Prob. Design an RC T-beam girder bridge to suit following data:

Clear width of road way = 7.5 m ✓

Span (c/c of bearings) = 16m / 20m ✓

Live load = IRC class AA tracked vehicle ✓

Avg thickness of wearing coat = 80 mm ✓

No. of main girders = 4 ✓

Spacing of " " = 2.5 m ✓

" " cross " = 4 m ✓

Width of kerb = 600 mm ✓

Materials : M25 ✓ & Fe415 ✓

Using Courbon's method, compute the design moments and shears, design the deck slab, main girder, and cross girders and sketch the typical details of reinforcements.

Soln. Step (1) : Data

Eff. span of T-beams = 16 m

Width of carriageway = 7.5 m

Thickness of w/coat = 80 mm

Materials = M25 & Fe415

Step (2) : Permissible stresses

$\sigma_{cb} = 8.3\ N/mm^2$ $\quad m = 10$

$\sigma_{st} = 200\ N/mm^2$ $\quad j = 0.9$

$\varphi = 1.1$

Step (3) : Cross-Section of Deck

3 main girders are provided @ 2.5 m c/c

Thickness of deck slab = 200 mm

Wearing coat = 80 mm

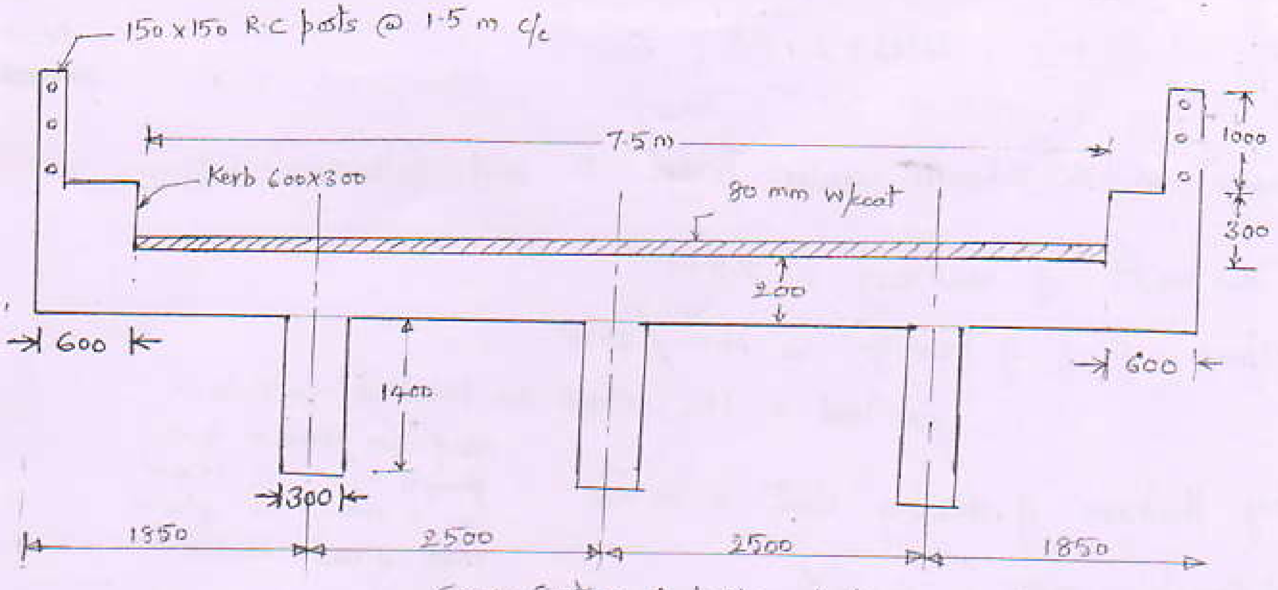
Width of main girder = 300 mm

Kerbs 600 mm wide × 300 mm deep are provided.

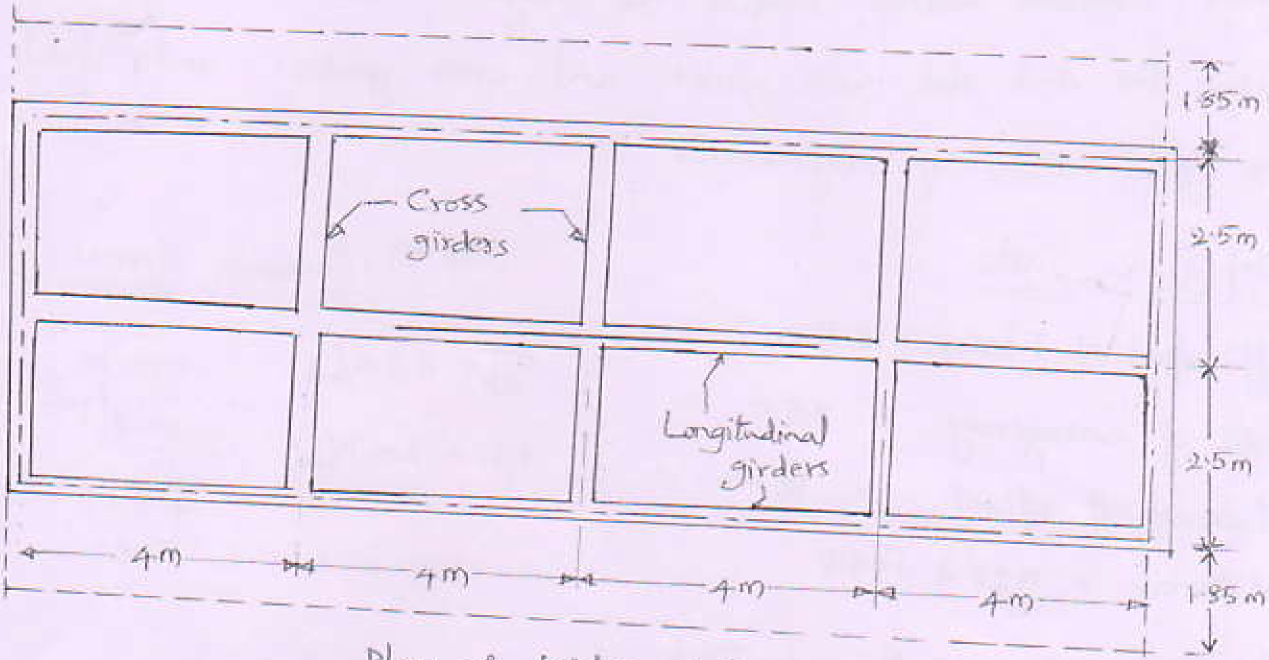
Cross girders are provided at every 4 m interval.

Breadth of cross girder = 300 mm

Depth of main girder = 1600 mm at the rate of 100 mm per metre of span.



Cross Section of bridge deck.



Plan of bridge deck.

Depth of cross girder is taken as equal to depth of main girder to simplify computations.

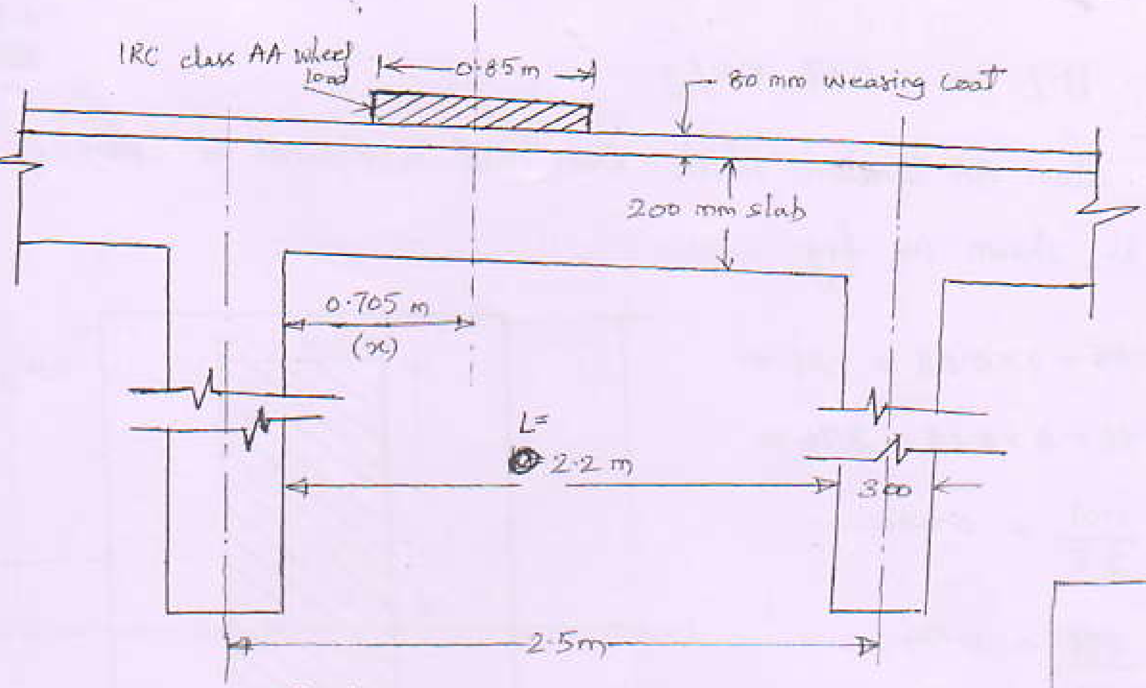
The cross-section of deck and plan showing the spacing of cross girders are shown in figures above.

Step ④ : Design of interior slab panels.

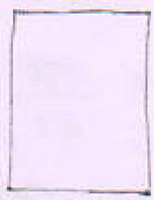
a) BM's

Dead weight of slab $= (\cancel{\ } \; 0.2 \times 24) = 4.8 \text{ kN/m}^2$

DL of wearing coat $= (0.08 \times 22) = 1.76 \text{ kN/m}^2$



Position of wheel load for max^{m} shear.

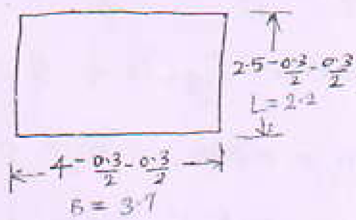


Eff width of slab $= kx\left(1 - \frac{x}{L}\right) + b_w$

Breadth of cross girder = 300 mm

Clear ~~length~~ width of panel B = 3·7 m

clear length L = 2·2
clear width B = 3·7



$\frac{B}{L}$ (width / L) $= \frac{3·7}{2·2} = 1·68$

$b_w = 3·6 + 2(0·08)$

From table 10·1 of N·Kr Raju k for continuous slab = 2·52

Eff width of slab $= \left[\overset{k}{2·52} \times \overset{x}{0·705}\left(1 - \frac{0·705}{2·2}\right) + \underbrace{3·6 + (2 \times 0·08)}_{b_w}\right] = 5\,m$

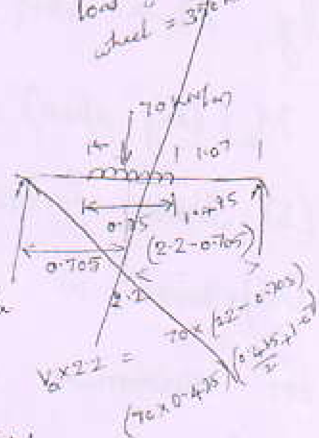
$\frac{700}{2} = 350$

∴ Load per metre width $= \left(\frac{350}{5}\right) = 70$ KN/m

(350 ↓ eff width of slab)

∴ Shear force $= 70\left[\frac{2·2 - 0·705}{2·2}\right] = 47·60$ KN

S·F with impact $= 1·25 \times 47·60 = 59·5$ KN



(c) Dead load BM & SF's

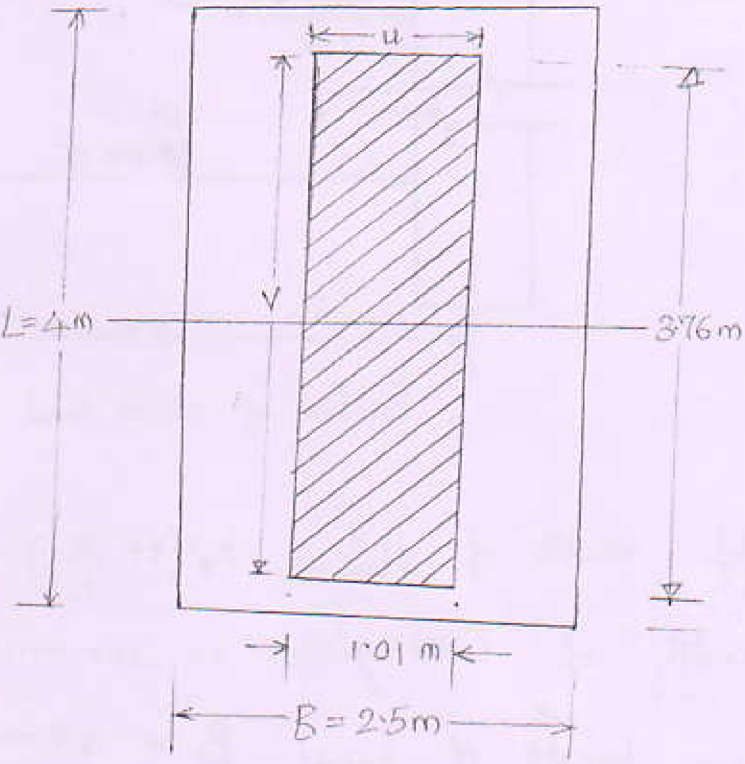
$D·L = 6·56\ KN/m^2$

Total load on panel $= (4 \times 2·5) \times 6·56 = 65·6$ KN

(panel size)

Total D·L = 6·56 KN/m²

LL is class AA tracked vehicle. One wheel is placed at centre of panel as shown in fig.



$u = 0{\cdot}85 + 2 \times 0{\cdot}08 = 1{\cdot}01\ m$

$V = 3{\cdot}60 + 2 \times 0{\cdot}08 = 3{\cdot}76\ m$

$\frac{u}{B} = \frac{1{\cdot}01}{2{\cdot}5} = 0{\cdot}404$

$\frac{v}{L} = \frac{3{\cdot}76}{4{\cdot}0} = 0{\cdot}94$

$k = \frac{B}{L} = \frac{2{\cdot}5}{4{\cdot}0} = 0{\cdot}625$

Referring to Pigeaud's curves Fig: 10.4 of N.Kr.R

$m_1 = 0{\cdot}085 \qquad m_2 = 0{\cdot}024$

Short span moment $M_B = W(m_1 + 0{\cdot}15 m_2) = 350(0{\cdot}085 + 0{\cdot}15 \times 0{\cdot}024) = 31{\cdot}01\ KN{\cdot}m$

As the slab is continuous, design BM = $0{\cdot}8 M_B$

Design BM including impact & continuity factor

M_B (short span) $= 1{\cdot}25 \times 0{\cdot}8 \times 31{\cdot}01 = 31{\cdot}01\ KN{\cdot}m$

Long span moment IIly $M_L = 350(0{\cdot}024 + 0{\cdot}15 \times 0{\cdot}085) = 12{\cdot}845\ KN{\cdot}m$

M_L (long span) $= 1{\cdot}25 \times 0{\cdot}8 \times 12{\cdot}845 = 12{\cdot}845\ KN{\cdot}m$

(b) Shear forces

Dispersion in direction of span (along 2·5) $= 0{\cdot}85 + 2(0{\cdot}08 + 0{\cdot}2) = 1{\cdot}41\ m$

For maximum shear, the load is kept such that the whole dispersion is in span. Load is placed @ $\frac{1{\cdot}41}{2} = 0{\cdot}705\ m$ from edge of beam as shown in fig.

Use 10 mm dia bars at 150 mm c/c ($A_{st} = 524\ mm^2$)

f) Check for shear stress

Shear stress $\tau_v = \dfrac{V}{bd} = \dfrac{66.716 \times 10^3}{1000 \times 180} = 0.37\ N/mm^2$

Permissible sh. st in slabs $\tau_c = k_1 k_2 \tau_{co}$

$k_1 = (1.14 - 0.7 \times 0.18) = \cancel{0.814}\ 1.017 \geq 0.5$ ($1.14 - 0.7d$)

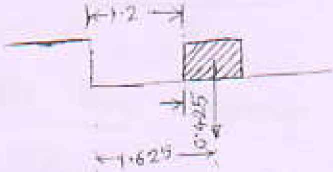
$k_2 = (0.5 + 0.25 p)$ & $p = \dfrac{100 A_s}{bd} = \dfrac{100 \times 1131}{1000 \times 180} = 0.63 \geq 1$

For M25 grade of concrete $\tau_{co} = 0.4\ N/mm^2$

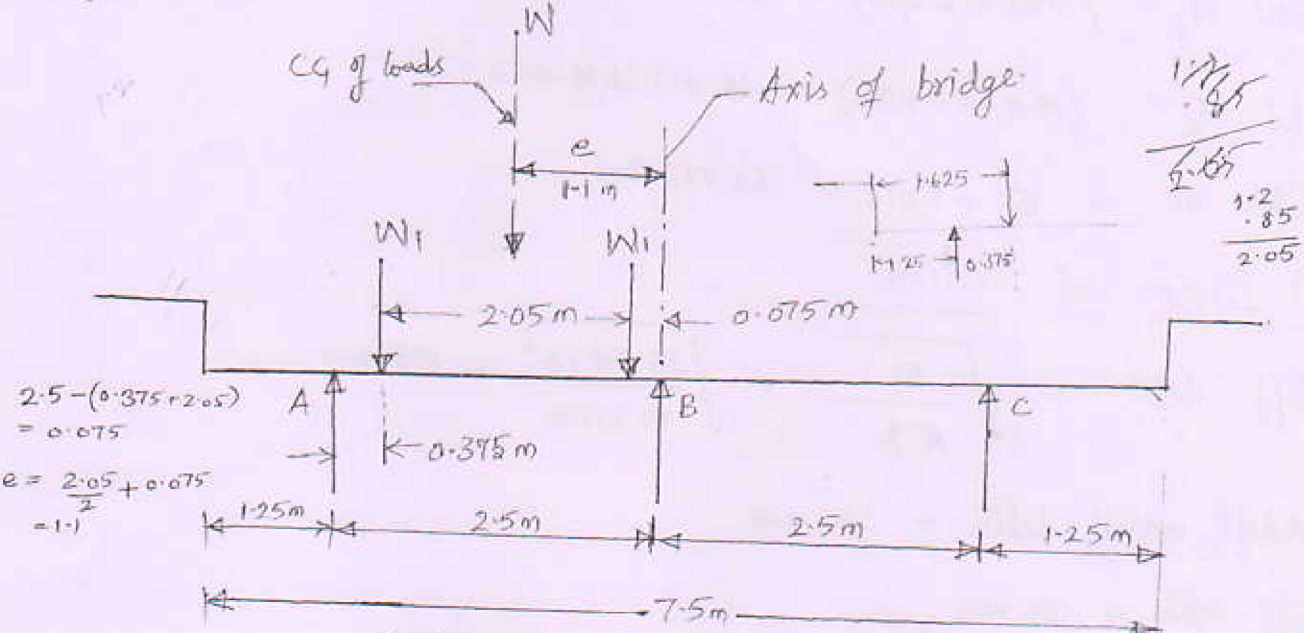
$\tau_c = k_1 k_2 \tau_{co} = \cancel{0.814}\ 1.017 \times 1 \times 0.4 = \cancel{0.35}\ 0.407\ N/mm^2$

Since $\tau_v \simeq \tau_c$; shear stresses are within permissible limits.

(5) Design of longitudinal girders.



a) Reaction ~~forces~~ factors



Critical position of IRC class AA tracked vehicle.

According to Courbon's method; Reaction factor $R_x = \dfrac{\Sigma W}{n}\left[1 + \left(\dfrac{\Sigma I}{\Sigma d_x^2\, I}\right) d_x\, e\right]$

Reaction factor for outer girder is;

$$R_A = \frac{2W_1}{3}\left[1 + \frac{3I \times 2.5 \times 1.1}{2I \times 2.5^2}\right] = 1.107\ W_1$$

Total conc. live loads $\Sigma W = 2W_1$,
n = no. of longi. girders
I = M.I. of each longi. girder
d_x = distance of girder under consideration from central axis of bridge
e = ecc. of LL w.r.t axis of bridge

When the intermediate and end longitudinal girders have same M.I., the quantity in brackets gets cancelled & $R_B = R_B\left(\frac{\Sigma W}{n}(1+0)\right)$

Reaction factor for inner girder is; $R_B = \dfrac{2W_1}{3}(1+0) = \dfrac{2W_1}{3}$

$\frac{u}{B} = 1$ & $\frac{v}{L} = 1$ as panel is loaded with udl

$k = \frac{B}{L} = \frac{2.5}{4} = 0.625$ and $\frac{1}{k} = 1.6$

From Pigeaud's curves (Fig 10.9 of N. Kr. Raju); $m_1 = 0.049$
$m_2 = 0.015$

$M_B = 65.6\,(0.049 + 0.15 \times 0.015) = 3.36$ kN·m

Taking continuity into effect $M_B = 0.8 \times 3.36 = 2.688$ kN·

$M_L = 65.6\,(0.015 + 0.15 \times 0.049) = 1.468$ kN·m

With continuity effect $M_L = 0.8 \times 1.468 = 1.174$ kN·m

DL SF $= \frac{6.56 \times 2.2}{2} = 7.216$ kN·

d) Design moments and SF's

Total $M_B = (31.01 + 2.688) = 33.698$ kN·m

Total $M_L = (12.845 + 1.174) = 14.019$ kN·m

Total SF $= 59 + 7.216 = 66.716$ kN.

e) Design of section

Eff. depth $= \sqrt{\frac{M}{R \cdot b}} = \sqrt{\frac{33.698 \times 10^6}{1.1 \times 1000}} = 175$ mm

Adopt overall depth = 200 mm

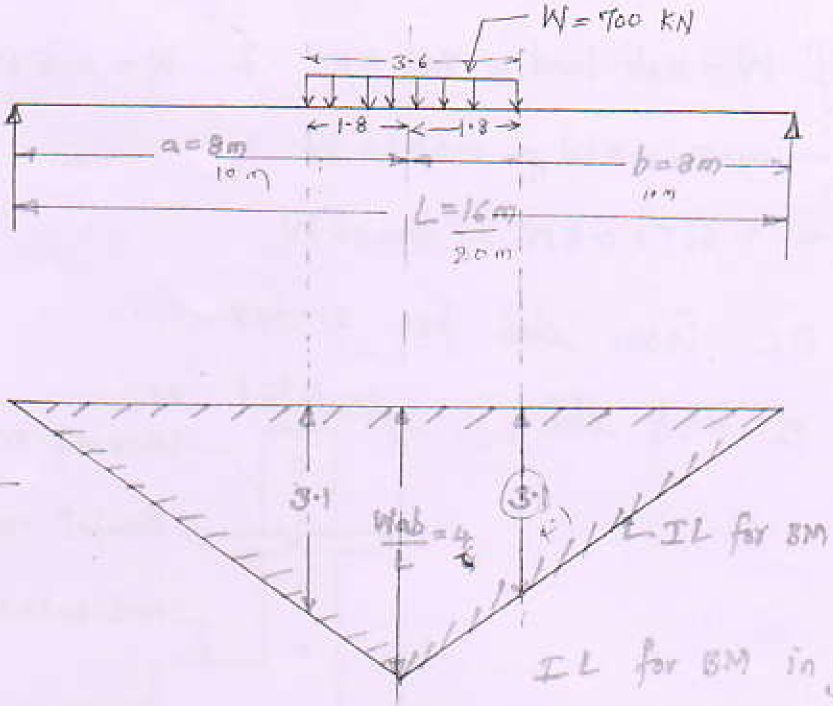
Eff. depth = 180 mm

A_{st} (short span) $= \frac{M}{\sigma_{st} \cdot j \cdot d} = \frac{33.698 \times 10^6}{200 \times 0.9 \times 180} = 1040\ mm^2$

Using 12 mm dia bars @ 100 mm c/c ($A_{st} = 1131\ mm^2$)

Eff. depth for long span using 10 mm dia bars $= (180 - 6 - 5) = 169$ mm

A_{st} (long span) $= \frac{14.019 \times 10^6}{200 \times 0.9 \times 169} = 461\ mm^2$



$\frac{4}{8} = \frac{y}{8-1\cdot8}$

$\Rightarrow y = 3\cdot1$

ILD for unit load;

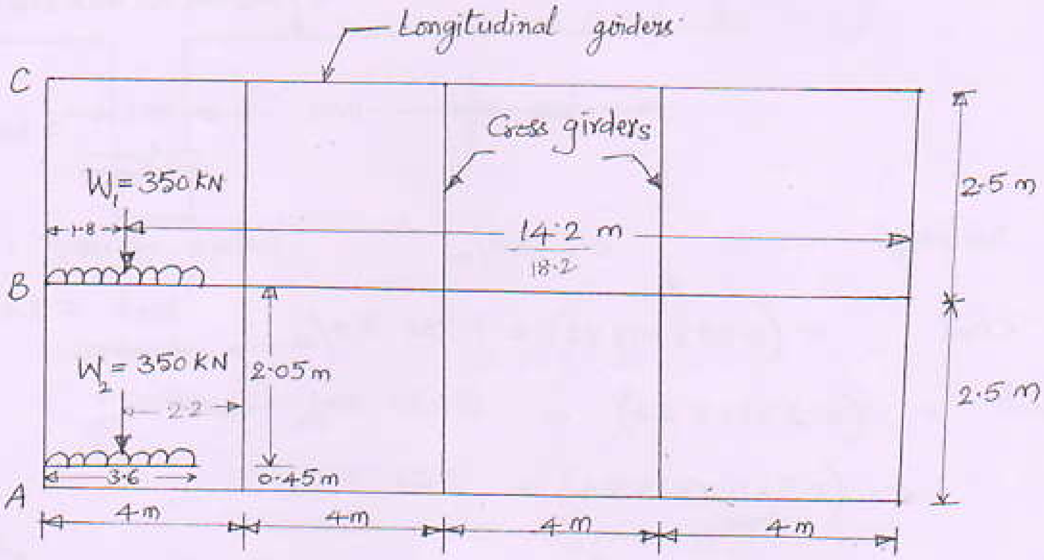
$\frac{Wab}{L} = \frac{1\times8\times8}{16} = 4$

$\frac{1\times10\times10}{20} = 5$

$\frac{5}{10} = \frac{y}{10-1\cdot8} \Rightarrow y = 4\cdot1$

IL for BM in girder.

$\frac{3\cdot6}{2} = 1\cdot8$



Position of IRC class AA loads for max^m shear.

$\left(\frac{5+4\cdot1}{2}\right)700$

$$BM = \left(\frac{4+3\cdot1}{2}\right)700 = 2485\ KN\cdot m$$

BM including impact and reaction factor for outer girder

$= \left(2485\times1\cdot1\times \cancel{0\cdot3333}\ 0\cdot5536\right) = \cancel{912\ KN\cdot m}\ 1513\ KN\cdot m$

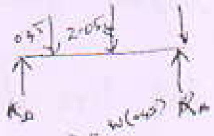
BM " " " " " " " inner girder $= 2485\times1\cdot1\times0\cdot3333 = 912\ KN\cdot m$

0·5536 & 0·3333 are reaction factors.

d) Live load shear.

For estimating max^m LL shear in the girders, IRC class AA loads are placed as shown in fig;

Reaction of W_2 on girder B (i.e. R_B) $= \frac{350\times0\cdot45}{2\cdot5} = 63\ KN$



$R_B(2\cdot5) = W(0\cdot45)$

$R_B =$

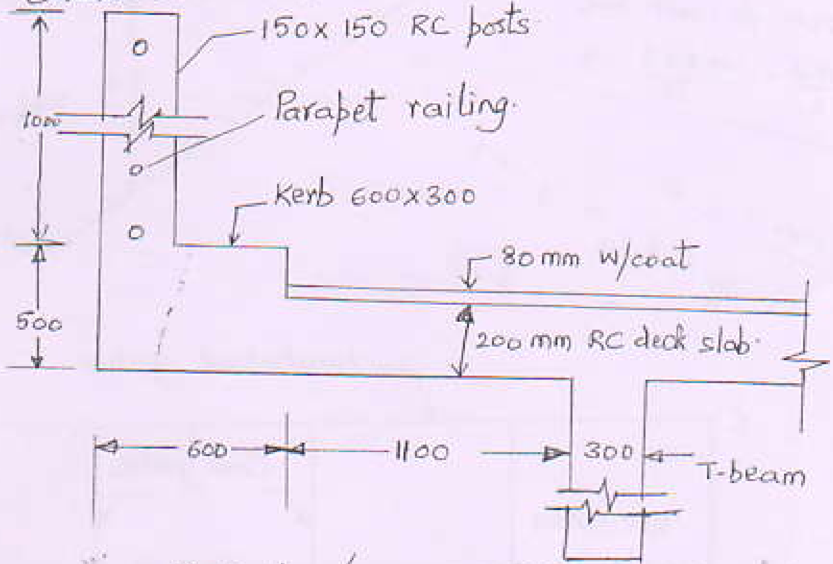
If W = axle load = 700 KN & $W_1 = 0.5\,W$

$R_A = 1.107 \times 0.5W = 0.5536\,W$

$R_B = 0.667 \times 0.5W = 0.333\,W$

(b) DL from slab per girder.

DL of deck slab is calculated as;



Details of deck slab, kerb + parapet.

i) Parapet railing = 0.7 KN/m ✓

ii) Wearing coat = $(0.08 \times 1.1 \times 22) = 1.936$ KN/m → ~~0.08×0.6×22~~

iii) Deck slab = $(0.2 \times 1.1 \times 24) = 5.280$ KN/m → ~~0.25×0.6~~

iv) Kerb = $(0.5 \times 0.6 \times 1 \times 24) = 7.20$ KN/m ✓

$0.6 \times 0.30 \times 24 = 7.92$

15.116 KN/m

Total DL of deck = $(2 \times 15.116) + (6.56 \times 5.3)$

(2×8.62) (8.01×7.5)

$= 65$ KN/m → 77.33

6.56 KN/m is DL from step ⓐ

$\frac{0.3}{2} + 2.5 + 2.5 + \frac{0.3}{2} = 5.3$

It is assumed that the DL is shared equally by all girders

$\therefore$ DL / girder $= \frac{65}{3} = 21.66$ KN/m → $\frac{77.33}{4} = 19.33$ KN/m

(c) Live load BM's in girders.

Span of girder = 16 m → 20 m

Impact factor (for class AA) = 10% ✓

The live load is placed centrally on span as shown in fig.

DL shear @ support $= \dfrac{31.74 \times 16}{2} + 25.2 + \dfrac{25.2}{2} = 292$ KN.

↳ Reaction $V_a = 292$ KN.

(f) Design BM and shear forces.

	BM (KN·m)		
	DLBM	LLBM	Total BM
Outer girder	ⓔ 1218	1513 ⓒ	2731
Inner girder	1218	912	2130

	SF (KN)		
	DLSF	LLSF	Total SF
Outer girder	step ⓔ 292	280.5 step(d)	572.5
Inner girder	292	402.6	694.6

The beam is designed as a T-section. Assuming an

$M_{max} = 2731$ KN·m

$V_{max} = 694.6$ KN

eff. depth $d = 1450$ mm

Approximate lever arm $= \left(1450 - \dfrac{200}{2}\right) = 1350$ mm

$A_{st} = \dfrac{2731 \times 10^6}{200 \times 1350} = 10114\ mm^2$

$\frac{\pi}{4}(32)^2 = 1018\ mm^2$

Max^m size of bars not to exceed 32 mm dia; as per IRC: 21-1987

Provide ~~12~~ 16 bars of ~~36~~ 32 mm diameter in three rows. $A_{st} = 12864\ mm^2$

Shear reinforcements are designed to resist max^m shear @ supports.

Nominal shear stress $\boxed{\tau_v = \dfrac{V}{bd}} = \dfrac{694.6 \times 10^3}{300 \times 1450} = 1.596\ N/mm^2$

$\not> 0.07 f_{ck} \not> 0.07 \times 25 \not> 1.75\ N/mm^2$

↓ Hence safe.

~~If 4 bars of 36 mm diameter are bend up near supports to resist shear,~~

~~$\boxed{\dfrac{100 A_{st}}{bd}} = \dfrac{100 \times 8 \times 1018}{300 \times 1450} = 1.8$~~

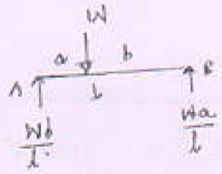
~~From tables; for M20 Concrete; $\tau_c = 0.47\ N/mm^2$.~~

Reaction of W_2 (i.e. R_A) on girder A $= \dfrac{350 \times 2.05}{2.5} = 287$ KN

Total load on girder B $= 350 + 63 = 413$ KN.

Max^m " " " B $= 413 \times 14.2/16 = 366.5$ KN.

Max^m reaction in girder A $= \dfrac{287 \times 14.2}{16} = 255$ KN



Max^m LL shears with impact factor in

inner girder $= 366 \times 1.1 = 402.6$ KN

outer girder $= 255 \times 1.1 = 280.5$ KN

(e) DL BM's + SF's in main girder.

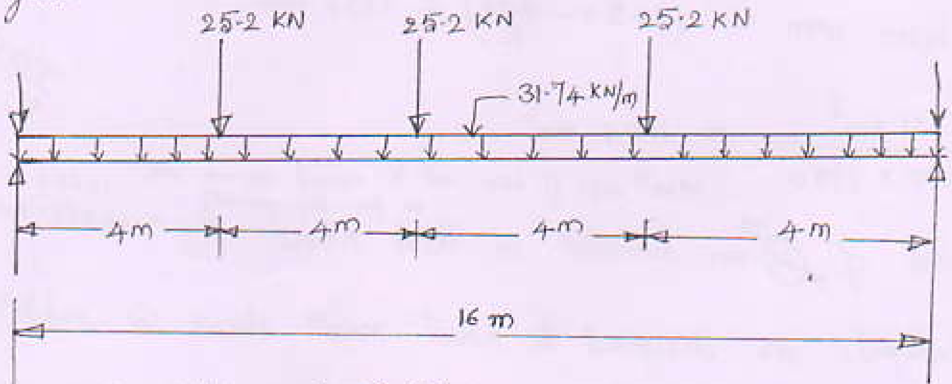
The depth of girder is assumed as 1600 mm (100 mm for every metre of span)

Depth of rib $= 1.4$ m

Width $= 0.3$ m

Weight of rib per metre $= (1 \times 0.3 \times 1.4 \times 24) = 10.08$ KN/m

The cross girder is assumed to have same c/s dimensions as main girder.



Weight of cross girder $= 10.08$ KN/m

Reaction on main girder $= 10.08 \times 2.5 = 25.2$ KN
→ wt of cross girder/rib x spacing of main girder

Reaction from deck slab on each girder $= 21.66$ KN/m (from step (b))
→ DL/girder

Total DL per metre on girder $= 21.6 + 10.08 = 31.74$ KN/m

Referring to the fig above; max^m BM @ centre is

$$M_{max} = \frac{31.74 \times 16^2}{8} + \frac{25.2 \times 16}{4} + \frac{25.2 \times 16}{4} = 1218 \text{ KN·m}$$

$V_a = 292$

$292 \times 8 - \left(31.74 \times 8 \times \frac{8}{2}\right) - 25.2 \times 4 = 1219.52$

In figure below:

Load on cross girder

$$= \frac{350(4-0.9)}{4} = 271.25 \text{ KN}$$

Assuming cross girder as rigid

Reaction on each longitudinal

Total load / no. of girders ← girder $= \frac{2 \times 271.25}{3}$ $\frac{2\times 271.25}{4}$

$= 180.83$ KN

Max^m BM in cross girder under the load

$5-2.05 = 2.95$, $2.95/2$ @ 1.475

$= (180.83 \times 1.475)$

$= 266.7$ KN·m

Live load BM (LLBM) including impact

$= 1.1 \times 266.7$

$= 293.37$ KN·m

Dead load BM (DLBM) @ 1.475 m from support;

30.47 is reaction on each cross girder

$$= 30.47 \times 1.475 - 18.28 \times \frac{1.475^2}{2}$$

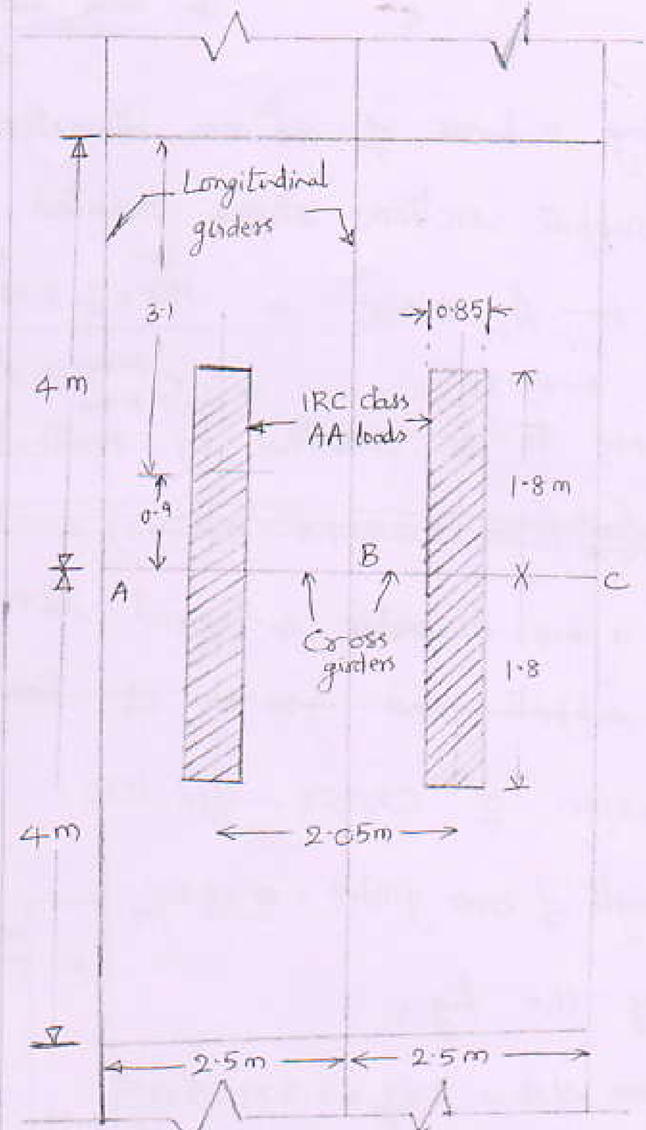
18.28 is total load on cross girders

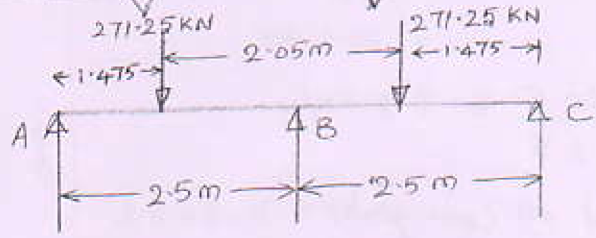
$= 25.10$ KN·m

Total design BM

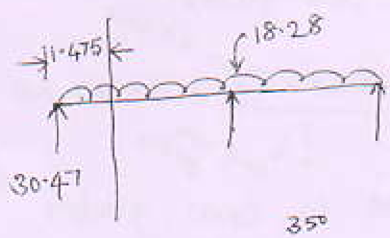
$= (293.37 + 25.10)$

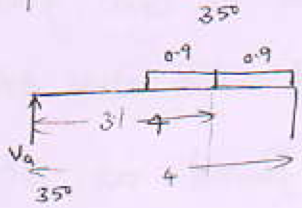
$= 318.47$ KN·m





Position of live loads for max^m BM in cross girders.





∴ ~~Shear taken by concrete~~ $\tau_c bd$

$= \cancel{\frac{0.47\times300\times1450}{1000}} = \cancel{204.45\ KN}$

Assuming 2 bars of 32 mm diameter to be bent up at any support section, shear resisted by bent-up bars is;

$V_s = \sigma_{sv} A_{sv} \sin\alpha = \frac{200\times2\times804\times1}{1000\times\sqrt{2}} = 227\ KN$

$\sin 45 = 1/\sqrt{2}$ — to convert to KN

∴ shear to be resisted by vertical stirrups; is computed as balance shear $= 694.6-227 = 467.7\ KN$

~~$V = (694.6 - 204.45 - 215.90) = 274.25\ KN$~~

Using 10 mm diameter 4-legged stirrups; ~~at 250 mm c/c~~ spacing $= S_v = \sigma_{sv} A_{sv}\, d/V$ ~~near supports and 450 mm c/c towards the centre of span.~~

$S_v = \frac{200\times4\times79\times1450}{467.7\times10^3} = 195\ mm\ c/c$

Provide 10mm ϕ 4 legged stirrups @ 150 mm c/c

694.6 / 227 / 7

⑥ Design of cross-girders

Self weight of cross girder = 10.08 KN/m

Referring the fig;

6.56 is DL from self 4-a

$DL\ from\ slab = 2\times\frac{1}{2}\times2.5\times1.25\times6.56$ (8.01)

$= 20.5\ KN$

Total DL of slab + wearcoat = 6.56 kN/m²

$Udl = \frac{20.5}{2.5} = 8.2\ KN/m$

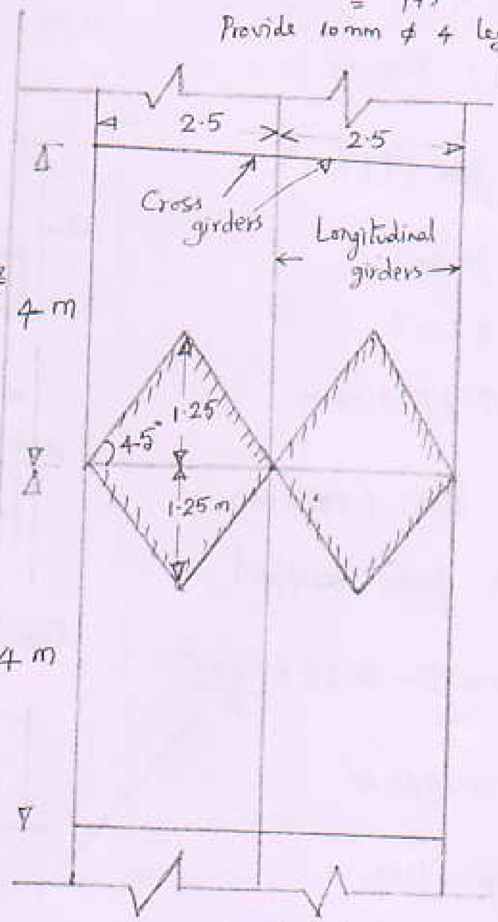
Total load on cross girders $= 10.08 + 8.2$

$= 18.28\ KN/m$

Assuming the cross girder to be rigid;

Reaction on each cross girder

$= \frac{18.28\times 5 \,(2.5+2.5)\ (7.5)}{3\ (4 \text{ no. of cross girders})} = 30.47\ KN$



For max^m BM in cross girder, the loads of IRC class AA should be placed as shown

Loads on cross girders

Live load shear including impact

$$= \left[\frac{2 \times 271.25}{3} \times 1.1 \right] = 198.917 \text{ KN}$$

or 180.83×1.1

DL shear = 30.47 KN

Total design shear = 198.917 + 30.47 = 229.39 KN

Assuming an eff. depth for cross girders as ~~1540~~ 1520 mm.

$$A_{st} = \frac{318.47 \times 10^6}{200 \times 0.9 \times 15\cancel{4}0} = \cancel{1148}\ 1163 \text{ mm}^2$$

Provide 4 bars of 20 mm diameter $(A_{st} = 1256 \text{ mm}^2)$

$$\text{Shear stress } \tau_v = \frac{229.39 \times 10^3}{300 \times 15\cancel{4}0} = \cancel{0.496}\ 0.503 \text{ N/mm}^2$$

$$\frac{100\, A_{st}}{bd} = \frac{100 \times 1256}{300 \times 1540} = 0.271$$

From tables; $\tau_c = 0.22 \text{ N/mm}^2$

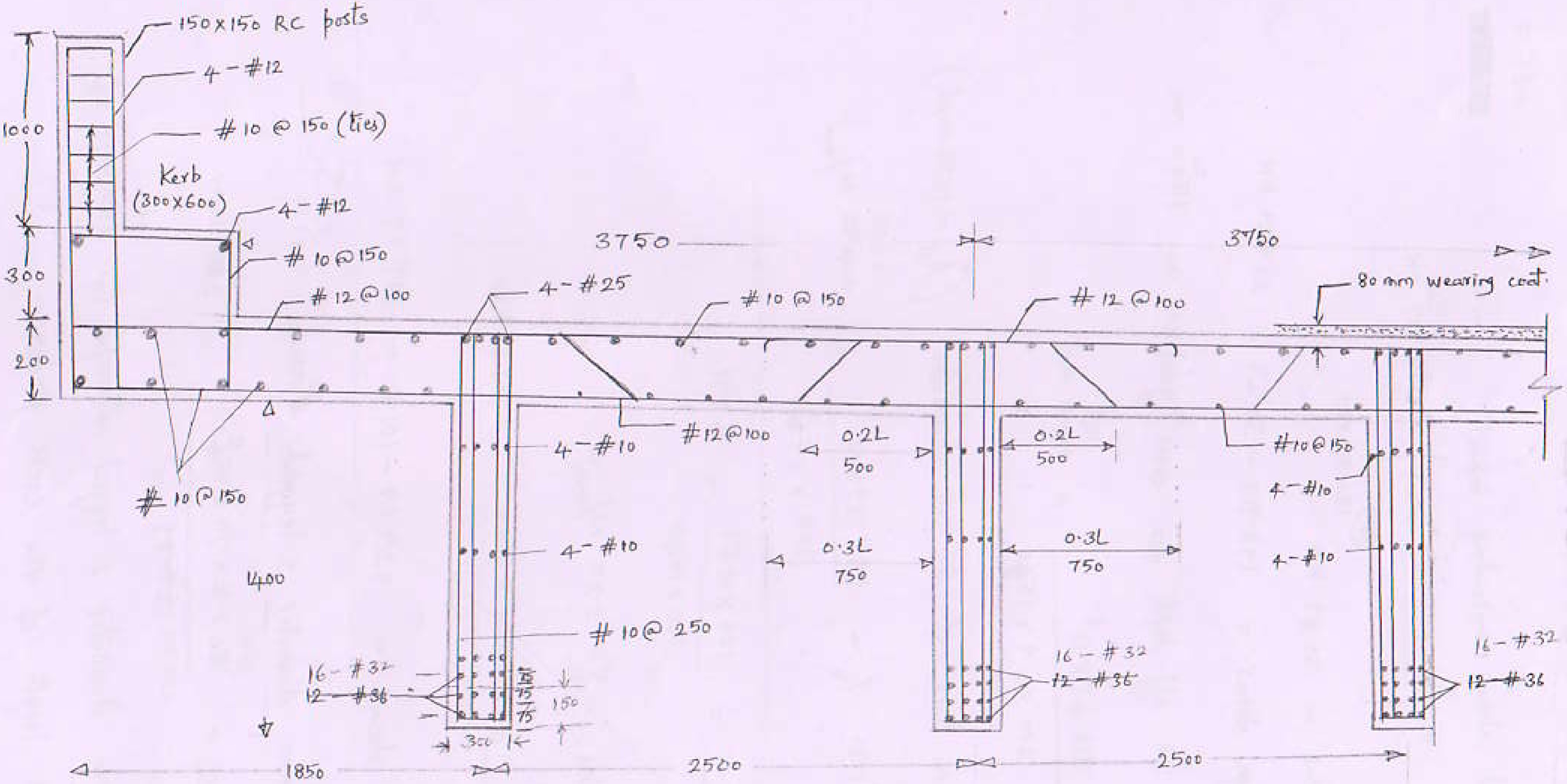
$$\tau_c bd = \frac{0.22 \times 300 \times 1540}{1000} = 101.6 \text{ KN}$$

Balance shear = 229.39 − 101.6 = 127.79 KN

Using 10 mm diameter 2-legged stirrups, $S_v = \frac{\sigma_{sv} A_{sv} d}{V}$

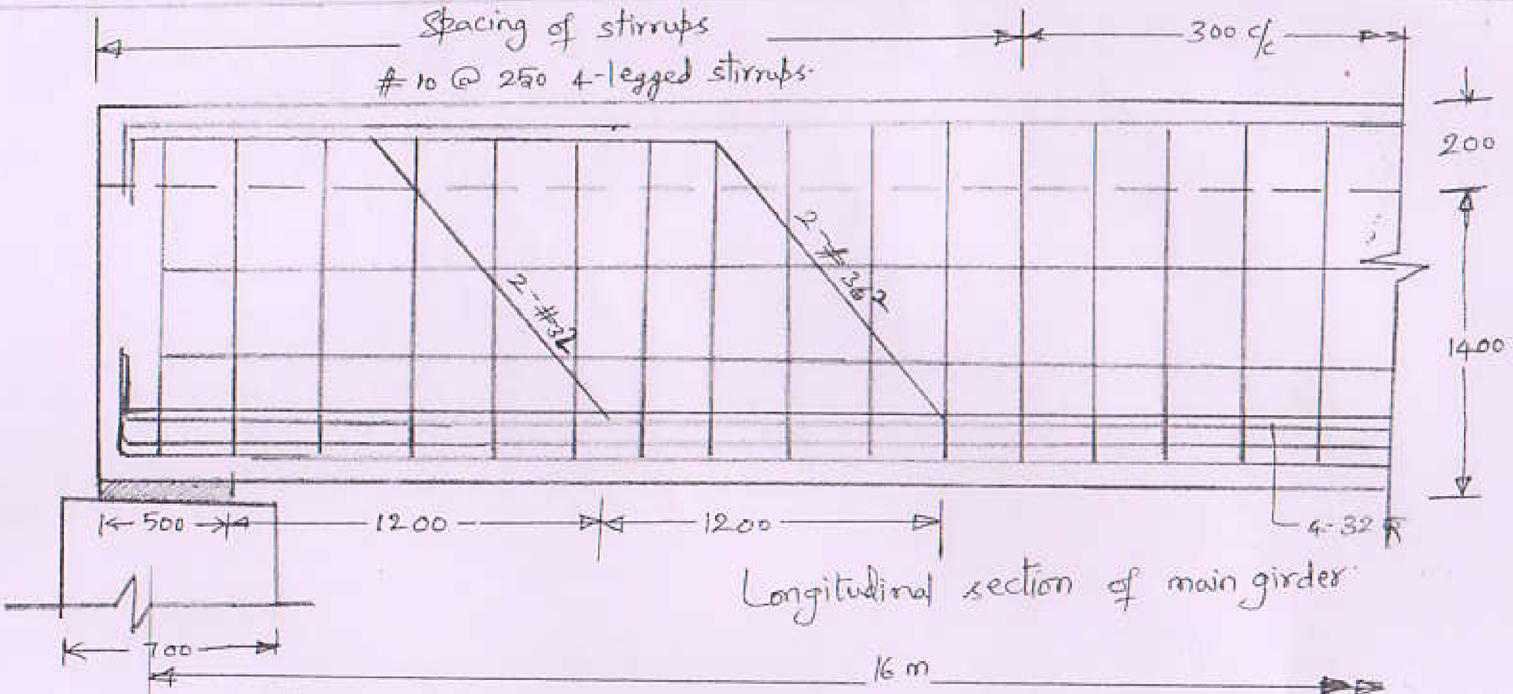
$$\text{Spacing } s_v = \frac{\cancel{150}\ 200 \times 2 \times 79 \times 15\cancel{4}0}{\cancel{127.79}\ 229.39 \times 10^3} = 2\cancel{8}5 \text{ mm}$$

Adopt 10 mm diameter 2-legged stirrups @ ~~250~~ 150 mm c/c thr out the length of the cross girder.

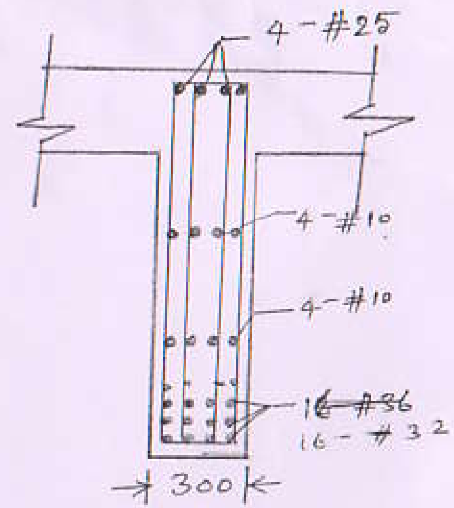


Cross-Section of T-beam and slab deck

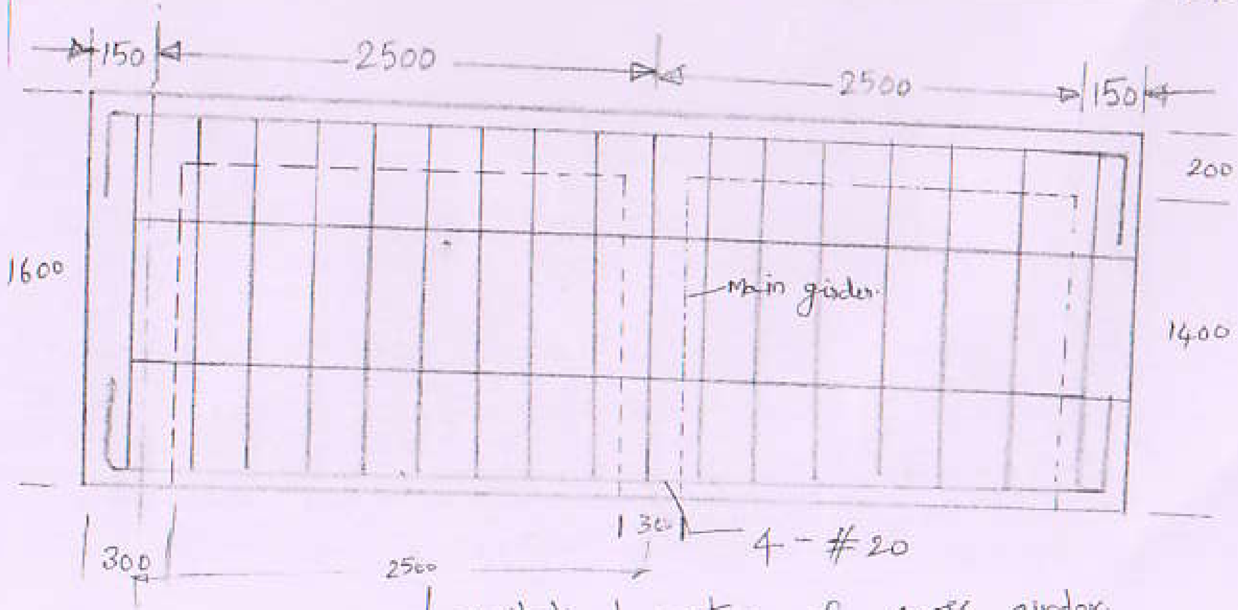
4-BE-8



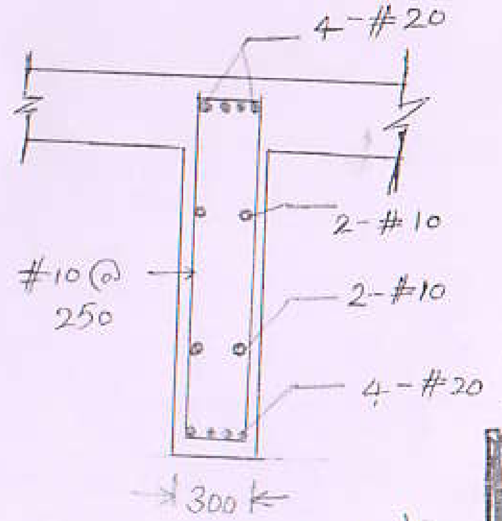
Longitudinal section of main girder



Cross-section



Longitudinal section of cross girder



Cross section