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CBCS SCHEME

USN

18CV72

Seventh Semester B.E. Degree Examination, July/August 2022
Design of RCC & Steel Structures

Time: 3 hrs. Max. Marks: 100

Note: 1. Answer any TWO full questions, choosing ONE full question from each module.
2. Use of IS456, IS-800, SP(6) and steel tables are permitted.

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.

Module-1

1 Design a combined footing slab beam type for two RCC column A and B separated by a distance of 4 m C/C. Column A is 500×500 mm and carries a load of 1250 kN and column B is 600×600 mm and carries a load of 1600 kN. Take SBC of soil is 200 kN/m^2. Use M20 concrete and Fe415 steel. Draw the sectional elevation. **(50 Marks)**

OR

2 Design a Cantilever retaining wall to retain earth embankment 5 m high above ground level. The density of earth is 18 kN/m^3 and its angle of repose is 30°. The embankment is horizontal at its top. The SBC of may be taken as 200 kN/m^2 and the co-efficient of friction between soil and concrete is 0.5. Adopt M20 grade concrete and Fe415 steel. Also draw cross sectional elevation showing reinforcement details. **(50 Marks)**

Module-2

3 A line diagram of a roof truss with internal loads and forces in each member are shown in Fig. Q3. Design the various members of the roof truss along with their end connection with bolt of property class 5.6. Also design the bearing plate at support for the reaction and anchor bolts for an uplift force of 15 kN. Also draw the Elevation of truss greater than half plan.

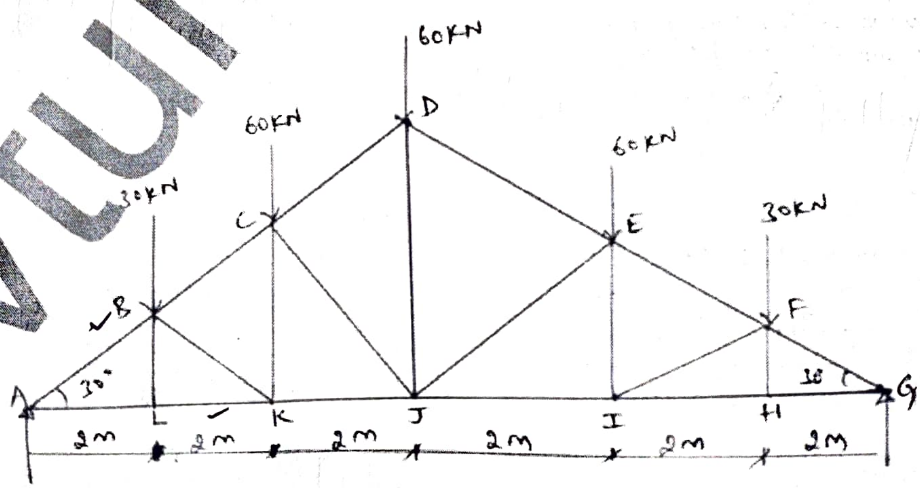

Fig. Q3

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Members	Length (m)	Force (kN)	Nature of force
AB, GF	2.31	240	C
BC, FE	2.31	210	C
CD, ED	2.31	160.04	C
AL, GH	2.0	207.84	T
LK, HI	2.0	207.84	T
KJ, IJ	2.0	181.82	T
BL, FH	1.154	0	-
BK, FI	2.31	30	C
CK, EI	2.31	15	T
CJ, EJ	3.05	66	C
DJ	3.46	60	C

(50 Marks)

OR

4 Design a simply supported gantry girder to carry an electrically operated crane with the following data:

Span of Crane bridge = 25 m
Span of gantry girder = 8 m
Wheel base = 3.5 m
Crane capacity = 200 kN
Weight of crane bridge = 150 kN
Weight of trolley = 75 kN
Min Hook distance = 1.0 m
Weight of rail = 0.30 kN/m
Height of rail = 105 mm
Draw the sectional elevation.

(50 Marks)

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Subject : Design of RCC & Steel Structures (18CV72)

Q1 Given Data : Col A - 500 x 500 mm - 1250 kN.
Col B - 600 x 600 mm - 1600 kN.

C/c between A & B = 4 m. M20 & Fe415 steel.
SBC of soil = 200 kN/m²

Total load A + B = 2850
10% of self wt = 285
3135 kN.

Area of footing = $\frac{Load}{SBC} = \frac{3135}{200} = 15.67\ m^2$.

Assume width = 2.2 m. $L = \frac{A}{B} = \frac{15.67}{2.2} = 7.12\ m$

Provide Area = 7.2 x 2.2 m = 15.84 m².
CG of footing coincide with CG of column load.

$x = \frac{1600 \times 4.0}{(2850)} = 2.24\ m$ from col A.

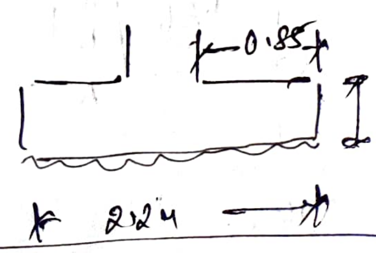
Project beyond A is at 2.24 = 7.2/2 a = 1.36 m

Project beyond B is b = 7.2 - (4 + 1.36) = 1.84 m

Ultimate upward pressure = $\frac{Load}{Area} = \frac{2850}{7.2 \times 2.2} = 180\ kN/m^2$

Design of bottom slab :

Cantilever projection = 0.85 m
Moment = 180 x 0.85²/2 = 65 kNm



Depth = $\sqrt{\frac{65\times10^6\times1.5}{2.76\times1000}} = 187\ mm$ For M20 & Fe415

$D = 190 + 50 + 10 = 250\ mm$ $d = 200\ mm$

Steel: $\frac{M_u}{bd^2} = \frac{97.5\times10^6}{1000\times200^2} = 2.43$.

$P_t = 0.81\%$. $A_{st} = 1620\ mm^2$ | SP 16 Table 2.

Using #16 $sp = \frac{201\times10^3}{1620} = 124\ mm$ $P_t\ (0.83\%)$

Provide #16 @ 120 mm c/c

Check for Oneway shear: $V_u = 180\times(0.85-0.2) = 117\ kN$

$\tau_v = \frac{117\times10^3}{1000\times200} = 0.57\ N/mm^2$. $\tau_c = 0.67$ hence ok

hence increase the depth of 50mm $D = 300$ & $d = 250\ mm$

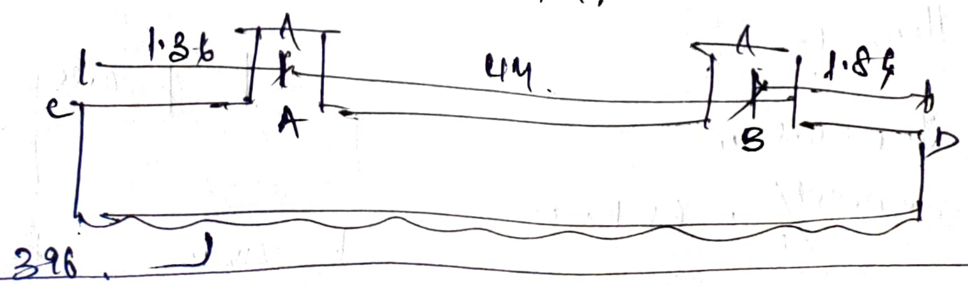
Distribution steel: $A_{st} = \frac{0.15\%\ GA}{} = \frac{0.15}{100}\times10^3\times300 = 450\ mm^2$

using #10 $sp = \frac{78.5\times10^3}{450} = 174\ mm$

provide #10 @ 170 mm c/c.

Design of Longitudinal Beam:

$w_u = 180\times2.2 = 396\ kN/m$.



portion AC $V_{AC} = 538$ kN $V_{AB} = 711$ kN.

$V_{BD} = 729$ kN $V_{BA} = 871$ kN,

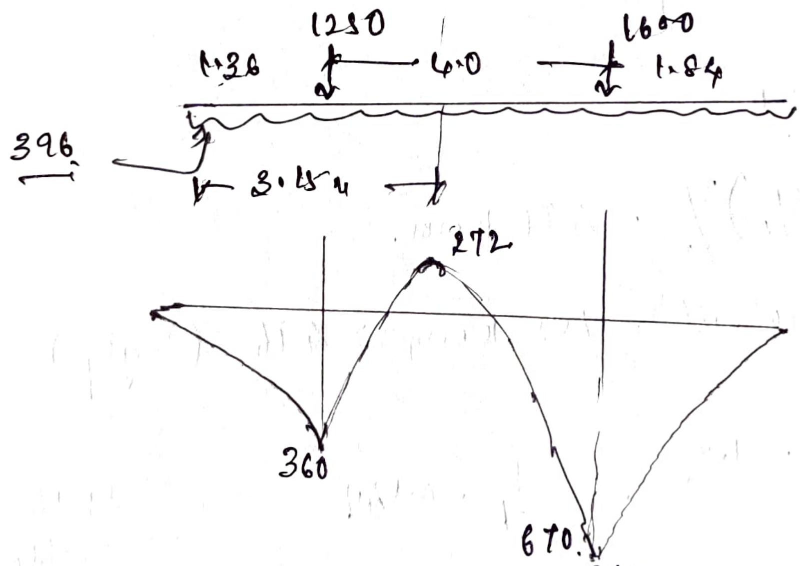
pt of zero shear from C

$x = \frac{1250}{396} = 3.15$ from C & 4.04 from D.

Max BM @ F $= 396 \times 3.15^2/2 - 1250\,(3.15 - 1.36) = -272$ kNm

BM @ A pt $= 396 \times 1.36^2/2 = 366$ kNm

— B $= 396 \times 1.84^2/2 = 670$ kNm.



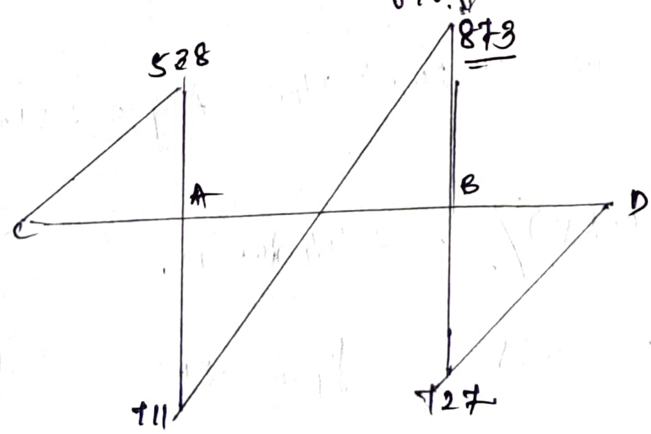
Check depth for Mom

$d = \sqrt{\frac{670 \times 10^6}{2.76 \times 1000}}$

$= 492$ mm

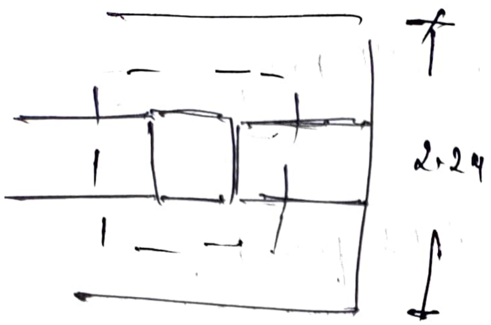
provide $D = 750$ mm

$d = 690$ mm



Check for Two way shear

$B = 600 \times 600$, $d_b = 690$, $d_s = 250$,



$(= D + d_b)$

Area resisting $= 2(600 \times 690 + 200 \times 200) + 2(600 + 690) \times 200$,

$\frac{A}{(b_o d)} = 908000 + 516000 = 1424000$

$\tau_v = \frac{P_u}{b_o d} = \frac{1600 - 180 (60 \cdot 00)}{1424000} = 0.99$

$\tau_{uc} = \tau_{uc} \times k_s = 1.11 \times 1 = 1.11$ $\quad \tau_{uc} = 0.25\sqrt{f_{ck}} = 1.11$

$\therefore \tau_{uc} > \tau_v$ hence safe

Area of steel : portion BD

$M_u = 396 \times (1.84 - 0.6/2)^2/2 = 496$ kNm

$M_{ulim} = 2.76 \times 600 \times 690^2 \times 10^{-6} = 788$ kNm > 496 (Singly)

$\frac{M_u}{bd^2} = \frac{498 \times 10^6}{(600 \times 690^2)} = 1.73$. $p_t = 0.547\%$ SP 16 Table 2.

$A_{st} = 2286$ mm² provide # 20 - 8 Nos $A_{sp} = 2512$ mm²

portion AC :-

$M_u = 396 \times (1.36 - 0.25)^2/2 = 244$ kNm

$\frac{M_u}{bd^2} = \frac{244 \times 10^6}{(600 \times 690^2)} = 0.85$ $\quad p_t = 0.25$ (SP-16 Table 2)

$A_{st} = 1035$ mm², provide # 16 - 6 Nos $A_{sp} = 1206$ mm²

Design between AB

$$\frac{M_u}{bd^2} = \frac{872 \times 10^6}{600 \times 688^2} = 0.95$$

Sp-16 Table 2.

$p_t = 0.28\%$. $A_{st} = 1160\,mm^2$.

Using # 12 mm $S_p = \frac{\pi/4 (12)^2 \times 10^3}{1160}$

provide ϕ 16 @ 7 Nos — $A_{st_p} = 1407\,mm^2$. p_t (0.299)

Design of Shear Reinforcement :

portion AB : $V_{max} = 873$

Shear @ column edge $873 - 396 \times 0.3 = 754$ kN.

$$\tau_v = \frac{754 \times 10^3}{600 \times 690} = 1.82\ MPa \qquad \tau_c = 0.42$$

Sp-16 Table 61.

provide Shear Reinforcement.

$$V_{us} = \frac{754}{69} - 0.42(600 \times 690) = 580\ kN.$$

$$\frac{V_{us}}{d} = \frac{580}{68} = 8.4$$

Sp-16 Table 62

provide 10 mm ϕ - 4 legged @ 180 mm c/c

portion BD :

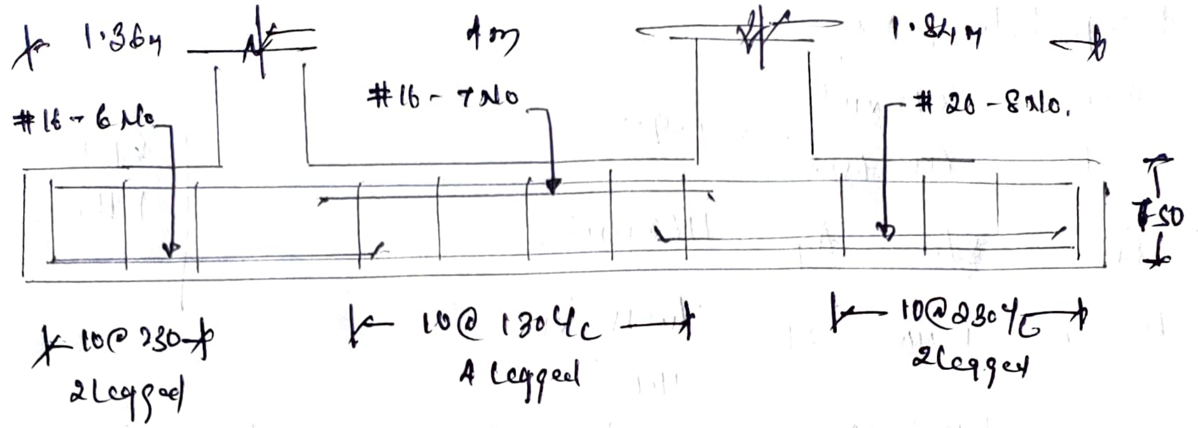
Shear @ BD : $827 - 396 (0.6/2 + 0.69) = 335$ kN.

$$\tau_v = \frac{335}{600 \times 690} = 0.81. \qquad \tau_c = 0.42.$$

$$V_{us} = 335 - 0.42(600 \times 690) = 161\ kN$$

$$\frac{V_{us}}{d} = \frac{161}{69} = 2.33$$

provide 10 mm ϕ 2 legged @ 280 mm c/c



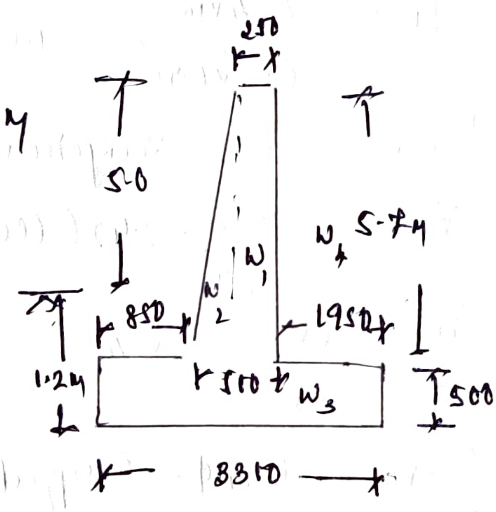
(2). Given Data: $h' = 5M$ $\gamma = 18\ kN/M^3$ $\phi = 30°$

SBC = 200 kN/M^2 $\mu = 0.5$ M-20 & Fe-415 steel.

Depth of foundation:

$$D_f = \frac{SBC}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 = 1.23 \simeq 1.2M$$

$H = 5+1.2 = 6.2M$.



Thk of slab $= \left(\frac{1}{10} + \frac{1}{14}\right)H = 0.5M$

width of slab $= (0.5\ to\ 0.6)H = 3.3M$

toe projection $= \left(\frac{1}{3}\ to\ \frac{1}{4}\right)B = 0.85M$

Stability analysis:

		Magnitude	LA	Moment
W_1 →	$0.25 \times 5.7 \times 25$	= 35.62	1.225	43.63
W_2 →	$\frac{0.25}{2} \times 5.7 \times 25$	= 17.81.	1.016	18.10.
W_3 →	$3.3 \times 0.5 \times 25$	= 41.25.	1.65	68.06.
W_4 →	$5.7 \times 1.95 \times 18$	= 200.10.	2.325	465.00.
		294.68		594.79

P_H = 115 kN. 2.067 237.66

Check for Overturning : $\frac{M_R}{M_O} = \frac{594}{237} = 2.51 > 1.55$ Safe

Sliding : $\frac{\mu \Sigma W}{P_H} = \frac{0.5 \times 294.7}{115} = 1.28 < 1.55$ provide Shear key of 300 × 300 below the stem part.

Check for Subsidence :

$P_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6 \times e}{b}\right]$

$e = b/2 - x$

$x = \frac{\Sigma M}{\Sigma W} = \frac{358}{295} = 1.21.$

$= \frac{295}{3.3}\left[1 + \frac{6 \times 0.66}{3.3}\right] = 161.$

$e = 3.3/2 - 1.21 = 0.44 \ngtr b/6$

$P_{min} = \frac{\Sigma W}{b}\left[1 - \frac{6 \times e}{b}\right] = 18$ kN/m²

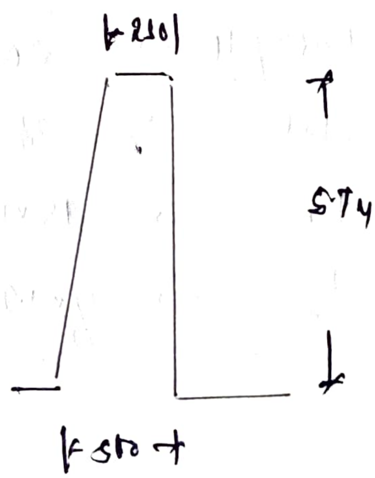
@ D =

E =

Design of stem :

$P_H = 1/2 \times K_a \cdot \gamma \cdot H^2$

$= 1/2 \times 1/3 \times 18 \times 5.7^2 = 97.5$ kN.



$M = P_H \cdot H/3 = 97.5 \times \frac{5.7}{3} = 185$ kNm

$M_u = 278$ kNm.

$\frac{M_u}{bd^2} = \frac{278 \times 10^6}{1000 \times 450^2} = 1.372$

$p_t = 0.415\%$ $A_{st} = 1867$ mm².

Using #16. $S_p = 105$ mm c/c.

$A_{st}(p) = 0.45\%$

Curtail : h_c = 1.03 from top.

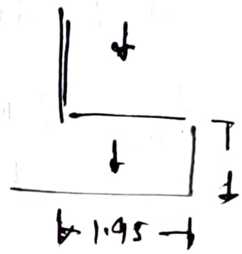
16 @ 100 upto 1.5 m.

& # 16 @ 200 from 1.5 to 3.0 m.

16 @ 300 from 3.0 to 5.7 m.

Distributory / Secondary steel (Temp)

<u>0.12% of GA</u> = $\frac{0.12}{100} \times 10^3 \times 500$ = 600 mm².



10 @ 125 mm c/c.

<u>Check Shear</u>: V_u = 97.5 × 1.5 = 146.25

$\tau_v = \frac{V_u}{bd} = \frac{146 \times 1000}{1000 \times 450}$ = 0.32. τ_c = 0.45 | SP 16 Table 61.

<u>hence ok</u>

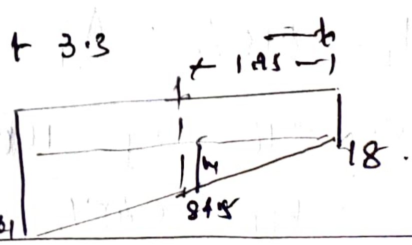
<u>Design of heel</u>

<u>Load</u>	Magnitude	LA	moment
Backfill →	200	0.975	195
Slab →	24.375	0.975	23.76
P_1 →	18 × 1.95 = 35.1	0.975	− 34.22
P_2 →	½ × 1.95 × 84.5 = 82.38	1.95/3	− 53.54
	+100.9		+131 kN/m

M_u = 131 × 1.5 = 196 kNm

$\frac{M_u}{bd^2} = \frac{196 \times 10^6}{1000 \times 450^2}$ = 0.97

$\frac{143}{3.3} = \frac{h}{1.95}$



p_t = 0.295%.

$$A_{st} = \frac{0.295 \times 1000 \times 450}{100} = 1327\ mm^2$$

#16 @ 150mm c/c

Check shear: $V_u = 107 \times 1.5 = 160.5$

$$\tau_v = \frac{V_u}{bd} = \frac{160.5 \times 10^3}{1000 \times 450} = 0.356 \qquad \tau_c = 0.39$$

SP16 Table 61

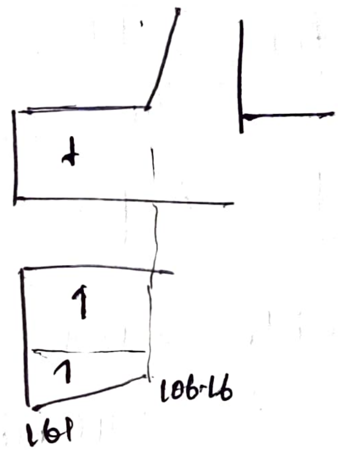
Hence Safe

Design of toe:

$$\frac{143}{3.3} = \frac{h}{2.45}$$

$h = 106.16$

load	magnitude	LA	moment
w_1 -	$0.85 \times 0.5 \times 25 = 10.62$	0.425	4.51
P_1 -	106.16 (90.23) × 0.425		90.23
P_2 =	$\frac{0.85}{2} \times \frac{54.84}{23.31}$	0.283	−6.596
			92.31

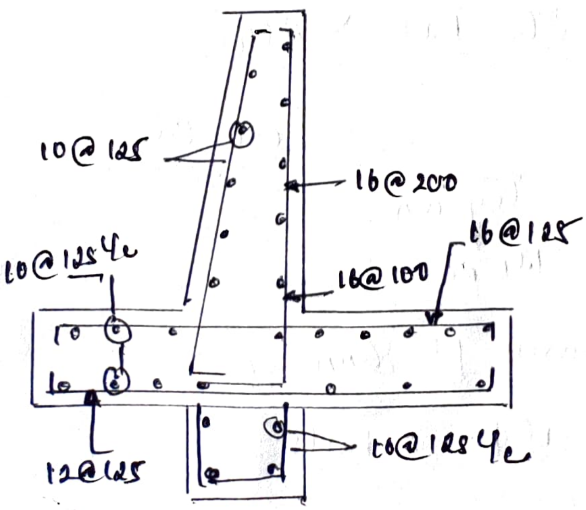


$M_u = 92.31 \times 1.5 = 138.46$ kNm

$$\frac{M_u}{bd^2} = \frac{138.46 \times 10^6}{1000 \times 450^2} = 0.683$$

$p_t = 0.187$ $A_{st} = 842\ mm^2$

provide #12 @ 125 mm c/c



Q3 Design of member

Top chord AB, BC & CD = 240 kN is max in comp.
length = 2.31 m
Effective length = 0.7 × 2.31 = 1.617 m
Try 2 ISA 65×65×8 placed back to back.

$A = 2 \times 976 = 1952\ mm^2$ $\quad r_{min} = 19.6\ mm$

$\lambda = \frac{kL}{r_{min}} = \frac{1617}{19.6} = 82.5 < 180.$

$\alpha = 0.58$ from IS 800-2007 table 8(c)

$f_{cd} = \frac{\alpha f_y}{\gamma_{m0}} = \frac{0.58 \times 250}{1.25} = 116\ N/mm^2$

$P_d = A \times f_{cd} = \frac{1952 \times 116}{1000} = \cancel{138}\ 226\ kN < 240.$
hence unsafe

provide 2 ISA 70×70×8

$A = 2116\ mm^2$ $\quad r_{min} = 21.2\ mm$

$\therefore \lambda = \frac{kL}{r_{min}} = 76.7 < 180$ $\quad \therefore \alpha = 0.63.$

$f_{cd} = \frac{0.63 \times 250}{1.25} = 126\ N/mm^2.$

$P_d = A \times f_{cd} = \frac{2116 \times 126}{10^3} = 266\ kN > 240$

hence ok

Member AL, LK & KJ = 207.84 Tension.
length = 2m effective length = 1.4 m
Try 2 ISA 65×65×8 mm by Gusset pl 8 mm &
16φ - 2 bolts.

$A = 1952\ mm^2$ $\quad r_{min} = 19.6\ mm$

i) Yield strength $\quad T_{dg} = \dfrac{A_g \cdot f_y}{\gamma_{m0}} = \dfrac{1952 \times 250}{1.1} = 443 \text{ kN}.$

ii) Rupture strength $\quad T_{dy} = \dfrac{\alpha \cdot A_n \cdot f_u}{\gamma_{m1}}$

$A_n = \left[(65 - 18)(8 \times 2) \right] = 752 \text{ mm}^2$

$T_{dy} = \dfrac{0.9 \times A_{nc} \cdot f_u}{\gamma_{m1}} + \dfrac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$ | cl 6.3.3 - 1S 800 - 2007

$A_{nc} = (65 - 18)\, 8 = 376 \text{ mm}$ Connected leg

$A_{go} = (65 - 8)\, 8 = 456 \text{ mm}$ Outstanding leg

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

$= 1.4 - 0.076 \left(\frac{65}{8}\right)\left(\frac{250}{410}\right)\left(\frac{65 + 30}{65}\right) =$

$= 1.4 - 0.076\,(8.125)\,(0.61)\,(1.461) = 0.85$

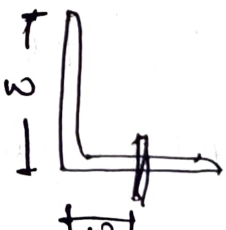
$T_{dy} = \dfrac{0.9 \times 376 \times 410}{1.25} + \dfrac{0.85 \times 456 \times 250}{1.1} = 210 > 207$

hence ok

iii) Strength of block shear :

$A_{vg} = 8(65 + 65) = 1040 \text{ mm}^2.$

$A_{vn} = 8(65 + 65) - (1.5 \times 18) = 1013 \text{ mm}^2$



$A_{tg} = 8 \times 32.5 = 260 \text{ mm}^2$

$b_s = w + w_1$

$A_{tn} = (8 \times 32.5 - 0.5 \times 18) = 251 \text{ mm}^2$

$T_{db1} = \dfrac{A_{vg} \cdot f_y}{\sqrt{3}\, \gamma_{m0}} + \dfrac{0.9 \times A_{tn} \cdot f_u}{\gamma_{m1}}$

$= \dfrac{1040 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 251 \times 410}{1.25} = 210 > 207 \text{ kN}$

hence ok

Member CJ : comp = 66 kN   length = 3.05   eff = 2.13 m

Try 2 ISA 50×50×6 mm placed back to back

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

$$\lambda = \frac{KL}{r_{min}} = \frac{2130}{15.1} = 141 < 180.$$

from IS 800-2007 cl 7.1.2.1

$\alpha = 0.296$

$$f_{cd} = \frac{\alpha \cdot f_y}{\gamma_{m0}} = \frac{0.296 \times 250}{1.25} = 59.2\ N/mm^2$$

$P_d = A \cdot f_{cd} = 1136 \times 59.2 = 67.25 > 66$ hence safe.

Member BK   comp 30 kN $l = 2.31$ eff = 1.617 m

Try 65×65×8.

$A = 976\ mm^2$ $r_{min} = 19.6\ mm$

$$\lambda = \frac{KL}{r_{min}} = \frac{1617}{19.6} = 82.5 < 180.$$

from IS 800-2007 cl 7.1.2.1

$\alpha = 0.56.$

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

$P_d = A \cdot f_{cd} = 976 \times 112 = 109\ kN > 30$ //

Design of end bearing :

Reaction @ A = 30 + 60 + 30 = 120 kN.

Bearing area = $\frac{120 \times 10^3}{0.8} = 150,000\ mm^2$ (800 kN/m² Bearing pressure assumed)

Assume 300 wall, provide 250 mm width of base plate

length of plate = $\frac{150000}{250} = 600\ mm$

Provide 600 × 250 mm base plate

Anchor bolts, uplift = 15 kN

provide 2 bolts of 16 mm ϕ.

uplift is balanced by wt of masonry activated by bolt length (d).

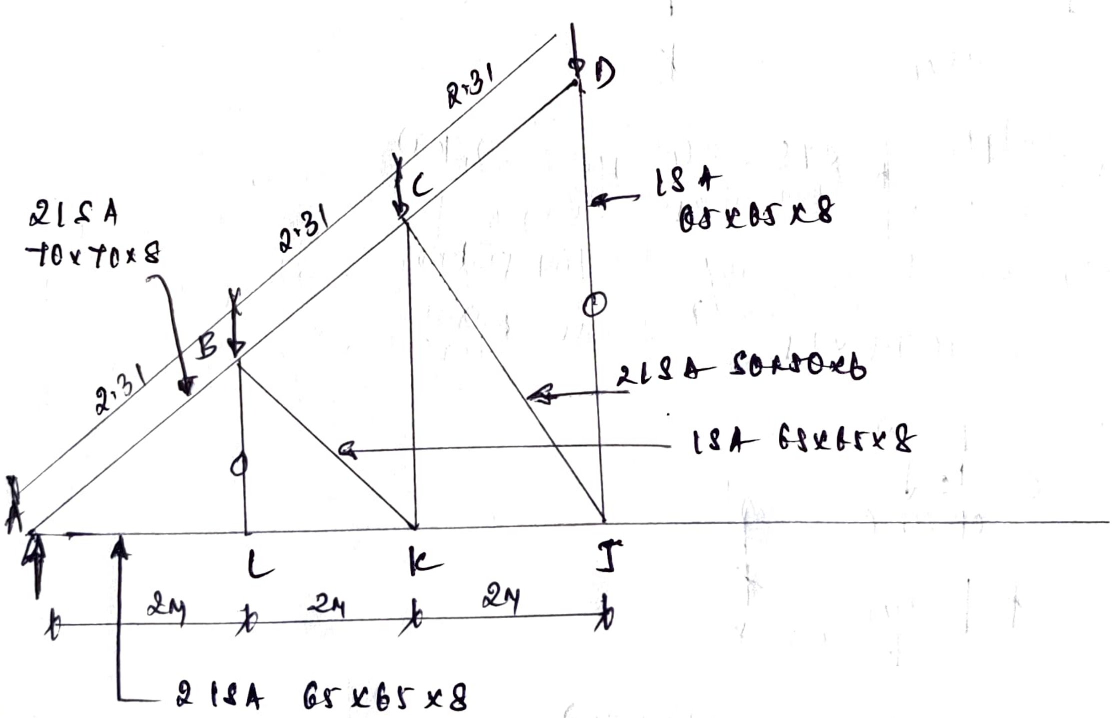
Assume dispersion along masonry @ 45°.

$$\text{wt of blocks} = \left[0.6 + (0.6 + 2d)\right] \times \frac{d}{2} \times 0.3 \times 16$$

$$= 2.88\,d + 4.8\,d^2 = 15\,000.$$

$\therefore d =$

Hence embed bolt in masonry by ___ cm.

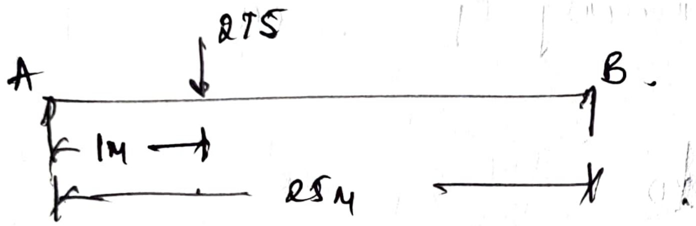


(4)

Span of GG = 8m
Wheel base = 3.5m
Wt of trolley = 75 kN
Hook dist = 1.0m

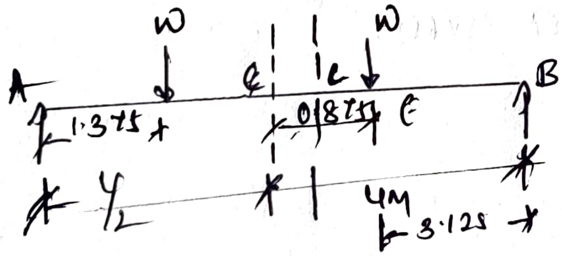
Span of bridge = 25m
Crane Capacity = 200 kN
Wt of Crane bridge = 180 kN.
Wt of rail = 0.3 kN/m
Ht of rail = 105 mm.

Wt of Capacity + trolley = 200+75 = 275 kN.
Self wt = 250 kN



Load on $R_A = \frac{180}{2} + \frac{275 \times 24}{25} = 339$ kN

Load on each wheel = $339/2$ = 169.5 kN
Factored load = 169.5×1.5 = 255 kN.



$R_B = \frac{255 \times 1.375 + 255\ (4+0.875)}{8} = 200$ kN

Max BM @ C = 200×3.125 = 625 kN.
Impact (moment) = 0.25×625 = 156 kN.
Self wt of rail = 0.3 kN/m
Self wt of Girder = 2.0.
2.3 kN/m

Factored DL = 2.3×1.5 = 3.45 kN/m
Moment due to DL = $3.45 \times 8^2/8$ = 27.6 kNm

Total moment $= M_x = 625 + 156 + 27.6 = 809$ kNm

Max Moment due to horz forces :

Horz force to rail = 10% of wt of trolley + load lifted.

$= \frac{10}{100}(275) = 27.5$ kN.

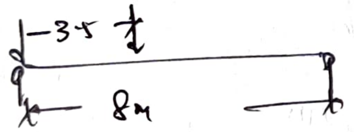
load on each wheel $= 27.5/4 = 6.875$ kN

factored horz force $= 1.5 \times 6.875 = 10.31$ kN.

Max Moment in $M_y = \frac{10.31}{255} \times 625 = 25.27$ kNm.

Shear forces

Vertical shear force $= 255 + \frac{255 \times 4.5}{8} = 398.6$ kN.



Impact $= 0.25 \times 398.6 = 99.65$ kN.

Vertical shear due to self wt $= 3.45 \times 8/2 = 13.8$ kN.

Total vertical shear $= 398.6 + 99.65 + 13.8 = 512$ kN

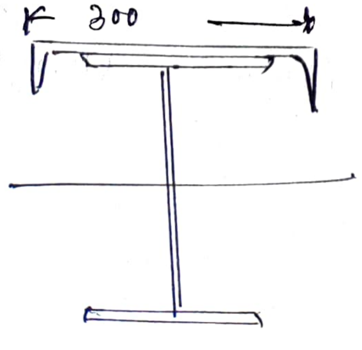
lateral shear $= \frac{10.31 \times 512}{255} = 20.7$ kN.

Primary Sections :

Depth $= L/12 = \frac{8000}{12} = 666$ mm

flang width $= L/25 = 320$ mm

Choose ISWB 600 @ 133. with ISMC 300.



	ISWB 600	ISMC 300
A =	17038	4564
b_f =	250	80.
I_{xx} =	106198.5×10^4	6362.6×10^6
I_{yy} =	4702.8×10^6	310.8×10^4
C_y =		23.6 mm

N A Section from Extreme fibre of tension flange ($\bar{Y}$)

$$\bar{Y} = \frac{17038 \times 300 + 4564\,(600 + 7.6 - 23.6)}{17036 + 4564} = 360 \text{ MM}$$

$$I_{xx} = 106198.5 \times 10^4 + 17038\,(60)^2 + 310.8 \times 10^4 + 4564\,(224)^2$$

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

$$\therefore Z_{ec} = \frac{1355.43 \times 10^6}{360} = 376.50 \times 10^6 \text{ MM}^3.$$

<u>Compr flange @ YY</u>

$$I = \frac{bd^3}{12} + I_{yy} = \frac{21.3 \times 250^3}{12} + 6302.6 \times 10^4$$

$$I_{x2} = 9136.03 \times 10^4 \text{ MM}^4$$

$$Z_{yy} \text{ @ flange} = \frac{913 \times 10^4}{150} = 609.07 \times 10^3 \text{ MM}^3.$$

<u>Plastic Modulus of Section</u>

NA at a distance Y_p from tension flange

$$(Y_p - 21.3) \times 11.2 + (250 \times 21.3) = A/2 \qquad (A = A_1 + A_2)$$

$$\therefore Y_p = 510.2 \text{ MM}$$

M_p = Σ Moment of force @ Yield plastic N A

$$= (21.3 \times 250)\,(510.2 - 21.3/2) \cdot f_y + (510.2 - 21.3)^2/2 \times 11.2 \times f_y$$

$$+ (600 - 21.3 - 510.2)^2/2 \times 11.2\, f_y + (21.3 \times 250) \cdot (600 - \tfrac{21.3}{2} - 510.2)$$

$$+ 4564\,(600 + 13.6 - 510.2 - 23.6) \cdot f_y$$

$$\therefore M_p = 4686450 \cdot f_y$$

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

For top flange :

$Z_{py} = bd^2/4 + bd^2/4 + CC_1$.

$Z_{py} = \frac{250^2 \times 21.3}{4} + \frac{1}{4}(300 - 2 \times 13.6)^2 \times 7.6 + 2(90 \times 13.6)(150 - 13.6/2)$

$Z_{py} = 824.76 \times 10^3 \text{ mm}^3$

Check for Moment Capacity

b/t for ISWB 600 $= \frac{250 - 11.2}{2 \times 21.3} = 5.6 < 8.4$

d/t for ISWB 600 $= \frac{600 - 2 \times 21.3}{11.2} = 49.76 < 84$

b/t for channel $= \frac{90 - 7.6}{13.6} = 6.06 < 8.4$

It is a plastic section

Moment capacity in bending vertical plane

$M_{dz} = \frac{f_y \cdot Z_p}{1.1} = \frac{250 \times 4686450}{1.1} = 1065.1 \times 10^6 \text{ Nmm}$

$\frac{1.2 \cdot Z_e \cdot f_y}{1.1} = \frac{1.2 \times 378.5 \times 10^4 \times 250}{1.1} = 1028.81$

$\therefore M_{dz} = 1065 \text{ kNm}$

top flange : $M_{dy} = \frac{f_y \cdot Z_y}{1.1} = \frac{250 \times 824.76 \times 10^3}{1.1} = 187.4 \text{ kNm}$

$\frac{1.2 Z_e \cdot f_y}{1.1} = \frac{1.2 \times 609.07 \times 10^3 \times 250}{1.1} = 166.11 \text{ kNm}$.

Check for local capacity

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} \le 1.$$

$$\frac{809}{1065} + \frac{25.27}{166.11} = 0.759 + 0.152 = 0.91 \le 1.0$$

Check for buckling resistance:

(cl. 8.2.2.1)

$M_d = \beta_b \cdot Z_p \cdot f_{bd}$ $\beta = 1.0$ - plastic section

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

$$f_{cr} = \frac{1.1 \pi^2 E}{(l_{LT}/r_y)^2} \left[1 + \frac{1}{20}\left[\frac{(l_{LT}/r_y)}{(h_f/t_f)}\right]^2\right]^{0.5}$$

$r_y = \sqrt{I_y/A}$ $I_{yy} = I_{yy} + I_{xx}$ (channel)

$= 11065.1 \times 10^6$

$$\therefore r_y = \sqrt{\frac{11065.1 \times 10^6}{21602}} = 71.57 \text{ mm}$$

from table 13 (a) IS 800-2007

$f_{cr} = 323.06 \text{ N/mm}^2$

$f_{cb} = 107.8 \text{ N/mm}^2$

$M_{dx} = 1.0 \times 107.8 \times 1168 \times 10^{?}$ $= 786 \times 10^6$ $\ngtr 809$ factored (1.5) value

(540) unfactored value

$\boxed{786 > 702}$ factored value (1.3 l_c)

hence safe

Check for shear

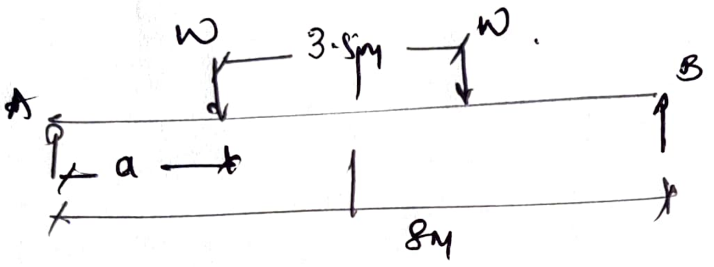
$V = 512$ kN

Shear capacity: $\frac{A_v \cdot f_{yw}}{\sqrt{3} \times 1.1} = \frac{600 \times 11.2 \times 250}{\sqrt{3} \times 1.1} = 913 > 512$

hence ok

Check for deflection :

$$\delta_{max} = \frac{WL^3}{6EI}\left[\frac{3a}{4L} - \frac{a^3}{L^3}\right]$$



$a = (8-3.5)/2 = 2.25\,m$

$L = 8m.$

$W = 170\,kN.$

$I = 1.35\times10^9$

$E = 2\times10^5.$

$$\therefore \delta_{Max} = \frac{170\times8000^3\times10^3}{6\times2\times10^5\times1.35\times10^9}\left[\frac{3(2250)}{4(8000)} - \frac{(2250)^3}{(8000)^3}\right]$$

$$\delta_{Max} = 9.96\,mm < L/750\left[\frac{8000}{750}\right] = 10.67\,mm$$

Hence Safe.

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