

Sixth Semester B.E. Degree Examination, Aug./Sept.2020
Design of Steel Structural Elements

Time: 3 hrs. Max. Marks: 80

Note: 1. Answer any FIVE full questions, choosing ONE full question from each module.
2. Use of IS-800-2007, SP(6)-1 or steel table is permitted.

Module-1

1 a. What are the advantages and disadvantages of steel structures? **(08 Marks)**
b. Explain limit state of strength and limit state of serviceability. **(08 Marks)**

OR

2 a. State upper bound, lower bound and uniqueness theorems. **(06 Marks)**
b. A propped cantilevers ABCD is loaded as shown in Fig.Q.2(b). Find the collapse load if the beam is of uniform cross section. **(10 Marks)**

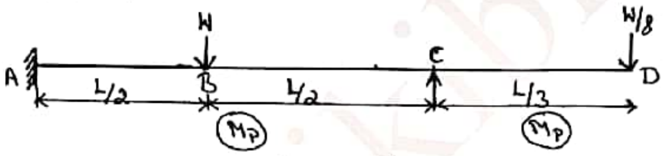


Fig.Q.2(b)

Module-2

3 a. Explain the phenomenon of load transfer in high strength friction grip bolts. **(06 Marks)**
b. A double cover butt joint is used to connect two flats 200 ISF 10 with 8mm cover plates. The two plates are connected by 9 bolts in chain bolting at a pitch of 60mm and edge distance of 40mm. The bolts are arranged in 3 rows with 3 bolts in each row as shown in the Fig.Q.3(b). Determine the strength and efficiency of the joint. The diameter of the bolts used is 20mm. Assume grade of bolt as 4.6. (Assume both thread and shank to interfere the shear plane). **(10 Marks)**

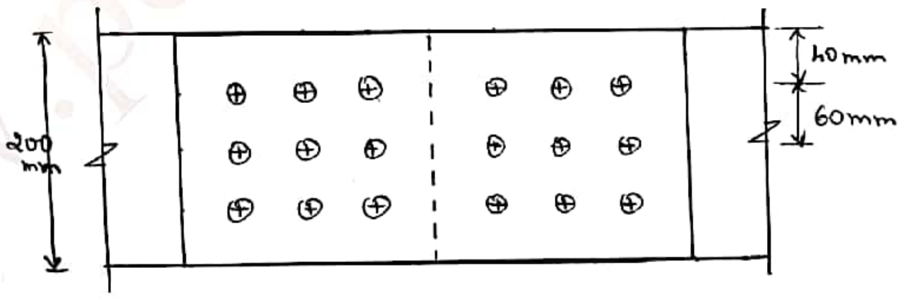


Fig.Q.3(b)

OR

4 a. Write the advantages of welded connections over bolted connections. **(06 Marks)**
b. A tie member consisting of an ISA 80 × 50 × 8mm (Fe 410 grade steel) is welded to a 12mm thick gusset plate at site. Considering the size of weld as 6mm, find the length of weld required to transmit load equal to design strength of the member. **(10 Marks)**

Module-3

5 a. Determine the design strength of ISHB300@ 0.588kN/m, used as stanchion. Effective length of stanchion is 3.0m. **(04 Marks)**

b. Design a compression member of a roof truss to carry an axial load of 150kN. Design the member using a single unequal angle and the corresponding connections to a gusset plate using 20mm diameter bolts of grade 4.6 grade, connecting the longer legs to the gusset plate of 8mm thick. Take effective length of the member as 2.5m. **(12 Marks)**

OR

6 The axial load on a steel column is 2000kN. The column of length 5m is effectively held in position at both ends and restrained in direction at the end. Design a suitable built-up column made of 2 I-sections spaced apart, adopting a single lacing system. Consider permissible stress (f_{cd}) = 180N/mm^2. **(16 Marks)**

Module-4

7 a. Explain: i) Lug angles ii) Shear Lag. **(06 Marks)**

b. Determine the design tensile strength of the plate 200 × 10mm with bolts as shown in Fig.Q.7(b). The yield and ultimate strengths of steel are 250MPa and 410MPa respectively. The diameter of bolt used is 20mm. **(10 Marks)**

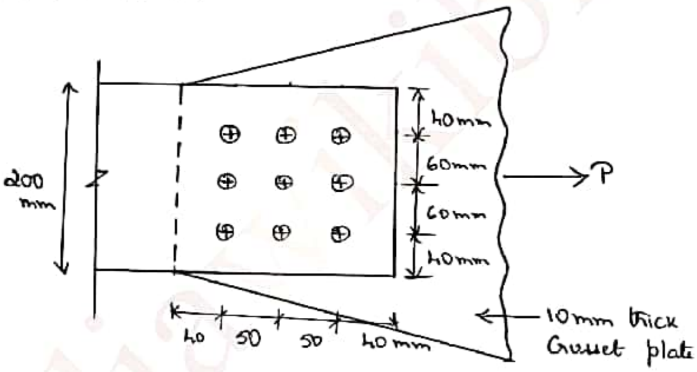


Fig.Q.7(b)

OR

8 a. With the help of neat sketches, explain the different types of column bases. **(06 Marks)**

b. Design a suitable slab base for a column carrying an axial load of 800kN. The section of the column is built up by ISHB250 @ 54.7 kg/m and 2 plates 300mm × 10mm one on each flange of the joint section. The bearing capacity of the soil is 250 kN/m^2. Consider grade of concrete as M20, thickness of weld as 8mm and bearing strength of concrete as 9N/mm^2. **(10 Marks)**

Module-5

9 a. Explain the factors affecting the lateral stability of beams. **(08 Marks)**

b. Calculate the moment and shear capacity of a laterally restrained beam ISLB350 @ 0.486kN/m. **(08 Marks)**

OR

10 a. Write a note on the ways to connect a beam and a column. **(04 Marks)**

b. Check the adequacy of a laterally restrained cantilever beam ISMB 550@ 1.037 kN/m to withstand a moment of 562.5 kN-m and shear force of 225kN, performing all checks necessary for design of a beam. **(12 Marks)**

Design of Steel Structural Elements (15CV62)
(Aug/ Sept 2020)

Solutions -

1.

a. What are the advantages and disadvantages of steel structures?

→ The following are the advantages of steel structures

① Steel members have high strength per unit weight. Therefore a steel member of a small section which has little self weight is able to resist heavy loads.

② Being light, steel members can be conveniently handled and transported. for this reason, prefabricated members can be frequently provided.

③ Properly maintained steel structures have long life.

④ The properties of steel mostly dont change with time. This makes steel most suitable material for structure.

⑤ Steel being a ductile material, does not fall suddenly but gives visible evidence of impending failure by large deflections.

⑥ Additions and alterations can be easily made to steel structures.

⑦ They can be erected at a faster rate.

Disadvantages of Steel Structures -

① Steel structures when placed in exposed conditions are subjected to corrosion. Therefore, they require frequent painting.

② Steel structures need fire proof treatment, which increases the cost.

b. Limit state of strength and limit state of serviceability.

→ Limit state of strength –

- Strength (yielding, buckling)
- Stability against overturning and sway of structure.
- Fracture of structural element due to fatigue.
- Plastic collapse of structure.
- Brittle fracture of structural element.

→ Limit state of serviceability –

- Deflection of structural element.
- Vibration of structure or some part of it.
- Fatigue checks including repairable damage due to fatigue.
- Corrosion on steel.
- Fire hazards in future.

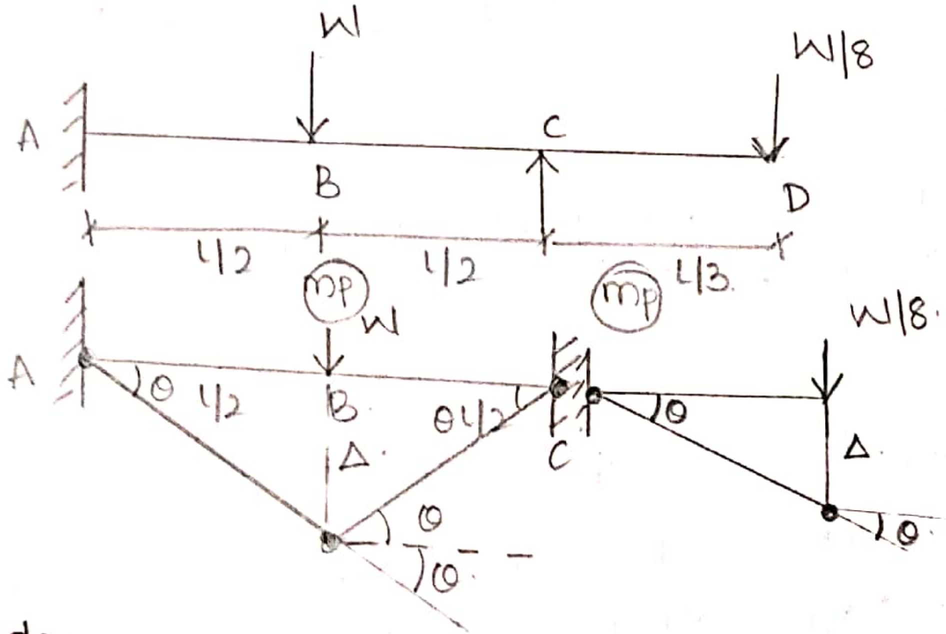
2.

a. Upper bound theorem – This theorem states that load computed from any assumed kinematically admissible mechanism is greater than or equal to the true collapse load. This theorem leads to kinematic or mechanism method of analysis This theorem also called the unsafe theorem satisfies equilibrium and mechanism condition

Lower bound theorem – This theorem states that the load computed from any distribution of BMD's in equilibrium with external loads so that the maximum BM in any member shall not exceed its plastic moment is less than or equal to the true collapse load. This theorem leads to equilibrium or static method of plastic analysis.

Uniqueness theorem – This theorem states that if the load evaluated by static and kinematic theorems is same, then it is the true collapse load. All the three conditions of plastic analysis is satisfied. According to this theorem, there is only one unique solution for a given structure, while there are innumerable possible solutions with other theorems.

b.



Consider span AC.

By kinematic method,

$$W_u \times \theta \times L/2 = M_p\theta + M_p\theta + M_p\theta$$

$$W_u \times \phi \times L/2 = 3M_p\phi$$

$$\boxed{W_u = \frac{6M_p}{L}}$$

Consider span CD,

$$\frac{W_u}{8} \times \theta \times \frac{L}{3} = M_p\theta + M_p\theta$$

$$\frac{W_u \times \phi \times L}{24} = 2M_p\phi \qquad \therefore \boxed{W_u = \frac{48M_p}{L}}$$

3.

a. Preloaded bolts exert a compressive stress on the connected plates. The compression gives rise to high frictional resistence, which enables load to be transferred between the connected parts. When the applied load F exceeds the frictional resistence which is developed between the plates, the plates will slip relative to each other allowing the bolt to act in bearing. Bolts which transfer load by friction are known as High Strength Friction Grip bolts. Controlled tightening of bolts allows the frictional action to be quantified for design.

b.

Data -

> Double cover butt joint, No. of bolts = 9, p = 60mm
e = 40mm., d = 20mm, do = 20+2 = 22mm
Grade of bolt = 4.6, fub = 400 MPa, fu = 410 MPa

> Strength of 1 bolt -

Strength of bolt in shear -

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right] / \gamma_{mb}$$

$$= \frac{400}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi}{4} \times 20^2 + 1 \times \frac{\pi}{4} \times 20^2\right] / 1.25$$

$$\boxed{V_{dsb} = 103.31 \text{ kN}}$$

Strength of bolt in bearing -

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 \times k_b \times d \times t \times f_{ub}}{1.25}$$

k_b is least of

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$\frac{e}{3d_0}$, $\frac{P}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$ or 1.

$\frac{40}{3\times22}$, $\frac{60}{3\times22} - 0.25$, $\frac{400}{410}$ or 1. ie 0.6, 0.66, 0.97, 1

$\therefore K_b = 0.6$.

$\therefore V_{dpb} = \frac{2.5\times0.6\times20\times8\times400}{1.25} = 76.8$ KN.

$\therefore$ Strength of 1 bolt = 76.8 KN.

> Strength of plate –

Strength due to yielding of gross section –

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{200\times10\times250}{1.1} = 454.54$ KN

Strength due to rupture of critical section –

$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$ $\quad A_n = (B - nd_0)\times t$

$= (200 - 3\times22)\times10$

$= 1340\ mm^2$

$\therefore T_{dn} = \frac{0.9\times1340\times410}{1.25}$

$T_{dn} = 395.56$ KN.

$\therefore$ Strength of plate = 395.56 KN

$\therefore$ Efficiency = $\frac{N\times \text{Strength of one bolt}}{\text{Strength of plate}}\times100$

$= \left(\frac{9\times76.8}{395.56}\right)\times100 = 174.73\%$.

4.

a. Advantages of welded connections –

> Permanent joint – It is a permanent joint. It does not allow dismantling of jointed parts without rupturing them.

> Leak proof joints – These joint components can be held tightly without leakage.

③ 100% strength of joint - The strength of welded joint is very high than other joints.

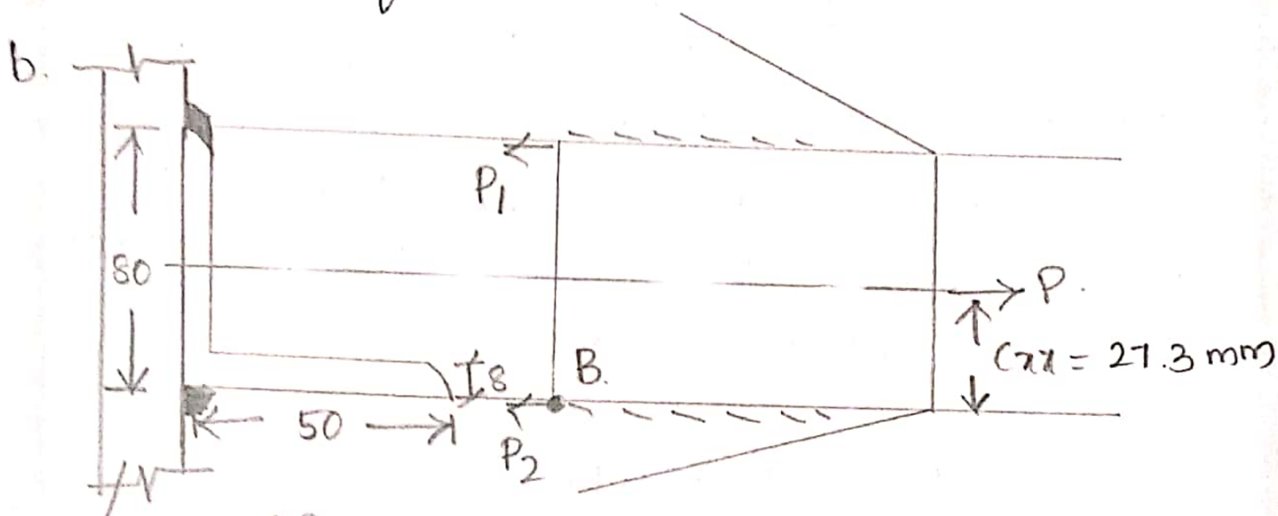
④ Lighter assembly - In welded joint no additional components are used. So they have lower weight than bolted joint.

⑤ High load carrying capacity -

⑥ Dissimilar metal joints.

⑦ Less time consuming - Welding process requires less time for connection.

b.



Size of weld = 6mm, $A_g = 978\ mm^2$ (from steel table)

① Strength of section due to yielding of gross sectn

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{978 \times 250}{1.1} = \underline{\underline{222.27\ kN}}.$$

② $\Sigma M_B = 0$.

$\Rightarrow P \times 27.3 - P_1 \times 80 = 0$

$\Rightarrow 222.27 \times 27.3 - P_1 \times 80 = 0$.

$\Rightarrow P_1 = 75.84$ kN.

$\Rightarrow P_1 + P_2 = P$

$\Rightarrow P_2 = P - P_1 = 221.27 - 75.84 = 146.43$ kN

(3) Design strength of fillet weld -

$$f_{wd} = \frac{f_u}{\gamma_{mw} \times \sqrt{3}} = \frac{410}{1.25 \times \sqrt{3}} = 189.44 \text{ N/mm}^2$$

(4) Design strength per mm length of weld -

$$= f_{wd} \times L_w \times t_t$$

$$= 189.44 \times 1 \times 0.7 \times 6.$$

$$= 795.48 \text{ N/mm}$$

(5) Lengths of weld -

$$L_1 = \frac{P_1}{795.48} = \frac{75.84 \times 10^3}{795.48} = 95.33 \text{mm}$$

$$L_2 = \frac{P_2}{795.48} = \frac{146.43 \times 10^3}{795.48} = 184.07 \text{ mm}.$$

5.

a. Section - ISHB 300 @ 0.588 KN/m.

$L_{eff} = 3$m.

Assuming both the ends are hinged.

> S/n details -

$h = 300$mm, $b_f = 250$mm, $t_f = 10.6$mm, $t_w = 7.6$mm

$r_{zz} = 12.95$ cm $= 129.5$mm $\quad r_{yy} = 5.41$ cm $= 54.1$mm

> Determining the buckling class -

$\frac{h}{b_f} = \frac{300}{250} = 1.2$ and $t_f = 10.6$mm

$r_{zz} > r_{yy}$ $\therefore$ Buckling class is 'c'.

> For buckling class 'c'; $\alpha = 0.49$.

> $$f_{cc} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{1 \times 3000}{54.1}\right)^2} = 641.92 \frac{N}{mm^2}$$

> $$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624.$$

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

$= 0.5[1+0.49(0.624-0.2)+0.624^2]$

$\phi = 0.798$

> $f_{cd} = \dfrac{\dfrac{f_y}{\gamma_{m0}}}{\phi+(\phi^2-\lambda^2)^{0.5}} = \dfrac{\dfrac{250}{1.1}}{0.798+(0.798^2-0.624^2)^{0.5}}$

$f_{cd} = 175.44$

> $P_d = A_c f_{cd}$

$= 7485 \times 175.44$

$P_d = \underline{\underline{1313\ KN}}.$

b. Given axial load = 150 KN

Factored load = $1.5 \times 150 = 225$ KN

d = 20mm $d_o = 20+2 = 22$mm

Grade 4.6 $f_{ub} = 400$ MPa.

$L_{eff} = 2.5$m.

> Design stress in compression member -

$f_{cd} = 0.4f_y - 0.6f_y.$

$= 0.4 \times 250 = 100\ N/mm^2 - 0.6 \times 250$

$\therefore f_{cd} = 100\ N/mm^2$. $= 150\ N/mm^2$

> Effective area $A_c = \dfrac{P_u}{f_{cd}} = \dfrac{225 \times 10^3}{100} = 2250\ mm^2$

Increase area by 25% to 40%.

$\frac{25}{100} \times 2250 + 2250 = 2812.5\ mm^2$

$\frac{40}{100} \times 2250 + 2250 = 3150\ mm^2$

$\therefore A_g = 2812.5\ mm^2$.

Adopt ISA 150X115X12

$A = 3038\ mm^2$

$r_{xx} = r_{yy} = 33.7$ mm. $r_{min} = 24.3$mm

> Connection details –

Assume 8mm thick gusset plate.

• Strength of bolt in shear –

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}} \times \frac{1}{\gamma_{mb}} \left(n_n A_{nb} + n_s A_{sb} \right) \quad \cancel{n_s} \; 0.$$

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

$$= 45.27 \text{ kN}.$$

$\therefore$ No. of bolts reqd $= \frac{225}{45.27} = 5$ nos.

> Effective length –

$L_e = 0.85 \times L = 0.85 \times 2500 = 2125$ mm

> Slenderness ratio –

$$\lambda = \frac{KL}{r_{min}} = \frac{2125}{24.3} = 87.44.$$

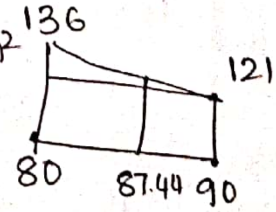
> Calculating buckling class – As per table no 10 of IS 800-2007, for angle section, buckling class is 'c'. $\therefore \alpha = 0.49$

> Calculating fcd –

$$\frac{KL}{r_{min}} = 87.44.$$

As per table 9(c), fcd =

$$\frac{15}{10} = \frac{?}{2.56} \qquad \therefore f_{cd} = 124.84 \text{ N/mm}^2$$



> Calculation of Pd

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

$$= 3038 \times 124.84$$

$$= 379.26 \text{ kN} > 225 \text{ kN}$$

Hence section is safe.

6.

a. Given data -

$L = 5m$, factored load $= 1.5 \times 2000 = 3000 kN$

$f_{cd} = 180 \ N/mm^2$

The column is restrained at both ends.

Assume $f_y = 250 MPa$.

> Design of lacing column -

Area reqd $= A_c = \frac{P_u}{f_{cd}} = \frac{3000 \times 10^3}{180} = 16666.67 \ mm^2$.

> Selection of trial section -

Area reqd for one column $= \frac{16666.67}{2} = 8333.3 \ mm^2$

Select 2 I sections ISHB 350 @ 72.4 N/m.

Area $= 9221 \ mm^2$. $\therefore A_g = 2 \times 9221 = 18442 \ mm^2$

Section properties -

$h = 350$, $b = 250mm$ $t_f = 10.6mm$ $t_w = 10.1 mm$.

$r_{xx} = 14.65 = 146.5mm$ $r_{yy} = 52.2mm$.

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

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

> Computing the effective length -

$L_e = K \times L = 1 \times L$

$= 1 \times 5000 = 5000 mm$.

Actual $\frac{KL}{r} = \frac{5000}{146.5} = 34.12$

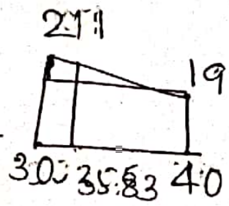
Increase actual $\frac{KL}{r}$ by 1.05

$\therefore \lambda = \frac{KL}{r} \times 1.05 = 34.12 \times 1.05 = 35.83$

> For any built up member, buckling class is 'c'

$\therefore$ For $\frac{KL}{r} = 35.83$

$f_{cd} = 203.42 \ N/mm^2$



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> Calculation of design load (P_d)

$P_d = A \times f_{cd}$

$= \cancel{8333.3}\ 18442 \times 203.42$

$P_d = 3751.4 \text{ KN} > 3000 \text{ KN}$

Hence section is safe.

> Design of lacing flat -

For 2-I sections,

$I_{xx} = 2 \times 19802.28 \times 10^4$

$I_{xx} = 39604.56 \times 10^4 \text{ mm}^4$

$I_{yy} = 2\left[2510.5 \times 10^4 + 9221 \times \left(\frac{d}{2} + \frac{10.1}{2}\right)^2\right]$

Equating $I_{xx} = I_{yy}$.

$39604.56 \times 10^4 = 2\left[2510.5 \times 10^4 + 9221\left(\frac{d^2}{4} + 25.50 + 2 \times \frac{d}{2} \times 5.05\right)\right]$

$\Rightarrow 39604.56 \times 10^4 = 2\left[2510.5 \times 10^4 + 2305.2\,d^2 + 235135.5 + 46566.05\,d\right]$

$\Rightarrow 19802.28 \times 10^4 = 2305.2 d^2 + 46566.05 d + 25.3 \times 10^6$.

$\Rightarrow 2305.2 d^2 + 46566.05 d - 172.72 \times 10^6 = 0$

$\Rightarrow d = 263 \text{ mm}$

Adopt $d = 250 \text{mm}$.

> Design of lacings -

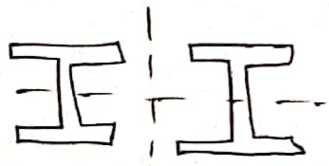
Provide lacings at 45° to horizontal -

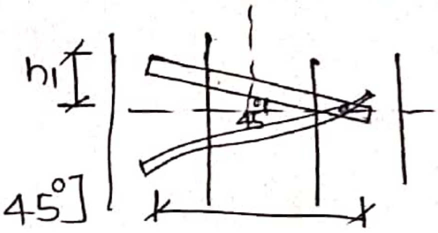
Horizontal spacing

$= d + \frac{10.1}{2} + \frac{10.1}{2}$

$= 250 + 10.1$

$= 260.1 \text{ mm}$





Vertical spacing

$= 2h_1 = 2[260.1 \tan 45°]$

$= 520.2mm$

> Radius of gyration of one I section -

$r_{min} = 52.2mm$

$\frac{KL}{r} = \frac{520.2}{52.2} = 9.96 < 50$ Hence ok.

> Designed transreverse shear -

$V_t = 2.5\%$ of axial load

$= \frac{2.5}{100} \times 3000 = 75 KN$. Shear resisted by each $= \frac{75}{2} = 37.5 KN$

Lacing length $= \frac{260.1}{\cos 45} = 367.83mm$

Minimum thickness of lacing flat $= \frac{1}{40} \times L$

$= \frac{1}{40} \times 367.83$

$= 9.19 mm.$

Provide 10mm flat lacing.

Providing 20mm dia bolts, min width $= 3 \times d$

$= 3 \times 20$

$= 60mm$

Provide 60 ISF 10mm lacing flats.

Area $= 60 \times 10 = 600 mm^2$.

$\therefore r_{min} = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{60 \times 10^3}{12}}{600}} = 2.88 mm$

$\frac{KL}{r} = \frac{367.83}{2.88} = 127.71 < 145$ Hence section is safe.

> Design of bolted connections -

$d = 20mm$

$d_0 = 20+2 = 22mm$

$f_{ub} = 400 MPa$

$t = 10mm.$

> Strength of bolts-

Assuming it in single shear - $\nearrow 0$

$$V_{dsb} = \frac{1}{\gamma_{mb}} \frac{f_{ub}}{\sqrt{3}} [n_n A_{nb} + \cancel{n_s A_{sb}}]$$

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} [1 \times 0.78 \times \frac{\pi}{4} \times 20^2]$$

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

Strength due to bearing-

$$V_{dpb} = \frac{1}{\gamma_{mb}} \times 2.5 \times k_b \times f_{ub} \times d \times t$$

k_b is min of

$e = 1.5 d_0 = 33\text{mm} \approx 40\text{mm}$

$P = 2.5 d = 50\text{mm} \approx 60\text{mm}$

$$\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606$$

$$\frac{P}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \quad \text{or } 1$$

$$\therefore k_b = 0.606.$$

$$V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 400 \times 20 \times 10 = 96.96 \text{ kN}$$

$\therefore$ Bolt value $= 45.27$ KN

$\therefore$ No. of bolts $= \frac{75/2}{45.27} = 0.828 \approx 1$

7.

a. <u>Lug angles</u> - It is a small piece of angle used to connect outstanding legs of the members to the gusset plate. The purpose of lug angles is to reduce the length of connection to the gusset plate and to reduce shear lag effect. If lug angle is used then the unconnected length of main angle behave like a connected leg and entire cross section area of the angle become effective in resisting tension. So if lug angle is used, then efficiency of the tension member increases because it reduces shear lag effect.

Shear lag - The non uniform stress distribution that occurs in a tension member adjacent to a connection in which all elements of the cross section are not directly connected is reffered as shear lag effect. This effect reduces the design strength of member because the entire cross section is not fully effective at critical section location.

b.

$e = 40mm$ $P = 60mm$, $f_y = 250 MPa$ $f_u = 410 MPa$

$d = 20mm$ $d_o = 20+2 = 22mm$

Design strength of plate due to yielding of gross section -

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{200 \times 10 \times 250}{1.1} = 454.54 KN$$

Design strength of plate due to rupture of critical section -

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$A_n = (B - n d_o) \times t = (200 - 3 \times 22) \times 10 = 1340 mm$$

$$\therefore T_{dn} = \frac{0.9 \times 1340 \times 410}{1.25} = 395.56 KN.$$

Design strength due to block shear failure -

$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$$ OR

$$T_{db} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

$A_{vg} = (40 + 2 \times 50) \times 10 = 1400\ mm^2$

$A_{vn} = (40 + 2 \times 50 - 2.5 \times 22) \times 10 = 850\ mm^2$

$A_{tg} = 2 \times 60 \times 10 = 1200\ mm^2$

$A_{tn} = (2 \times 60 - 2 \times 22) \times 10 = 760\ mm^2$

$$\therefore T_{db} = \left[\frac{1400 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 760 \times 410}{1.25}\right] = 408\ KN$$

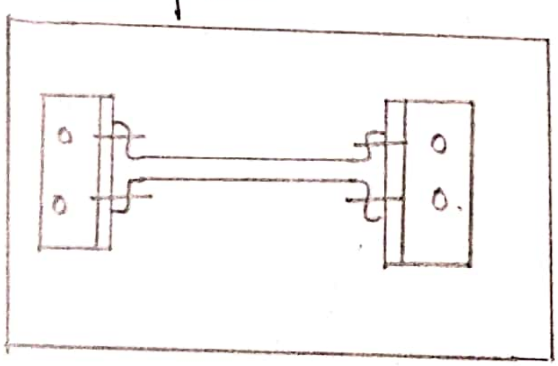
Or

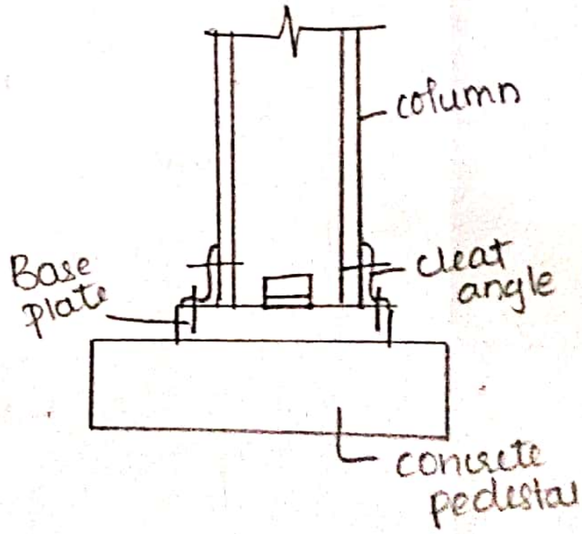
$$T_{db} = \left[\frac{0.9 \times 850 \times 410}{\sqrt{3} \times 1.25} + \frac{1200 \times 250}{1.1}\right] = 417.13\ KN$$

$\therefore$ Strength of plate = 395.56 KN.

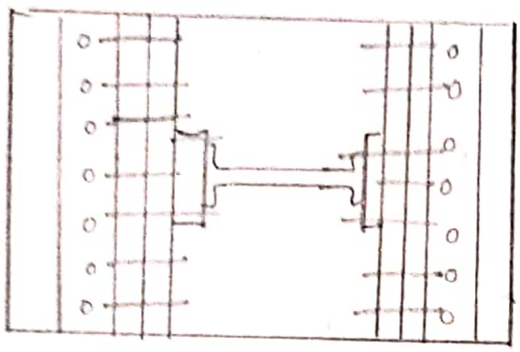
8.

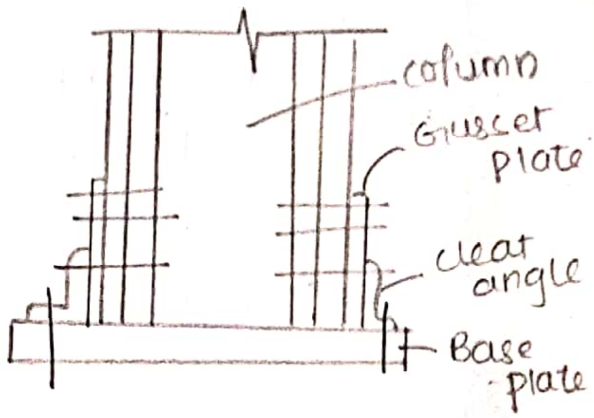
a. <u>Slab base</u> - Slab bases are used where the columns have independent concrete pedestals. A thick base plate and two cleat angles connecting the flanges of the column to the base plate. The ends of the column and also the base plate should be mechanized so that the column load is wholely transferred to the base plate.





b. <u>Gussetted base</u> - Gussetted bases are provided for columns carrying heavier loads requiring large base plates. A gusseted base consists of a base of reduced thickness and two gusset plates are attached one to each flange of column. The gusset plates, cleat angles and fastenings with bearing area of shank shall be sufficient to take all loads.





b. Axial load = 800 kN

factored load = 1.5 X 800 = 1200 kN.

ISHB 250 @ 54.7 kg/m.

Bearing capacity of soil = 250 kN/m^2.

Grade of concrete = M20.

thickness of weld = 8mm

Bearing strength of concrete = 9 N/mm^2.

> Sectional properties of column –

$A = 6971\ mm^2$ $h = 250mm$ $b = 250mm$ $t_f = 9.7mm$ $t_w = 8.8mm$

> Area of base plate $= \frac{P_u}{9} = \frac{1200 \times 10^3}{9} = 133333.3\ mm^2$

Provide 350 mm X 400 mm size plate.

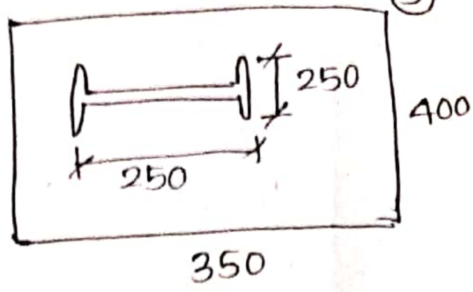
Area provided = 350 X 400 = 140000 mm^2.

> Pressure intensity $w = \frac{1200 \times 10^3}{140000} = 8.57\ N/mm^2$

> Projections

$$a = \frac{350-250}{2} = 50\ mm$$

$$b = \frac{400-250}{2} = 75\ mm.$$



> Thickness of base -

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

$$= \sqrt{\frac{2.5 \times 8.57 (50^2 - 0.3 \times 75^2) \times 1.1}{250}} > 9.7$$

$$= 8.75 \ngtr 9.7$$

∴ provide 12mm thick base plate.

> Connecting 350 x 400 x 12 mm plate to concrete pedestal Use 4 bolts of 20mm diameter 300mm long to anchor the plate.

> Design of welds -

Total length of weld $= 250 + 250 + 250 + 250 - 8.8 - 8.8 + 250 + 250 - 4 \times 9.7$

$= 1443.6\ mm.$

Available length after deducting end returns

$= 1443.6 - 2 \times 8 \times 12$

$= 1251.6\ mm.$

Strength of weld $= \dfrac{L_w \times t \times f_{wn}}{\sqrt{3}\, \gamma_{mw}} = \dfrac{L_w \times 8 \times 410}{\sqrt{3} \times 1.25}$

$= 1514\ L_w.$

∴ $1514\ L_w = 1200 \times 10^3.$

∴ $L_w = 792\ mm < 1251.6$

Hence design is safe.

9.

a. Factors affecting lateral stability of beams are -

> The slenderness of the member between adequate lateral restraints

> The shape of cross section

> The variation of moment along the beam.

> the form of end restraints provided.

> the manner in which the load is applied.

b.

ISLB 350 @ 0.486 KN/m.

$f_y = 250$ MPa.

From steel table, $A = 6301\ mm^2$ $h = 350mm$

$b = 165mm$ $Z_p = 851.11 \times 10^3\ mm^4$ $t_w = 7.4mm$

Assuming plastic section,

① $$M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}}$$

$$= \frac{1 \times 851.11 \times 10^3 \times 250}{1.1} = 193.43 \times 10^6\ mm^4.$$

② Shear capacity of section -

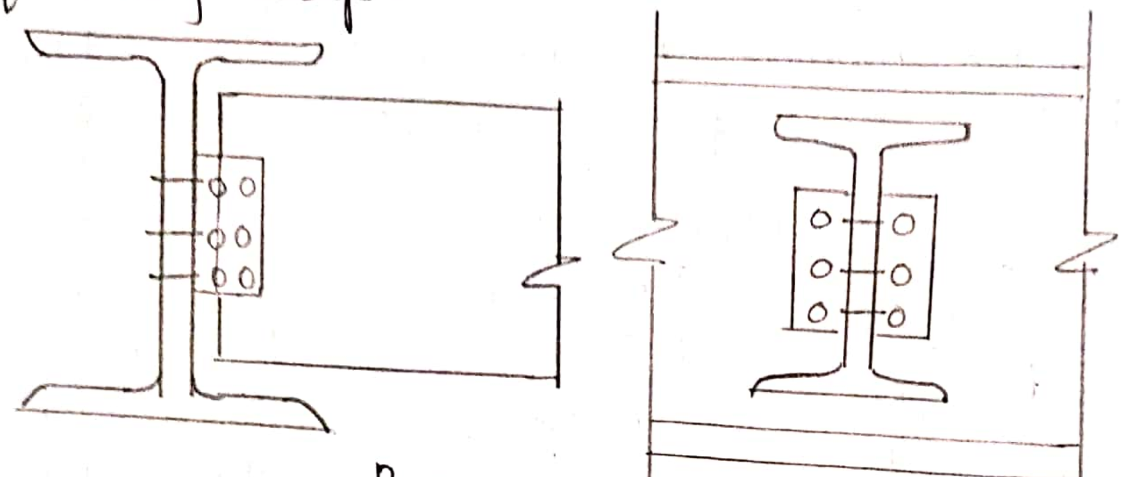
$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times \text{shear area} = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 350 \times 7.4$$

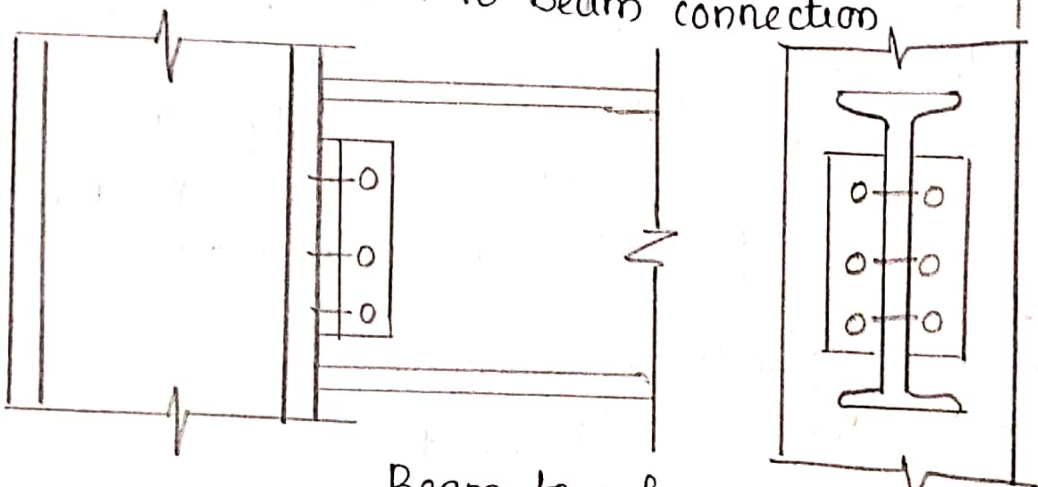
$$= 339.84\ KN.$$

10.

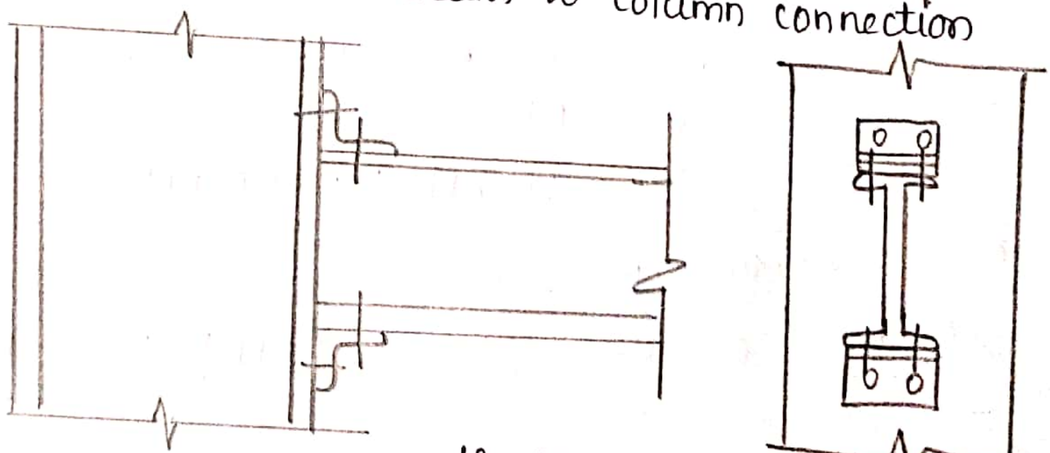
a. The beam and the column can be connected in the following ways-



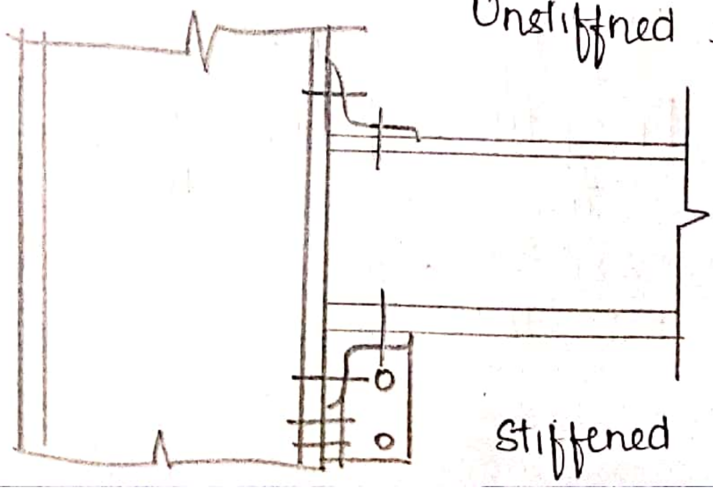
Beam to Beam connection



Beam to column connection



Unstiffned seated connection



Stiffened seated connection

b. Cantilever beam ISMB 550 @ 1.031 kN/m

$M = 562.5$ kNm

$V = 225$ kN.

Factored moment $M = 1.5 \times 562.5 = 843.75 \times 10^6$ Nmm

Factored shear force $V = 1.5 \times 225 = 337.5 \times 10^3$ N.

> Section classification –

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.$$

$$\frac{b}{t_f} = \frac{190}{19.3} = 9.84 < 15.7\epsilon$$

$$\frac{d}{t_w} = \frac{550 - 2(19.3+18)}{11.2}$$

$$= 42.4 < 84\epsilon.$$

Sectional details –

$h = 550$ mm

$b = 190$ mm

$t_f = 19.3$ mm

$t_w = 11.2$ mm

$I_{xx} = 64893.6 \times 10^4$ mm^4

$I_{yy} = 1833.8 \times 10^4$ mm^4

$r_1 = 18$ mm

∴ It is classified as semi compact (class 3) section.

> Shear capacity of section –

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 550 \times 11.2$$

$$= 808.29 \text{ kN} > 337.5 \text{ kN}.$$

Hence section is safe

$0.6V_d = 0.6 \times 808.29 = 484$ kN > 337.5 kN.

> Moment capacity of section –

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \quad \left[\text{since } \frac{d}{t_w} < 67\epsilon \text{ \& } V < 0.6V_d\right]$$

$$\beta_b = \frac{Z_e}{Z_p} = \frac{2359.8 \times 10^3}{2711.98 \times 10^3} = 0.87$$

$$M_d = 0.87 \times 2711.98 \times 10^3 \times 250 \times \frac{1}{1.1}$$

$$= 536.23 \times 10^6 \text{ Nmm } \cancel{>} < 843.75 \times 10^6.$$

Hence the section fails to resist moment.