step 1:- preliminary dimension.

i) 5 no cross girder @ 3.75 m c/c.

ii) Assume 3 main girder @ 2.5 m c/c

iii) slab thickness = 200 mm

iv) width of the main girder = 300 mm

v) depth of the main girder = $l/d = 10$

$$\frac{15}{10} = d \quad d = 1.5\,m.$$

vi) Dimension of the footpath = 0.3 X 0.6 m

Design of interior pannel

Size of the pannel = 2.5 X 3.75 m.

i) Load calculation

Self wt of slab = 0.2 X 24 = 4.8 kN/m².

Self wt of wearing coat = 0.08 X 22 = 1.76 kN/m²

6.56 kN/m²

Load on each pannel = 2.5 X 3.75 X 6.56 kN/m²

= 61.5 kN

ii) BM & SF due to D.L

$K_1 = B/L = \frac{2.5}{3.75} = 0.66$

$1/K = L/B = \frac{3.75}{2.5} = 1.5.$

$m_1 = 0.048$

$m_2 = 0.016$

moment in shorter span

$\alpha = 0.15$

$m_B = TL (m_1 + \alpha m_2)$

$= 61.5(0.048 + 0.15 \times 0.016)$

$= 3.09$ kN-m

moment in longer span

$m_L = TL (m_2 + \alpha m_1)$

$= 61.5(0.016 + 0.15 \times 0.048)$.

$= 1.42$ kN-m.

Considering continuity of the slab.

$m_B = 3.09 \times 0.8$

$= 2.47$ kN-m

$m_L = 1.42 \times 0.8$

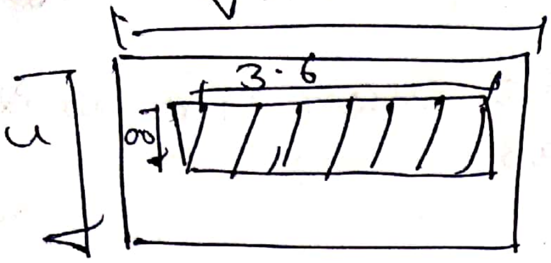
$= 1.13$ kN-m.

$SF = \frac{wl}{2}$

$= \frac{6.56 \times 2.2}{2}$

$= 7.2$ kN.

iii) BM & SF due to L.L



$u = 0.8\cancel{8}^{5} + 2(0.08)$

$= \cancel{0.98}\ m\ \ 1.01m$

$v = 3.6 + 2(0.08)$

$= 3.76m.$

$k = B/L$

$= 0.66 \simeq 0.7$

$\frac{V}{L} = \frac{3.76}{3.75} = 1.00$

$\frac{U}{B} = \frac{1.01}{2.5} = 0.40$

$m_1 = 0.086$

$m_2 = 0.035$

moment in shorter span

$m_B = TL \times (m_1 + \alpha m_2)$

$= 350 \times (0.086 + 0.15 \times 0.035)$

$= 31.93$ KN-m

moment in longer span

$m_L = TL \times (m_2 + \alpha m_1)$

$= 16.76$ KN-m

Considering impact factor and continuity of the slab.

Impact factor span less than 9m is 25%.

moment in shorter span

$M_B = 31.93 \times 0.8 \times 1.25$

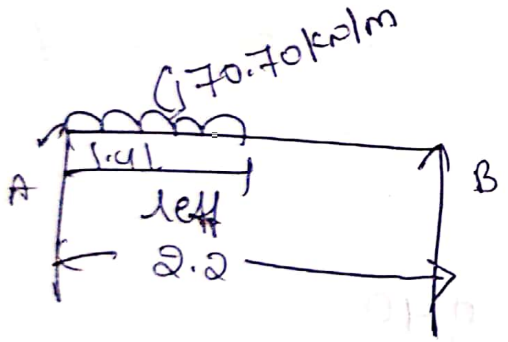
$= 31.93\ kN\text{-}m$

moment in longer span.

$M_L = 16.76 \times 0.8 \times 1.25$

$= 16.76\ kN\text{-}m.$

SF due to L-L



$l_{eff} = 0.85 + 2(0.2 + 0.08)$

$= 1.41m.$

$a = \frac{1.41}{2} = 0.705m.$

$b_1 = 3.6 + 2(0.08)$

$= 3.76m.$

$\alpha = \frac{B}{L} \Rightarrow \frac{3.45}{2.2}$

$= 1.56$

$\alpha = 2.49.$

$b_{eff} = \alpha a \left(1 - \frac{a}{l_0}\right) + b_1$

$= 2.49 \times 0.705 \left[1 - \frac{0.705}{2.2}\right] + 3.76$

$= 4.95m.$

Load per 'm' length $= \frac{350}{4.95}$

$= 70.70 kN/m$

BM @ RB = 0

$RA \times 2.2 = 70.7 \times 1.41 \times \left(2.2 - \frac{1.41}{2}\right)$

$RA = 67.74 kN$

Considering the impact factor 25%.

max SF = 67.74×1.25 from IRC 6 pg no 21

$= 84.67 kN.$

Design BM & SF.

	Due DL	Due L.L	TL
BM kN-m	2.47	31.93	34.4
SF kN	7.2	84.67	91.87.

CMR

Step3:- Structural Design

i) check for depth

$$d = \sqrt{\frac{M}{Qb}}$$

$\sigma st = 200 N/mm^2$ — from IRC 21

$\sigma cb = 10 N/mm^2$ — pg 18 & 19.

$Q = 0.5 \sigma cb \, n j$

$$n = \frac{1}{\left(1 + \frac{\sigma st}{m \sigma cb}\right)} \quad m = 10$$

$= 0.33$

$$j = 1 - \frac{n}{3}$$

$= 0.89$

$Q = 1.46$

$$D = \sqrt{\frac{34.4 \times 10^6}{1.46 \times 1000}}$$

$D = 153.49 mm$

$D_{req} < D_{prov}$ hence safe.

ii) Ast calculation

$$Ast\ main = \frac{M}{\sigma_{st} \times j \times d}$$

$d = 200 - 20 - 12/2$

$= 174mm$

$$= \frac{34.4 \times 10^6}{200 \times 0.89 \times 174}$$

$= 1110.68 mm^2$

$$Sv = \frac{\pi/4 \times 12^2}{1110.68} \times 1000$$

$= 101.82 \simeq 100mm$

provide #12Ø @ 100mm c/c

$Ast\ prv = 1130.97 mm^2$

Distribution steel

M = 0.3 moment due to LL + 0.2 moment to DL

$= 0.3 \times 31.93 + 0.2 \times 2.47$

$= 10.07 kN\text{-}m$

$d = 174 - 12/2 - 10/2$

$= 163mm.$

$$Ast_{dist} = \frac{10.07\times10^6}{200\times163\times0.89}$$

$$= 347.073 mm^2$$

~~Sv~~ = spacing $= \frac{\pi/4\times10^2}{347.073}\times1000$

$$= 226.29 mm$$

$$\simeq 225 mm$$

provide #10ϕ @ 225 mm c/c.

step 4:- check for shear

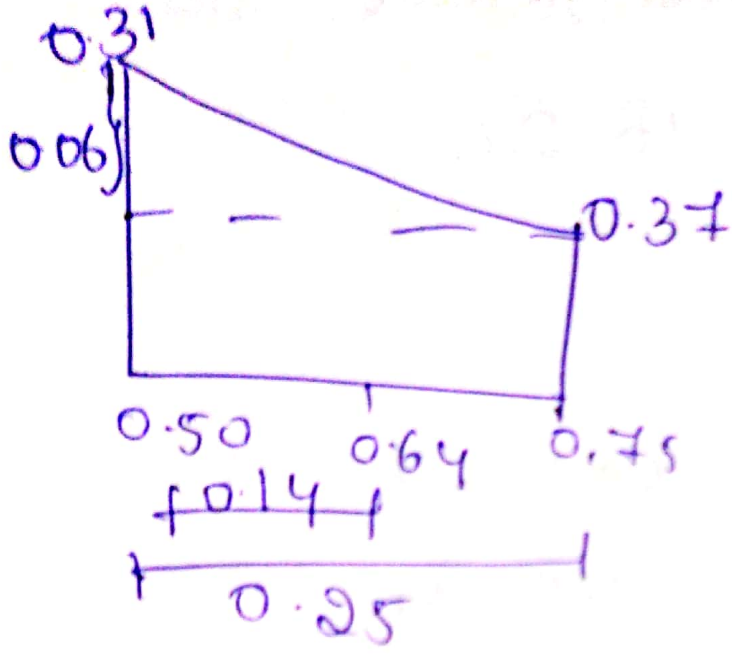
$$\tau_v = \frac{V_u}{bd}$$

$$= \frac{91.87\times10^3}{1000\times174}$$

$$= 0.52 N/mm^2$$

$$p_t = \frac{Ast_{prv}\times100}{bd}$$

$$= \frac{1130.97\times100}{1000\times174} = 0.64.$$



$\tau_c = 0.34 \times k$

$= 0.34 \times 1.2$

$= 0.40\ N/mm^2$

$\tau_c < \tau_v$

Hence it is not safe

it has to design for shear.

CMR

Design of longitudinal girder

Step 1: Bm & SF due to D.L

Load calculation

Self wt of slab = 0.2 X 24 X 7.5 X 15
= 540 KN.

Self wt of wearing coat = 0.08 X 22 X 7.5 X 15
= 198 KN

wt of footpath = 0.6 X 0.5 X 2 X 24 X 15
= 216 KN.

wt of parapit = 2 X 0.8 X 15 = 24 KN

978 KN.

Total load on deck slab = $\frac{978}{15}$
= 65.2 KN/m

Total load on each girder = $\frac{65.2}{3}$
= 21.73 kN/m

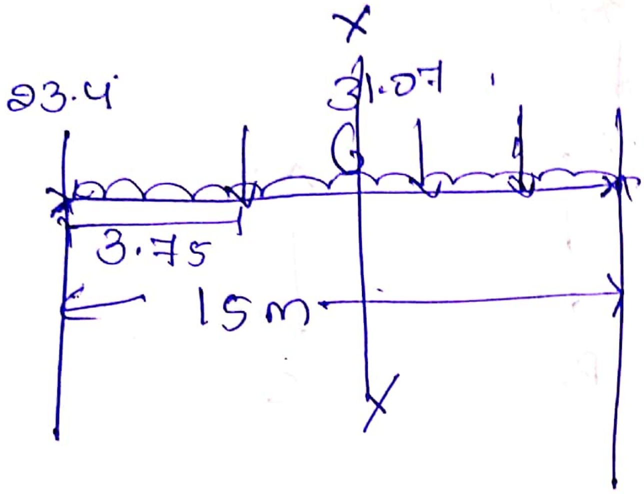
Self wt of main girder = 1.3 X 0.3 X 24
= 9.34 KN/m.

Total udl acting on main girder = 21.73 + 9.34
= 31.07 kN/m

Load from cross girder to main girder

$= 1.3 \times 0.3 \times 24 \times 2.5$

$= 23.4 \text{ kN}$



$RA = RB = 23.4 \times 5 + 31.07 \times 15$

$= 583.05 \text{ kN}.$

max SF $= \frac{583.05}{2}$

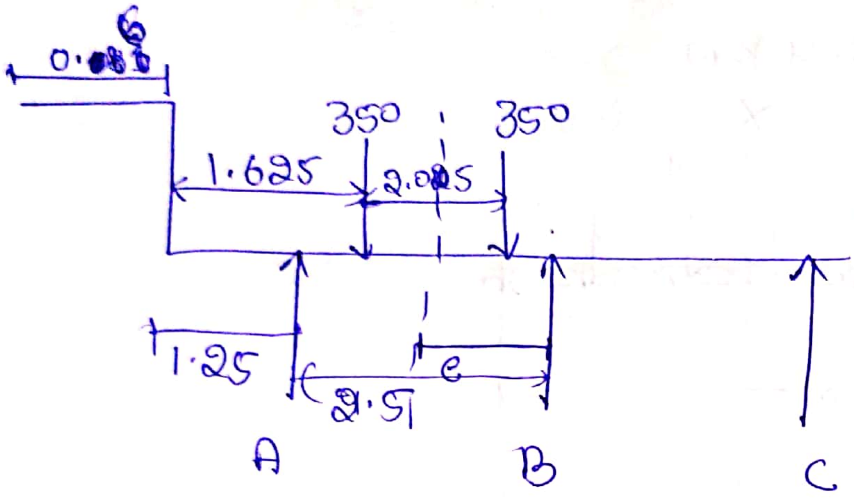
SF $= 291.52 \text{ kN}.$

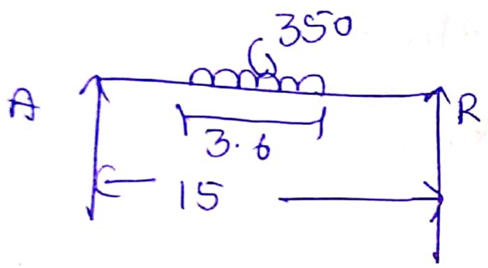
Bm @ mid span

$291.52 \times 7.5 - 23.4 \times 7.5 - 23.4 \times 3.75 - 31.07 \times 7.5 \times \frac{7.5}{2}$

Bm $= 1049.30 \text{ kN-m}.$

Step 2:- BM & SF due to L.L





$R_A = R_B = \frac{350}{2}$

$= 175\,kN.$

BM @ mid span

$175 \times 7.5 - \frac{350}{2} \times \frac{3.6}{2} \times \frac{3.8}{2} = 0.$

$BM = 1029\ kN\text{-}m$

$R_x = \frac{\Sigma w}{n}\left[1 + \frac{n e x}{\Sigma x^2}\right]$

$e = (0.8 + 1.25 + 2.5) - \left(0.8 + 1.625 + \frac{2.025}{2}\right)$

$= \sout{4.55 - 3.43}$

$= 4.35 - 3.25 = 1.1\,m$

$$R_A = R_C = \frac{2\times1029}{3}\left(1+\frac{3\times1.1\times2.5}{2\times2.5^2}\right)$$

$= 1138.76\ kN\text{-}m.$

$$R_B = \frac{\Sigma w}{n}\times1$$

$$= \frac{2\times1029}{3}\times1$$

$= 686\ kN\text{-}m$

Impact factor for BM span is more than 9m is 10%. from IRC 6 pg 21.

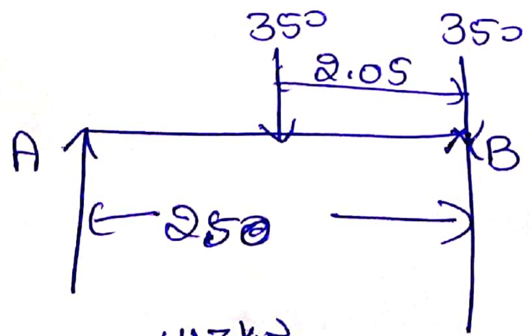
$R_A = R_C = 1138.76\times1.1$

$= 1252.63\ kN\text{-}m$

$R_B = 686\times1.1$

$= 754.6\ kN\text{-}m$

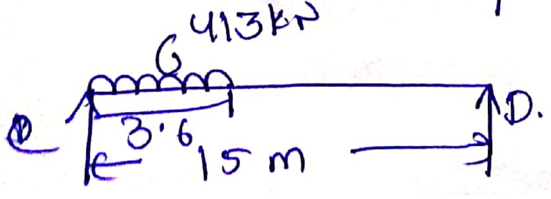
SF due to L.L



moment $R_A = 0$

$R_B\times2.5 = 350\times2.5 + 350\times0.45$

$R_B = 413\ kN$



Bm @ D = 0

$R_C \times 15 = 413 \times \left(15 - \frac{3.6}{2}\right)$

SF = R_C = 363.44 kN.

SF max including impact factor 10%.

SF = 363.44 × 1.1

= 399.78 kN.

Design Bm & SF.

	Due DL	Due LL	TL
Bm kN-m	1049.30	1252.63	2301.93
SF kN	291.52	399.78	691.3

step 3:- Ast calculation

$$A_{st} = \frac{M}{\sigma_{st} \times j \times d}$$

d = 1.5 - 0.15 - 0.1

= 1.25 m

$$A_{st} = \frac{2301.93 \times 10^6}{200 \times 0.89 \times 1250}$$

= 10345.75 mm²

Assume 32mm ϕ bars

No of bars = $\frac{10345.75}{\pi/4 \times 32^2}$

= 12.86 ≃ 16 bars.

~~prv~~ ^

Ast prv = $16 \times \pi/4 \times 32^2$

= 12867.96 mm^2.

Step 4:- check for shear.

$\tau_v = \frac{V_u}{bd} < 0.07 f_{ck}$

$= \frac{691.3 \times 10^3}{30 \times 1350}$

d = 1.5 − 0.15

= 1.35 m

= ~~1706.91~~ 1.7 N/mm²

= ~~2 N~~ 0.07 fck

= 2.1 N/mm²

$\tau_v < 0.07 f_{ck}$

Hence safe.

Shear resistance

Bend up 2 bars of 32 ϕ mm bars

~~Bo~~ $V_B = \sigma_{sd} A_{sv} \sin\alpha$

$= 2 \times \pi/4 \times 32^2 \times 200 \times \sin(45)$

= 227.47 kN.

shear resistance of vertical stirrups p. the balance

shear = 691.3 - 227.47

V_u = 463.83 kN.

provide 4LVS of dia 10mm

spacing $S_V = \frac{A_{sv} \sigma_{st} d}{V_u}$

$= \frac{4 \times \pi/4 \times 10^2 \times 200 \times 1350}{463.83}$

$= 182.87 \simeq 175$ mm.

provide 4LVS @ 175mm c/c.