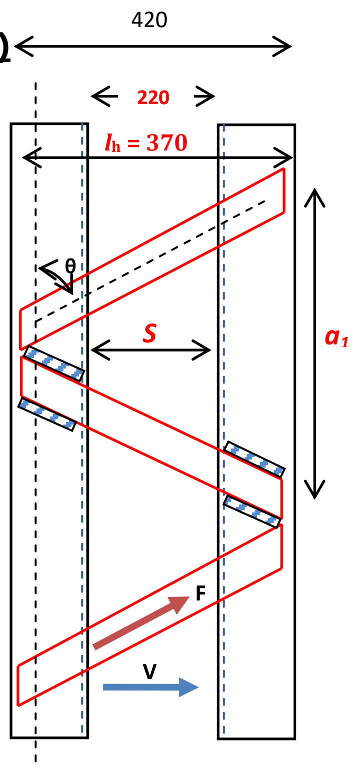
$$= \frac{177.30 \times 10732}{1000} = 1902.80\ KN > 1300\ KN$$

Safe

Provide 2 - ISMC 350 @ 84.2 Kg / m.

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$\frac{a_1}{r_1} \not> 50$ or $0.7\lambda.7$ builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

The min radius of gyration for ISMC 350

$r_1 = 2.83cm = 28.3\ mm$

$\therefore$ Horizontal length of lacing $l_h = 100 + 220 + 100 - 50 = 370$ mm

Spacing of lacing is c/c distance of adjacent bars

$= a_1 = 2 \times 370 = 740$ mm

$$\frac{a_1}{r_1} = \frac{740}{28.3} = 26.15 < 50 \text{ and } < 0.7 \times 53.81 \Rightarrow 37.45$$

The local buckling of the column does not occur,

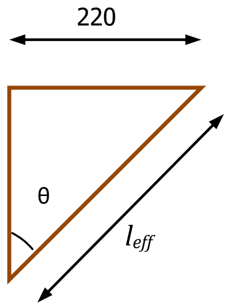
Hence single lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming, Width of lacing = 60 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)



$t \nless \frac{1}{40}$ of distance of inner welds

$$\sin\theta = \frac{220}{l_{eff}}$$

$$l_{eff} = \frac{220}{\sin 45} = 311.13\text{mm}$$

$l_{eff} = 311.13\text{mm}$ For single lacing system

$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 311.13 = 7.77$ mm Say 8 mm

Try a lacing bar of 60 mm width and 8 mm thick

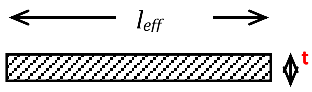
i.e., 60 ISF 8

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Note: (P- 50, Cl 7.6.3) Double lacing system $t = \frac{1}{60} \times l_{eff}$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)



$$\lambda = \frac{l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{311.13 \times \sqrt{12}}{8}$$

$= 134.72 < 145$ Safe

Note: - (P-50, Cl 7.6.3)

For double lacing system l_{eff} = 0.7 x Length of lacing bar b/w inner bolts.

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1300 = 32.5 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{32.5}{2 \times \sin 45} = 22.98 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system, n = 2 for double lacing system

Ref Page 42, Table 9(c)

λ	f_{cd}
130	74.3
134.72	?
140	66.2
10	8.1
4.72	?

$$f_{cd} = 74.3 - \frac{8.1 \times 4.72}{10} = 70.48 \text{ N/mm}^2$$

Compressive Stress

for $\lambda = 134.72$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \not< F$$

$$\text{Compressive Force} = \frac{70.48 \times 60 \times 8}{1000} = 33.82\text{KN} > 22.98\text{KN} \qquad \underline{\text{Safe}}$$

$\underline{\text{Tensile Force (P - 32)}}$

$$T_{dn} = \frac{0.9 A_n f_u}{Y_{m1}} \not< F$$

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{Y_{m1}} = \frac{0.9 \times 60 \times 8 \times 410}{1.25 \times 1000} = 141.70\text{KN} > 22.98 \text{ KN} \qquad \underline{\text{Safe}}$$

Provide 60 ISF 8 as lacing bar

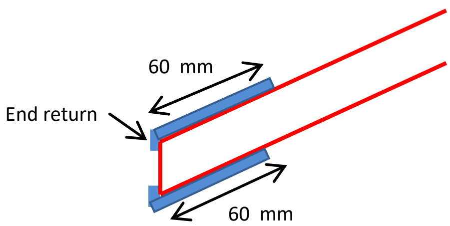
Welded connection:

Max Size of weld S = 8 - 1.5 = 6.5 mm

Say S = 5 mm

Force = Strength of the weld

$f_u = 410 \text{ N/mm}^2$.



$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 5 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 669.43\, l \text{ N - mm}$$

$$\text{Effective length of weld} = \frac{22.98 \times 10^3}{669.43} = 34.33 \text{ mm} \qquad \text{Say 40 mm}$$

Length of weld on each side of flat $= 40/2 = 20$ mm.

Length of longitudinal weld : It is the max of the following

1. Overlap length a) 4t = 4 x 8 = 32mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

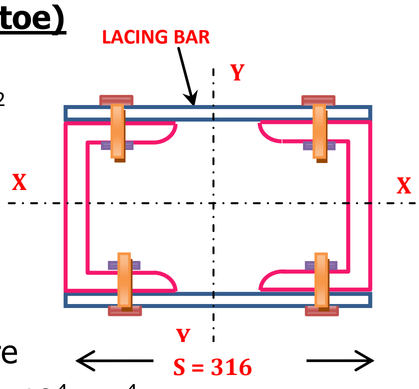
The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

Problem:

Design a built up member to carry an factored load of 1400 KN and effective length in both planes is 6.5m. The column is restrained in position but not in direction at both the ends. Provide double lacing system with bolted connections. Assume steel of grade Fe 410 and bolts of grade 4.6. Design the column with two channels placed toe- to – toe.

Solution:

Design of compression member (Channels toe to toe)



Factored Load= 1400 KN

Assuming permissible stress $= 0.6\, f_y = 0.6 \times 250 = 150\ N/mm^2$

$$\text{Area of 2 channels} = \frac{\text{Load}}{\sigma_{ac}} = \frac{1400 \times 10^3}{150}$$

$= 9333.33\ mm^2 = 93.33\ cm^2$

Try 2 – ISMC 350 @ 42.1 kg /m. Properties of each channels are

$a = 53.66\ cm^2 = 5366\ mm^2$ $\qquad I_{yy} = 430.6\ cm^4 = 430 \times 10^4 mm^4,$

$I_{zz} = 10008\ cm^4 = 10008 \times 10^4 mm^4\ ,$ $\qquad C_{yy} = 2.44\ cm = 24.4\ mm$

Spacing (S): Equate $I_{ZZ} = I_{YY}$ of builtup sections

$$2 \times I_{ZZ} = 2 \times I_{yy}$$

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

$$2 \times 10008 \times 10^4 = 2 \times \left[430 \times 10^4 + 5366 \times \left(\frac{S}{2} - 24.4 \right)^2 \right]$$

$S = 316mm$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section } (\lambda) = 1.05 \times \frac{KL}{r}$$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$KL = L = 6.5\ \ m = 6500\ mm$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2 \times 10008 \times 10^4}{2 \times 5366}} = 136.57$$

44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for $f_y = 250\ N/mm^2$.

Compressive stress about ZZ- axis, (f_{cd-zz})

$$\lambda_{ZZ} = 1.05 \times \frac{6500}{136.57} = 50$$

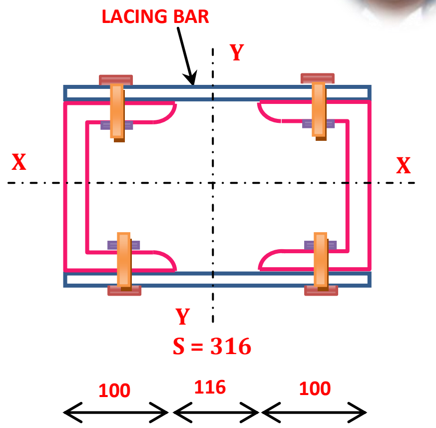
Compressive stress $f_{cd} = 183\ N/mm^2$.

$$\text{Load carrying capacity} = f_{cd} \times \text{Area} = \frac{183 \times 2 \times 5366}{1000} = 1964\ KN > 1400\ KN$$

Safe *Provide 2 - ISMC 350*

Design Of Lacing: (Double lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$\frac{a_1}{r_1} \not> 50$ or $0.7\lambda.7$ builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45^\circ \; \left(40^0 \prec \theta \prec 70^0\right)$

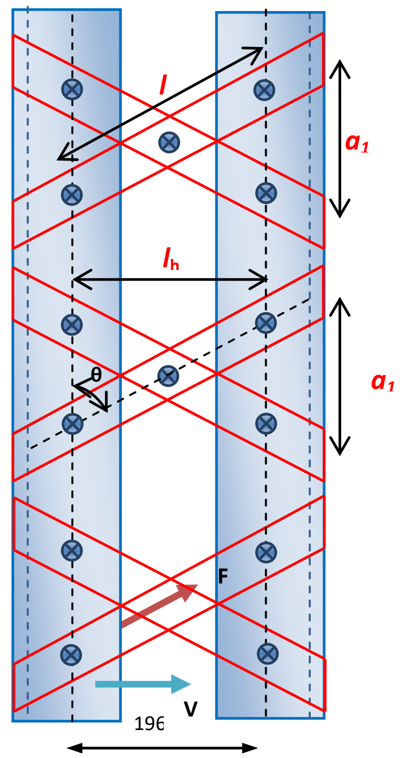
The gauge distance 'g' for ISMC 350 is 60 mm.

$r_1 = 2.83\text{cm}$

$\therefore$ Horizontal length of lacing b/w bolts $l_h = 316 - 60 - 60 = 196$ mm

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 196$ mm



$$\frac{a_1}{r_1} = \frac{196}{28.3} = 6.93 < 50 \text{ and} < 0.7 \text{ x}50 \Rightarrow 35$$

The local buckling of the column does not occur,

Hence double lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming dia of bolt = 16 mm

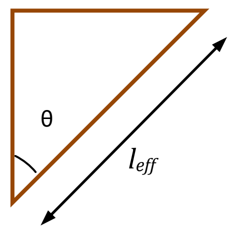
Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 50 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{60}$ of distance of inner bolts

$$\sin\theta = \frac{196}{l_{eff}}$$

$$l_{eff} = \frac{196}{\sin 45} = 277.20\text{mm}$$



For double lacing system

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 277.20 = 4.62 \text{ mm} \qquad \text{Say } 6 \text{ mm}$$

Try a lacing bar of 50 mm width and 6 mm thick

i.e., 50 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \not> 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 277.20 \times \sqrt{12}}{6} = 112.03 < 145 \quad \underline{\text{Safe}}$$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1400 = 35\ KN$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{35}{4 \times \sin 45} = 12.37\ KN$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Ref Page 42 , Table 9(c)	
λλ	f_{cd}
100	107
112.03	?
110	94.6
10	12.4
2.03	?

$$f_{cd} = 107 - \frac{2.03 \times 12.4}{10} = 104.48\ N/mm^2$$

Compressive Stress for $\lambda = 112.03$, $f_{cd} = 104.48\ N/mm^2$

Compressive Force = Compressive Stress × Area of lacing bar $\not<$ F

$$\text{Compressive Force} = \frac{104.48 \times 50 \times 6}{1000} = 31.34 > 12.37 KN \quad \text{Safe}$$

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (50 - 18) \times 6 \times 410}{1.25 \times 1000}$$

$= 56.68 KN > 12.37\ KN$ Safe

Provide 50 ISF 6 as lacing bar

Connection Details:

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 16 mm.

Dia of hole (d_0) = 16 + 2 = 18 mm

1) Strength of one bolt in Single shear :

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming thread is interfering the shear plane

$n_n = 1$ $n_s = 0$, $\gamma_{mb} = 1.25$, $A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83 mm^2$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83}{1.25 \times 1000}\right) = 28.97 KN$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 =27 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.68$ P = 2.5 x 16 = 40 mm Say 50 mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t → Min of 1) Thickness of flange of channel (13.5) and*
2) thickness of lacing bar (6 mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 6^* \times 400}{1.25 \times 1000} = 49.92 KN$$

Bolt value (BV) = 28.97 KN.

No of bolts = $\dfrac{12.37}{28.97} = 0.43$ Say 2 No's (Min) One on each side

Problem:

Design a built up member to carry an factored load of 1400 KN and effective length in both planes is 6.5m. The column is restrained in position but not in direction at both the ends. Provide double lacing system with fillet field weld connections. Assume steel of grade Fe 410. Design the column with two channels placed toe – to – toe.

Solution:

Design of compression member (Channels toe to toe)

Factored Load= 1400 KN

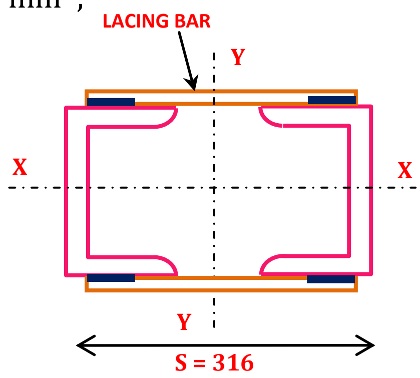
Assuming permissible stress $= 0.6\,f_y = 0.6 \times 250 = 150\ N/mm^2$

Area of 2 channels $= \dfrac{Load}{\sigma_{ac}} = \dfrac{1400 \times 10^3}{150} = 9333.33\ mm^2 = 93.33\ cm^2$

Try 2-ISMC 350 @ 42.1Kg/m Properties of each channels are

$a = 53.66\ cm^2 = 5366\ mm^2$ $I_{yy} = 430.6\ cm^4 = 430 \times 10^4\ mm^4$,

$I_{ZZ} = 10008\ cm^4 = 10008 \times 10^4\ mm^4$, $C_{yy} = 2.44\ cm = 24.4\ mm$



Spacing (S): Equate $I_{ZZ} = I_{YY}$ of builtup sections

$$2 \times I_{ZZ} = 2 \times I_{yy}$$

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

$$2 \times 10008 \times 10^4 = 2 \times \left[430 \times 10^4 + 5366 \times \left(\frac{S}{2} - 24.4 \right)^2 \right]$$

$S = 316mm$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section}\,(\lambda) = 1.05 \times \frac{KL}{r}$$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$$KL = L = 6.5 \ \text{m} = 6500 \ \text{mm}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2 \times 10008 \times 10^4}{2 \times 5366}} = 136.57$$

44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05 \times \frac{6500}{136.57} = 50$$

Compressive stress f_{cd} = 183 N/mm^2.

$$\text{Load carrying capacity} = f_{cd} \times \text{Area} = \frac{183 \times 2 \times 5366}{1000}$$

$= 1964$ KN > 1400 KN Safe

Provide 2 - ISMC 350

Design Of Lacing: (Double lacing system)

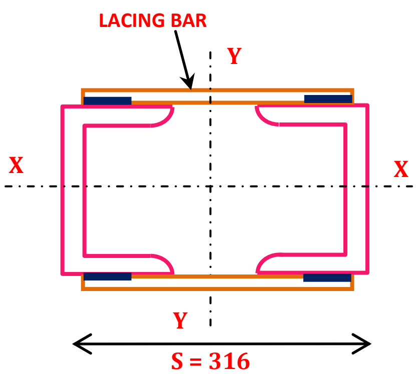
Check for local buckling of column section (P -50, cl 7.6.5)

$$\frac{a_1}{r_1} \not> 50 \ \text{ or } 0.7\lambda.7 \ \text{builtup section, whichever is less.}$$

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45^\circ \ \left(40^0 \prec \theta \prec 70^0\right)$



The radius of gyration for single ISMC 350

$$r_1 = 2.83\text{cm} = 28.3\text{mm}$$

$\therefore$ Horizontal length of lacing $l_h = 316 - 50 = 266$ mm

Spacing of lacing $= a_1 = 266$ mm

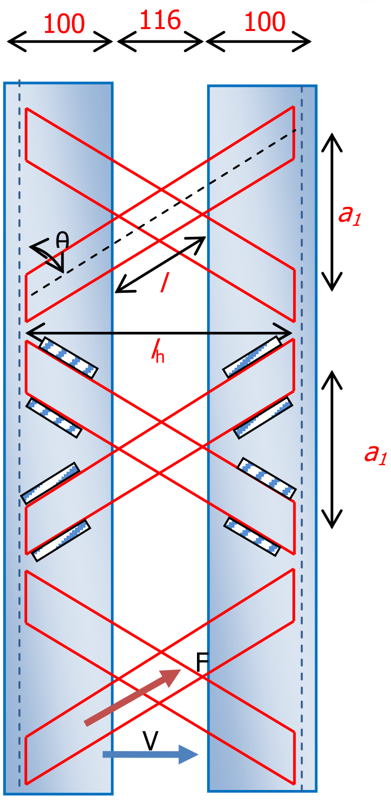
$$\frac{a_1}{r_1} = \frac{266}{28.3} = 9.40 < 50 \text{ and } < 0.7 \text{ x}50 \Rightarrow 35$$

The local buckling of the column does not occur,

Hence double lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)



Assuming,

Width of lacing = 60 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \nless \frac{1}{60}$ of distance of inner welds

Horizontal length b/w inner welds $= 316 - 100 - 100 = 116\text{mm}$

$$\sin\theta = \frac{116}{l_{eff}}$$

$$l_{eff} = \frac{116}{\sin 45} = 164.05\text{mm}$$

For double lacing system

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 164.05 = 2.73 \text{ mm} \qquad \text{Say 6 mm}$$

Try a lacing bar of 60 mm width and 6 mm thick

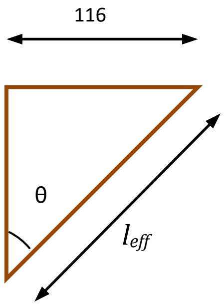
i.e., 60 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 164.05 \times \sqrt{12}}{6} = 66.30 < 145 \qquad \underline{\text{Safe}}$$



Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1400 = 35 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{35}{4 \times \sin 45} = 12.37 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \operatorname{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Compressive Stress

for $\lambda = 66.30$

Compressive Force $=$ Compressive Stress $\times$ Area of lacing bar $\nless$ F

$$\text{Compressive Force} = \frac{157.92 \times 60 \times 6}{1000} = 56.85 > 12.37\,KN$$

Safe

Ref Page 42, Table 9(c)

λ	f_{cd}
60	168
66.30	?
70	152
10	16
6.30	?

$$f_{cd} = 168 - \frac{6.30 \times 16}{10} = 157.92\ \text{N/mm}^2$$

Tensile Force (P-32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \nless F$$

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times 60 \times 6 \times 410}{1.25 \times 1000}$$

$T_{dn} = 106.27$ kN > 12.37 kN Safe

Provide 60 ISF 6 as lacing bar

Welded connection:

Max Size of weld S = 6 - 1.5 = 4.5 mm

Say S = 4 mm

Force = Strength of the weld

$f_u = 410\ \text{N/mm}^2$.

$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\, l\ \text{N-mm}$$

$$\text{Effective length of weld} = \frac{12.37 \times 10^3}{535.54} = 23\ \text{mm} \qquad \text{Say } 30\ \text{mm}$$

Length of weld on each side of flat $= 30/2 = 15$ mm.

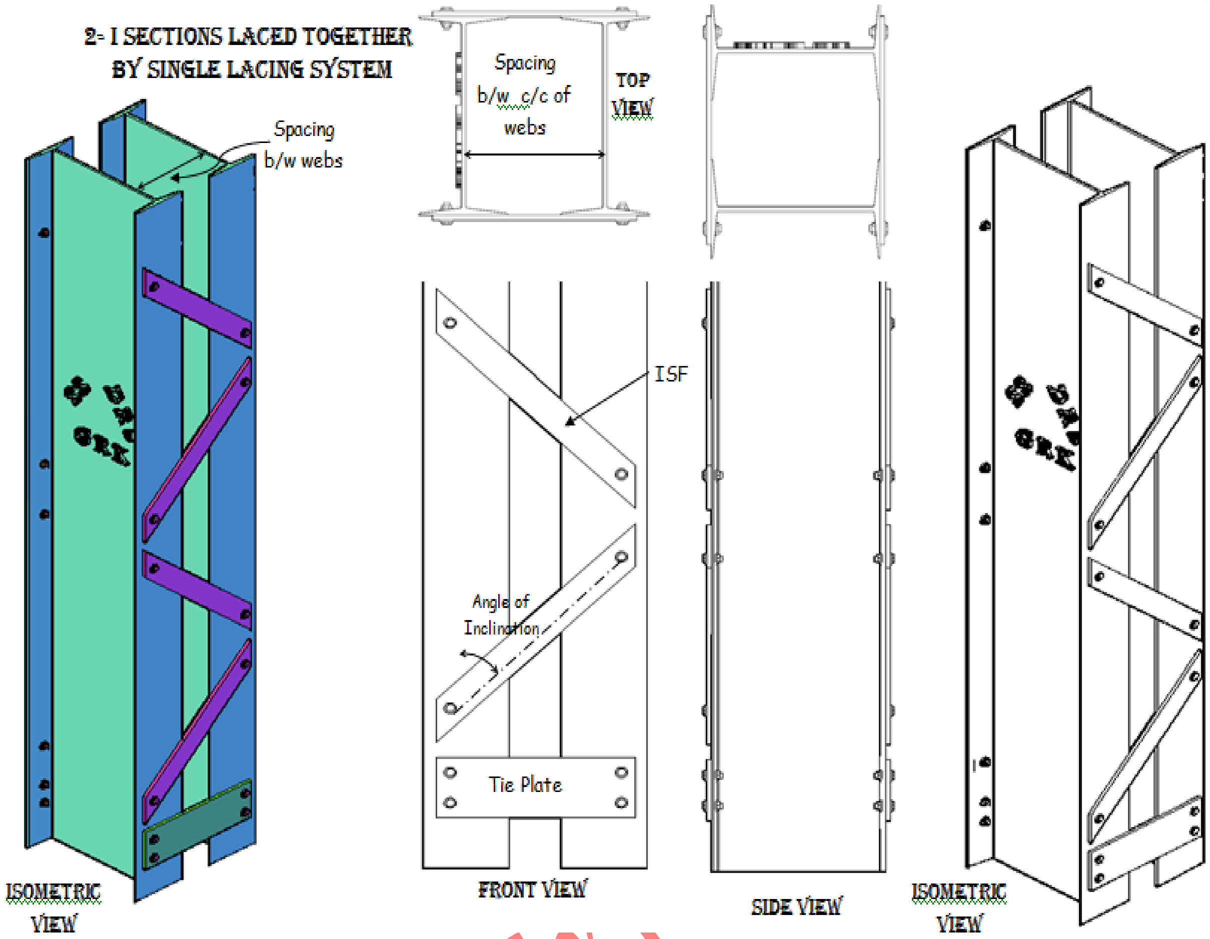
Length of longitudinal weld : It is the max of the following

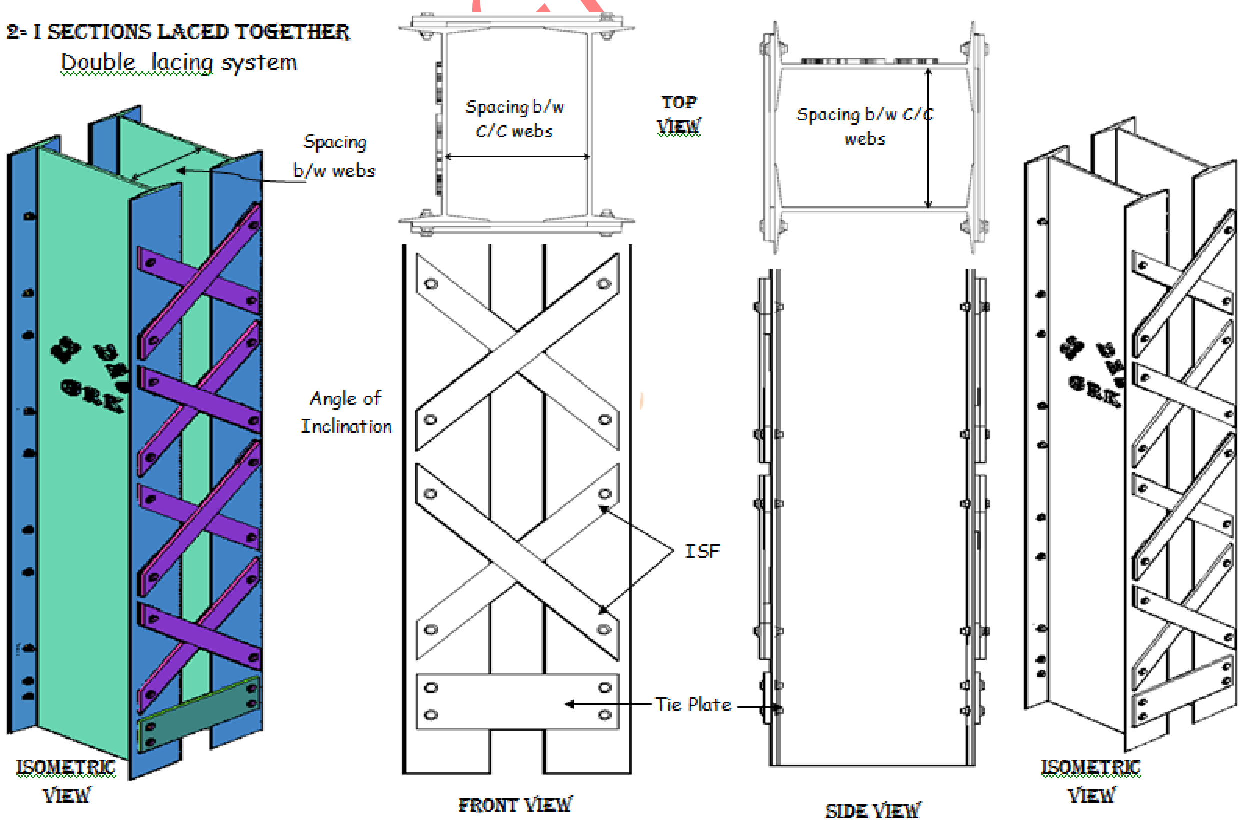
1. Overlap length a) 4t = 4 x 6 = 24mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

Problems on two – I sections laced together


2- I SECTIONS LACED TOGETHER
BY SINGLE LACING SYSTEM
Spacing
b/w webs
Spacing
b/w c/c of
webs
TOP
VIEW
ISF
Angle of
Inclination
Tie Plate
ISOMETRIC
VIEW
FRONT VIEW
SIDE VIEW
ISOMETRIC
VIEW


2- I SECTIONS LACED TOGETHER
Double lacing system
Spacing
b/w webs
Spacing b/w
C/C webs
TOP
VIEW
Spacing b/w C/C
webs
Angle of
Inclination
ISF
Tie Plate
ISOMETRIC
VIEW
FRONT VIEW
SIDE VIEW
ISOMETRIC
VIEW

Problem:

The axial load on a steel column is 2000 KN. The column of length 5 m is effectively held in position at both ends and restrained in direction at one end. Design a suitable built up I-section for the column adopting single lacing and sketch the elevation and plan of the column. Permissible stresses confirm to the specification of IS 800 – 2007.

Solution:

Design of compression member

Axial Load= 2000 KN

Factored load = 1.5 x 2000 = 3000 KN

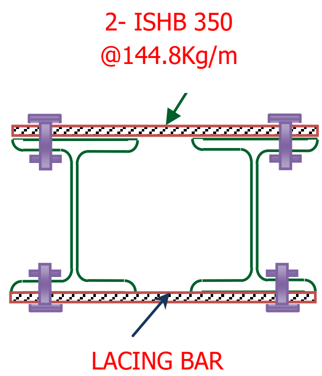
$$\text{Assuming permissible stress}(f_{cd}) = 180\ \text{N/mm}^2$$

$$\text{Area of 2 I-sections} = \frac{\text{Load}}{\sigma_{ac}} = \frac{3000\times10^3}{180}$$

$$= 16666.66\ \text{mm}^2 = 166.67\ \text{cm}^2$$

Try 2-ISHB 350 @ 144.8Kg/m with spacing b/w webs $S = 275$ mm

$$\left[\begin{array}{l}\text{P-48, Cl: 7.6.1.1}\\ \text{Note:- The spacing is choosen in such a way that}\\ \qquad I_{xx} \approx I_{yy} \quad \& \ r_{yy} > r_{xx}\end{array}\right]$$



$a = 184.42\ \text{cm}^2 = 18442\ \text{mm}^2$

$r_{ZZ} = 14.65\ \text{cm} = 146.5\ \text{mm},$

$r_{yy} = 14.71\ \text{cm} = 147.1\ \text{mm}$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section}(\lambda) = 1.05\times\frac{KL}{r}$$

Effective length (Table11, Cl: 7.2.2, P-45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (One end fixed and one end Hinged)

$$KL = 0.8\times L = 0.8\times5 = 4\ \text{m} = 4000\ \text{mm}$$

P-44, Table 10, Buckling curve class about any axis ‘c’.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05\times\frac{4000}{146.5} = 28.67$$

Compressive stress f_{cd} = 215.73 N/mm^2.

$\lambda\lambda$	f_{cd}
20	224
28.67	?
30	211
10	13
8.67	?

$$f_{cd} = 224 - \frac{8.67\times13}{10} = 215.73\ \text{N/mm}^2$$

Load carrying capacity $= f_{cd} \times \text{Area}$

$$= \frac{215.73 \times 18442}{1000} = 3978.50 \text{ KN} > 3000 \text{ KN}$$

Safe

Provide 2 - ISHB 350 @ 144.8 Kg / m.

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5.1)

$\frac{a_1}{r_1} \not> 50$ or $0.7\lambda.7$ builtup section, whichever is less.

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45°$ $\left(40^0 \prec \theta \prec 70^0\right)$

The gauge distance 'g' for ISHB 350 is 140 mm.

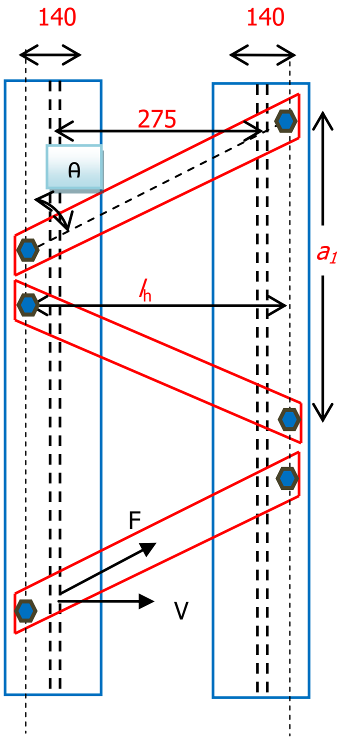
$r_1 = 5.22\text{cm} = 52.2 \text{ mm}$

$\therefore$ Horizontal length of lacing $= 140/2 + 275 + 140/2$

$l_h = 415$ mm

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \text{ x } 415 = 830$ mm



$$\frac{a_1}{r_1} = \frac{830}{52.2} = 15.9 < 50 \text{ and} < 0.7 \text{ x } 28.67 \Rightarrow 20.07$$

The local buckling of the column does not occur,

Hence single lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming dia of bolt = 20 mm

Width of lacing = 3 x dia of bolt = 3 x 20 = 60 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{40}$ of distance of inner bolts

$$\sin\theta = \frac{415}{l_{eff}}$$

$$l_{eff} = \frac{415}{\sin 45} = 586.9\text{mm}$$

$l_{eff} = 586.9$mm For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 586.9 = 14.67 \text{ mm}$$ Say 16 mm

Try a lacing bar of 60 mm width and 16 mm thick

i.e., 60 ISF 16

Note: (P- 50, Cl 7.6.3) Double lacing system $t = \frac{1}{60} \times l_{eff}$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{l_{eff}}{r_{\min}} \ngtr 145$$

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{586.9 \times \sqrt{12}}{16} = 127.1 < 145$$

Safe

$$r_{\min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{\min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

Transverse Shear $(V_t) = 2.5$ % of axial load

$$= \frac{2.5}{100} \times 3000 = 75 \text{ KN}$$

$$\text{Force } (F) = \frac{V_t}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{75}{2 \times \sin 45} = 53.03 \text{KN}$$

Compressive Stress

for $\lambda = 127.1$

Ref Page 42, Table 9(c)

λ	f_{cd}
120	83.7
127.1	?
130	74.3
10	9.4
7.1	?

$$f_{cd} = 83.7 - \frac{9.4 \times 7.1}{10} = 77.03 \text{ N/mm}^2$$

Compressive Force = Compressive Stress × Area of lacing bar $\nless$ F

$$\text{Compressive Force} = \frac{77.03 \times 60 \times 16}{1000} = 73.95 > 53.03 KN$$

Safe

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \nless F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9(60 - 22) \times 16 \times 410}{1.25 \times 1000} = 179.48 \text{KN} > 53.03 \text{ KN} \quad \underline{\text{Safe}}$$

Provide 60 ISF 16 as lacing bar

Connection Details:

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 20 mm.

Dia of hole (d_0) = 20 + 2 = 22 mm

P-75, Cl: 10.3.3

1) Strength of one bolt in Single shear:

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming shank is interfering the shear plane

$n_n = 0 \qquad n_s = 1 \quad , \gamma_{mb} = 1.25, \qquad A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 314.16}{1.25 \times 1000} \right) = 58.04 \text{KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.57$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 20 = 50 mm

3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

k_b = 0.51

t → Min of 1) Thickness of flange of I-section (11.6) and*

2) Thickness of lacing bar (16 mm)

$$V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 11.6^* \times 400}{1.25 \times 1000} = 94.66 \text{ kN}$$

Bolt value (BV) = 58.04 KN.

No of bolts = $\frac{53.03}{58.04} = 0.91$ $\boxed{\text{Say 2 No's (Min) One on each side}}$

Problem:

The axial load on a steel column is 2000 KN. The column of length 5 m is effectively held in position at both ends and restrained in direction at one end. Design a suitable built up I-section for the column adopting single lacing system with site welded connection and sketch the elevation and plan of the column. Permissible stresses confirm to the specification of IS 800 – 2007.

Solution:

Design of compression member

Axial Load= 2000 KN

Factored load = 1.5 x 2000 = 3000 KN

Assuming permissible stress$(f_{cd}) = 180 \text{ N/mm}^2$

$$\text{Area of } 2\,\text{I-sections} = \frac{\text{Load}}{\sigma_{ac}} = \frac{3000 \times 10^3}{180} = 16666.66 \text{ mm}^2 = 166.67 \text{ cm}^2$$

Try 2 - ISHB 350 @ 144.8Kg/m with spacing b/w webs S = 275 mm

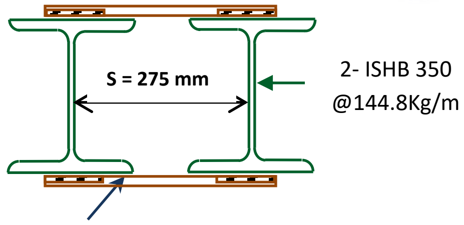
[P - 48, Cl: 7.6.1.1]

Note : - The spacing is choosen in such a way that

$$I_{xx} \approx I_{yy} \quad \& \; r_{yy} > r_{xx}$$

$a = 184.42\ cm^2 = 18442\ mm^2$, $r_{zz} = 14.65\ cm = 146.5\ mm$,

$r_{yy} = 14.71\ cm = 147.1\ mm$



P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \dfrac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (One end fixed and one end Hinged)

$$KL = 0.8 \times L = 0.8 \times 5 = 4\ m = 4000\ mm$$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, (f_{cd-zz})

$$\lambda_{zz} = 1.05 \times \frac{4000}{146.5} = 28.67$$

Compressive stress f_{cd} = 215.73 N/mm^2.

Load carrying capacity $= f_{cd} \times$ Area

$$= \frac{215.73 \times 18442}{1000} = 3978.50\ KN > 3000\ KN$$

Safe

Provide 2 - ISHB 350 @ 144.8 Kg / m.

λ	f_{cd}
20	224
28.67	?
30	211
10	13
8.67	?

$$f_{cd} = 224 - \frac{8.67 \times 13}{10} = 215.73\ N/mm^2$$

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5.1)

$\dfrac{a_1}{r_1} \ngtr 50$ or $0.7\lambda.7$ builtup section, whichever is less.

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

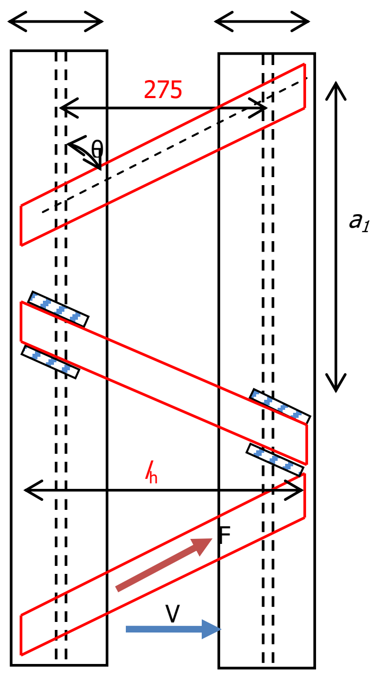
$r_1 = 5.22cm = 52.2mm$

The horizontal distance (l_h):

$l_h = 250/2 + 275 + 250/2 - 250/2 - 50 = 475\,mm$

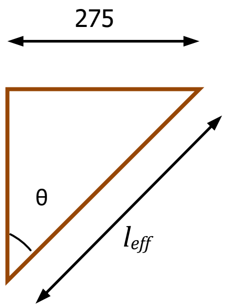
Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \times 475 = 950$ mm



$$\frac{a_1}{r_1} = \frac{950}{52.2} = 18.20 \;< 50 \text{ and} < 0.7 \text{ x } 28.67 \Rightarrow 20.10$$

The local buckling of the column does not occur,
Hence single lacing system can be adopted.



Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming ,

Width of lacing = 60 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{40}$ of distance of inner welds

$$\text{horizontal distance b/w inner welds} = 275 - 2\left(\frac{b_f}{2}\right) = 275 - 250 = 25$$

$$\sin\theta = \frac{25}{l_{eff}}$$

$$l_{eff} = \frac{25}{\sin 45} = 35.36mm$$

$$l_{eff} = 35.36mm$$

For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 35.36 = 0.88 \text{ mm} \quad \text{Say } 6 \text{ mm}$$

Try a lacing bar of 60 mm width and 6 mm thick

i.e., 60 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{l_{eff}}{r_{min}} \not> 145$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{35.36 \times \sqrt{12}}{6} = 20.42 < 145$$

Safe

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

$$\text{Transverse Shear } (V_t) = 2.5\text{ \% of axial load}$$

$$= \frac{2.5}{100} \times 3000 = 75 \text{ KN}$$

$$\text{Force } (F) = \frac{V_t}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

Ref Page 42, Table 9(c)

λ	f_{cd}
20	224
20.42	?
40	211
10	13
0.42	?

$$f_{cd} = 224 - \frac{0.42 \times 13}{10} = 223.45 \text{ N / mm}^2$$

$$F = \frac{75}{2 \times \sin 45} = 53.03 KN$$

Compressive Stress

for $\lambda = 20.42$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \not< F$$

$$\text{Compressive Force} = \frac{223.45 \times 60 \times 6}{1000} = 80.44 > 53.03 \text{ kN} \qquad \underline{\text{Safe}}$$

$\underline{\text{Tensile Force } (P - 32)}$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times 60 \times 6 \times 410}{1.25 \times 1000} = 106.27 KN > 53.03 KN \qquad \underline{\text{Safe}}$$

Provide 60 ISF 6 as lacing bar

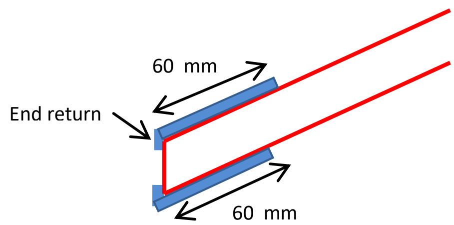
Welded connection:

Max Size of weld S = 6 - 1.5 = 4.5 mm

Say S = 4 mm

Force = Strength of the weld

$f_u = 410 \text{ N/mm}^2$.



$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 535.54 \, l \, \text{N - mm}$$

$$\text{Effective length of weld} = \frac{53.03 \times 10^3}{535.54} = 99.02 \text{ mm} \qquad \text{Say } 100 \text{ mm}$$

Length of weld on each side of flat $= 100/2 = 50$ mm.

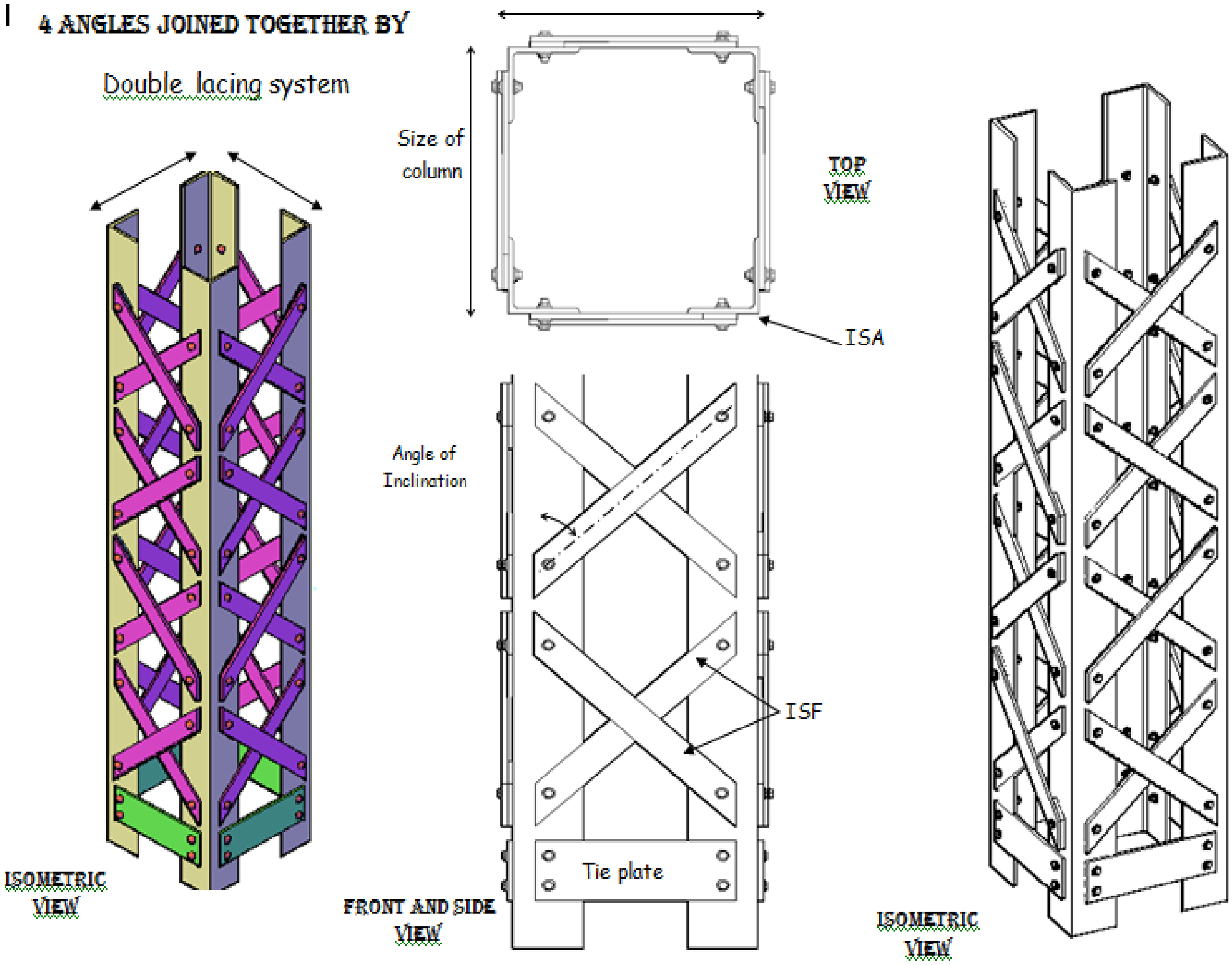
Length of longitudinal weld: It is the max of the following

1. Overlap length a) 4t = 4 x 6 = 24mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

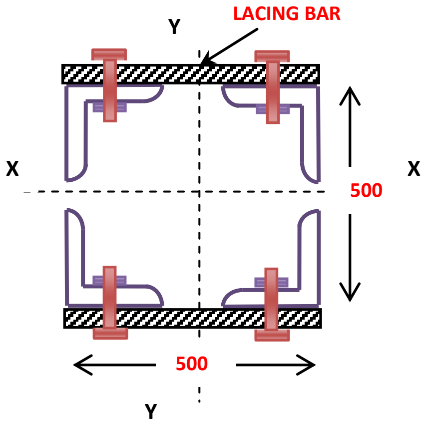
Problem on double lacing system joined together by 4 angles



Problem: 1996 Aug

A mild steel built-up column is to be designed to carry an axial load of 1800 KN. The height of the column is 7 m. The column is considered to be held effectively in position at both the ends and restrained in direction at one end. The column is to be designed using 4-angle section suitably laced together.

Solution:



Design of compression member

Design of compression member

Axial Load= 1800 KN

Factored load = 1.5 x 1800 = 3000 KN

$$\text{Assuming permissible stress}(f_{cd}) = 180\ \text{N/mm}^2$$

$$\text{Area od 4-angles} = \frac{\text{Load}}{f_{cd}} = \frac{2700 \times 10^3}{180} = 15000\text{mm}^2 = 150\text{cm}^2$$

$$\text{Area of each angle} = \frac{150}{4} = 37.5\text{cm}^2$$

Try 4 - ISA $130 \times 130 \times 158$ mm @ 28.9 Kg / m

having the following properties

$$a = 36.81\text{cm}^2 = 3681\text{mm}^2$$

$$I_{ZZ} = I_{yy} = 574.6\text{cm}^4 = 574.6 \times 10^4\ \text{mm}^4,$$

$$C_{xx} = C_{yy} = 3.78\ \text{cm} = 37.8\ \text{mm}$$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section } (\lambda) = 1.05 \times \frac{KL}{r}$$

Effective length (Table11, Cl: 7.2.2, P-45)

End condition : Effectively held in postion at both ends and restained in direction at one end.

$$l_{eff} = 0.8 \times l_{act} = 0.8 \times 7\ \text{m} = 5.6\ \text{m} = 5600\ \text{mm}$$

Let the size of the built-up column with 4 angles be 400 mm x 400 mm

$$I_{ZZ\ \text{of built up column}} = 4\left[574.6 \times 10^4 + 36.81\left(\frac{400}{2} - 37.8\right)^2\right]$$

$$= 410.36 \times 10^6\ \text{mm}^4$$

$$\text{Radius of gyration } r_{ZZ} = r_{yy} = \sqrt{\frac{410.36 \times 10^6}{4 \times 3681}} = 166.94\ \text{mm}$$

$$\text{Slenderness ratio } \lambda = 1.05 \times \frac{5600}{166.94} = 35.22$$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, (f_{cd-zz})

Compressive stress $f_{cd} = 204.21$ N/mm^2.

λ	f_{cd}
30	211
35.22	?
40	198
10	13
5.22	?

$$f_{cd} = 211 - \frac{5.22 \times 13}{10} = 204.21\ \text{N/mm}^2$$

$$\text{Load carrying capacity} = f_{cd} \times \text{Area}$$

$$= \frac{204.21 \times 4 \times 3681}{1000} = 3007\ \text{KN} > 2700\ \text{KN}$$

Safe

Provide 4 - ISA 130 x 130 x 15 mm @ 28.9 Kg / m.

Design Of Lacing:

Assuming Double lacing system:

Check for local buckling of column section (P -50, cl 7.6.5)

$\frac{a_1}{r_1} \ngtr 50$ or $0.7\lambda.7$ builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45° \ (40^0 \prec \theta \prec 70^0)$

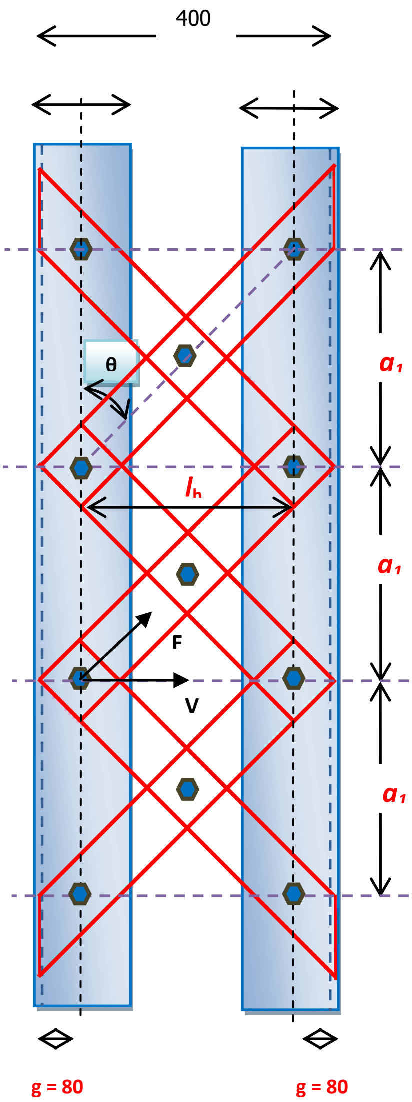
The gauge distance 'g' for ISA 130 x 130 x 15 is 80 mm.

$\therefore$ Horizontal length of lacing $l_h = 400 - 80 - 80 = 240$ mm

Spacing of lacing is c/c diatance of adjacent bolts

$= a_1 = 240$ mm

r'_{min} for single channel ISA 130 x130 x 15 $= 2.53$ cm $= 25.3$ mm



$$\frac{a_1}{r_1} = \frac{240}{25.3} = 9.49 < 50 \text{ and} < 0.7 \text{ x } 35.22 \Rightarrow 24.65$$

The local buckling of the column does not occur, hence Double lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

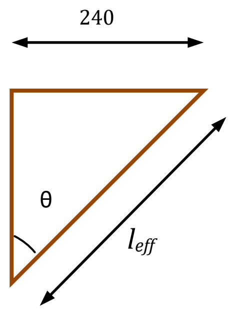
Assuming dia of bolt = 16 mm

Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 0 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$$t \nless \frac{1}{60} \text{ of distance of inner bolts}$$

$$\sin\theta = \frac{240}{l_{eff}}$$

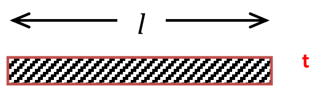


$$l_{eff} = \frac{240}{\sin 45} = 339.41\text{mm}$$

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 339.41 = 5.66 \text{ mm} \qquad \text{Say 8 mm}$$

Try a lacing bar of 60 mm width and 8 mm thick

i.e., 60 ISF 8



Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 339.41 \times \sqrt{12}}{8} = 102.88 < 145 \qquad \underline{\text{Safe}}$$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

Transverse Shear $(V_t) = 2.5\ \%$ of axial load

$$= \frac{2.5}{100} \times 2700 = 67.5 \text{ KN}$$

$$\text{Force } (F) = \frac{V_t}{n \times \sin\theta}$$

$n = 2$ for single lacing system

$n = 4$ for double lacing system

$$F = \frac{67.5}{4 \times \sin 45} = 23.86\text{KN}$$

Ref Page 42 , Table 9(c)

λλ	f_{cd}
100	107
102.88	?
110	94.6
10	12.4
2.88	?

$$f_{cd} = 107 - \frac{2.88 \times 12.4}{10} = 103.43\ \text{N/mm}^2$$

Compressive Force = Compressive Stress × Area of lacing bar $\nless$ F

$$\text{Compressive Force} = \frac{103.43 \times 60 \times 8}{1000} = 49.65 > 23.86\text{KN}$$

Safe

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9A_n f_u}{\gamma_{m1}} \nless F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (60 - 18) \times 8 \times 410}{1.25 \times 1000} = 99.20KN > 23.86\ KN \quad \underline{Safe}$$

Provide 60 ISF 8 as lacing bar

Connection Details: No of bolts $= \frac{F}{BV}$

Dia of bolt = 16 mm.

Dia of hole (d_0) = 16 + 2 =18 mm

P-75, Cl: 10.3.3

1) Strength of one bolt in Double shear :

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Assuming both thread and shank is interfering the shear plane

$n_n = 1$, $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83 mm^2$$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 16^2 = 201.06 mm^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83 + 1 \times 201.06}{1.25 \times 1000}\right) = 66.12KN$$

2) Strength of bolt in Bearing $V_{dpb} = \frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 =27 mm say 35 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.68$

P = 2.5 x 16 = 40 mm Say 50 mm

3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t^* → Min of 1) Thickness of angle (15) and

2) Thickness of lacing bar (8mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 8^* \times 400}{1.25 \times 1000} = 66.56\ KN$$

Bolt value (BV) = 66.12 KN.

No of bolts = $\frac{23.86}{66.12} = 0.35$ Say 2 No's (Min) One on each side

Problem:

The c/s of a 6 m long, pin ended column consists of 4- ISA 100 x 100 x 10 mm suitably connected with lacing bars. The angles face inwards and the outside dimensions of the c/s are 350 x 350 mm.

1. Determine the safe axial compressive load for the column
2. Also design the lacing bars and their connection of angles.