TE-sem-VI - CBSGS - civil - DDSS 18/5/1

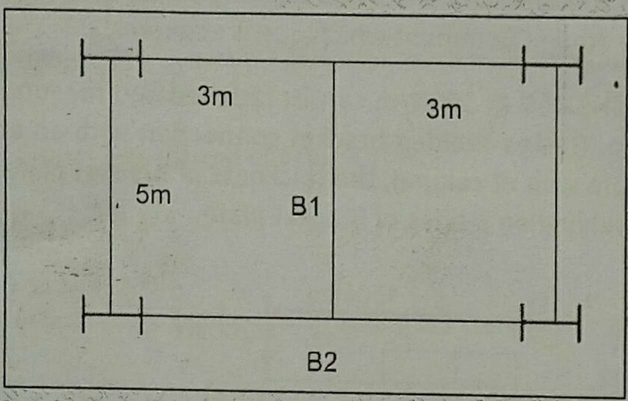
Q. P. Code: 21685

(4 Hr) Maximum Marks - 80

N.B.
1. Question No.01 is compulsory, attempt any **three** out of remaining questions
2. Draw neat and proportionate **sketches** whenever necessary.
3. Use of **IS 800:2007 and steel table** is permitted.
4. Assume **suitable data** if necessary and justify the same.
5. Use steel of **Grade Fe410** and bolt of **grade 4.6**

Q.1 Design **Beam B1 and B2** using ISMB section and **beam to beam connection**, assuming top flange of beam embedded in slab. The flooring plan is as shown, Design flooring system for following data, provide cover plates to Beam B2 if Necessary **32**

- Thickness of Slab - 15cm
- Thickness of wall - 200mm
- Height of wall over all beams - 1.3m
- Unit weight - (Concrete-25 $N/mm^3$ , Brick Wall - 20 $N/mm^3$)




**OR**

Q.1 Find panel point load for given roof truss for DL,LL and WL and design member AB,AL and BL. the structure is situated in Mumbai industrial area with rise 1/4. **32**

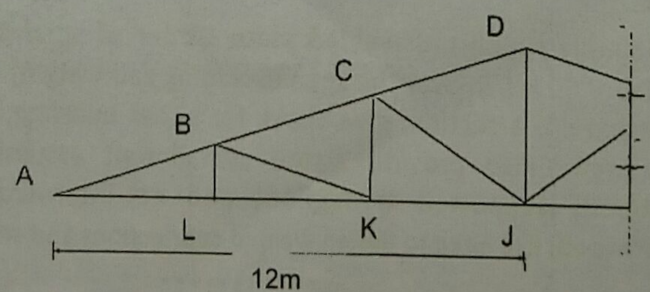
-Spacing between trusses - 3m,

-Span of truss – 12 m

-Self weight of Purlin - 220 N/m

-wt of GI sheets - 150 $N/m^2$

The values of $K_1 = 1.0$, $K_2 = 0.98$, $K_3 = 1.0$ and $(C_{pe} - C_{pi}) = -0.3$,



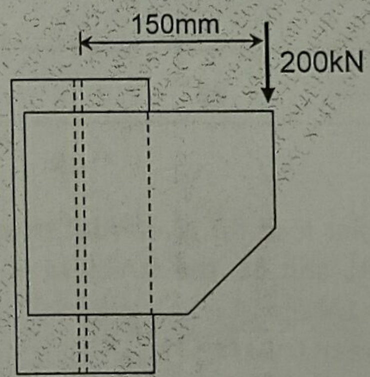
**Q.2** a) Design a **built -up column** with two channel sections which are placed face to face to support factored axial compressive load of 1600 kN, If the effective length of column is 6.0 m, Design appropriate section, spacing between channel and suitable bolted lacing system for d=20mm. **10**

b) **Design a column** using ISHB Section. Column is of length 4.8m supports factored load of 700kN, the column is effectively held in position and direction at both the ends. **06**

**Q.3** a) A Column ISHB 300@576.83 N/m strengthened with two cover plates of size 350 x 20mm to carry factored axial load of 2000kN, calculate Size, Thickness and number of bolts required for the **Gusset base** assuming M20 concrete grade and 24mm bolt diameter, draw diagrams showing all details. **10**

b) A column is Consisting of ISHB 300@576.83 N/m carries axial factored load of 800kN, Design a **Square and a Rectangular slab base** considering M15 concrete grade . Comment which one is economical. **06**

**Q.4** a) A column of ISHB 150 @ 300N/m carries factored end reaction of 200kN due to a Beam. **Design Welded bracket connection** with an eccentricity of 175 mm from web of column, the thickness of bracket plate is 12mm, and Provide welding on 3 sides of bracket plate. **08**




**Q.5** b) A ISLB 350 @486 N/m used to design a laterally unsupported beam with length of 3.5, Determine design bending strength ($M_d$) by using IS code table, also determine **safe UDL** that can be applied over beam. **08**

Q.5 **a)** A simply supported **welded plate Girder** of span 24m is subjected to UDL of 50 kN/m over the span excluding self weight, **16**
Design cross section, give check for shear buckling and design bending strength, also provide 2-step curtailment assuming plate girder is laterally supported throughout and no intermediate stiffeners are provided. (No need to design welded connections and stiffeners)

***********

Correction inQP code 21685

1. **Q.1** Unit weight ( Concrete – 25KN/$m^3$, Brick wall – 20KN/$m^3$)

**Instead of** Unit weight ( Concrete – 25N/$mm^3$, Brick wall- 20N/$mm^3$)

OR

2. **Q1** Span of truss- 24m **instead of 12m.**

2. **Correct Q5 b as Q4 b**

3. **Correct Q 5a as Q 5**

4. **Q4 a** Value of Eccentricity shall be 150mm **instead of 175mm.**


Correction inQP code 21685

1. **Q.1** Unit weight ( Concrete – 25KN/$m^3$, Brick wall – 20KN/$m^3$)

**Instead of** Unit weight ( Concrete – 25N/$mm^3$, Brick wall- 20N/$mm^3$)

OR

2. **Q1** Span of truss- 24m **instead of 12m.**

2. **Correct Q5 b as Q4 b**

3. **Correct Q 5a as Q 5**

4. **Q4 a** Value of Eccentricity shall be 150mm **instead of 175mm.**

Correction inQP code 21685

1. **Q.1** Unit weight ( Concrete – 25KN/$m^3$, Brick wall – 20KN/$m^3$)

**Instead of** Unit weight ( Concrete – 25N/$mm^3$, Brick wall- 20N/$mm^3$)

OR

2. **Q1** Span of truss- 24m **instead of 12m.**

2. **Correct Q5 b as Q4 b**

3. **Correct Q 5a as Q 5**

4. **Q4 a** Value of Eccentricity shall be 150mm **instead of 175mm.**

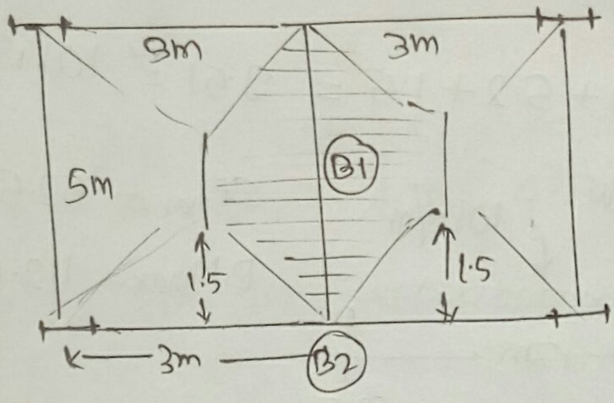
TE - CE - SEM - VI - CBSGS - DDSS

Civil Engineering, → Q.P. Code - 21685

HAWELIKAR S.C. (AIKTC Panvel)

**Q.1** Design Beam B1 & B2

| | |
|---|---|
| Load Calculations | - 06 |
| Beam Design | = 10+10 |
| Connection with diagram. | = 06 |
| | 32 |




Slab thickness = 15 cm

Wall thickness = 200 mm

Wall height = 1.3 m



* Load Calculations

⇒ Load on Beam (B1)

= Dead load due to slab.

$= \left[(3\times5) - 2\left(\frac{3\times1.5}{2}\right)\right]\frac{\times0.15\times25}{5} \Rightarrow 7.87 \cong 8\,kN/m$

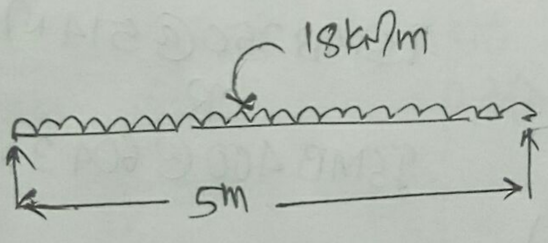
⇒ assuming LL = 2 kN/m²

$= \left[(3\times5) - 2\left(\frac{3\times1.5}{2}\right)\right]\times 2/5 = 4.2\,kN/m$

⇒ Wall load (as its acting on all Beam)

$= 1.3\times0.2\times20 = 5.2\,kN/m$

Total UDL $= 7.87 + 4.2 + 5.2 = 17.27\,kN/m \cong 18\,kN/m$




Beam B1

$SF_{max} = \frac{wl}{2} = \frac{18\times5}{2}$

$= 45\,kN$

$BM_{max} = \frac{wl^2}{8} = 56.25\,kNm$

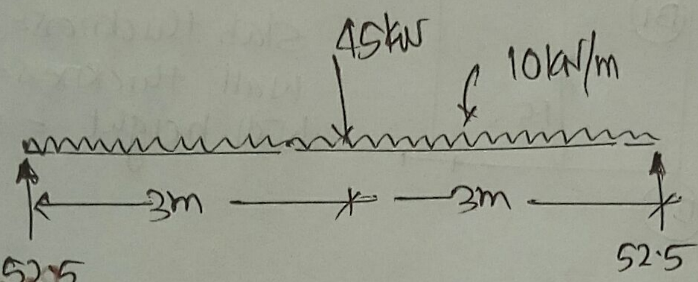
→ Calculations for Beam B2

$D.L = \frac{(1.5 \times 3)}{2} \times \frac{0.15 \times 25}{3} = 2.81 \text{ kN/m}$

Wall load = 5.2 kN/m

$LL = \frac{(1.5 \times 3)}{2} \times 2/3 = 1.5 \text{ kN/m}$

Total UDL = 2.81 + 5.2 + 1.5 = 9.51 ≅ 10 kN/m




$SF_{max} = 52.5$

$BM_{max} = 112.5 \text{ kNm}$

Pg.02

10 52.5

| B1 | SF | BM | B2 | SF | BM |
|---|---|---|---|---|---|
| Service | 45 kN | 56.25 kNm | Service | 52.5 kN | 112.5 kNm |
| Factored | 68 kN | 85 kNm | factored | 80 kN | 170 kNm |

$Z_{pz}$ Required $= \frac{M \gamma_{mo}}{f_y}$

$= 85 \times 1.1/250$

$= 374 \times 10^3 \text{ mm}^3$

Only ISMB

ISMB 250 @ 365.91 N/m
(or next larger)

$Z_{pz}$ Req $= \frac{M \gamma_{mo}}{f_y}$

$= 170 \times 1.1/250$

$= 748 \times 10^3 \text{ mm}^3$

ISMB 350 @ 514 N/m
OR
ISMB 400 @ 604.3 N/m

Q2 (a)

## Design of Built up Column

$P = 1600\ kN$, $l_e = 6\ m$, $d = 20\ mm$

**Face to face**

assuming $f_{cd} = 152\ N/m$

Area required $= 1600 \times 10^3 / 152 = 10526\ mm^2$

single channel $= 10526/2 = 5264\ mm^2$

Providing 2 ISMC 350 @ 413 N/m — $A_g = 5366\ mm^2$

OR 2 ISMC 400 @ 481.6 N/m — $A_g = 6293\ mm^2$

Pg. 03

For 2 ISMC 350 @ 413 N/m.

$A = 5366\ mm^2$, $r_{xx} = 136.6\ mm$, $r_{yy} = 28.3\ mm$

$I_{xx} = 10{,}008 \times 10^4\ mm^4$, $I_{yy} = 430.6 \times 10^4\ mm^4$, $C_{yy} = 24.4\ mm$

Area provided $= 2 \times 5366 = 10732\ mm^2 > 10526\ mm^2$

$\Rightarrow$ ~~Area provided~~ $= \lambda e = \frac{6000}{136.6} = 43.92$

For $\lambda = 43.92$, $f_y = 250 \longrightarrow f_{cd} = 192.12\ N/mm^2$

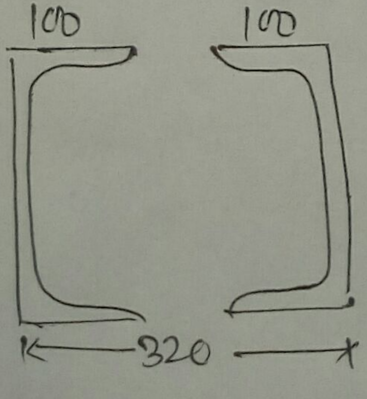
$P_d = A_e \times f_{cd} = 2 \times 5366 \times 192.12 = 2061\ kN > 1600\ kN$ OK

$\Rightarrow$ Spacing $\Rightarrow 2 \times 10.008 \times 10^4 = 2\left[430.6 \times 10^4 + 5366\left(S/2 - 24.4\right)^2\right]$

**$S = 316 \simeq 320\ mm$**

$a_1 = 2(320 - 60 - 60)\cot 45$
$= 400\ mm$

$\frac{a_1}{r_{yy}} = \frac{400}{28.3} = 14.13 < 50$
$< 0.7(\lambda e)$




Force Calculations

$V_t = \frac{1600 \times 2.5}{100} = 40\ kN$

Transverse shear

$V_t/N = 40/2 = 20\ kN$

$V_b = \frac{V_t}{N} \operatorname{cosec}\theta = 28.28\ kN$

⇒ d = 20 mm , do = 22 mm
e = 35 mm , p = 50 mm

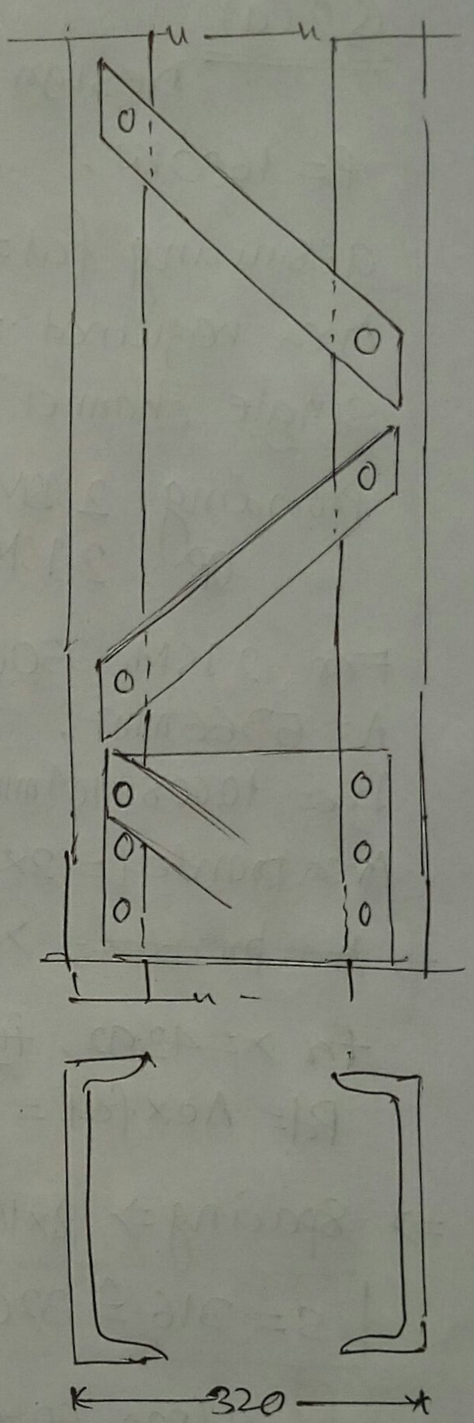
⇒ thickness of lacing flat } 3(do) = 3(20
= 60 mm

⇒ effective length of lacing flat =
= (320 − 60 − 60) Cosec45
= 282.84 mm

⇒ Length of lacing flat
= 282.84 + 2(35)
= 352.84 ≅ 360 mm

⇒ Min thickness of lacing flat
= 1/40 (le) = 1/40 (282.84)
= 7.07 mm
≅ 8 mm

360×60×8




Secn Design = 03 marks
proper Diagram = 02 Marks
Spacing & force Calculn. - 03
lacing dimension = 02

10 Marks

Q.2(b)

Design of ISHB column...

$L = 4.8\,m$, $P = 700\,kN$

$L_e = K \cdot L = 4.8 \times 0.65 = 3.12\,m$.

assuming $\lambda = 100$.

for I-section $r_{xx} > r_{yy}$ & buckling class 'b'

$f_{cd} = 118\,N/mm^2$

Area required $= \frac{700 \times 10^3}{118} = 5932\,mm^2$

providing ISHB 250 @ 500.31 N/m, $A = 6496\,mm^2$

$r_{xx} = 109.1\,mm$, $r_{yy} = 54.9\,mm$

$\lambda_e = \frac{K \cdot L}{r_{min}} = \frac{3120}{54.9} = 56.83$

For buckling class 'b', $f_y = 250$ & $\lambda = 56.83$

$f_{cd} = 185.12$

$P_d = A_e \times f_{cd} = 6496 \times 185.12$

$= 1202\,kN > 700\,kN$ OK

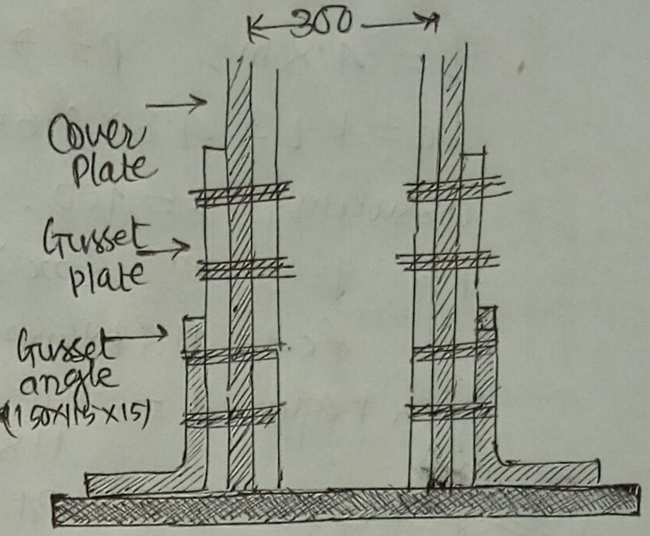
Design = 03 Marks
check = 03 Marks } 06 marks

Pg. 05

Q.3 (a)

Design of Gusset Base.

ISHB 300@576.83,
P = 2000 kN, d = 24mm
Cover plate = (350×20)
M20 Concrete Grade




$$A = \frac{2000 \times 10^3}{0.45(20)}$$

$= 222.222 \times 10^3 \text{ mm}^2$

assuming Gusset plate thickness = 16mm

Minimum width

$= 300 + 2(16+20+115) = 602\text{mm} \cong 610\text{mm}$

projection = 4mm

Length of Base plate $= \frac{222.222 \times 10^3}{610} = 369 \cong 380\text{ mm}$

**Size = 610×380**

$w_1 = 2000 \times 10^3/(610 \times 380) = 8.62 < (0.45 f_{ck})$

$M_x = w_1 c_1^2/2 = 8.62 \times [4+115-15]^2/2 = 46616.96 \text{ kNmm}$

$45.45t^2 = 46616.96 \Rightarrow t = 32 \text{ mm}$

$t_b = 32 - 15 = 17\text{mm} \cong 20\text{ mm}$

No of bolts required assuming (double shear)

$V_{sb} = 65 \times 2 = 130 \text{kN}$

Bolts required $= 0.5 \times 2000/130 = 7.69 \cong$ **16 No**

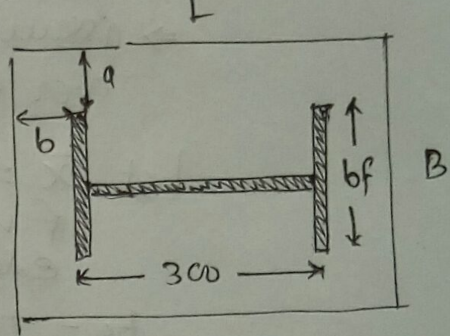
Req. area Calculation = 02 Marks
actual size calculations = 03 Marks
Bolts calculation = 03 Marks
Diagram = 02

Q.3 (b)

ISHB 300 @ 576.83 | $b_f = 250$ mm, $h = 300$ mm

$P = 800$ kN, $f_{ck} = 15$ N/mm²

$$\text{Req Area} = \frac{800 \times 10^3}{0.45(15)} = 118519 \text{ mm}^2$$




Small projection = b

Large projection = a

⊛ Providing <u>Square</u> Base plate

$L = B = \sqrt{A}$

$L = B = \sqrt{118519} = 344.26$

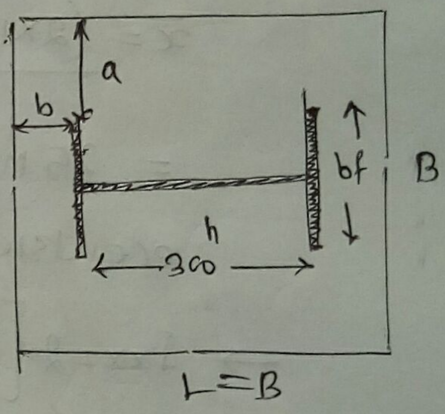
$= 360$ mm

$b = (360 - 300)/2 = 30$ mm

$a = (360 - 250)/2 = 55$ mm

$t_s = \sqrt{2.5 \times w_1 (a^2 - 0.3b^2) \gamma_{mo} / f_y}$

$t_s = 13.67$ mm $\cong 16$ mm




⊛ Rectangular Base plate

$A = (D + 2b) * (b_f + 2a)$

$= (D + 2a) * (b_f + 2a) = 118519$

$= (300 + 2a) * (250 + 2a) = 118519$

$\Rightarrow a = b = \underline{36 \text{ mm}} \cong 40$ mm

$L = 300 + 2(40)$

$= 380$ mm

$B = 250 + 2(40)$

$= 330$ mm

$w_1 = 800 \times 10^3 / (380 \times 330) = 6.37$ N/mm² $< 0.45 f_{ck}$ "<u>ok</u>"

$t_s = 8.85$ mm $\cong 10$ mm (Rectangular Base plate is economical than the Square Base plate)

[Square – 03, Rectangular 03] = 06 marks

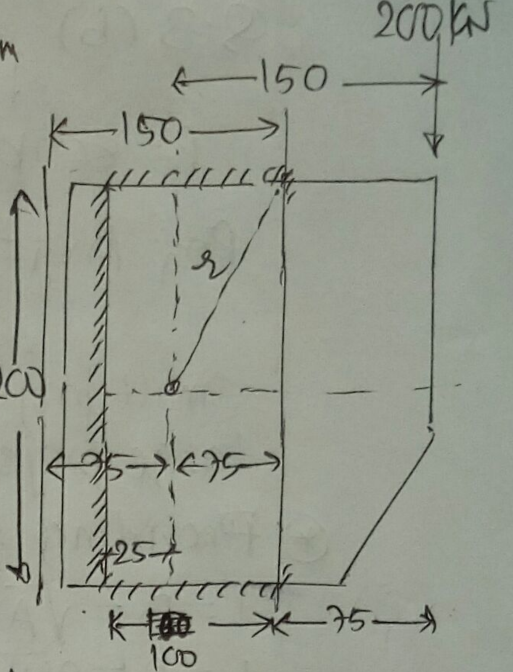
Q. 4 (a) ISHB 150 @ 300 N/m

$e = 150$ mm, bracket plate $= 12$ mm

$D = 150$ mm, $b_f = 150$ mm

$t_f = 9$ mm, $t_w = 8.4$ mm

⇒ assuming overlaping of plate over Column = 100 mm




Let $\bar{x}$ = distance of Centroide of weld group from left edge of bracket plate

$t_t$ = throat thickness.

$$\bar{x} = \frac{(2\times100\times t_t)\times 50 + (200\times t_t\times 0)}{2\times100\times t_t + 200\times t_t}$$

$= 25$ mm

$r = \sqrt{100^2 + 75^2}$

$r = 125$ mm

eccentricity of load $= (100-25)+75 = 150$ mm

→ $I_{xx} = 2\times\left[\frac{100\times t_t^3}{12} + (100\times t_t)\times 100^2\right] + \frac{t_t\times 200^3}{12}$

$= (266.66\times10^4 \text{ mm}^4)\, t_t$

→ $I_{yy} = 2\left[\frac{t_t\times 100^3}{12} + (t_t\times100)(50-25)^2\right] + \frac{200\times t_t^3}{12} + 200\times t_t\times 25^2$

$= (41.66\times10^4 \text{ mm}^4)\, t_t$

→ $I_p = I_{xx} + I_{yy} \Rightarrow 266.66 + 41.66 = (308.32\times10^4 \text{ mm}^4)\, t_t$

$\cos\theta = (75/125) = 0.6$

Direct shear $= 200\times0.5/(2\times100+200)\,t_t = 0.025/t_t$

$q_1 = 250/t_t$

shear stress $= q_2 = \frac{0.5\times200\times150\times10^3\times125}{308.32\times10^4\, t_t}$

$= q_2 = 608/t_t$

Pg. 07

$$= \frac{608}{t_t} \leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$\frac{608}{t_t} = \frac{410}{\sqrt{3}(1.25)}$$

$$t_t = 3.21 \text{ mm}$$

We know that,

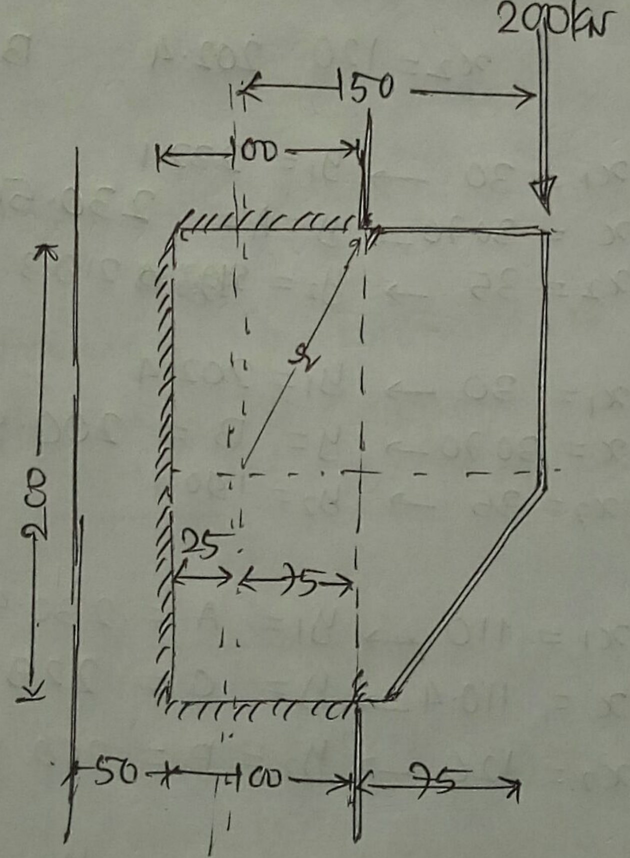
$t_t = K \cdot S$

$3.21 = 0.7(S)$

$S = 4.58$ mm

**S = 5 mm**

Providing 5mm weld size...



Q.4. (b) ⇒ ISLB-350 @ 486 N/m

$L_e = 3.5$ m, $h = 350$, $b_f = 165$ mm,

$t_f = 11.4$ mm, $t_w = 7.4$ mm, $r_{yy} = 31.7$

$KL/r = \frac{3500}{31.7} = 110.4$

$h/t_f = 350/11.4 = 30.7$

| KL/r \ h/tf | 30 | 30.70 | 35 |
|---|---|---|---|
| $x_1 = 110$ | 232.1 | A | 219.3 |
| $x = 110.4$ | | C | |
| $x_2 = 120$ | 202.4 | B | 190.1 |

For $f_y = 250$

$KL/r = 110.4$

$h/t_f = 30.70$

$f_{cr,b} = ?$

For (A) $x_1 = 30 \rightarrow y_1 = 232.1$

$x = 30.70 \rightarrow y = A = 230.56$

$x_2 = 35 \rightarrow y_2 = 219.3$

For (B) $x_1 = 30 \rightarrow y_1 = 202.4$

$x = 30.70 \rightarrow y = B = 200.92$

$x_2 = 35 \rightarrow y_2 = 190.1$

Md Calculation ⇒ 06 Marks

UDL Calculation = 02 Marks

Total = 8.

For (C) $x_1 = 110 \rightarrow y_1 = A = 230.56$

$x = 110.4 \rightarrow y = C = 229.37 = f_{cr,b}$

$x_2 = 120 \rightarrow y_2 = B = 200.92$

P9-

For,

$f_{cr,b} = 229.37$ N/mm²

$f_y = 250$ N/mm²

$\alpha_{LT} = 0.21$ then,

$f_{bd} = 144.79$

$x_1 = 250 \rightarrow y_1 = 152.3$

$x = 229.37 \rightarrow y = 144.79$ N/mm²

$x_2 = 200 \rightarrow y_2 = 134.1$

$M_d = \beta_b \cdot Z_p \cdot f_{bd}$

$= 1.0 \times 851.11 \times 10^3 \times 144.79$

**Md = 123 kNm**

Capacity to bear UDL

$M_R = WL^2/8 \Rightarrow 123 = W(3.5)^2/8$

$W = 96.09$ kN/m

Service $= 96/1.5 = 64$ kN/m

Q.05 Design of plate girder

span = 24m  UDL = 50 kN/m

Total Imposed UDL = 50 kN/m

Self wt $= WL/400 = \frac{(50 \times 24)24}{400}$

$= 72$ kN

Self wt Intensity $= 72/24 = 3$ kN/m

Total UDL acting $= 50 + 3 = 53 \cong 55$ kN/m

factored UDL $= 55 \times 1.5 = 82.5$ kN/m

$\cong 83$ kN/m

$SF_{max} = \frac{83 \times 24}{2} = 996$ kN

$BM_{max} = \frac{83 \times 24 \times 24}{8} = 5976$ kNm

SFD & BMD are necessary.

→ Assuming No - Intermidiate stiffners

$d/t_w = K = 150 < 200\epsilon$

$d = \left[\frac{5976 \times 10^6 \times 150}{250}\right]^{0.33} = 1422 \cong 1500$ mm

$t_w = \left[\frac{5976 \times 10^6}{250 \times 150^2}\right]^{0.33} = 9.96 = 16$ mm

$A_f = \left[\frac{5976 \times 10^6 \times 1.1}{250 \times 1500}\right] = 17530$ mm²

$b_f = 0.3(1500) = 450 \cong 500$ mm

$t_f = 17530/500 = 35 \cong 40$ mm

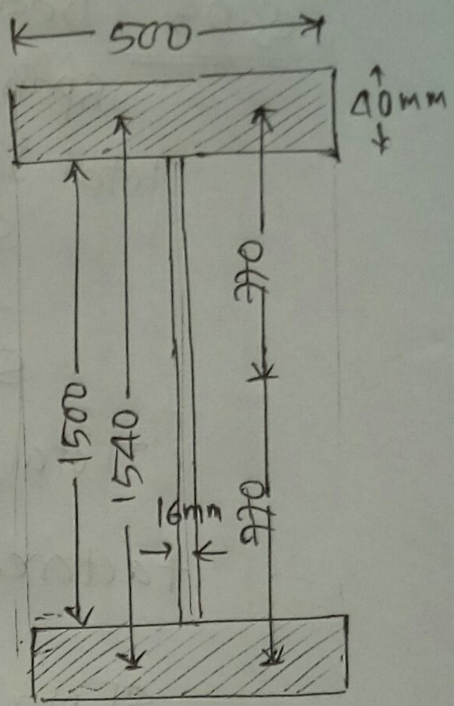
⇒ classification of Sectn

$b = \frac{500 - 16}{2} = 242$ mm

$b/t_f = \frac{242}{40} = 6.02 < 8.4\epsilon$

Sectn is plastic . . . $\beta_b = 1.0$

⊛- Check for Bending Strength ⇒




$M_d = \beta_b \cdot Z_{p2} \cdot f_y / \gamma_{m0}$

$= 1.0 \times 2(500 \times 40 \times 770) \times 250/1.1$

$= 7000$ kNm $> 5976$ kNm

'OK'

⊛ Shear Capacity of web

$d/t_w = \frac{1500}{16} = 93.75$

$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2) \times (93.75)^2}$

$= 109.92$ N/mm²

$\lambda_w = \sqrt{\frac{250}{\sqrt{3} \times 109.92}} = 1.31 > 1.2$ from IS 800:2007

$\tau_b = f_{yw} / \lambda_w^2 \sqrt{3} = \frac{250}{1.31 * \sqrt{3}} = 110.18$ N/mm²

$V_{cr} = d \cdot t_w \cdot \tau_b = 1500 \times 16 \times 110.18$

$= 2644$ kN $> 996$ kN 'OK'

⊛ - Providing 2 step. Curtailment

$t_f = 40$ mm.

1st ⇒ 40 mm → 30 mm

2nd ⇒ 30 mm → 20 mm

## * First Curtailment

$t_f = 30mm$

$$M_d = \frac{1.0\,[500\times30\times765]\times2\times250}{1.1}$$

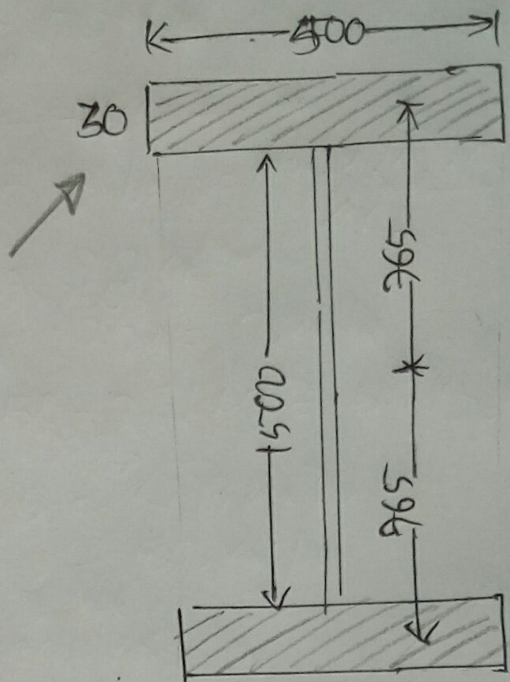
$= 5215\ kNm$

$M_d = M_R$ (at Midspan)

$996\times x - 83\,(x^2/2) = 5215$

$x = 7.71\ m$

$x = 7.0\ m$ from End




## * 2nd Curtailment

$t_f = 20mm$

$M_d = 1.0\times(500\times20\times760)\times2\times250/1.1$

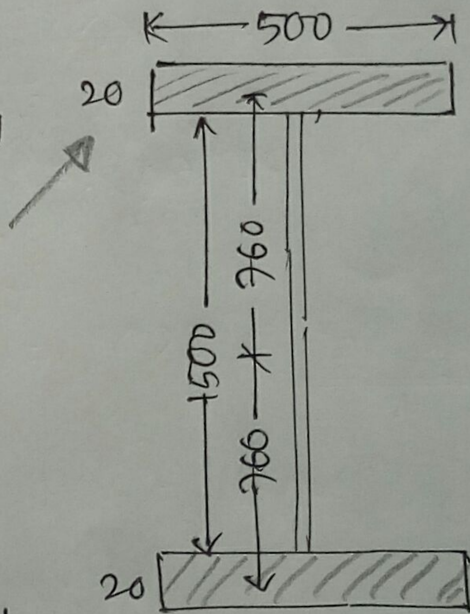
$= 3454\ kNm$

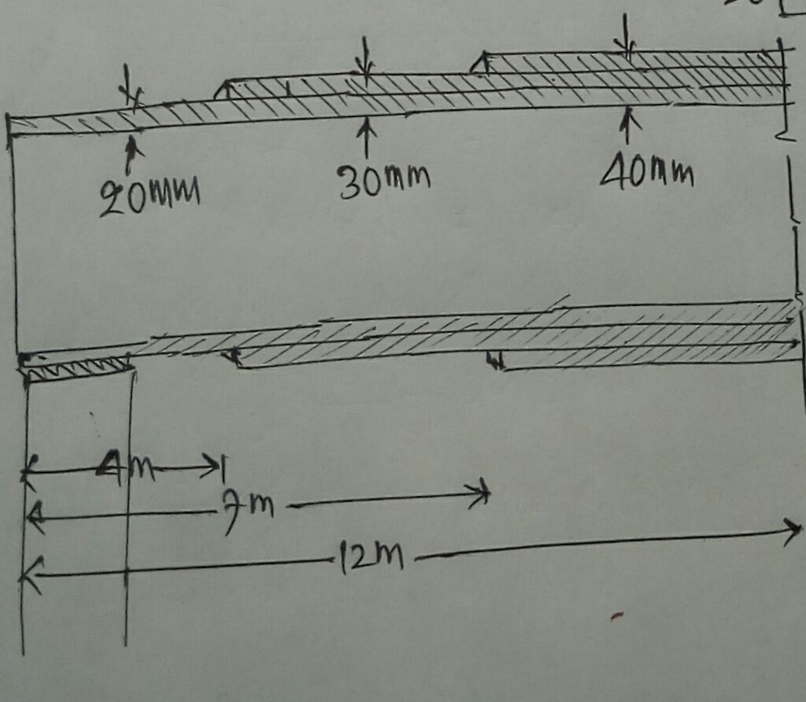
$M_d = M_R$ (at midspan)

$996x - 83\,(x^2/2) = 3454$

$x = 4.2m$

$x = 4.0\ m$ from End




16

load Calculation = 02
c/s Calculations = 04
Drawing
Check Shear (03) + Bending (02) = 05
Curtailment + Drawing = 05