Box culvert :-

* Reinforced Concrete rigid frame box culverts Consisting of two horizantal and vertical slabs built monolithically are ideally suited for a road (or) railway bridge crossing with high embankments, crossing a stream with limited flow.

* Box culverts of square (or) rectangular vent spans of upto 4 m are Commonly used for crossing small rivers.

* The height of the vent rarely exceeds 3m.

* The box culvert generally comprises the following structural Components.

  (i) Solid barrel (or) box section of Sufficient length to accommodate the road width of the carriage way along with Kerbs and foot paths.

  (ii) In the case of deep embankments, wing walls splayed at 45° are used to guide the flow of water in the stream through the box culvert.

* Box culverts are economical due to their rigidity and monolithic action and seperate foundations are not required since the ~~sta~~ bottom slab resting directly on the soil serve as a raft foundation to the culvert.

* For small discharges, single celled box culvert is used and for larger flow, multi celled box culvts are used.

* The barrel of the box culvert should be of sufficient length to support the entire width of the carriage way.

Design Principles:-

1. Hydraulic design:-

* The vent way required to carry the discharge in the stream is computed by examining the discharge records over a period of time at bridge site.

* The ratio of span to height of the vent way lies between 1:1 and 1·5:1.

2. Structural Design:-

* It consists of detailed analysis of the rigid frame for moments, shear forces and thrusts developed in the various structural elements of the box culvert due to the various type of loading conditions given below.

(i) Concentrated load :- (case 1)

* In cases where the top slab forms the deck of the bridge, concentrated loads due to the wheel loads of the IRC class AA or A type loading have to be considered.

If W = Concentrated load on the slab

P = wheel load

I = impact factor

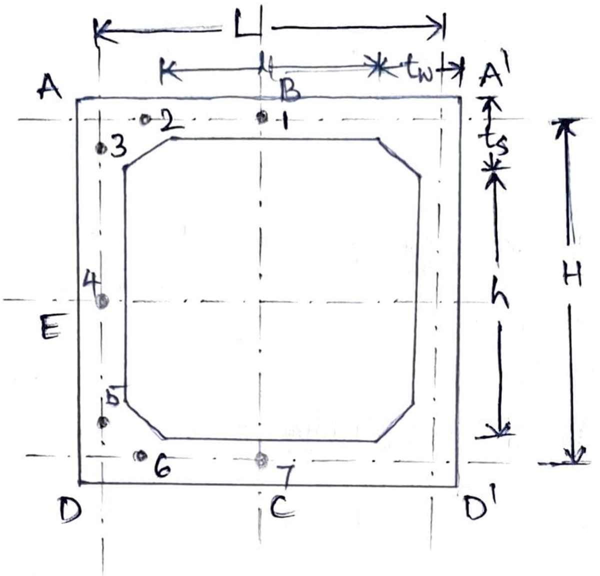
e = effective width of dispersion

Hence $$W = \frac{PI}{e}$$

* The soil reaction of the bottom slab is assumed to be uniform.

L = Span of culvert

H = Height of culvert

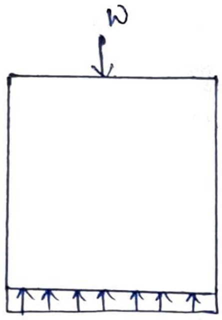


$L = l + t_w$

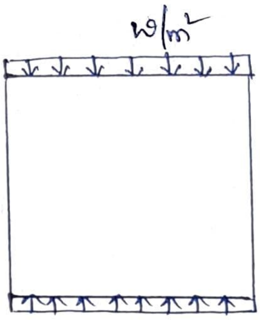
$H = h + t_s$

$K = \frac{H}{L}\left(\frac{t_s}{t_w}\right)^3$

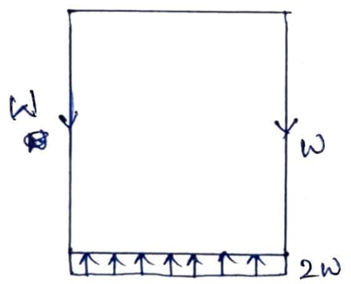
Notations



(a) Case 1 (Concentrated load)

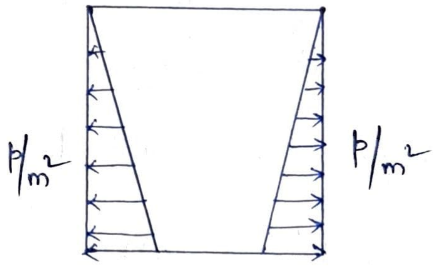


(b) Case 2 (UDL)



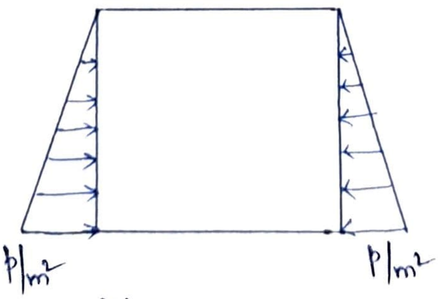
(c) Case 3

(weight of two side walls)



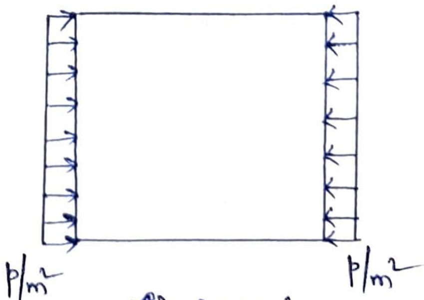
(d) Case 4

(water pressure inside culvert)



(e) Case 5

(Earth Pressure on vertical side walls)



(f) Case 6

(uniform lateral load on side walls)

(ii) Uniform distributed load :- (Case 2)

* The weight of embankment, wearing Coat and deck slab and the track load are considered to be uniformly distributed loads on the top slab with the uniform soil reaction on the bottom slab.

(iii) weight of side walls :- (Case 3)

* The self weight of side walls acting as Concentrated loads are assumed to produce uniform soil reaction on the bottom slab.

(iv) water pressure inside culvert :- (Case 4)

* when the culvert is full with water, the pressure distribution on side walls is assumed to be trainglar with a maximum pressure intensity of $P = wh$ at the base, where

w = density of water

h = depth of flow

(v) Earth pressure on vertical side walls :- (Case 5)

* The earth pressure on the vertical side walls of the box culvert is computed according to the coloumb's theory.

(vi) uniform lateral load on side walls :- (Case 6) :-

* Uniform lateral pressure on vertical side walls has to be considered due to the effect of live load surcharge.

* Also, trapezoidal pressure distribution on side walls due to embankment loading can be obtained by Combining cases 5 and 6.

Design moments, shears and Thrusts :-

* The box culvert is analysed for moments, shear forces and axial thrusts developed due to various loading Conditions by any of the classical methods such as moment distribution, slope deflection (or) Column analogy Procedures.

* Alternatively Coefficients for moments, shears and thrusts Compiled by Victor. J.D are very useful in the Computation of the various force Components for the different loading Conditions.

* (The fixed end moments developed for six different loading cases are given in Table 1 of N. Krishna Raju Text book)

* The moment, shear and thrust Coefficients for various loading cases are given in Table

Design of critical sections :-

* The maximum design moments resulting from the Combination of the various loading cases are determined.

* The moments at the centre of span of top and bottom slabs and the Support sections and at the centre of the vertical walls are determined by suitably Combining the different loading patterns.

* The maximum moments generally develop for the following loading Conditions.

(i) When the top slab supports the dead and live load and the culvert is empty.

(ii) when the top slab supports the dead and live load and the culvert is running full.

(iii) when the sides of the culvert do not carry the live load and the culvert is running full.

* The slabs of the box culvert is reinforced on both faces with fillets at the inside corners.

* The critical sections of the box culvert have to be designed for the limit states of strength and serviceability according to the specifications of IRC: 112 - 2011.

Coefficients for Moment M, Thrust N and Shear V in Transverse sections of unit width

L : H	Section			Loading case 1	2	3	4	5	6
			Factors For						
			M	WL	wL^2	WL	pL^2	pL^2	pL^2
			N	W	wL	W	pL	pL	pL
			V	W	wL	W	pL	pL	pL
1 : 1	B	1	M	+0.182	+0.083	+0.021	+0.019	−0.019	−0.042
			N	0	0	0	−0.167	+0.167	+0.500
	A	2	M	−0.068	−0.042	+0.021	+0.019	−0.019	−0.042
			N	0	0	0	−0.167	+0.167	−0.500
			V	+0.500	+0.500	0	0	0	0
	A	3	M	−0.068	−0.042	+0.021	+0.019	−0.019	−0.042
			N	+0.500	+0.500	0	0	0	0
			V	0	0	0	+0.167	−0.167	−0.500
	E	4	M	−0.052	−0.042	−0.042	−0.043	−0.043	+0.083
			N	+0.500	+0.500	+0.500	0	0	0
	D	5	M	−0.036	−0.042	−0.104	+0.023	−0.023	−0.042
			N	+0.500	+0.500	+1.000	−0.333	+0.333	0
			V	0	0	0	0	0	+0.500
	D	6	M	−0.036	−0.042	−0.104	+0.023	−0.023	−0.042
			N	0	0	0	0	0	+0.500
			V	−0.500	−0.500	−1.000	−0.333	+0.333	0
	C	7	M	+0.088	+0.083	+0.146	+0.023	−0.023	−0.042
			N	0	0	0	−0.333	+0.333	+0.500
1.5 : 1	B	1	M	+0.170	+0.075	+0.018	+0.015	−0.015	−0.033
			N	0	0	0	−0.167	+0.167	+0.500
	A	2	M	−0.079	−0.050	+0.018	+0.015	−0.015	−0.333
			N	0	0	0	−0.167	+0.167	+0.500
			V	+0.500	+0.500	0	0	0	0
	A	3	M	−0.079	−0.050	+0.018	+0.015	−0.015	−0.033
			N	+0.500	+0.500	0	0	0	0
			V	0	0	0	+0.167	-0.167	−0.500
	E	4	M	−0.062	−0.050	−0.050	−0.047	+0.047	+0.092
			N	+0.500	+0.500	+0.500	0	0	0
	D	5	M	−0.045	−0.050	−0.118	+0.018	−0.018	−0.033
			N	+0.500	+0.500	+1.000	0	0	0
			V	0	0	0	−0.333	+0.333	+0.500
	D	6	M	−0.045	−0.050	−0.118	+0.018	−0.018	−0.033
			N	0	0	0	−0.333	+0.333	+0.500
			V	−0.500	−0.500	−1.000	0	0	0
	C	7	M	+0.079	+0.075	+0.132	+0.018	−0.018	−0.033
			N	0	0	0	−0.333	+0.333	+0.500

Note: 1. Positive moment indicates tension on inside face
2. Positive thrust indicates compression on the section
3. Positive shear indicates that the summation for forces at the left of the section acts outward when viewed from within

(Pb) Design a reinforced concrete box culvert having a clear vent way of 3m x 3m. The superimposed dead load on the culvert is 12.8 KN/m². The live load is estimated as 50 KN/m². The density of soil at site is 18 KN/m³. Angle of repose = 30°. Adopt M_{20} grade concrete and Fe 415 HYSD bars. Sketch the details of reinforcements in the box culvert. The design should confirm to the specifications of IRC: 112-2011.

Sol:- Step 1:- Given data :-

clear span, L = 3m

Height of vent, h = 3m

dead load = 12.8 KN/m²

live load = 50 KN/m²

Density of soil = 18 KN/m³

Angle of repose = 30°

$f_{ck} = 20 \text{ N/mm}^2$, $f_y = 415 \text{ N/mm}^2$

Assume $E_s = 200 \text{ Gpa} = 2 \times 10^5 \text{ N/mm}^2$

$E_c = 30 \text{ Gpa} = 3 \times 10^4 \text{ N/mm}^2$

$\alpha_e = \dfrac{E_s}{E_c} = 6.66$

Step-2:- Dimensions of box culvert :-

Adopting the thickness of slab as 100 mm/m span,

Thickness of slab = $t_s = t_w = 3 \times 100 = 300$ mm (Generally 1/10th to 1/15th of span).

∴ Effective span, $L = 3000 + \dfrac{300}{2} + \dfrac{300}{2} = 3300$ mm.

Step - 3:- Loads :-

Self weight of top slab $= 0.3 \times 24 = 7.2\ KN/m^2$

Superimposed dead load $= 12.8\ KN/m^2$

live load $= 50\ KN/m^2$

Total load $= w = 50 + 12.8 + 7.2 = 70\ KN/m^2$

weight of vertical side walls $= 0.3 \times 3.3 \times 24 = W = 24\ KN$

Angle of repose $= \phi = 30^\circ$

Height of Soil fill $= h = 3.3\ m$

Soil Pressure $= p = wh\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$

$$= 18 \times 3.3 \left(\frac{1 - \sin 30^\circ}{1 + \sin 30^\circ}\right) = 20\ KN/m^2$$

Uniform lateral pressure due to the effect of superimposed dead and live load surcharge is

$$p = (50 + 12.8)\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) = 62.8 \times 1/3 = 21\ KN/m^2$$

Uniform lateral pressure due to the effect of super imposed dead load surcharge only is evaluated as

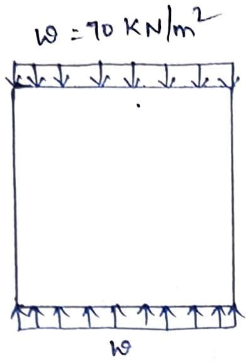
$$p = 12.8\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) = 12.8 \times 1/3 = 4.26\ KN/m^2$$

weight of water

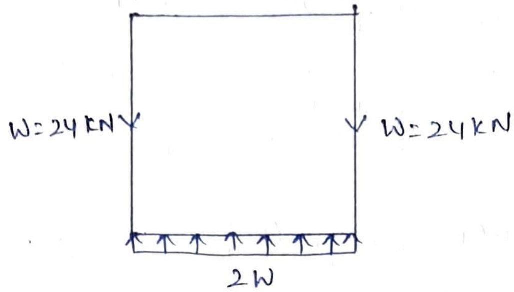
Intensity of water pressure $= p = wh = 10 \times 3.3 = 33\ KN/m^2$

Step - 4:- Analysis of moments, shears and thrusts :-

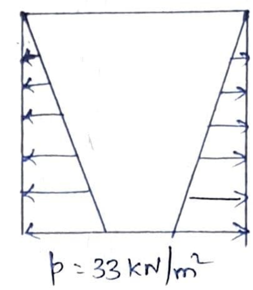
The various loading patterns considered are shown below.



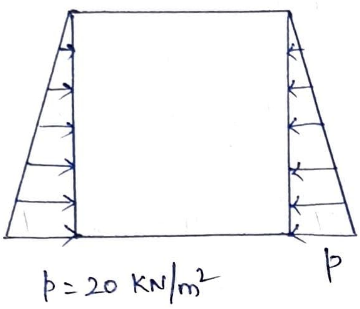
Case 2



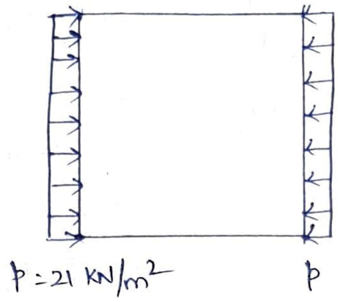
Case 3



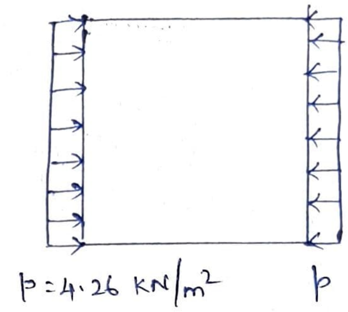
Case 4



Case 5



Case 6 (a)



Case 6(b)

The moments, shears and thrusts corresponding to the loading cases are evaluated using the coefficients ~~and~~ ~~obtained~~ are given below.

Section	Forces	Loading case					
		Case 2	Case 3	Case 4	Case 5	Case 6(a)	Case 6(b)
B-1	M (KNm)	63.27 ($70 \times 3.3^2 \times 0.083$)	1.66 ($24 \times 3.3 \times 0.021$)	6.82 ($33 \times 3.3^2 \times 0.019$)	-4.13 ($20 \times 3.3^2 \times -0.019$)	-9.6 ($21 \times 3.3^2 \times -0.042$)	-1.94 ($4.26 \times 3.3^2 \times -0.042$)
	N (KN)	0 ($70 \times 3.3 \times 0$)	0 (24×0)	-18.18 ($33 \times 3.3 \times -0.167$)	+11.02 ($20 \times 3.3 \times 0.167$)	+34.65 ($21 \times 3.3 \times 0.5$)	+7.02 ($4.26 \times 3.3 \times 0.5$)
A-2	M (KNm)	-32.01	1.66	6.82	-4.13	-9.6	-1.94
	N (KN)	0	0	-18.18	+11.02	+34.65	-7.02
	V (KN)	115.5	0	0	0	~~34.6~~ 0	0
A-3	M	-32.01	1.66	6.82	-4.13	-9.6	-1.92
	N	115.5	0	0	0	0	0
	V	0	0	18.18	-11.02	-34.65	-7.02
E-4	M	-32.01	-3.32	-15.45	-9.36	+18.98	+3.85
	N	115.5	~~+34.6~~ +12	0	0	0	0

		(2)	(3)	(4)	(5)	(6a)	(6b)
D-5	M	-32.61	-8.236	8.26	-5.00	-9.6	-1.94
	N	115.5	~~+79.2~~ +24	-36.26	+21.97	0	0
	V	0	0	0	0	+34.65	+7.03
D-6	M	-32.61	-8.236	8.26	-5.00	-9.6	-1.92
	N	0	0	0	0	+34.65	+7.03
	V	-115.5	~~-79.2~~ -24	-36.26	+21.97	0	0
C-7	M	63.27	11.56	8.26	-5.00	-9.60	-1.92
	N	0	0	-36.26	+21.97	+34.65	+7.03

The service load design forces resulting from the combination of various cases yielding maximum moments and forces at the supports and mid-span sections are given below.

Section	Loading Combination Cases	Moment (M) (KNm)	Thrust (N) (KN)	Shear force (V) (KN)
D-6	2+3+5+6 (a)	-54.84	+34.65	~~-172.8~~ -117.53
A-2	2+3+5+6 (a)	-44.08	-23.65	+115.5
B-1	2+3+4+5+6 (b)	65.68	-0.14	0
C-7	2+3+4+5+6 (b)	76.10	-7.26	0
E-4	2+3+4+5+6 (b)	-56.29	+127.5 ~~+155.1~~	0

The maximum positive moments develop at the centre of bottom ~~and top~~ slabs for the condition that the sides of the culvert not carrying the live load and the culvert is running full with water.

The maximum negative moments develop at the ~~centre of bottom~~ ~~side wall~~ support sections of the ~~bottom slab~~ bottom slab for the Condition that culvert is empty and the ~~top~~ slab carries the dead and live loads. (by considering only top and bottom slab excluding side wall)

Step - 5 :- Design of Reinforcements :-

(a) Section C-7 (mid span of bottom slab)

Service load moment = $M_w = 76.17$ kNm

Design moment = $M_u = 1.5 \times 76.17 = 114.25$ kNm

Service load thrust = $N = -7.28$ kN (Tension)

Design thrust = $N_u = 1.5 \times 7.28 = -10.89$ kN (Tension)

The parameters $N_u/f_{ck} bD$ and $M_u/f_{ck} bD^2$ for using the design charts of SP-16.

$$\frac{N_u}{f_{ck}\, bD} = \frac{10.89 \times 10^3}{20 \times 1000 \times 300} = 0.0018$$

$$\frac{M_u}{f_{ck}\, bD^2} = \frac{114.25 \times 10^6}{20 \times 1000 \times 300^2} = 0.0634$$

Adopting effective cover as $d' = 50$ mm, $d = 300 - 50 = 250$ mm

Ratio $d'/D = \frac{50}{300} = 0.16$

From design chart - 70 of SP-16 for tension and bending

$$\frac{p}{f_{ck}} = 0.05$$

$$\therefore p = \frac{100 A_{st}}{bd} = 0.05 \times 20 = 1$$

$$\Rightarrow A_{st} = \frac{bd}{100} = \frac{1000 \times 250}{100} = 2500 \text{ mm}^2/\text{m}$$ (equally distributed on both faces)

$\therefore$ A_{st} on each face $= \frac{2500}{2} = 1250 \text{ mm}^2$

Using 16 mm dia bars, Spacing $= \frac{a_{st}}{A_{st}} \times 1000$

$$= \frac{\pi/4 \times 16^2}{1250} \times 1000 = 160.87 \text{ mm}$$

$\therefore$ Provide 16 mm dia bars @ 150 mm c/c on each face as main reinforcement

$$A_{st} \text{ Provided} = 2 \times \frac{\pi/4 \times 16^2}{150} \times 1000 = \underline{\underline{2681.1 \text{ mm}^2}}.$$

Provide 10 mm dia bars @ 150 mm c/c on both faces as distribution reinforcement.

The serviceability limit state of cracking is easily satisfied since the diameter and spacing of bars are 16 mm and 150 mm respectively and the stress in steel at working loads will be less than 240 N/mm² for a Permissible crack width of 0.3mm, according to tables 12.2 and 12.3.

(b) <u>Section D-6 (Support Section)</u>:-

Service load moment $= M_w = -54.84$ KNm

Design moment, $M_u = 1.5 \times 54.84 = -82.26$ KNm

Service load thrust $= N = 34.65$ KN (Compression)

Ultimate thrust, $N_u = 1.5 \times 34.65 = 51.97$ KN (Compression)

$$\frac{N_u}{f_{ck} b D} = \frac{51.97 \times 10^3}{20 \times 1000 \times 300} = 0.0086$$

$$\frac{M_u}{f_{ck} b D^2} = \frac{82.26 \times 10^6}{20 \times 1000 \times 300^2} = 0.0457$$

Referring chart 33 of SP-16, $f_y = 415 \text{ N/mm}^2$, $d'/D = 0.15$, for Compression and bending

$$\frac{p}{f_{ck}} = 0.03$$

$\therefore p = \frac{100 A_{st}}{bd} = 0.03 \times 20 = 0.6$

$A_{st} = \frac{0.6 \times bd}{100} = \frac{0.6 \times 1000 \times 250}{100} = 1500 \text{ mm}^2$

Using 12 mm dia bars, spacing $= \frac{a_{st}}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 12^2}{750} \times 1000 = 150.81$ mm

Provide 12 mm dia bars @ 150 mm c/c both faces with 10 mm diameter at 150 mm c/c as distribution reinforcement.

Both strength and serviceability criteria (or) requirements are easily satisfied with the provided bar dia and spacing. from tables 6.12 and 6.13 of IRC: 112-2011,

(c) Section E-4 (vertical side wall):-

Service load moment $= M_w = -56.89$ KNm

Design ultimate moment $= M_u = 1.5 \times 56.89 = -84.43$ KNm

Service load thrust $= N = \cancel{155.1}$ 127.5 KN (Compression)

Ultimate thrust, $N_u = 1.5 \times \cancel{155.1}$ 127.5 $= \cancel{232.65}$ 191.25 KN (Compression).

$\frac{N_u}{f_{ck} bD} = \frac{191.25 \times 10^3}{20 \times 1000 \times 300} = 0.0318$

$\frac{M_u}{f_{ck} bD^2} = \frac{84.43 \times 10^6}{20 \times 1000 \times 300^2} = 0.0469$

Referring chart - 33 of SP-16 for $f_y = 415 \text{ N/mm}^2$, $d'/D = 0.15$ for compression and bending.

$\frac{p}{f_{ck}} = 0.025$

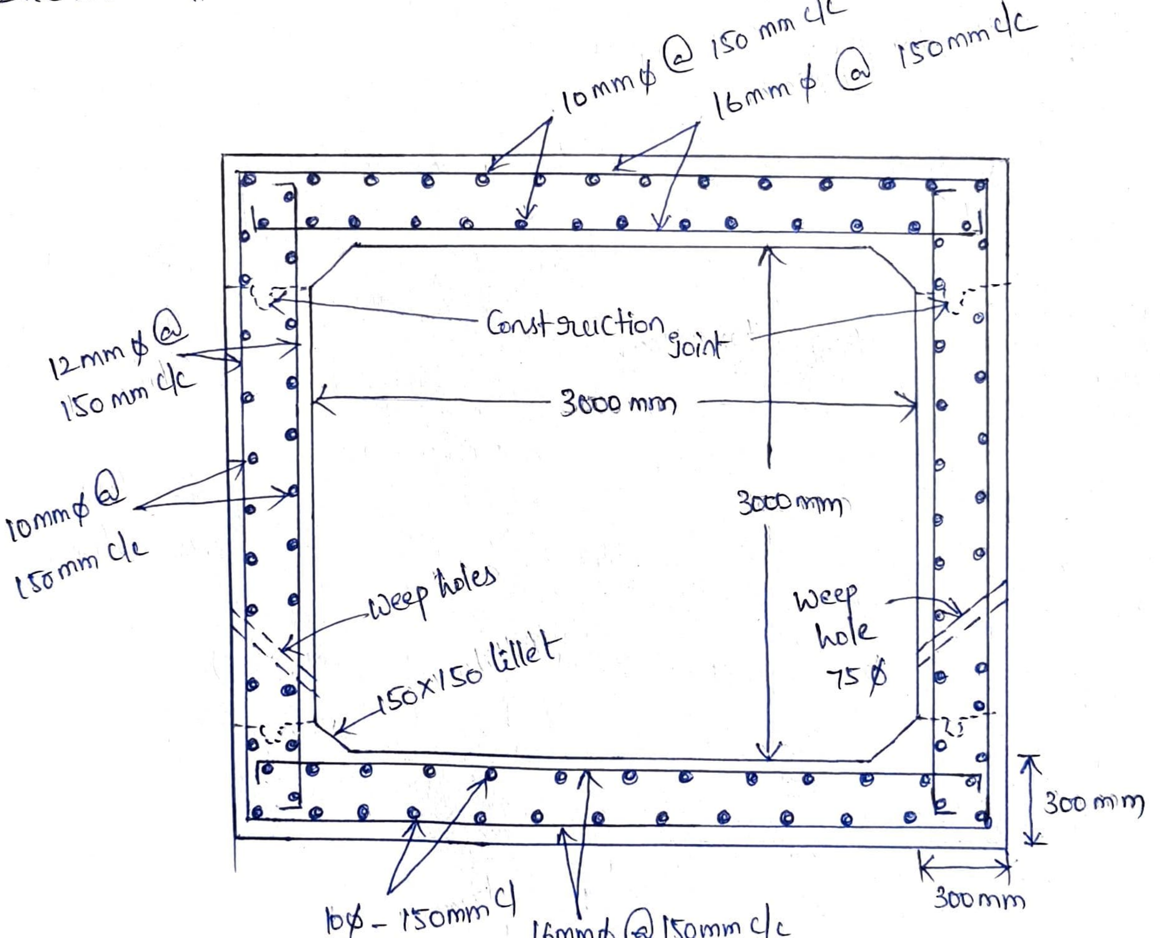
$\therefore p = \frac{100 A_{st}}{bd} = 0.025 \times 20 = 0.5$

$A_{st} = \frac{0.5 \; bd}{100} = \frac{0.5 \times 1000 \times 250}{100} = 1250 \text{ mm}^2$

Using 12 mm dia bars, spacing $= \frac{\pi/4 \times 12^2}{(1250/2)} \times 1000 = 180.97\,mm$

Provide 12 mm dia bars @ 150 mm c/c on both faces with 10 mm dia bars @ 150 mm c/c as distribution reinforcement.

Both strength and serviceability requirements are satisfied with the Provided dia of bar and spacing from tables 6.12 and 6.13 of IRC:112-2011.



Reinforcement Details of Box culvert

UNIT-II

Deck Slab Bridge:-

* Reinforced Concrete slab type decks are Commonly used for small spans.
* This type of superstructure is economical for spans upto 8m.
* For larger spans, Pre stressed Concrete slab decks are preferred since the thickness of the slab can be reduced.
* The deck slab is designed as a one way slab to support the dead and live loads with impact.
* National highway bridge decks slabs are generally designed support the IRC class AA (or) A type vehicle loads whichever gives the worst effect.
* According to the new IRC Code (IRC :112 -2011), RC bridge decks should be designed to confirm two basic groups of limit states.

  (i) Ultimate limit state (or) limit state of strength in which the structural element is designed to withstand safely the ultimate design loads obtained by applying suitable partial safety factors to the service loads.

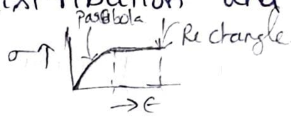
  (ii) Serviceability limit state in which the structural element should ~~perform~~ Perform its intended function satisfactorily at service loads without excessive deflection (or) displacement, or local cracking.
* The factor of safety for equilibrium, design strength and serviceability are Table 3.1 (Pg 75), Table 3.2 (Pg 77) and Table 3.3 (Pg 78) respectively in IRC : 6 - 2014.
* New IRC code also permits the use of high strength concrete (M_{30} to M_{90}) and high strength steel (Fe 415 to Fe 600).

* The strength and deformation characteristics of normal Concrete generally used for bridge Construction are given in Table 6.5, Pg 38, IRC:112-2011.

Flexural strength of Reinforced Concrete bridge decks:-

Basic Assumptions:-

1. Plane sections normal to the axis remain plane after bending.
2. The maximum strain in concrete at the outmost Compression fiber is taken 0.0035 in flexure.
3. The relation between the compressive stress distribution and strain in Concrete is assumed to be rectangular parabola.



4. The tensile strength of Concrete is ignored.
5. The stresses in the reinforcement are derived from representative stress - strain curve for the type of steel used.
6. The maximum strain in tension reinforcement in the section at failure shall be not less than that Computed by the relation

$$\epsilon_{su} = \left(\frac{f_y}{1.15 E_s} + 0.002\right) = \left(\frac{0.87 f_y}{1.15 E_s} + 0.002\right)$$

(All other moment equations are similar to Design of RC sections as per IS:456-2000.)

Shear strength of Reinforced Concrete bridge decks:- (Pg 85, IRC:112-2011)

* shear failures are likely to occur near the supports of bridge decks where maximum shear forces develop due to vehicle loads.

* The most Common types of shear failures are

(i) Diagonal tension

(ii) ~~Flexurde~~ Flexure - shear

(iii) shear - Compression

(iv) shear - Bond

(v) shear - Friction

* The ultimate shear strength of reinforced concrete beam (or) slab sections depends upon factors like percentage reinforcement ratio, grade of concrete and depth of slab.

* IS:456-2000 and IRC:112-2011 has ~~factor~~ incorporated the shear strength enhancing factor 'K' which depends on depth of slab.

* IS Code specifies the values of 'K' ranging from 1.00 to 1.30. for slab depths ranging from 300 mm to 150mm. respectively.

* The IRC Code specifies an empirical equation

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.00$$, where d = effective depth in mm.

* As per IRC bridge design code, The design shear resistance ($V_{Rd.c}$) of the member without shear reinforcement is given by

$$V_{Rd.c} = \left[0.12\,K\,(80\,P_1\,f_{ck})^{0.33}\right] b_w d$$ (Pg 88, cl.10.3.2, IRC:112-2011)

where, $P_1 = \left(\frac{A_{sl}}{b_w \cdot d}\right) \leq 0.02$

A_{sl} = Area of longitudinal reinforcement in the member

b_w = width of member in slabs (or) width of rib in beams

d = Effective depth of member.

(Note: → If nominal shear stress exceeds permissible value, the depth of slab is increased to avoid the use of shear reinforcements.)

Analysis of slab decks:-

* Reinforced Concrete slab decks used for small span culverts are generally spanning in one direction and hence the moments due to dead and live loads are critical in longitudinal direction i.e the direction of moving loads.

* Analysis of slabs with different support conditions are detailed ~~under~~ below.

1. Solid slabs spanning in one direction :- (Annexure B 3.2, Pg 278, IRC:112-2011)

(i) Single Concentrated load :-

* In this case, the dead load moments are directly computed assuming the slab to be simply supported between the bearings.

* live loads of vehicles transmitted through wheels are considered as Concentrated loads spread over the contact area of tyres with the deck slab.

* The bending moment per unit width of slab is calculated by assuming the width of slab considered as effective.

* The effective width of slab, for a single concentrated load calculated as

$$b_e = Kx\left(1-\frac{x}{L}\right)+b_w$$

where, b_e = Effective width of slab on which load acts

L = Effective span

x = Distance of C.G of load from nearer support

b_w = breadth of concentration area of load. i.e dimension of the tyre (or) track contact area over the road surface of slab in a direction at right angles to the span + Twice the thickness of wearing coat (or) surface finish above the structural slab.

K = Constant depending upon ratio of (B/L)

(Pg 279, IRC:112-2011)

* The effective width shall not exceed the actual width of slab

* In the case of a load near the unsupported edge of a slab, the effective width shall not exceed the above value nor half the above value + The distance of the load from the unsupported edge.

(ii) Two (or) more Concentrated loads in-line in the direction of span:-

* when two (or) more concentrated loads are positioned in a line in the direction of span, the bending moment per unit width of slab shall be calculated seperately for each load according to its appropriate effective width of slab as specified under the single concentrated load.

(iii) Two (or) more Concentrated loads not in line in the direction of span:-

* In cases where the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the resultant effective width for the two loads equals to the sum of effective widths for each load minus the width of overlap, Provided that the slab so designed is tested for the two loads acting seperately.

2. Solid Cantilever slab:- (Pg 279, IRC: 112-2011)

* The effective width of slab for a single Concentrated load is computed as

$$b_e = 1.2x + b_w$$

where x = Distance of C.G of load from face of Cantilever support.

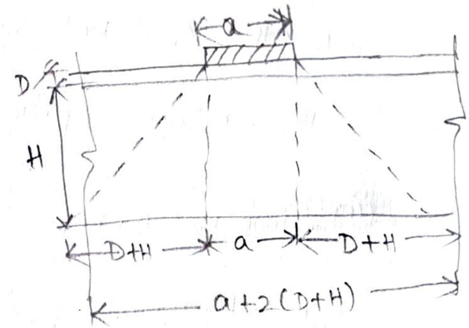
b_w = Dimension of tyre over the road surface of slab parallel to the supporting edge of Cantilever + 2x thickness of wearing Coat (or) surface finish.

* The effective width should be limited to one-third length of the cantilever slab measured parallel to the support.

3. Dispersion of loads Along the span :- (cl B.3.3, Pg 280, IRC : 112-2011)

* The effective length of slab

$$l = a + 2(D + H)$$



where,

D = Depth of the wearing coat

H = Depth of the slab

a = wheel load contact area along the span

l = Effective length of dispersion along the span.

Minimum and Maximum Reinforcements in slabs :- (cl 16.6.1, IRC : 112-2011 Pg 181 [cl 16.5.1, Pg 175])

* The minimum reinforcements in solid bridge deck slab is

$$A_{s,min} = 0.26 \left(\frac{f_{ctm}}{f_{yk}}\right) b_t d \not< 0.0013\, b_t d$$

where, b_t = Mean width of the tension zone in slabs and width of web in beams

f_{ctm} = Mean value of axial tensile strength of Concrete (Table 6.5, IRC : 112-2011)

d = Effective depth.

* The maximum reinforcement should not be greater than $0.025\, A_c$ at sections other than laps.

$$A_{s,max} \not> 0.025\, A_c$$

* Secondary transverse reinforcement of area not less than 20% of the main reinforcement should be provided for one way slabs.

* The maximum spacing of main reinforcements should be the lesser of 2 x thickness of slab (or) 250 mm.

* The maximum spacing of secondary reinforcement in one way slabs should be the lesser of 3 times the thickness of slab (or) 400 mm.

Control of Cracking in Bridge Decks:- (cl 12.3, Pg 121, IRC:112-2011)

* The maximum permissible width of cracks in reinforced and prestressed concrete bridge decks depending upon the conditions of exposure given as below in Table 12.1 of IRC:112-2011

Condition of exposure as per cl 14.3.1	Reinforced members and Prestressed members with un-bonded tendons Quasi-permanent load Combination (mm)	Prestressed members with bonded tendons Frequent load Combination (mm)
1. Moderate	0.3	0.2
2. Severe	0.3	0.2
3. Very severe and extreme	0.2	0.2 and decompression

* Minimum reinforcement requirements for crack control is given in cl 12.3.3 of IRC:112-2011.

Calculation of Crack width:-

* The width of cracks in the tension zone of RC bridge depends upon the maximum spacing of bars (Table 12.3, IRC:112-2011) and the mean strain in reinforcement and concrete between cracks.

* The design surface crack width is computed as

$$w_k = S_{r,max}(\epsilon_{sm} - \epsilon_{cm})$$

where, w_k : Design surface crack width

$S_{r,max}$: The maximum crack spacing

ϵ_{sm} : The mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations, restrained thermal shrinkage effects allowing for the effects of tension stiffening of the concrete. For prestressed members only, the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered.

ϵ_{cm} : Mean strain in concrete between the cracks.

$$\epsilon_{sm} - \epsilon_{cm} = \left[\frac{\sigma_s - K_t\left(\frac{f_{ct,eff}}{\rho_{p,eff}}\right)(1 + \alpha_e \rho_{p,eff})}{E_s}\right] \geq 0.6\left(\frac{\sigma_s}{E_s}\right)$$

where,

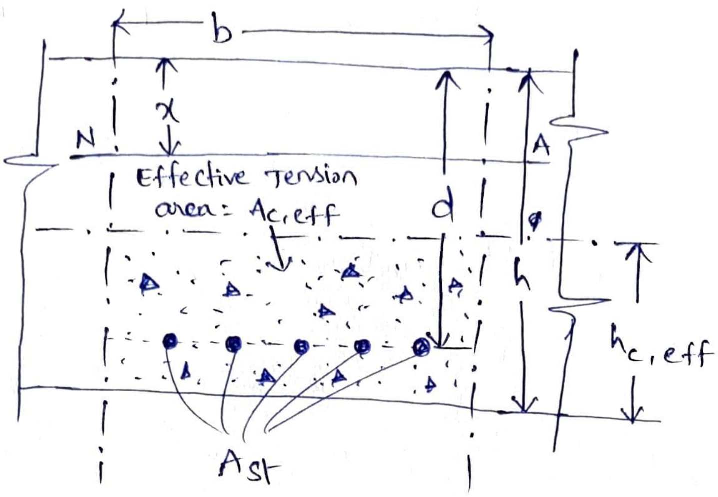
σ_s : Stress in tension reinforcement calculated using the cracked concrete section

K_t : a factor that accounts for the duration of loading which may be taken as 0.5

$\rho_{p,eff} = \frac{A_s}{A_{c,eff}}$ is the effective reinforcement ratio based on an effective concrete tension area ($A_{c,eff}$)

$f_{ct,eff}$ = Concrete tensile strength at the time of cracking (Table 6.5, IRC:112-2011)

α_e : Modular ratio $= \frac{E_s}{E_{cm}}$



$h_{c,eff}$ is the least of

(a) $2.5\,(h-d)$

(b) $(h-x)/3$

(c) $h/2$

* The maximum crack spacing $S_{r,max}$ is calculated as

$$S_{r,max} = 3.4c + (0.425\, k_1 k_2\, \phi)/\rho_{p,eff}$$

where, ϕ = bar diameter (avg of bars if different sizes are used)

c = Cover to the reinforcement

k_1 = 0.8 for high bond bars and 1.6 for Plain bars

k_2 = 0.5 for flexure and 1 for direct tension

For the case of deformed bars associated with bending

$$S_{r,max} = 3.4c + 0.425 \times 0.8 \times 0.5\, \phi / \rho_{p,eff}$$

$$= 3.4c + \frac{0.17\phi}{\rho_{p,eff}}$$

* upper bound limit to the crack width may be calculated by assuming maximum crack spacing

$$S_{r,max} = 1.3\,(h-x)$$

where reinforcement spacing exceeds $5\,(c+\phi/2)$

where, h: effective depth and x = depth of neutral axis from the Compression face.

Control of Deflection in Bridge Decks :- (cl 12.4, Pg 131, IRC : 112-2011)

Limiting values of Deflection :-

* The following deflection limits under live load by taking into account the nature of the super structure, bridge deck furniture and functional needs of the bridge are given.

(i) vehicular - span/800

(ii) vehicular and pendestrian (or) pedestrian alone - span/1000

(iii) Vehicular on cantilever - $\frac{\text{Cantilever span}}{300}$

(iv) Vehicular and pedestrian (or) pedestrian only on cantilever arms - Cantilever span/375

Calculation of deflection due to sustained loads :-

* The computation of deflection considered in two parts

(i) Instantaneous (or) short term deflections occuring on application of the loads.

(ii) Long term deflections resulting from differential shrinkage and creep due to sustained loading.

* In case of cracked members, appropriate value of cracked moment of inertia should be used in the computations.

* If actual value of cracked moment of Inertia cannot be determined, the code permits the use of 70 % of gross moment of inertia for computations.

* Deflection due to creep effects of sustained loads over a long period is calculated by using the effective modulus of elasticity for concrete as

$$E_{c,eff} = \left[\frac{E_{cm}}{1+\phi(\infty, t_0)}\right]$$

where, E_{cm} = Secant modulus of elasticity

$\phi(\infty, t_0)$ = Creep coefficient relavent for the load and time interval (Table 6.9, Pg 47, IRC:112-2011)

* The Deflection due to shrinkage is expressed as

$$a_{cs} = K\left(\frac{1}{r_{cs}}\right)L^2 = K\,\varphi_{cs}\,L^2$$

where, K = a constant depending upon the support conditions

= 0.5 for cantilevers

= 0.125 for simply supported members

= 0.086 for members continuous at one end

= 0.063 for fully continuous members.

$\varphi_{cs} = \frac{1}{r_{cs}}$ = shrinkage curvature

$= \epsilon_{cs}\,\alpha_e\left(\frac{S}{I}\right)$

ϵ_{cs} = free shrinkage strain = $(\epsilon_{cd} + \epsilon_{ca})$ (Pg 44, cl 6.4.2.6)

α_e = Effective modular ratio = $(E_s/E_{c,eff})$

S = First moment of area of the reinforcement about centroid of the section

I = Second moment of area of the section (Moment of Inertia).

ε_{ca} = Auto genous shrinkage strain (Table 6.6, Pg 45, IRC:112-2011)

Grade of Concrete	M30	M35	M45	M50	M60	M65	M25
Autogenous shrinkage strain ($\varepsilon_{ca} \times 10^{-6}$)	35	45	65	75	95	105	30

ε_{cd} = unrestrained Drying shrinkage strain (cl. 6.4.2.6 (4), Pg 45, IRC:112-2011)
(Table 6.8, Pg 46, IRC:112-2011).

= $K_h \, \varepsilon_{cd}$ (Final value of drying shrinkage strain).

where, K_h is a factor depends upon the notional size h_o.
(Table 6.7, Pg 45)

h_o = Notional size (mm) of the cross section = $2A_c/u$

A_c = Concrete c/s area

u = Perimeter of that part of cross section which is exposed to drying.

* The development of drying shrinkage strain in time can be taken as

$$\varepsilon_{cd}(t) = \left(\beta_{ds}(t, t_s) \, K_h \, \varepsilon_{cd}\right)$$

where, $$\beta_{ds}(t, t_s) = \left[\frac{t - t_s}{(t - t_s) + 0.04\sqrt{h_o^3}}\right]$$

t = Age of Concrete in days at the time Considered

t_s = Age of Concrete in days at the beginning of drying shrinkage, normally this is at the end of curing (28 days)

Minimum grade of Concrete and Cover requirements :-

* Bridge structures have to be designed for a service life of atleast 100 years and the new code specifies the maximum w/c ratio, minimum Cement Content and grade of Concrete along with the Cover requirements for different types of environmental exposure Conditions.

(Table 14.2, Pg 142, IRC:112-2011)

Exposure Condition	Maximum w/c ratio	Min. Cement Content (kg/m^3)	Minimum Grade of Concrete	Minimum Cover (mm)
1. Moderate	0.45	340	M25	40
2. Severe	0.45	360	M30	45
3. Very severe	0.40	380	M40	50
4. Extreme	0.35	400	M45	75

(Pb) Design the reinforced Concrete slab deck and sketch the details of reinforcement in the longitudinal and cross section of the slab for a reinforced Concrete slab culvert for a national highway crossing to suit the following data.

Carriage way - Two lane (7.5 m wide)

Foot Paths - 1 m on either side

clear span = 6m

wearing Coat = 80 mm

Width of bearing = 400 mm

Materials - M_{25} grade Concrete and Fe_{415} grade HYSD bars

Loading - IRC class AA tracked vehicle

The design should Confirm to the specifications of IRC:6-2014 and IRC:112-2011.

Sol:- step-1:- Given data:-

clear span = 6m

width of bearing = 400mm

IRC class AA Tracked vehicle loading

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$

Assume $E_s = 200\ GPa = 2\times10^5\ N/mm^2$

$E_c = 30\ GPa = 3\times10^4\ N/mm^2$

$\therefore \alpha_e = \frac{E_s}{E_c} = 6.66$

Step-2:- Depth of slab and effective span

Assume the thickness of the slab to be 80 mm/m span of the bridge deck.

(Note: Thickness of slab = 80 to 90 mm per meter span of deck

The larger depth is needed for small spans inorder to satisfy shear criteria)

$\therefore$ Overall depth of slab = $80 \times 6 = 480$ mm $\simeq 500$ mm

Assuming moderate exposure Conditions, clear Cover = 40 mm (Table 14.2, Pg 142)

Using 20 mm diameter HYSD bars as main reinforcements with a clear Cover of 40 mm,

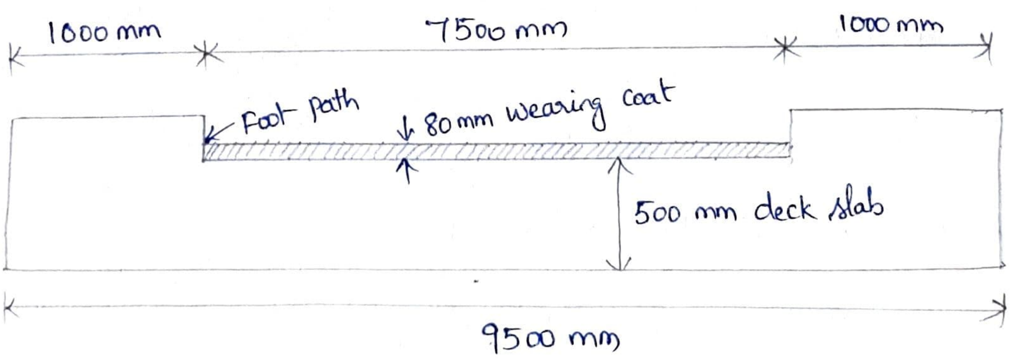
Effective depth $= 500 - 40 - 20/2 = 450$ mm

width of bearing = 400 mm

Effective span is the least of

(i) clear span + effective depth $= 6 + 0.45 = 6.45$ m

(ii) Centre to Centre of bearings $= 6 + 0.4 = 6.4$ m



Cross section of deck slab

Step-3:- Dead load bending moments:-

Dead weight of slab $= 0.5 \times 24 = 12$ kN/m^2

Dead weight of wearing coat $= 0.08 \times 22 = 1.76$ kN/m^2

$\therefore$ Total load $= 12 + 1.76 = 13.76 \ KN/m^2$

$\therefore$ Dead load bending moment $= \dfrac{13.76 \times 6.4^2}{8} = 70.45 \ KNm$

step - 4 :- Live load bending moments :-

Generally the bending moment due to live load will be maximum for IRC class AA ~~load~~ tracked vehicle.

The impact factor for IRC class AA Tracked vehicle is 25% for spans upto 5m linearly reducing to 10% for spans upto 9m.

(cl 208.3, a, i, Pg 25, IRC:6-2014)

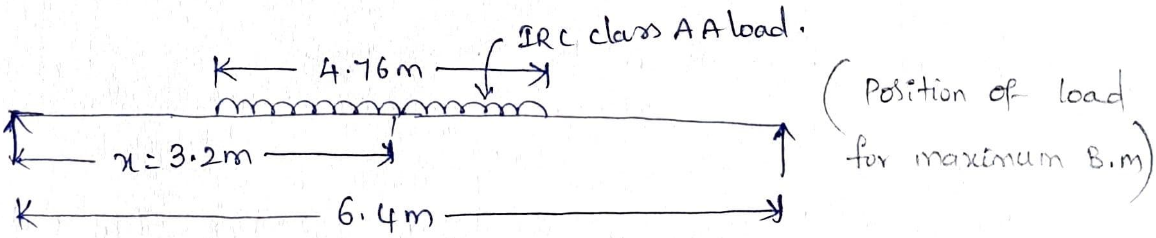
$\therefore$ For 6.4m span, Impact factor $= 25 - \dfrac{25-10}{9-5}(6.4-5)$

$= 19.75\%$

Assume
The tracked vehicle is placed symmetrically on the span

(cl B3.3, Pg 280, IRC:112-2011)

Effective length of load $= [3.6 + 2(0.5 + 0.08)] = 4.76 \ m.$



Effective width of slab perpendicular to ~~slab~~ span

$b_e = Kx\left(1 - \dfrac{x}{L}\right) + b_w$

$x = 3.2m, \ L = 6.4m, \ B = 9.5m$ (including foot paths)

$B/L = \dfrac{9.5}{6.4} = 1.48$

$b_w = (0.85 + 2 \times 0.08) = 1.01 m$

Refer figure of class AA loading

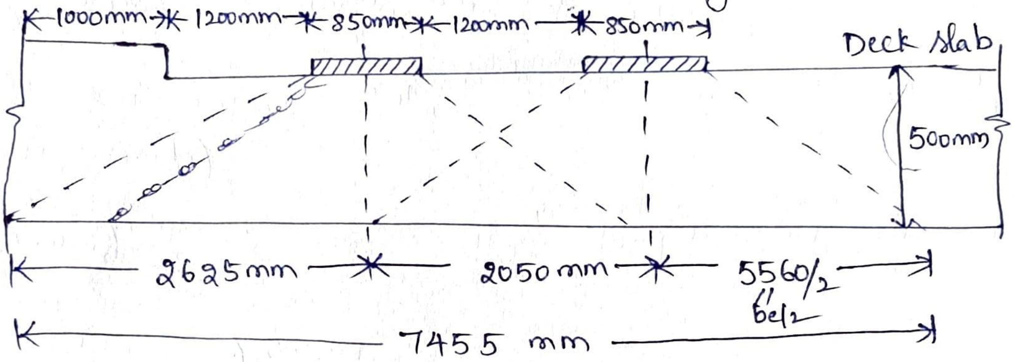
From Table in Pg 279 of IRC : 112-2011,

for $B/L = 1.48$ and simply supported slab, $K = 2.84$

$\therefore b_e = 2.84 \times 3.2 \left(1 - \frac{3.2}{6.4}\right) + 1.01$

$= 5.56$ m

Assume the tracked vehicle is placed close to the kerb with the required minimum clearance as shown in figure below.



Net effective width of dispersion $= 7.455$ m

Total load of two tracks with impact $= 700 \times 1.1975 = 838.25$ kN

Average intensity of load $= \frac{838.25}{4.76 \times 7.455} = 23.62$ kN/m²

Maximum bending moment due to live load (at centre)

$M_{max} = \left[\underbrace{\frac{1}{2} \times 23.62 \times 4.76}_{W} \times \underbrace{3.2}_{4/2}\right] - \left[\frac{1}{2} \times 23.62 \times 4.76 \times \frac{4.76}{4}\right]$

$= 113$ kNm

$\therefore$ Total design bending moment $= M_d + M_L$

$= 70.45 + 113 = 183.45$ kNm

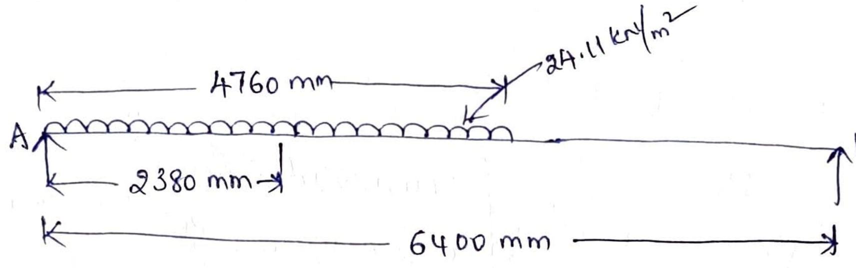
Total design ultimate moment $= 1.35\, M_d + 1.5\, M_L$ (from Table 3.2, Pg 77, IRC:6-2014)

$= 1.35 \times 70.45 + 1.5 \times 113$

$= 264.6$ kNm/m

Step-5:- Shear due to class AA Tracked vehicle :-

For maximum shear at support, the IRC class AA Tracked vehicle is arranged as shown.



(Position of load for maximum shear)

Effective width of dispersion is given by

$$b_e = K x \left(1 - \frac{x}{L}\right) + b_w$$

$x = 2.38\,m$, $B = 9.5\,m$, $L = 6.4\,m$, $B/L = 1.48 \Rightarrow K = 2.84$

$$\therefore b_e = 2.84 \times 2.38 \left(1 - \frac{2.38}{6.4}\right) + 1.01$$

$$= 5.256\ m$$

$\therefore$ width of dispersion $= 2625 + 2050 + \frac{5256}{2} = 7303$ mm

Average intensity of load $= \frac{838.25}{4.76 \times 7.303} = 24.11\ kN/m^2$

$\therefore$ shear force $= V_A = \frac{24.11 \times 4.76 \times (6.4 - 2.38)}{6.4} = 72\ kN$

Dead load shear $= \frac{1}{2} \times 13.76 \times 6.4 = 44.03\ kN.$

$\therefore$ Total design shear force $= 72 + 44.03 = 116.03\ kN.$

Total design ultimate shear force $= 1.35\,V_d + 1.5\,V_L$

$= 1.35 \times 44.03 + 1.5 \times 72$

$= 167.44\ kN/m.$

Step-6 :- Design of Deck slab :-

Using M_{25} grade concrete and Fe_{415} HYSD bars,

Limiting moment of resistance for singly reinforced sections is expressed as

$$M_{ulim} = 0.138\, f_{ck}\, b d^2$$

$$d = \sqrt{\frac{M_u}{0.138\, f_{ck}\, b}} = \sqrt{\frac{264.6 \times 10^6}{0.138 \times 25 \times 1000}} = 276.94 \text{ mm}$$

Since the effective depth ~~assu~~ provided is 450 mm, the section is under reinforced.

$$\therefore M_u = 0.87\, f_y\, A_{st}\, d \left(1 - \frac{f_y A_{st}}{f_{ck}\, b d}\right)$$

$$264.6 \times 10^6 = 0.87 \times 415 \times A_{st} \times 450 \times \left(1 - \frac{415\, A_{st}}{25 \times 1000 \times 450}\right)$$

$$= 162472.5\, A_{st} - 5.99\, A_{st}^2$$

$$\Rightarrow A_{st} = 1740 \text{ mm}^2$$

Using 20 mm dia bars as main reinforcement, the spacing is.

$$S = \frac{1000\, a_{st}}{A_{st}} = \frac{\pi/4 \times 20^2}{1740} \times 1000 = 180.5 \text{ mm}$$

Provide 20 mm dia bars @ 150 mm c/c.

$$A_{st\,prov} = \frac{a_{st} \times 1000}{S} = \frac{\pi/4 \times 20^2}{150} \times 1000 = 2094 \text{ mm}^2$$

The distribution reinforcement should be designed to resist transverse moment.

$$\text{Transverse moment} = (0.3\, M_{UL} + 0.2\, M_{UD})$$

$$M_{UL} = 1.5 \times 113 = 169.5 \text{ KNm}$$

$$M_{UD} = 1.35 \times 70.45 = 95.1 \text{ KNm}$$

$\therefore$ Transverse moment $= 0.3 \times 169.5 + 0.2 \times 95.1$

$= 69.87$ KNm

Area of distribution reinforcement $= \frac{2094}{264.6} \times 69.87$

$= 552.93 \text{ mm}^2$

$\therefore$ Provide 12 mm dia bars @ 200 mm c/c.

step - 7 :- check for ultimate flexural strength :-

$$M_u = 0.87 \times 415 \times 2094 \times 450 \left(1 - \frac{415 \times 2094}{25 \times 1000 \times 450}\right)$$

$= 314$ KNm > 264.6 KNm

Hence Safe

step - 8 :- check for ultimate shear strength :-

The ultimate shear strength of reinforced concrete deck slab is calculated as

$$V_{Rdc} = \left[0.12K(80\,\rho_1 f_{ck})^{0.33}\right] b_w d$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2 = 1 + \sqrt{\frac{200}{450}} = 1.66$$

$$\rho_1 = \frac{A_{sl}}{b_w d} = \frac{2094}{1000 \times 450} = 0.0046$$

$$\therefore V_{Rdc} = 0.12 \times 1.66 \left(80 \times 0.0046 \times 25\right)^{0.33} \times 1000 \times 450$$

$= 186.44$ KN > 169.44 KN

Hence safe

step - 9 :- check for Serviceability limit states :-

(a) Limit state of cracking :-

The width of cracks, $w_k = S_{r,max} (\epsilon_{sm} - \epsilon_{cm})$

$$\epsilon_{sm} - \epsilon_{cm} = \frac{\sigma_s - k_t \left(\frac{f_{ct,eff}}{\rho_{p,eff}}\right)(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6 \left(\frac{\sigma_s}{E_s}\right)$$

$k_t = 0.5$

$f_{ct,eff} = f_{ctm} = 2.2\ N/mm^2$ for M_{25} grade (Table 6.5, IRC:112-2011)

$\alpha_e = 6.66$

For the cracked section using the parameters

$h = 500mm$, $b = 1000mm$, $d = 450mm$, $A_s = 2094\ mm^2$, $\alpha_e = 6.66$

Neutral axis depth can be computed as

$$0.5\, b x^2 = \alpha_e A_s (d - x)$$

$$0.5 \times 1000 \times x^2 = 6.66 \times 2094 (450 - x)$$

$$\Rightarrow x = 98.95\ mm$$

$$\rho_{p,eff} = \frac{A_s}{A_{c,eff}} = \frac{A_s}{b \times h_{c,eff}}$$

$h_{c,eff}$ is the least of

(a) $2.5(h-d) = 2.5(500-450) = 125\ mm$

(b) $\frac{(h-x)}{3} = \frac{500-98.95}{3} = 133.6\ mm$

(c) $h/2 = 500/2 = 250\ mm$

$\therefore h_{c,eff} = 125\ mm$

$$\therefore \rho_{p,eff} = \frac{2094}{1000 \times 125} = 0.0167$$

~~$\therefore (\epsilon_{sm} - \epsilon_{cm}) =$~~

σ_s = stress in steel at service load = $\dfrac{M}{\left(d-\frac{x}{3}\right) A_{st}}$

$= \dfrac{183.45 \times 10^6}{\left(450 - \frac{98.95}{3}\right) \times 2094} = 210.08\ N/mm^2$

$\therefore (\epsilon_{sm} - \epsilon_{cm}) = \dfrac{210.08 - 0.5\left(\frac{2.2}{0.0167}\right)(1 + 6.6 \times 0.0167)}{2 \times 10^5}$

$= \underline{\underline{6.847 \times 10^{-4}}} \geq 0.6 \dfrac{\sigma_s}{E_s} = 0.6\left(\dfrac{210.08}{2 \times 10^5}\right) = 6.302 \times 10^{-4}$

Max. spacing of cracks, $S_{r,max} = 3.4C + 0.17\ \phi/\rho_{p,ef}$, If spacing is less than $5(C + \phi/2)$

$5(C + \phi/2) = 5(40 + 20/2) = 250mm$

spacing provided $= 150\,mm < 250\,mm$, Hence the above equation can be used.

$\therefore S_{r,max} = 3.4 \times 40 + \dfrac{0.17 \times 20}{0.0167} = 339.59\ mm$

$\therefore$ width of crack $= S_{r,max}(\epsilon_{sm} - \epsilon_{cm})$

$= 339.59 \times 6.847 \times 10^{-4} = 0.23\,mm$

~~$0.23 \ngeq 0.6\left(\frac{\sigma_s}{E_s}\right) = 0.6\left(\frac{210.08}{2 \times 10^5}\right) > 0.0006\,mm$~~

The maximum crack width is less than the permissible value 0.3 mm
Hence Safe.

Simple calculation for Control of Cracking

Service load moment $= 183.45\ KNm$

$\sigma_s = 210.08\ N/mm^2$

The slab is reinforced with 20 mm and spacing 150 mm c/c.

Referring to tables 12.2 and 12.3 of IRC : 112 - 2011, bar size and spacing are with in limits.

Hence Safe.

(b) Limit state of deflection :-

(i) Deflection due to shrinkage

$$a_{cs} = K \psi_{cs} L^2$$

$K = 0.125$ for simply supported beam

ψ_{cs} = shrinkage curvature = $\varepsilon_{cs} \, \alpha_e \left(\frac{S}{I}\right)$

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \, K_h \, \varepsilon_{cd}$$

$$\beta_{ds}(t, t_s) = \frac{t - t_s}{(t - t_s) + 0.04\sqrt{h_0^3}}$$

$$h_0 = \frac{2A_c}{u} = \frac{2 \times 1000 \times 500}{2000}$$

foot path area ; foot path ; (1000 + 1000)

$= 500$ mm

$t = 365$ days , $t_s = 28$ days

$$\beta_{ds}(t, t_s) = \frac{365 - 28}{(365 - 28) + 0.04\sqrt{500^3}} = 0.429$$

$K_h = 0.70$ (Table 6.7, IRC : 112 – 2011)

ε_{cd} for relative humidity of 50% for M_{25} grade Concrete

From Table 6.8 , $\varepsilon_{cd} = 535 \times 10^{-6}$

$\therefore \varepsilon_{cd}(t) = 0.429 \times 0.70 \times 535 \times 10^{-6} = 1.60 \times 10^{-4}$

$\varepsilon_{ca} = 30 \times 10^{-6}$

$\therefore$ Total shrinkage strain, $\varepsilon_{cs} = 1.60 \times 10^{-4} + 30 \times 10^{-6}$

$= 1.9 \times 10^{-4}$

S = First moment of area of the reinforcement about centroid of section

$= 2094 \times 200 = 418.8 \times 10^3$ mm^3

$$I_g = \frac{bh^3}{12} = \frac{1000 \times 500^3}{12} = 10.4 \times 10^9 \text{ mm}^4$$

$$\varphi_{cs} = \cancel{2.02}\ 1.90 \times 10^{-4} \times 6.66 \times \frac{\cancel{580.8}\ 418.8 \times 10^3}{10.4 \times 10^9}$$

$$= \cancel{7.51}\ 5.09 \times 10^{-8}$$

$$\therefore a_{cs} = 0.125 \times \cancel{7.51}\ 5.09 \times 10^{-8} \times 6400^2$$

$$= 0.260 \text{ mm}$$

(ii) Long term deflection due to sustained loads (dead loads)

Total dead load, $g = 13.76$ kN/m (Per 1 m width)

$= 13.76$ N/mm

Effective span, $L = 6.4$ m $= 6400$ mm

Modulus of elasticity of Concrete $= 30 \times 10^3$ N/mm^2

Maximum short term deflection due to dead load $= a_g = \dfrac{5gL^4}{384 E_c I_{eff}}$

$$I_{eff} = \frac{I_r}{1.2 - \left(\frac{M_r}{M}\right)\left(\frac{z}{d}\right)\left(1-\frac{x}{d}\right)\left(\frac{b_w}{b}\right)}$$

$$I_r = \frac{bx^3}{3} + m A_{st}\, r^2 \quad , \quad r = d - x = 450 - 98.95 \leftarrow \text{step-a)}$$

$$= 351.05 \text{ mm}$$

$$= \frac{1000 \times 98.95^3}{3} + 6.66 \times 2094 \times 351.05^2$$

$$= 2.04 \times 10^9 \text{ mm}^4$$

$$z = \text{lever arm} = d - x/3 = 450 - \frac{98.95}{3} = 417.01 \text{ mm}$$

$$M_r = \frac{f_{cr} I_g}{y_t} = \frac{3.5 \times 10.4 \times 10^9}{500} = 72.8 \times 10^6 \text{ Nmm}$$

$$\left(f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{25} = 3.5 \text{ N/mm}^2\right)$$

$$M = \frac{wL^2}{8} = \frac{13.76 \times 6.4^2}{8} = \cancel{64.63}\ 70.45 \text{ kNm}$$

$$\therefore I_{eff} = \frac{2.04 \times 10^9}{1.2 - \left(\frac{72.8\times10^6}{\cancel{69.63}\ 70.45 \times 10^6}\right)\left(\frac{417}{450}\right)\left(1-\frac{98.95}{450}\right)\left(\frac{1000}{1000}\right)}$$

$$= \cancel{2.82}\ 4.5 \times 10^9\ mm^4$$

$$\therefore \text{deflection},\ a_g = \frac{5 \times 13768 \times 6400^4}{384 \times 30\times10^3 \times \cancel{2.82}\ 4.5 \times 10^9}$$

$$= \cancel{3.55}\ 2.22\ mm$$

For computing long term deflection, effective modulus of elasticity, involving the effect of creep has to be used.

$$E_{c,eff} = \frac{E_c}{1+\phi}$$

ϕ value depends upon the notional size of the member $= 2A_c/u$

$= 500\ mm$

From Table 6.9, Pg.47, IRC:112-2011, for age at loading of 28 days and relative humidity of 50%, $\phi = 2.6$ (By interpolation)

Hence, $E_{ceff} = \frac{E_c}{1+2.6} = \frac{E_c}{3.6}$

$\therefore$ Long term deflection due to permanent loads $= \frac{5gL^4}{384\ E_{ceff}\ I_{eff}}$

$$= 3.6\left(\frac{5gL^4}{384\ E_c\ I_{eff}}\right)$$

$$= 3.6 \times \cancel{3.55}\ 2.22$$

$$= \cancel{12.78}\ 7.99\ mm$$

(iii) Deflection due to live loads:-

The live load due to the IRC class AA Tracked vehicle is computed as 23.61 KN/m² spread over a length of 4.76 m at the centre of span 6.4 m.

Assuming the entire span is loaded with an U.D.L of 23.61 kN/m, a conservative estimate of the maximum deflection is computed as

$$a_q = \frac{5qL^4}{384 E_c I_{eff}} = \frac{5 \times 23.61 \times 6400^4}{384 \times 30000 \times \cancel{2.82}\,4.50 \times 10^9}$$

$$= \cancel{6.08}\ 3.82 \text{ mm}$$

∴ The total deflection = deflection due to shrinkage + deflection due to dead loads + live loads

$$= \cancel{0.384}\ 0.260 + \cancel{12.78}\ 7.99 + \cancel{6.08}\ 3.82$$

$$= \cancel{19.244}\ 12.07 \text{ mm}$$

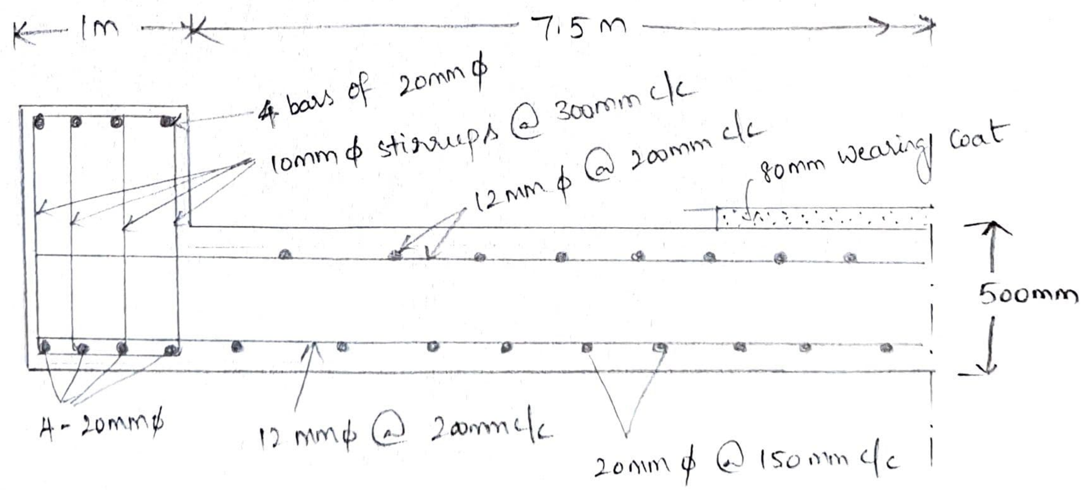
Maximum deflection due to live loads $\leq \frac{\text{Span}}{800} = \frac{6400}{800} = 8 \text{ mm}$

$\cancel{6.08}\ 3.82 \text{ mm} \leq 8 \text{ mm}$

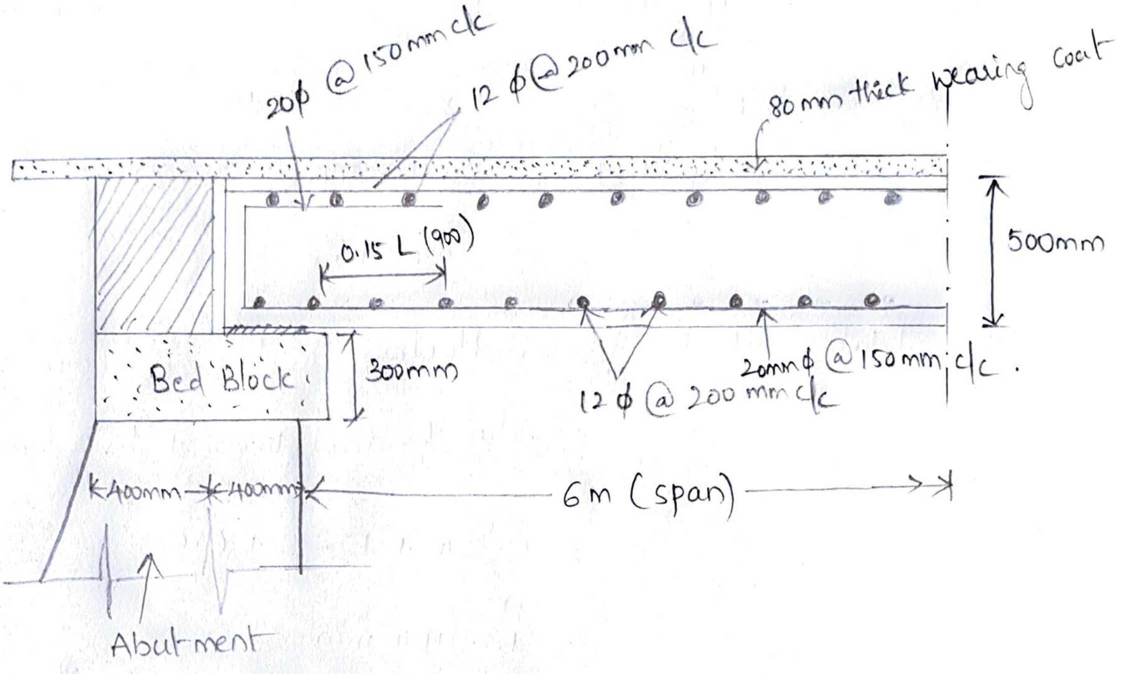
Total maximum deflection $\leq \frac{\text{span}}{250} = \frac{6400}{250} = 25.6 \text{ mm}$

$\cancel{19.24}\ 12.07 \leq 25.6 \text{ mm}$

Hence the design is safe



Cross section of Deck slab



Longitudinal section of deck slab

Reinforcement Details in deck slab

Design of Kerb:-

The kerb may be designed for a live load of 4 KN/m² (pedestrian load).

The minimum height of the kerb may be taken 225 mm above the road level.

Total depth of kerb = 500 + 80 + 225 = 805 mm

Live load / metre run of the kerb = $\underset{\text{width}}{1} \times \underset{\text{length}}{1} \times 4 = 4$ KN/m

Dead load of the kerb = $\underset{\text{depth}}{0.805} \times \underset{\text{width}}{1} \times 24 = 19.32$ KN/m

weight of the railings = 0.5 KN/m

Total load = 23.82 KN/m

∴ Bending moment = $\frac{23.82 \times 6.4^2}{8} = 121.96$ KNm

As the kerb is also a part of the deck slab, the vehicular load will have influence in generating bending moment in the kerb. This bending moment is normally taken as 50% of the live load bending moment obtained for the slab.

∴ Live load bending moment generated in the kerb = 0.5 × 113 = 56.5 KNm

Design bending moment = D.L B.M + L.L B.M

= 121.96 + 56.5 = 178.46 KNm

Hence, effective depth required = $\sqrt{\frac{178.46 \times 10^6}{1000 \times 25 \times 0.138}}$ = ~~84.49 mm~~ 227.43 mm < 805 mm

Hence safe.

Using 20 mm dia bars, available depth = 805 − 40 − 10 = 755 mm

~~∴ Area of steel = Ast 178.46×10^6~~

$Mu = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$.

$$\Rightarrow 178.46 \times 10^6 = 0.87 \times 415\, A_{st} \times 755 \left(1 - \frac{415\, A_{st}}{25 \times 1000 \times 755}\right)$$

$$\Rightarrow A_{st} = 664.38\ mm^2$$

$$\text{No. of bars} = \frac{664.38}{\pi/4 \times 20^2} = 2.11 \simeq 4 \text{ bars}$$

Provide 4 bars of 20 mm dia as main reinforcement.

4 legged stirrups of 10 mm dia are provided at a nominal spacing of 300 mm.