CBCS SCHEME

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18CV61

Sixth Semester B.E. Degree Examination, July/August 2022
Design of Steel Structural Elements

Time: 3 hrs. Max. Marks: 100

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

Module-1

1 a. What are the advantages and disadvantages of steel structures? (10 Marks)
b. Distinguish between working stress design and limit state design of steel structures. (10 Marks)

OR

2 a. Calculate the shape factor of triangle. (10 Marks)
b. Calculate 'M_p' for the continuous beam shown in Fig.Q2(b). Take load factor 1.5.

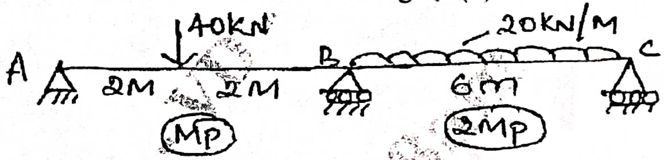

Fig.Q2(b) (10 Marks)

Module-2

3 a. Explain the failure modes of bolted connection. (10 Marks)
b. Design a bolted connection for a lap joint of plate thickness 16 mm and 12 mm to carry a factored load of 160 kN. Use M_{16} and 4.6 grade bolts. (10 Marks)

OR

4 a. What are the advantages and disadvantages of welded connection? (10 Marks)
b. A tie member of Truss consisting of angle section ISA 65 × 65 × 6 mm of Fe 410 grade is welded to 8 mm gusset plate. Design a weld to transmit a factored load of 150 kN. (10 Marks)

Module-3

5 a. Explain the failure modes of axial loaded column. (10 Marks)
b. Determine the design compressive strength of ISHB300@576.8 N/m, Length of column is 3.5 m and both ends are pinned. (10 Marks)

OR

6 Design a single angle discontinuous strut to carry a factored load of 65 kN. The length of strut is 3m, between intersection. It is connected to 12 mm thick gusset plate by 20 mm diameter, 4.6 grade bolts. (20 Marks)

Module-4

7 a. Explain the factors effecting strength of tension members. (10 Marks)
b. Design a tension member to carry factored load of 400 kN connected to shorter leg back to back. Length of member is 3m. (10 Marks)

Important Note : 1. On completing your answers, compulsorily draw diagonal cross lines on the remaining blank pages.
2. Any revealing of identification, appeal to evaluator and /or equations written eg, 42+8 = 50, will be treated as malpractice.

OR

8 a. Explain Lug angles and column splices. **(10 Marks)**

b. Design slab base for a column made of ISHB250@536 N/m to carry axial working load of 520 kN. The grade of concrete is M_{20} and grade of steel Fe 410. **(10 Marks)**

Module-5

9 a. Explain the factors effecting lateral stability of beams. **(10 Marks)**

b. Calculate the load carrying capacity of laterally restrained simply supported beam with ISMB500@86.9 kg/m section for an effective span of 5m. **(10 Marks)**

OR

10 Design a steel beam section for supporting hall for the following data:

Clear span = 6.5 m

End bearing = 200 mm

c/c spacing of beams = 3 m

Live load on beams = 12 kN/m^2

Dead load = 3 kN/m^2 **(20 Marks)**

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AY: 2020-21

Department: Civil Engg | IA Test No:
Subject with Sub. Code: Design Of Steel Structural Elem 18CV61 | Semester / Division: VI
Name of Faculty: Prof. Parvati B. Oni

Q.No.	Solution and Scheme	Marks
1. a.	Structural steel sections are usually used for construction of buildings, transmission lines etc. They also find in manufacturing of automotive vehicles, ships etc. Steel exhibits desirable physical properties with which it is one of the most versatile structural material in use. Its great strength, uniformity light weight, easy of use and many other desirable properties makes it a material of choice. Elasticity – Steels follow hook's law very accurately. Ductility – Steel can withstand extensive deformation without failure under high tensile pressure. Toughness – Steel has both strength and ductility. Additions to existing structures – New bays or even entire new wings can be added to existing frame buildings and steel bridges can be easily widened.	

Disadvantages of Steel Structures -

> Maintainence cost - Steel structures are susecptible to corrosion when exposed to air.

> Fire proofing cost - steel is an incombustible material however its strength is reduced tremendously at high temperature due to common fires.

> Fatigue - The strength of structural steel members can be reduced if the member is subjected to cyclic loading.

> Brittle fracture - Under certain conditions steel loses its ductility and brittle fracture may occur at places of stress concentration.

b.

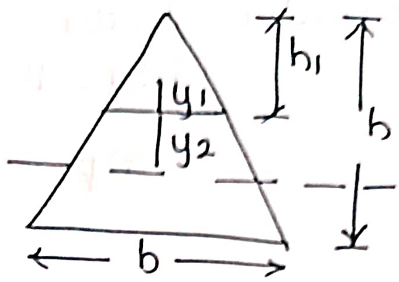
Working stress method	Limit state method.
1. It is easy for calculation	1. It requires more calculation so it makes some difficulty to design
2. Material strengths are not fully utilised in designing the member of structure	2. Material strengths are fully utilised in designing the member of structure.
3. It does not give idea about the excess load which a structure can carry beyond the working load without collapse.	3. It gives idea about excess load which a structure can carry beyond the working load without collapse.
4. In this method, concrete and steel are considered elastic and strain curves is linear for both	4. This method is based on the actual stress strain curves for steel and concrete.
5. No factor of safety is used for loads.	5. Design loads are obtained by multiplying partial safety factors of load to the working loads.

2.

a. Consider a triangle of base b and height h.

We have shape factor $= \frac{Z_p}{Z_y} = \frac{Z_p}{Z_e}$

$Z_p = \frac{A}{2}[y_1 + y_2]$ $\quad Z_e = \frac{I}{y_{max}}$



From equal area section,

$\frac{1}{2} \times b_1 \times h_1 = \left(\frac{1}{2} \times b \times h\right) \times \frac{1}{2}$

$b_1 = \frac{h}{h_1} \times \frac{b}{2}$ — ①

By similar triangle, $\frac{b_1}{b} = \frac{h_1}{h}$.

$\Rightarrow \frac{h}{h_1} \times \frac{\cancel{b}}{2} = \cancel{b}\left[\frac{h_1}{h}\right] \Rightarrow h_1^2 = \frac{h^2}{2} \Rightarrow \boxed{h_1 = \frac{h}{\sqrt{2}}}$

$\therefore b_1 = \frac{h}{h_1} \times \frac{b}{2} = \frac{h}{\frac{h}{\sqrt{2}}} \times \frac{b}{2} = \frac{b \times \sqrt{2}}{2} = \frac{b}{\sqrt{2}}$

$\therefore Z_p = \frac{A}{2}[y_1 + y_2]$ $\quad A = \frac{1}{2} \times b \times h$ $\quad y_1 = \frac{h_1}{3} = \frac{h}{3\sqrt{2}} = 0.235h$

$y_2 = \left(\frac{2b_1 + b}{b_1 + b}\right)\left(\frac{h - h_1}{3}\right) = \left(\frac{2 \times \frac{b}{\sqrt{2}} + b}{\frac{b}{\sqrt{2}} + b}\right)\left(\frac{h - \frac{h}{\sqrt{2}}}{3}\right)$

$= \left(\frac{2.414b}{1.707b}\right)(0.097h)$

$\therefore y_2 = 0.138h$

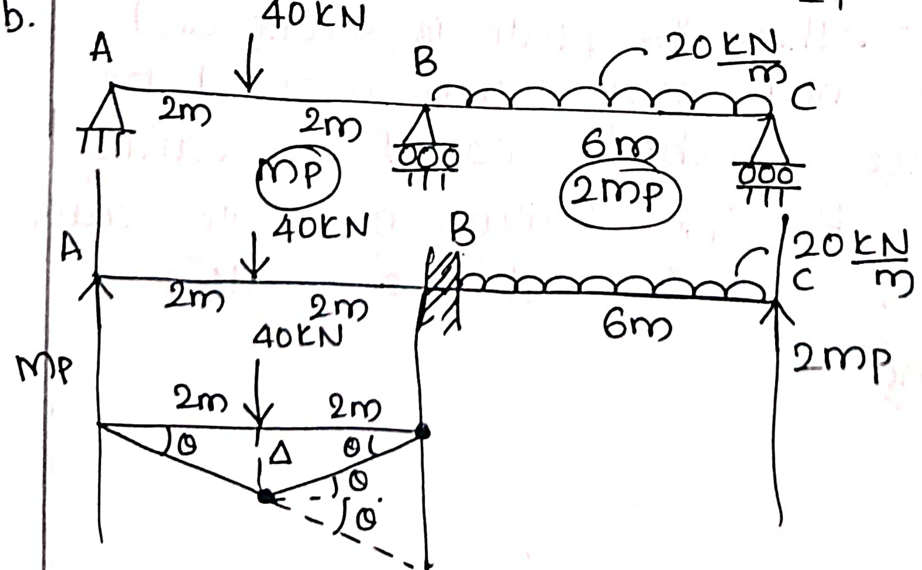
$\therefore Z_p = \frac{1}{2} \times b \times h \times \frac{1}{2}(0.235h + 0.138h) = 0.093bh^2$

$Z_e = \frac{I}{y_{max}} = \frac{bh^3}{36}$ $\quad y_{max} = \frac{2h}{3}$

$\therefore Z_e = \frac{bh^3}{36} \times \frac{3}{2h} = \frac{bh^2}{24}$

$\therefore$ Shape factor $= \frac{Z_p}{Z_e} = \frac{0.093bh^2}{\frac{bh^2}{24}} = 2.23.$

b.



$1.5 \times 40 \times 2 \times \theta = M_p\theta + M_p\theta + M_p\theta$ Consider beam AB

$80\phi = 3M_p\phi$

$\therefore M_p = 26.67 \text{ kNm} \times 1.5 = 40 \text{ kNm}$

Consider beam BC.

$W_u = 11.59 \frac{M_p}{L}$

$\therefore M_p = \frac{W_u \times L}{11.59} = \left(\frac{1.5 \times 20 \times 6 \times 6}{11.59}\right) 2 = 186.36 \text{ kNm}$

3.

a. The bolted joints may fail in one of the failure modes -

① Shear failure of bolt.
② Tension failure mode
③ Bearing failure mode

① Shear failure of bolt - This type of failure occurs when factored shear force generated on bolt due to applied load exceeds the shear strength capacity of bolt. When the strengths of plate is less than the shearing strength of bolt, the tearing failure of plate may occur. To avoid this type of failure minimum edge distance shall be provided.

② Tension failure mode - The bolt subjected to tensile force fails if applied factored force is greater than tensile strength of bolt. Tension failure of plates occurs when the bolt is strong and plate is weak.

③ Bearing failure - When the plate is strong and bolt is weak, the bolt may crush around the half circumference which is called as bearing failure of bolt. Bearing failure of plate occurs when the bolt is strong and the plate is weak in bearing.

b. Factored load = 160 KN

$d = 16\,mm$

$d_o = 16+2 = 18\,mm$

$f_{ub} = 400\,N/mm^2$ $\quad f_u = 410\,N/mm^2$

> Shear strength of bolt -

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

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

Assume shear plane passing through thread portion

$n_s = 0$, $n_n = 1$

$$V_{nsb} = \frac{400}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi}{4} \times 16^2\right] = 36.21\,KN$$

$$\therefore V_{dsb} = \frac{36.21}{1.25} = 28.97\,KN.$$

> Bearing strength of bolt -

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$V_{npb} = 2.5\,k_b d t f_{ub}$ $\quad k_b$ is least of

$$\frac{e}{3d_o},\ \frac{p}{3d_o} - 0.25,\ \frac{f_{ub}}{f_u},\ 1$$

$e = 1.5\,d_o = 1.5 \times 18$ $\quad p = 2.5\,d = 2.5 \times 16 = 40$

$= 27 \approx 30\,mm$ $\quad \approx 60\,mm$

$$\frac{30}{3 \times 18} = 0.55,\ \frac{60}{3 \times 18} - 0.25 = 0.86,\ \frac{400}{410} = 0.97,\ 1$$

$\therefore k_b = 0.55$

$$\therefore V_{npb} = 2.5 \times 0.55 \times 16 \times 12 \times 400 = 105.6\,KN$$

$$\therefore V_{dpb} = \frac{105.6}{1.25} = 84.48\,KN.$$

$\therefore$ Strength of one bolt $= 28.97\,KN.$

$$\therefore \text{No. of bolts reqd} = \frac{\text{Factored load}}{28.97}$$

$$= \frac{160}{28.97} = 5.52 \approx 6\ \text{nos}.$$

4\. a. Advantages of welded connections –

- With welded connection, there is 100% guarantee that the connection is strong enough to withhold pressure.
- Welding is a method that provides a more rigid connected for steel structures.
- Transfer of forces between elements are more direct.
- Requires little additional elements
- Shorter length of joints
- Rigid connections are easy to achieve.

Dis Advantages of welded connections –

- Requires skilled manpower.
- Requires special equipments
- Not easy to achieve at difficult locations less ductile
- Prone to defects & fatigue cracks under cyclic loading.

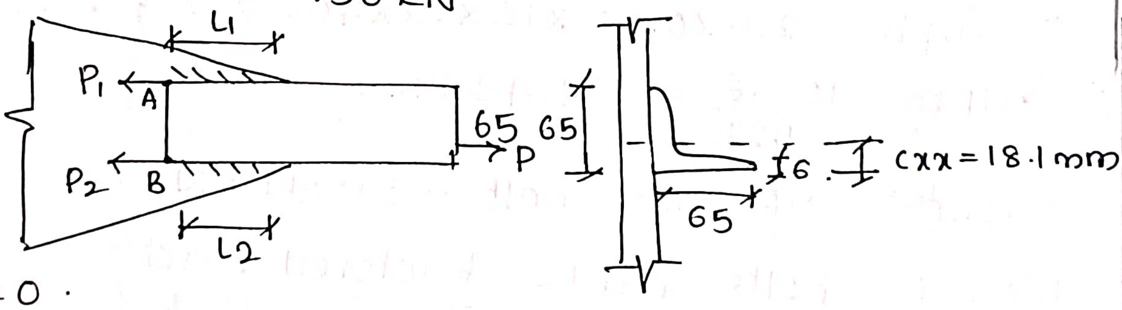
b. From steel table,

ISA 65X65X6 mm.

$A_g = 7.44\ cm^2 = 744\ mm^2$

$C_{xx} = 1.81\ cm = 18.1\ mm$

Factored load = 150 kN



$\Sigma M_B = 0$.

$P \times C_{xx} - P_1 \times 65 = 0 \Rightarrow 150 \times 18.1 = P_1 \times 65 \Rightarrow P_1 = 41.76\ kN$

$P_1 + P_2 = 150 \Rightarrow P_2 = 150 - P_1 = 150 - 41.76 = 108.23\ kN$

Design of fillet weld

$$f_{wd} = \frac{f_u}{\gamma_{mw} \times \sqrt{3}} = \frac{410}{1.25 \times \sqrt{3}} = 189.4\ \frac{N}{mm^2}$$

Design strength / mm length of weld

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

$= 189.4 \times 0.7 \times 6$

$= 795.48 \ N/mm^2$

$\therefore L_1 = \frac{P_1}{795.48} = \frac{41.76 \times 10^3}{795.48} = 52.49 \ mm$

$L_2 = \frac{P_2}{795.48} = \frac{108.23 \times 10^3}{795.48} = 136.05 \ mm$

5.

a. Based on the slenderness ratio of the column, there are three modes of failure of columns.

> Column failure due to pure compression – When the RCC columns are axially loaded, the reinforcement steel and concrete experiences stresses. When the loads are high compared to c/s area of column, the steel and concrete reach the yield stress and column fails without undergoing any lateral deformation.

> Column failure due to combined compression and bending – Long columns undergo lateral deflection and bending even when they are only axially loaded. Under such circumstances when the stresses in steel and concrete reach their yield stress, material failure happens due to combined compression and bending.

> Column failure due to elastic instability – When the RCC columns are subjected to even small loads, they tend to become unstable and buckle to any side. So, the reinforcement steel and concrete in such cases reach their yield stress even for small loads and fail due to lateral elastic buckling.

b. ISHB 300 @ 576.8 N/m

$l = 3.5m$ Both ends hinged.

$A_g = 7485 mm^2$ $h = 300mm$, $b = 250mm$, $t_f = 10.6mm$

$t_w = 7.6mm$ $r_{zz} = 129.5mm$ $r_{yy} = 54.1mm$

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

∴ From IS 800-2007 table 10, The buckling class is 'c'.

$\alpha = 0.49$.

$f_{cc} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 E}{(KL/r)^2}$

$\frac{KL}{r} = 1 \times L = 1 \times 3500 = 3500 mm$

$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{min}}\right)^2} = \frac{\pi^2 E}{\left(\frac{3500}{54.1}\right)^2} = 471.6 N/mm^2$

$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{471.6}} = 0.728$

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

$\phi = 0.5[1 + 0.49(0.728 - 0.2) + 0.728^2] = 0.894$

$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + (\phi^2 - \lambda^2)^{0.5}} = \frac{250/1.1}{0.894 + (0.894^2 - 0.728^2)^{0.5}}$

$= 160.85 N/mm^2$.

∴ $P_d = A_g f_{cd} = 7485 \times 160.85 = \underline{1204 kN}$.

6. Factored load = 65 kN $l = 3m$, $d = 120mm$, $f_{ub} = 400 \frac{N}{mm^2}$

Assume $f_{cd} = 0.4 f_y$ to $0.6 f_y$

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

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

∴ Take $f_{cd} = 100 N/mm^2$

$A_c = \frac{P_u}{f_{cd}} = \frac{65 \times 10^3}{100} = 650 mm^2$

Increase area by 25% to 40%.

$\frac{25}{100} \times 650 + 650 = 812.5$ to

$\frac{40}{100} \times 650 + 650 = 910 mm^2$

$\therefore A_g = 820\ mm^2$

Adopt ISA (80x80x8) $A_g = 1221\ mm^2 > 820\ mm^2$

$A = 1221\ mm^2$ $C_{xx} = 22.7mm$ $r_{xx} = 24.4mm$ $r_{uu} = 30.8mm$

$r_w = 15.5mm$ $\therefore$ Adopt $r_{min} = 15.5mm$

Connection design - $d = 20mm$ $d_o = 20+2 = 22mm$

Shear strength of bolt $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}] = \frac{400}{\sqrt{3}}[1 \times 0.78 \times \frac{\pi}{4} \times 20^2]$

$= 56.59\ kN$

$\therefore V_{dsb} = 45.27\ kN$

Bearing strength of bolt -

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

$e = 1.5 d_o = 1.5 \times 22 = 33\ mm \simeq 40\ mm$

$P = 2.5 d = 2.5 \times 20 = 50\ mm \simeq 60\ mm$

k_b is least of $\frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.606$

$\frac{P}{3d_o} - 0.25 \Rightarrow \frac{60}{3 \times 22} - 0.25 = 0.659$ $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$ or 1

$\therefore k_b = 0.606.$

$\therefore V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 12 \times 400}{1.25} = 116.35\ kN.$

$\therefore$ strength of bolt $= 45.27\ kN$

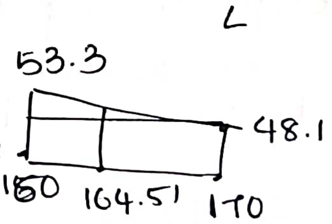
$\therefore$ No. of bolts $= \frac{65 \times 10^3}{45.27 \times 10^3} = 1.43 \simeq 2$

$\therefore L_e = 0.85 \times L = 0.85 \times 3000 = 2550\ mm.$

$\lambda = \frac{KL}{r_{min}} = \frac{2550}{15.5} = 164.51$

For buckling class c, $\alpha = 0.49$

$\therefore$ for $\frac{KL}{r_{min}} = 164.51$, $f_{cd} = 50.95\ N/mm^2$



$\therefore P_d = A_g f_{cd} = 820 \times 50.95 = 41.78\ kN < 65\ kN$

Hence redesign

7.

a. Factors affecting strength of tension members are connection length, fastener size and spacing, net area of cross section, fabrication type, connection eccentricity and shear lag at end connection.

b. Factored load = 400 kN $L = 3m$

Load on one angle section $= \frac{400}{2} = 200\,kN$

$$A_{g\,reqd} = \frac{T_{dg}\,\gamma_{m0}}{f_y} = \frac{200\times10^3\times1.1}{250} = 880\,mm^2$$

Adopt 2 ISA 80×50×10 mm angles placed back to back

$A_g = 2\times1202 = 2404\,mm^2$

Assume 20 mm bolts of 4.6 grade.

Design shear strength of bolt $= \frac{1}{1.25}\times\frac{400}{\sqrt{3}}\left[1\times0.78\times\frac{\pi}{4}\times20^2 + 1\times\frac{\pi}{4}\times20^2\right]$

$V_{dsb} = 103.31\,kN.$

$V_{dpb} = 2.5\times k_b\times d\times t\times f_{ub}/1.25$

$e = 1.5\times22 = 33 \approx 40mm$ $p = 2.5d = 2.5\times20 = 50 \approx 60mm$

k_b is least of $\frac{e}{3d_0} = \frac{40}{3\times22} = 0.606$ $\frac{60}{3\times22} - 0.25 = 0.659$

$\frac{400}{410} = 0.975$ or 1 $k_b = 0.606.$

$\therefore V_{dpb} = 2.5\times0.606\times20\times10\times400/1.25 = 96.97\,kN.$

$\therefore$ strength of one bolt $= 96.97\,kN$

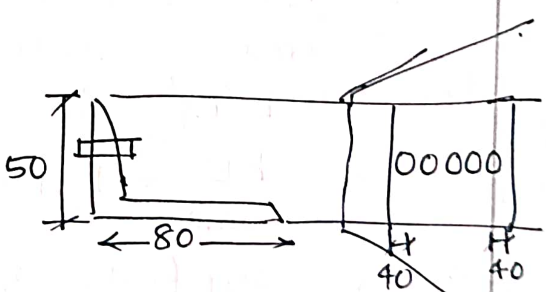
No. of bolts $= \frac{400}{96.97} = 4.124 \approx 5.$

Check for strengths –

> Rupture of critical section

$w = 80\,mm$ $w_i = 25mm$

$t = 10\,mm$ $b_s = 80+25-10$
$= 95mm$



$L_c = 4\times60 = 240\,mm$

$$\beta = 1.4 - 0.076\times\frac{80}{10}\times\frac{250}{410}\times\frac{95}{240} = 1.25$$

$$T_{dn} = \frac{0.9\,A_{nc}f_u}{\gamma_{m1}} + \frac{\beta\,A_{go}f_y}{\gamma_{m0}}$$

$A_{nc} = 50\times10 - 10\times20 - \frac{1}{2}\times10\times10 = 250\,mm^2$

$A_{go} = 80\times10 - 1/2\times10\times10 = 750\,mm^2$

$$T_{dn} = \frac{0.9 \times 250 \times 410}{1.25} + \frac{1.25 \times 750 \times 250}{1.1} = 73.8 + 213.06$$

$$= 286.86 \text{ kN} > 200 \text{ kN}$$

Hence s/n is safe.

> Block shear failure –

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \qquad T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$A_{vg}^{n} = (1 \times 40 + 4 \times 60 - 4.5 \times 20) \times 10$ $\quad A_{vg} = (1 \times 40 + 4 \times 60) \times 10$

$= 1900 \text{ mm}^2$ $\quad = 2800 \text{ mm}^2$

$A_{tn} = (25 - 0.5 \times 20) \times 10$ $\quad A_{tg} = 25 \times 10 = 250 \text{ mm}^2$

$= 150 \text{ mm}^2$

$$T_{db} = \frac{2800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 150 \times 410}{1.25} = 367.4 + 44.28 = 411.68 \text{ kN} > 200 \text{ kN}$$

$$T_{db} = \frac{0.9 \times 1900 \times 410}{\sqrt{3} \times 1.25} + \frac{250 \times 250}{1.1} = 323.82 + 56.81 = 380.63 > 200 \text{ kN}$$

Hence the assumed section is safe.

8.

a. Lug angles – These are sometimes used to reduce the length of connections. It is an angle section of short length which connects the outstanding legs of angle section or member to the gusset plate. It reduces the length of connection to gusset plate and also reduces the shear lag effect. When the lug angle is used the unconnected length of main angle acts like a connected leg and hence entire cross section area starts resisting the tension. Lug angle provides extra guage length to accommodate the no. of bolts.

Column splices – Multi storey structures generally require the column to be spliced in order to extend their length for full height of the structure. Splices may be either bolted or welded and is always assumed that welded splices are shop welded. They must resist any horizontal moments that may be incurred by lateral displacement, consequently column splices are generally designed to be moment resisting.

The column splice should be placed as close to a point of lateral restraint as possible and away from the points of maximum deflection.

b. Sectional details - ISHB 250 @ 536 N/m

$b = 250$ $b = 250\,mm$ $t_f = 9.7\,mm$ $t_w = 8.8\,mm$

Bearing strength of concrete $= 0.45 f_{ck} = 0.45 \times 20$

$F_L = 520\,kN$. $= 9\,N/mm^2$

Area of base plate $= \frac{520}{0.45 f_{ck}} = \frac{520}{9} = 57.77 \times 1000 = 57777.7\,mm^2$

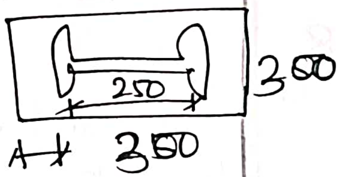
Provide 300×300 mm base plate.

Area $= 300 \times 300 = \cancel{62500}\ 90000\ mm^2$.

Pressure intensity $= w = \frac{\cancel{62500}\ 520 \times 10^3}{\cancel{62500}\ 90000} = \cancel{8.32}\ 5.77\ N/mm^2$

Projections $a = \frac{300-250}{2} = 25\,mm$

$b = \frac{300-250}{2} = 25\,mm$



Thickness of base -

$t_s = \left[\frac{2.5 \times 5.77\,(25^2 - 0.3 \times 25^2) \times 1.1}{250}\right]^{0.5}$

$= 5.26 > 9.7\,mm$ Hence provide 12mm thick base

Connecting 300×300 mm plate to concrete foundation using 4 bolts of 20mm dia 300mm long to anchor plate.

Size of weld $= 6\,mm$.

Length of weld $= (300 + 300 - 8.8 + 250 - 9.7) = (831.5) \times 2\,mm$
$= 1663\,mm$

Available length of weld $= 1663 - 2 \times 6 \times 12 = \cancel{1419.6}\ 1519\,mm$

Strength of weld $= \frac{L_w \times t \times f_{wn}}{\gamma_{mw}} = \frac{L_w \times 0.7 \times 6 \times 410}{\sqrt{3} \times 1.25}$

$= 795.35\,L_w$

$\therefore$ $795.35\,L_w = 520 \times 10^3$

$L_w = 653.8\,mm < 1519\,mm$.

Hence design is safe.

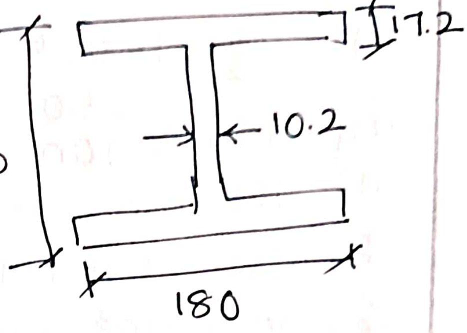
9.

a. The factors affecting lateral stability of beams are

① Shape of cross section

② Support conditions

3. Effective length
4. Level of application of loads.
5. Material used.



1b. ISMB 500 @ 86.9 kg/m

$I_{zz} = \frac{1}{12}\left[180 \times 500^3 - 169.8 \times 465.6^3\right]$

$= 446.7 \times 10^6 \ mm^4$

$Z_e = \frac{446.7 \times 10^6}{500/2} = 1.78 \times 10^6 \ mm^3.$

$M_p = (180 \times 17.2 \times 482.6 + 232.8 \times 10.2 \times 232.8) f_y$

$\therefore Z_p = \frac{M_p}{f_y} = 2.04 \times 10^6 \ mm^3.$

Section classification – $\epsilon = 1$

$\frac{b}{t_f} = \frac{180}{17.2} = 10.4$ b/w $9.4t$ & $13.6t$.

$\frac{d}{t_w} = \frac{465.6}{16} = 29.1 < 84\epsilon$. Hence it is semicompact.

Trial s/n – Assuming $V < 0.6 V_d$

$M_d = \beta_b Z_p f_y \frac{1}{\gamma_{mo}} = Z_e f_y \frac{1}{\gamma_{mo}} = \frac{1.78 \times 10^6 \times 250}{1.1} = 404.5 \times 10^6$ kNmm

Let factored UDL be w kN/length.

$M = \frac{wL^2}{2} = \frac{w \times 5^2}{2} = 12.5w.$

Equating it to M_d

$12.5w = 404.5$

$\therefore \boxed{w = 32.36 \ kN/m}$

10. Wt of RC slab $= 0.1 \times 1 \times 3 \times 25 = 7.5$ kN/m.

FL $= 1.5 \times 3 = 4.5$ kN/m

DL $= 3$ kN/m   self wt $= 0.8$ kN/m

$\therefore$ Total DL $= 4.5 + 3 + 0.8 = 8.3$ kN/m $+7.5$ $= 15.8$ kN/m.

LL $= 12$ kN/m

$\therefore$ Total load factored $= 1.5 \times 12 + 1.5 \times 15.8$

$= 41.7$ kN/m.

Eff span $= 6.5 + 0.2 = 6.7$ m.

Design moment = $M = \frac{wL^2}{8} = \frac{41.7 \times 6.7^2}{8} = 233.98$ kNm

Design SF = $\frac{41.7 \times 6.7}{2} = 139.69$ kN.

∴ S/n modulus reqd = $\frac{M}{f_y} \times \gamma_{mo}$

$Z_p = \frac{233.98 \times 10^6 \times 1.1}{250} = 1.029 \times 10^6$ mm³.

Try ISMB 400 for which $Z_p = 1175.2 \times 10^3$ mm³.

h = 400 mm, b = 140 mm, A = 7846 mm², t_f = 16 mm, t_w = 8.9 mm

d = 400 − 2(16+14) = 340 mm, $I_{zz} = 20458.4 \times 10^4$ mm⁴

$Z_e = 1020 \times 10^3$ mm⁴.

Check for shear str. — $V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times$ shear area.

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9 = 467.12 > 139.69$ Hence safe.

$0.6 V_d = 0.6 \times 467.128 = 280.7 > 139.69$ kN.

Check for design moment capacity —

$\frac{d}{t_w} = 38.2$ Hence $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$

$M_d = 1.0 \times 1175.2 \times 10^3 \times \frac{250}{1.1} = 267.09 \times 10^6$ Nmm > 233.98

Hence adequate.

Check for deflection —

$$\delta = \frac{5}{384} \frac{wL^4}{EI}$$

Total working load = 12 + 15.8 = 27.8 kN/m

$$\delta = \frac{5}{384} \times \frac{27.8 \times 6{,}700^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

= 17.82 mm.

Permissible deflection = $\frac{Le}{300} = \frac{6700}{300} = 22.33$.

Hence safe. ∴ Provide ISMB 400.

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