CBCS SCHEME

USN | | | | | | | | | | |

17CV72

Seventh Semester B.E. Degree Examination, Feb./Mar. 2022

Design of RC and Steel Structures

Time: 3 hrs. Max. Marks: 100

Note: 1. ***Answer any TWO full questions choosing one from each module.***
2. ***Use of IS456, IS800, SP(6), Steel tables are permitted.***
3. ***Assume any Missing data suitably.***

Module-1

1 Design a slab type rectangular combined footing for two columns of size 300mm × 450mm and 300mm × 600mm, subjected to axial loads of 650kN and 900kN respectively. The columns are spaced at 3.6m C/C. The width of the footing is restricted to 1.8m. Use M_{20} grade concrete and Fe415 grade steel. Assume SBC of soil ⇒160 kN/m^2. **(50 Marks)**

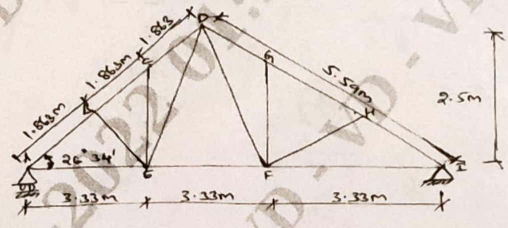
OR

2 An RC portal frame with a hinge base is required to suit the following data :
Spacing of portal frames ⇒ 4m C/C ; Height of column ⇒ 4m
Difference between column centers ⇒ 10m ; Live load on roof ⇒ 1.5 kN/m^2.
The RC slab is continuous over the portal frame, SBC of soil ⇒ 200 kN/m^2.
Materials : M_{20} and Fe415 steel are used. Design the slab, portal frame and foundation. **(50 Marks)**

Module-2

3 Design a roof truss shown in Fig. Q3, with forces in each member of the truss is given in table 1. The size of RC column supporting the truss is 300mm × 300mm. Use M20 grade concrete for column. Design the truss using bolt of M16, property class 4.6 for connections and also design anchor bolts. **(50 Marks)**

Fig.Q3



Member	Design force in kN	
	Compression	Tension
Top chord	54.25	-
Bottom chord	-	48.31
Diagonal (DF, DE)	14.35	-
Member (BE, HF)	-	24.50
Member (CE, GF)	12.40	-

Table – 1.

OR

4 Design a simply supported crane gantry girder for the following data :
Span of crane girder ⇒ 20m ; Span of gantry girder ⇒ 7m ; Capacity of the crane ⇒ 220kN
Self weight of crane excluding the crab ⇒ 200 kN ; Weight of crab ⇒ 60kN
Wheel base distance ⇒ 3.4m ; Min hook approach ⇒ 1.10m ; Self weight of rail ⇒ 0.3kM/mm
Height of rail ⇒ 70mm. **(50 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.

Scheme of Valuation

Sub: Design of RC and Steel Structures (17CV72)

Module 1

Q1. Given Data.

column A = 300 x 650   650 kN.

Column B , 300 x 600   900 kN

spacing between column = 3.6 m c/c

width of footing = 1.8 m , SBC = 160 kN/m²

M20 & Fe 415.

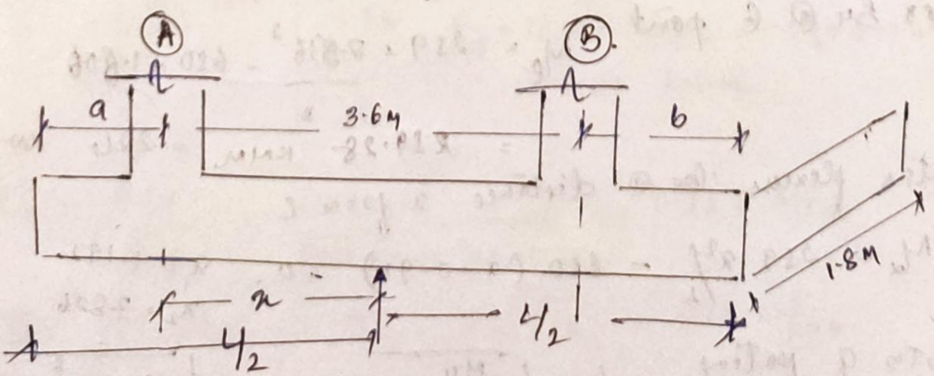
load from Column (A+B) = 1550 kN.

Self wt (10%) = 155

Total load = 1705 kN.

Required Area = $\frac{\text{load}}{SBC} = \frac{1705}{160} = 10.65\ m^2$

length of footing = $\frac{\text{Area}}{\text{width}} = \frac{10.65}{1.8} = 5.92\ m \simeq 6\ m$



$x = \frac{\text{col } B \times 3.6}{\text{col } A + \text{col } B} = \frac{900 \times 3.6}{900+650} = 2.09$ from A.

$\therefore (a+x) = L/2 \quad (a+2.09) = 6/2 \quad \&\ a = 0.91\ m$

III^{ly} $((3.6-x)+b) = L/2 \quad (1.51+b) = 3 \quad \&\ b = 1.49\ m$

Uplift pressure = $\frac{\text{Total load}}{\text{Area (pr)}}$ = $\frac{1550}{6 \times 1.8}$ = 143.5 ≈ 144

Design of slab for logitudinal bending.

as width = 1.8 m. w = 143.5 × 1.8 = 259 kN/m.

Calculation of Shear force & Bending Moment

V_{AC} = 259 × 0.91 = 235.7 V_{AB} = 650 − 235.7 = 414.3 kN

V_{BD} = 259 × 1.49 = 385.91 V_{BA} = 900 − 385.91 = 514.09

Shear become zero @ E pt. 259 (x) − 650 = 0.

∴ $x = \frac{650}{259}$ = 2.510 from footing end.

d = 1.600 from column A center.

M_A = 259 × $0.91^2/2$ = 107.2 kN-m.

M_B = 259 × $1.49^2/2$ = 287.5 kN m.

Max BM @ E point. M_E = 259 × $\frac{2.516^2}{2}$ − 650 × 1.600

= 224 kN-m

Contra flexure be @ distance x from C.

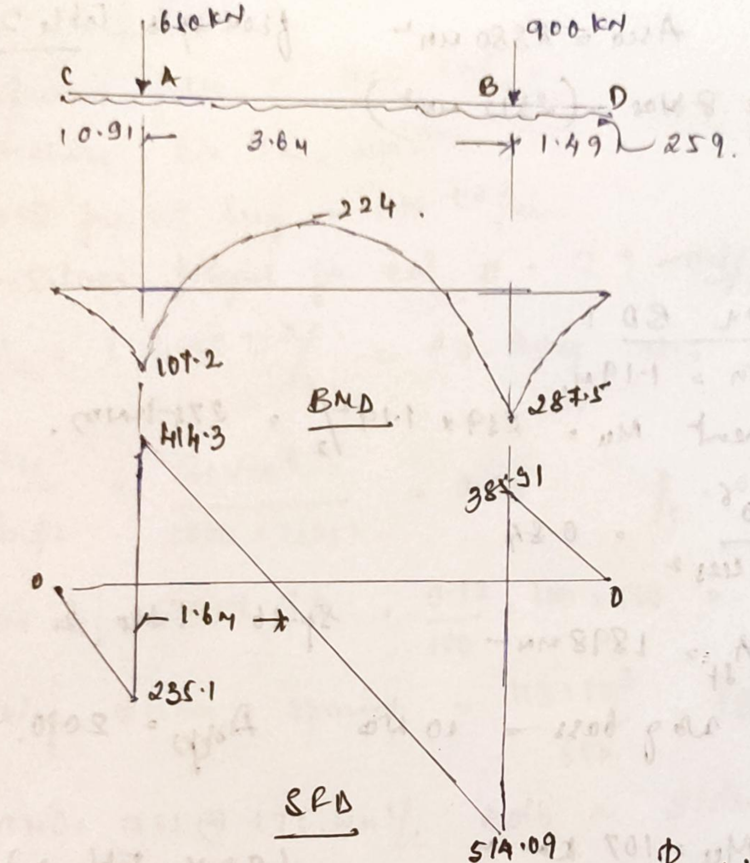
∴ M_x = 259 $x^2/2$ − 650 (x − 0.91) = 0. x_1 = 1.192

x_2 = 3.826

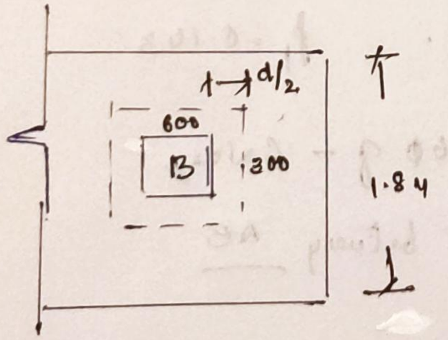
Depth of footing $d = \sqrt{\frac{M_u}{f_{ck} \times b \times 0.138}}$ = $\sqrt{\frac{288 \times 10^6 \times 1.5}{2.76 \times 1800}}$

= 294 + 75 = 369 mm

To have shear take D = 500 & d = 425 mm.



Punching Shear



load on heavy column

$P_u = 900 \times 1.5 = 1350$ kN.

$P_{ud} = 1350 - 143.52 \times 1.5\,(0.6 + 0.42)^2$

$= 1126$ kN

$$\tau_v = \frac{P_{ud}}{b_o d} = \frac{1126 \times 10^3}{(725 + 1025) \times 2 \times 425} = 0.757 \text{ MPa}$$

$\tau_c' = \tau_c \times k_s$ $\qquad \tau_c = 0.25\sqrt{20} = 1.11$ N/mm²

$$k_s = 0.5 + \frac{300}{600} = 1.0$$

$\therefore \tau_c' = 1.11$ N/mm² $> \tau_v$ hence safe

Area of Reinforcement:

Region AB. $\dfrac{M_u}{bd^2} = \dfrac{224 \times 10^6 \times 1.5}{1800 \times 425^2} = 1.03.$

$p_t = 0.311$ Area = 2380 mm^2 from sp16 table 2

provide 20ϕ of 8 Nos. (2512 mm^2)

Cantilever portion BD :

Cantilever length = 1.19 m

ultimate moment $M_u = 259 \times 1.19^2/2 = 275$ kN m.

$$\frac{M_u}{bd^2} = \frac{275 \times 10^6}{1800 \times 425^2} = 0.84$$

$p_t = 0.248$ $A_{st} = 1898\ mm^2$ Sp-16 Table 2.

using #16ϕ. No of bars = 10 No A_{st}(p) = 2010.

portion AC

$M_u = 107$ kN-m

(Sp16 table 2)

$$\frac{M_u}{bd^2} = \frac{160.5 \times 10^6}{1800 \times 425^2} = 0.493.$$ $p_t = 0.143$

$A_{st} = 1094\ mm^2$ provide 16ϕ of - 6 Nos.

Check for Shear Reinforcement between AB

$V_u = 514 \times 1.5 = 771$ kN.

Shear @ Center closure = $T_v = 771 - 388\ (0.282)$

$$\tau_v = \frac{V_u}{bd} = \frac{661 \times 10^3}{(1800 \times 425)} = 0.864 < \tau_{c\,max}$$

Along steel 2512 mm^2 = $p_t < 0.33$ $\therefore \tau_c = 0.41$

Shear reinforcement required. Sp16 Table 61.

$$\frac{V_{us}}{d} = \frac{(0.864 - 0.41)}{42.5} \times 1800 \times 425/1000 = 8.12/2 = 4.09$$

Sp16 Table 62.

provide #12 - 4 legged 180 mm c/c.

Design of Slab :

Upward force = 144 kN/m²

Consider b = 1m width.

Load per unit length = 144 kN/m

Cantilever project for col B = $0.9 - 0.3/2$ = 0.75 m

$M_u = 144 \times 0.75^2/2 = 40$ kNm $\times 1.5 = 61$ kNm

$\frac{M_u}{bd^2} = \frac{61 \times 10^6}{1000 \times 425^2} = 0.33$ $\quad p_t = 0.099$

Min A_{st} = 0.12% GA = $\frac{0.12}{100} \times 1000 \times 500 = 600$ mm²

Using #12mm spacing = $\frac{113 \times 10^3}{600} = 188$ mm

Provide #12 @ 175 mm c/c both as distribution & main.

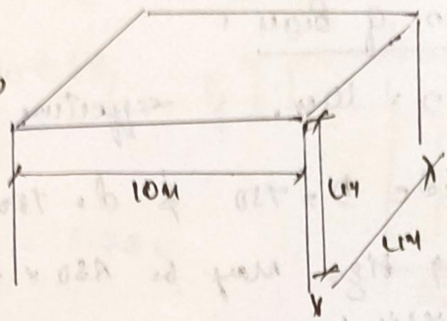
Q2. Given Data:

Spacing of frame = 4m   Ht of column = 4m

Span of frame = 10m   SBC of soil = 200 kN/m²

Live load = 1.5 kN/m²

Slab Design:



Assume slab depth = 120mm

Effective depth = 100mm

Self wt (0.12×24) = 2.88

finishing wt = 0.75

Total dead wt = 3.63 kN/m²

Live load = 1.50 kN/m²

$$\text{Max BM} = \frac{W_d L^2}{10} + \frac{W_L \times L^2}{9} = 8.5 \text{ kN-m} \quad M_u = 12.75 \text{ kN/m}$$

$$M_{ulim} = Q_{lim}\, b \cdot d^2 = \frac{2.76 \times 1000 \times 100^2}{1 \times 10^6} = 27.6 > 12.75$$

$$\frac{M_u}{bd^2} = \frac{12.75 \times 10^6}{1000 \times 100^2} = 1.275 \quad \text{from Sp16 Table 2}$$

$$P_t = 0.384 \qquad A_{st} = \frac{0.384}{100} \times 1000 \times 100 = 384 \text{ mm}^2$$

Using #10

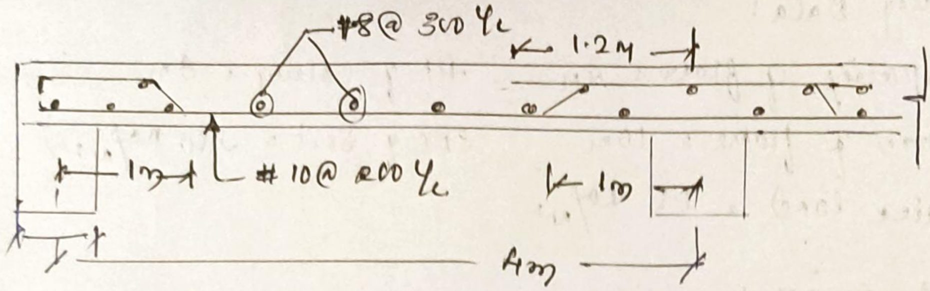
$$\text{Spacing} = \frac{10^2 \times \pi/4}{384} \times 1000 = 204$$

provide $\phi 10$ @ 200 c/c.

$$\text{Area of distributing steel} = \frac{0.12 \times bD}{100} = \frac{0.12}{100} \times 1000 \times 120.$$

Area = 144 mm²

provide 8mm @ 300 mm c/c.



Design of Beam :

Span = 10 m. effective depth = $\frac{span}{13}$ = 769 ≈ 750

provide D = 750 & d = 700 mm, breath b = 450 mm

column size may be 450 × 600 mm.

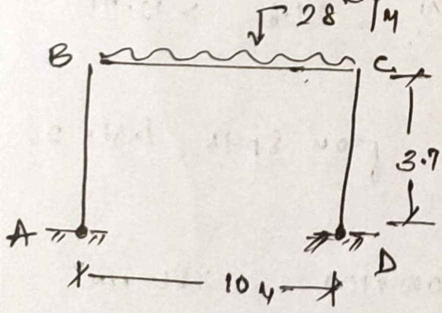
Analysis :

load from slab = (3.05 + 1.5) × 4.5 = 20.52 kN/m

self wt of rib = 0.45 × 0.63 × 24 = 6.80

Total = 27.32 ≈ 28.0

Ht of beam above hinge = 4 + 0.1 − 0.75/2 = 3.72 m.



$I_{AB} = \frac{450 \times 600^3}{12} = 81 \times 10^8\ mm^4$

$I_{BC} = \frac{450 \times 750^3}{12} = 158.2 \times 10^8$

Stiffness factor. $K = I/L$

$K_{BA} = \frac{I_{BA}}{L_{BA}} = \frac{81 \times 10^8}{3720} = 21.77 \times 10^5$ & $K_{BC} = \frac{I_{BC}}{L_{BC}} = 15.8 \times 10^5$

Distribution factor $D_{BA} = D_{BC} = \frac{K_{BA}}{\Sigma K_B} = 0.5$

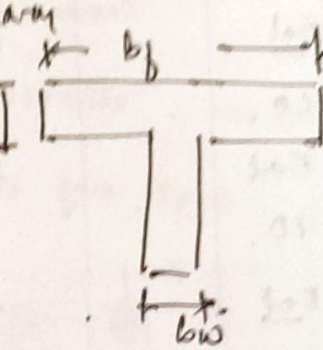
fixed end moments

$M_{FAB} = M_{FBA} = M_{FCD} = M_{FDC} = 0$.

$M_{FBC} = M_{FCB} = -wl^2/12 = -\frac{28 \times 10^2}{12} = 233$ kNm

Intermediate Beam is Designed during become Critical Mid-span Section is designed as T-beam & ends are designed as Rectangular Section.

Mid span Beam (T) : $M_u = 291$ kNm



flange width $b_f = \frac{l_0}{6} + b_w + 6D_f$

$l_0 = 0.7 \times L = 0.7 \times 10 = 7$ m

$b_f = \frac{7}{6} + 0.45 + 6 \times 0.12 = 2.33$ m

$\frac{b_f}{b_w} = 5.2$ $\frac{D_f}{d} = 0.17$ from SP16 Table 58.

$K_T = 0.43$

$M_{u\,lim} = K_T \cdot b_w \cdot d^2 \cdot f_{ck} = 0.43 \times 450 \times 700^2 \times 20 = 1896$ kNm

Since $M_{u\,lim} > M_u$ Safe & Single beam

$\frac{M_u}{bd^2} = \frac{291 \times 10^6}{(450 \times 700^2)} = 1.8$ from SP-16 Table 2.

$p_t = 0.392$

$A_{st} = \frac{0.392}{100} \times 450 \times 700 = 1234$ mm^2

Using #20 : No of bars $= \frac{1234}{\pi/4 (20)^2} = 3.93$

provide #20 of 4 No @ Midspan bottom.

End span : $\frac{M_{uB}}{bd^2} = \frac{234 \times 10^6}{450 \times 700^2} = 1.1$ $p_t = 0.327$

$A_{st} = \frac{0.327}{100} \times 450 \times 700 = 1030$. Using #20.

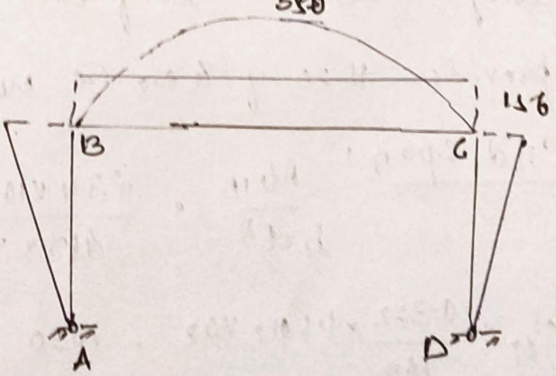
No of bar $= 3.2$

provide #20 of 4 Nos @ top in end.

Moment Distributing Table :

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	-	0.5	0.5	0.5	0.5	-
FEM	-	-	-233	233	-	-
Bal		116.5	116.5	-116.5	-116.5	-
CO.			-58.25	58.25		
Bal		29.13	29.13	-29.13	-29.13.	
CO.			-14.57	14.57		
Bal		7.29	7.29	-7.29	-7.29	
CO			-3.65	3.65		
Bal		1.83	1.83	-1.83	1.83	
CO.			-0.92	0.92		
Bal		0.46	0.46	-0.46	-0.46	
Total		155.21 (156)	-156.	156	-156.	

Bending Moment Diagram



$M_B = 156$ kN-m

$M_{uB} = 1.5 \times 156 = 234$ kN-m

Mid-Span Moment $= wl^2/8 - 156$

$= \frac{28 \times 10^2}{8} - 156 = 194$

$M_u = 1.5 \times 194 = 291$ kN-m

Max Shear @ B or C $= wl/2 = 28 \times 10/2 = 140$ kN.

$V_u = 1.5 \times 140 = 210$ kN.

Check for shear: $\tau_v = \frac{V_u}{bd} = \frac{210 \times 10^3}{450 \times 700} = 0.67$

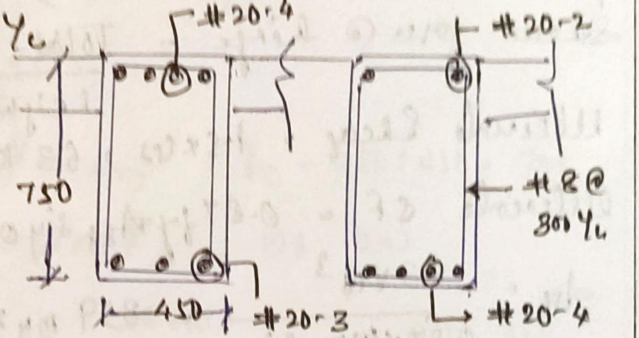
$p_t = \frac{100\, A_{st}}{bd} = \frac{100 \times 1256}{450 \times 700} = 0.39 \approx 0.4$ from Table 61.

$\tau_c = 0.432 \not< \tau_v$ provide shear reinforcement.

$V_{us} = V_u - V_{uc} = 74$ kN. $V_{uc} = \frac{0.432 \times 450 \times 750}{1000} = 136$ kN

$\therefore \frac{V_{us}}{d} = \frac{74}{70} = 1.0$ — Table 62 from SP-16.

provide #8 @ 300 mm c/c



@ support @ mid-span

Design of Column:

Ultimate axial load $= P_u = 1.5 \times 140 = 210$ kN.

Ultimate moment $= M_u = 1.5 \times 156 = 234$ kN-m

Assume cover $= 50$ mm $\therefore d'/D = 0.1$

$\frac{M_u}{f_{ck} b D^2} = \frac{234 \times 10^6}{20 \times 450 \times 600^2} = 0.07$

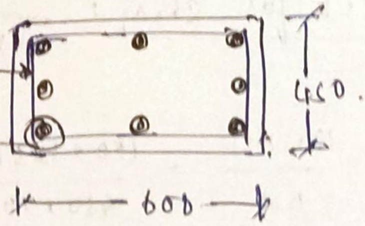
$\frac{P_u}{f_{ck} b D} = \frac{210 \times 10^3}{20 \times 450 \times 600} = 0.04$

Refer chart 22 from SP-16. $\frac{p}{f_{ck}} = 0.04$ $p = 0.8\%$

$A_{st} = \frac{0.8 \times 450 \times 600}{100} = 2160$ mm² No of bars $= \frac{A}{A_b} = 7$

provide 8 No of 20 ϕ.

provide 8 mm ϕ @ 300 c/c @ ties



Design of hinge

Permissible comp stress in concrete @ hinge = $0.8 \times 0.4 f_{ck}$ = 16 N/mm²

Factored thrust = 210 kN.

C/s area of hinge required = $\frac{210 \times 10^3}{16} = 13125$ mm²

Provide 200 × 100 (20000) for hinge.

Shear force @ hinge = $\frac{\text{Total Moment}}{\text{height of Col}} = \frac{156}{3.72} = 42$

Ultimate shear = 1.5 × 42 = 63 kN

Ultimate SF = $0.87 f_y A_{st} \sin\theta$

$A_{st} = \frac{63 \times 10^3}{0.87 \times 415 \times \sin 31^\circ} = 339$ mm²

$\tan\theta = \frac{30}{50}$

$\theta = 31^\circ$

Provide #16 - 4 Nos (A = 804 mm²)

Design of foundation,

Axial load = 140 kN

Column wt = 24 kN. ($0.45 \times 0.6 \times 3.72 \times 24$)

Self wt (10%) = 16 kN

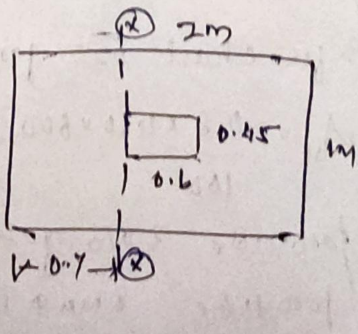
Total load = 180 kN

App area = $\frac{\text{load}}{\text{SBC}} = \frac{180}{200} = 0.9$ m²

Let assume area = 2 × 1 m

$q_{max} = \frac{P}{A} + \frac{M}{Z} = 153$ kN/m².

$q_{min} = \frac{P}{A} - \frac{M}{Z} = 27$ kN/m²



Avg pressure $q = \frac{153+27}{2} = 90 \ kN/m^2$

BM @ $\alpha - \alpha = 90 \times 1 \times 0.7^2/2 = 22$ kNm $\quad M_u = 33$

assume $D = 300$ & $d = 250$ mm

$\frac{M_u}{bd^2} = \frac{33 \times 10^6}{1000 \times 250^2} = 0.528.$

$p_t = 0.15\%$. $\quad A_{st} = 301 \ mm^2$ $\quad$ from table 2 sp16

provide #12 @ 300 c/c both ways.

<u>Check for punching shear:</u>

$a_0 = 600 + 250 = 850$ mm $\quad b_0 = 450 + 250 = 700.$

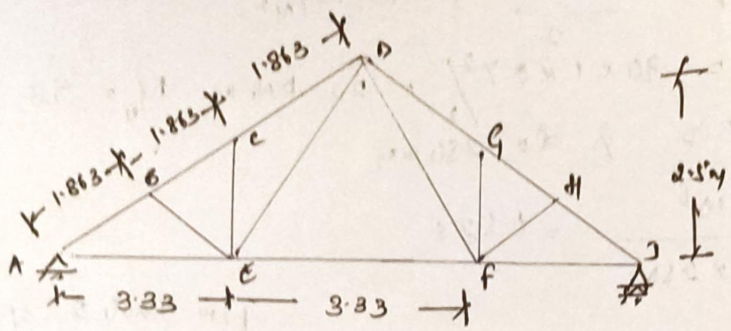
$V_p = 180 - 90 \times (0.85 \times 0.7) = 126$ kN.

$\tau_p = \frac{V_p}{2(a_0+b_0)d} = \frac{126 \times 10^3}{2(850+700)250} = 0.16$ MPa

permissible shear stress $= 0.25\sqrt{f_{ck}} = 1.118$ MPa $> \tau_p$.

Module 2

Q3.



(i) Design of Top chord. AB : compression force = 54.25 kN.

length = 1.863 m Effective length = (KL) = 0.7×L = 1.304 m

Try ISA 50×50×6 - 2 Nos placed back to back.

$KL/r = \frac{1304}{15.1} = 86.3 < 180$. from table $A = 1136\ mm^2$, $r = 15.1\ mm$

from IS 800-2007 Table 8 (c) $f_y = 250\ N/mm^2$.

$\chi = 0.56$.

$f_{cd} = \frac{\chi f_y}{\gamma_{m0}} = \frac{0.56 \times 250}{1.25} = 112\ N/mm^2$.

$\therefore P_d = A \cdot f_{cd} = \frac{1136 \times 112}{1000} = 127\ kN > 54.25\ kN$

(ii) Bottom chord AE : Tension force = 48.31 kN.

length = 3.33 m Effective length = 0.7×L = 2.33 m.

try ISA 50×50×6 - 2 Nos connected by Gusset pt of 8mm thick using #16 spaced 50mm c/c.

$A = 1136\ mm^2$. $r_{min} = 15.1\ mm$

a) Design strength due to yielding $T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$

$T_{dg} = \frac{1136 \times 250}{1.1} = 258\ kN$.

(b) Design strength governed by tearing. Cl. 6.3.3.

$T_{dg} = \alpha . A_g \frac{f_y}{\gamma_{m_1}}$ $A_g = [(50-18)(6\times2)] = 384\ mm^2$

$\alpha = 0.6$.

$T_{dg} = \frac{0.6 \times 384 \times 410}{1.25} = 75.5\ kN > 48$. hence safe.

(iii) Member DE: compressive load = 14.25 kN.

Effective length 3m. Try $50 \times 50 \times 5$ mm.

$A = 479\ mm^2$ $r_{min} = 15.2\ mm$ $\lambda = \frac{3000 \times 0.7}{15.2} = 140$.

Table 8(a) from IS 800-2007.

$\chi = 0.291$

$f_{cd} = \frac{0.291 \times 250}{1.25} = 58.2$.

$P_d = A . f_{cd} = 479 \times 58.2 = 27.8\ kN > 14.25$

hence safe.

(iv) Member DC = Tension force = 24.5 kN.

Try single $50 \times 50 \times 5$ mm. – 6mm thk Gusset plate & 2-No of 16 bolts @ 50 mm c/c.

$A = 479\ mm^2$. $r_{min} = 15.2\ mm$

Using #16 bolts $A_{nc} = (50-18)5 = 160\ mm^2$

$A_{go} = (50-5)5 = 225\ mm^2$

$A_g = 479\ mm^2$

(a) strength governed by yielding $T_{dg} = \frac{A_g . f_y}{\gamma_{m0}}$.

$T_{dg} = \frac{479 \times 250}{1.1} = 108\ kN$.

(b) strength governed by Rupture.

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m_1}} + \beta . A_{go} f_y / \gamma_{m0}$

$\beta = 0.7$ $\qquad$ $\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$

$$T_{dg} = \frac{0.9 \times 100 \times 410}{1.25} + \frac{0.7 \times 225 \times 250}{1.10} = 83.0 \text{ kN}$$

(c) Strength governed by Block shear

$A_{vn} = 5(50+50) - (1.5 \times 18) = 473 \text{ mm}^2$ $\qquad$ $A_{vg} = 5(50+50) = 500 \text{ mm}^2$

$A_{tn} = [(5 \times 25) - (0.5 \times 18)] = 116 \text{ mm}^2$ $\qquad$ $A_{tg} = 5 \times 25 = 125 \text{ mm}$

$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \left[\frac{500 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 116 \times 410}{1.25}\right] = 99.2 \text{ kN}$$

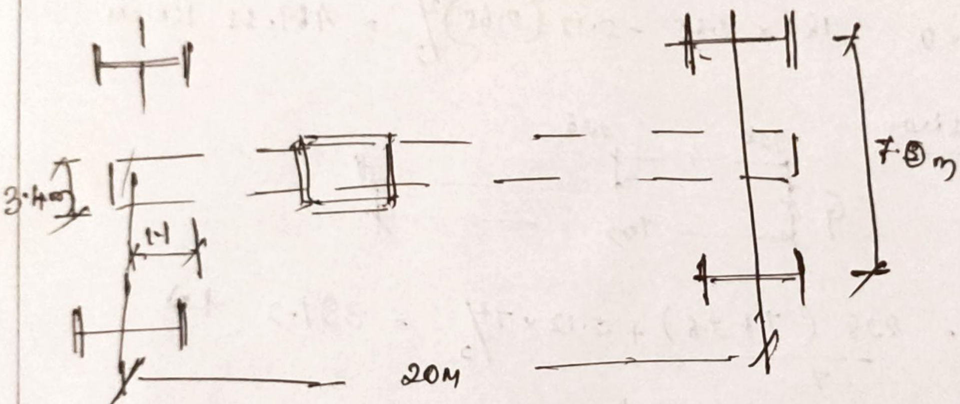
$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\cdot\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

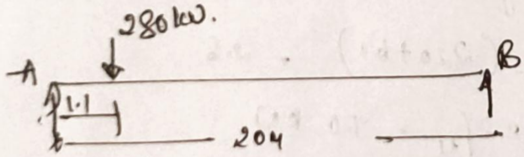
$$= \frac{0.9 \times 473 \times 410}{\sqrt{3} \times 1.25} + \frac{125 \times 250}{1.1} = 109.1 \text{ kN}$$

Among all $T_{dg} = 83.0$ is lowest > 26 hence safe.

Q1. Given Data : Span of Girder = 20m.
Span of G. Girder = 7m.

Capacity of Crane = 220 kN.
Self wt of Crane excluding Crab = 200 kN.
wt of Crab = 60 kN. wheel base distance = 3.4m
hook approach = 1.10m Self wt of Rail = 0.3 kN/m
Ht of Rail = 70mm.





$R_A + R_B = 280 + 200 = 480$ kN.

$\therefore R_A = 200/_2 + 280\frac{(18.9)}{20} = 364.6$ kN $R_B = 115.4$.

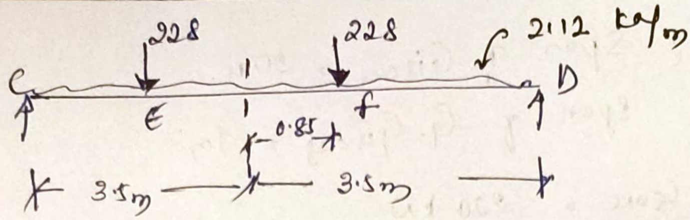
Load on each wheel $= 365/_2 = 182.5$ kN.

Add 25% Impact $= 45.62$

Total Load = 228. kN.

Self wt of Girder $w_1 = \frac{2 \times 228}{250} = 1.825$

wt of Rail = 0.3 kN/m   Total load = 2.12 kN/m



Max BM :

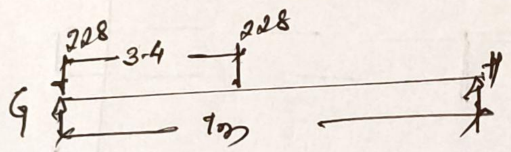
$R_C + R_D = 2 \times 228 + 2.12 \times 7.0 = 478.84$ kN

$R_D = \frac{228 \times 0.95}{7.0} + 228(3.5 + 0.85) + 2.12 \times 7^2/2 = 180$

$\therefore R_C = 290.80$ kN.

$\Sigma M_f = 0 \quad 180 \times 2.65 - 2.12 (2.65)^2/2 = 469.55$ kN-m

Max shear



$\therefore R_G = \frac{228(7 + 3.6)}{7} + 2.12 \times 7^2/2 = 397.2$ kN.

Calculation of horizontal force

Horz force on rail $= \frac{10}{100}(220 + 60) = 2.8$

load on each wheel $= 28/4 = 7.0$ kN.

factored load $= 10.5$ kN.

$\therefore M_y = \frac{10.5 \times 469.55}{228} = 21.62$ kN-m

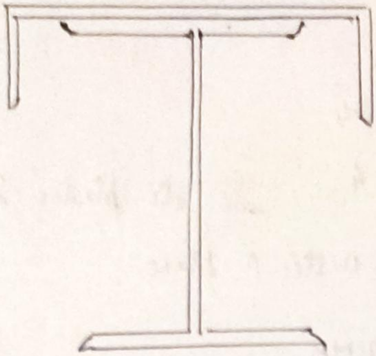
Vertical shear due to Impact $= 0.25 \times 397 = 99.25$

total shear $= 496.25$ kN

preliminary section $d = l/12 = \frac{7000}{12} = 583$ mm

$b = l/25 = \frac{7000}{25} = 280$ mm provide ISWB 600 @ 133.3

& ISMC 300.



ISWB 600		ISMC 300
17038	A	4564
250	b_f	80
$1061 98.1 \times 10^6$	I_{xx}	636.26×10^5
4702.1×10^6	I_{yy}	310.8×10^6
	C_{yy}	23.6 mm

$$\bar{y} = \frac{17038 \times 300 + 4564\,(600 + 7.6 - 23.6)}{17038 + 4564} = 360 \text{ mm.}$$

$I_{xx} = 1355.43 \times 10^6 \text{ mm}^4$

$$Z_{xx} = \frac{I_{xx}}{y} = \frac{1355.43 \times 10^6}{360} = 3.765 \times 10^6 \text{ mm}^3.$$

Compression flange @ YY $\quad I = \frac{bd^3}{12} + I_{yy} = 9136.0 \times 10^6$

$$Z_{cy} \text{ @ flange} = \frac{I_{yy}}{y} = \frac{9136 \times 10^6}{150} = 609.07 \times 10^3 \text{ mm}^3.$$

Plastic Modulus of Section :

N.A @ a distance Y_p from tension flange

$(Y_p - 21.3) \times 11.2 + (250 \times 21.3) = A/2 \quad \therefore Y_p = 510.2 \text{ mm}$

$M_p = \Sigma$ Moment of force @ yield plastic N.A :

$\therefore M_p = 4686450\, f_y.$

$\therefore Z_p = M_p / f_y = 4686450 \text{ mm}^3.$

for top flange : $\quad Z_{py} = \underbrace{\frac{bd^2}{4}}_{\text{flange}} + \underbrace{bd^2/4 \times C_q}_{\text{web}}.$

$\therefore Z_{py} = 824.76 \text{ mm}^3.$

Check for Moment Capacity

b/t of ISWB 600 = 5.6 ≤ 8.4

d/t —"— = 49.74 < 84.

b/t of channel = 6.06 < 8.4. So its plastic section

Moment Capacity of bending vertical plane.

$$M_{dz} = \frac{f_y \cdot Z_p}{1.1} = 1065.1 \times 10^6 \text{ N-mm}.$$

$$M_{dz} = \frac{1.2 Z_e \cdot f_y}{1.1} = 1026.81 \text{ kNm} \qquad \therefore M_z = 1065 \text{ kNm}$$

Top flange: $M_{dy} = \frac{f_y \cdot Z_p}{1.1} = 187.4$ kNm

$$\frac{1.2 Z_e \cdot f_y}{1.1} = 166.11 \text{ kNm}$$

Check for local Capacity:

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \le 1 \qquad \frac{704}{1065} + \frac{21.62}{166.11} = 0.66 + 0.13 = 0.73 \le 1.$$

Check for buckling resistance:

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

$$\therefore M_d = Z_p \cdot f_{bd} \qquad f_{cr} = \frac{1.1 \pi^2 E}{(L/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_T/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

$\therefore r_y = \sqrt{I_y/A} = 71.57$ mm $\qquad f_{cr} = 286$ N/mm²

from Table 13(a) $\quad f_{bd} = 159$ N/mm²

$\therefore M_{dz} = 765$ kNm >

Hence safe.

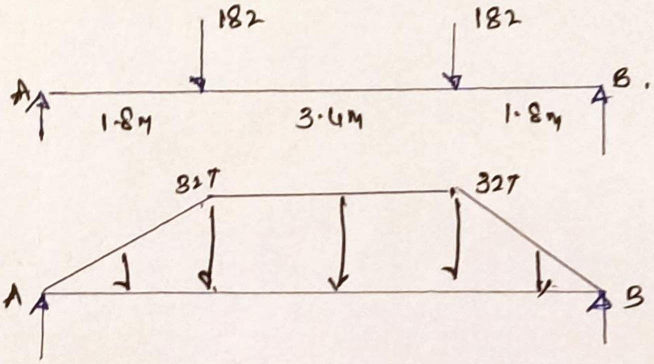
Check for Shear :

$V_z = 496$ kN.

Shear Capacity $= \dfrac{A_v \cdot f_{yw}}{\sqrt{3} \times 1.1} = 913 > 496$ hence ok

Check for Deflection :

At working load deflection $= L/750$.



Reaction @ Conjugate beam $= \frac{1}{2} \cdot$ total $\frac{M}{EI}$ Diagram

$= \left(\frac{1}{2} \times 1.8 \times \frac{327}{EI}\right) + \frac{327}{EI} \times 1.7 = 851/EI$

$EI\Delta = 851 \times 7/2 - (0.9 \times 327) \times 2.3 - 327 \times 1.7^2/2$

$EI\Delta = 1829.$

$\therefore \Delta = \dfrac{1829}{200 \times 1127.42} \times 10^{-3} = 8.14$ mm

Actual Allowable $= L/750 = \dfrac{7000}{750} = 9.33$

$\therefore$ Defletion is satisfied.

07-04-2022

Dean, Academics
KLS VDIT, HALIYAL

(Gr.C)