Bridges are structures built for carrying the road/railway traffic or other moving loads over a depression or gap or obstruction such as a river, channel, canyon, valley, road or railway. If a

COMPONENT PARTS OF A BRIDGE

A bridge is divided into the following component parts:

1. Substructure : Foundation, Piers, abutments, wing walls & approaches.
2. Bearings.
3. Superstructure.
4. Ancillary works such as protective works, etc.

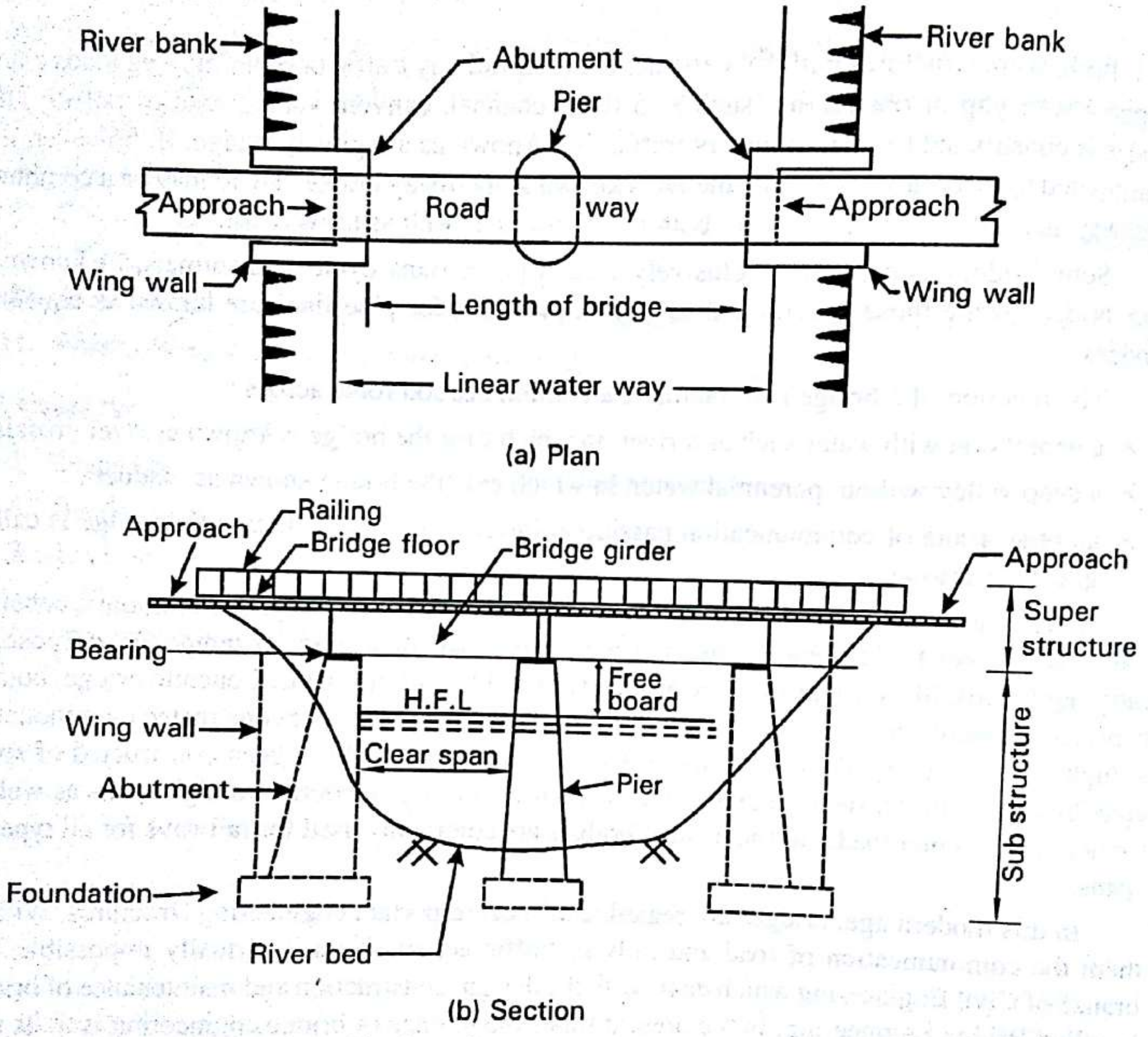

Fig. 9.1 : Component parts of a bridge

1. Substructure

(The function of substructure is to support the superstructure and to provide access to the traffic to the level of bridge superstructure through approaches.)

Thus the substructure supports the super structure and distributes the loads to the soil below through foundation. The sub-structure consists of foundation, piers, abutments, wing walls and approaches. They all support the superstructure of the bridge.

2. Bearings

The devices fixed on abutments and piers to allow for free expansion, contraction and deflection of the bridge superstructure are known as bridge bearings.

The structure below the bearings is known as substructure and the portion of bridge above the bearing is known as superstructure.

3. Superstructure

The upper part of a bridge consisting of structural system in the form of beams, girders, arches, suspension cables, parapet or railing, footway slab, etc. carrying the communication routs is called superstructure.

The function of superstructure is to provide carriageway over which the traffic moves with safety.

4. Ancillary works such as protective works, etc. : River training works like revetment for slopes of abutments, aprons for beds upstream and domstream, guide bunds, etc.

SELECTION OF BRIDGE SITE

Before constructing a bridge at a particular site, it is essential to consider the factors such as, need of the bridge, present and future traffic volume, characteristics of the stream, sub soil conditions, cost of the project, alternative sites available and their relative merits and aesthetics, etc.

The aim of the investigation is to select a suitable site for the construction of the bridge. The site for a bridge is governed by engineering factors, economics, demands of traffic, condition of stream, and aesthetics, etc. In the case of old alignments the bridge site is governed by existing road way or railway alignments, while in case of new alignments social and commercial benefits govern the site selection. The success of the project depends on the thoroughness of the information furnished through investigations.

Following are the factors to be carefully considered while selecting the ideal site for a proposed bridge:

1. The stream at the bridge site should be well defined and as narrow as possible.
2. There should be a straight reach of stream at bridge site.
3. The stream at bridge site should have permanent firm, straight and high banks. Firm banks also provide good foundation for the construction of towers of suspension bridges.
4. The site should be geologically sound i.e. it should be away from fault zone, and should have unyielding, non erodible foundation for abutments and piers. Hard rock or soil should be available near the bed for the economic construction of foundations.

5. The flow of water in the stream at the bridge site should be in steady regime condition.
6. There should be no confluence of large tributaries in the vicinity of bridge site.
7. The axis of stream at bridge site should be crossing at right angles to the centre line of the communication route as far as possible.
8. There should be no scouring and silting of the stream at bridge site. It should be free from whirls and cross-currents.
9. There should be no need for costly river training works in the vicinity of bridge site.
10. There should be minimum obstruction of natural waterway so as to have minimum afflux.
11. There should be easy availability of cheap labour, construction materials and transport facilities near to the bridge site.
12. In order to have minimum foundation cost, the bridge site should be such that no excessive work is to be carried inside the water.
13. The bridge site should provide sound, economical and straight approaches.
14. There should be no sharp curves in the approaches.
15. The bridges are constructed to connect the roads on either side of a river. The bridge site should therefore form a proper link between the roads on either side of a river.
16. The bridge site should be such that adequate vertical height and waterway is available underneath the bridge for navigational use.
17. There should be no adverse environmental impact.

Actually an ideal site is never found in practice. At every site some of the above mentioned ideal conditions are lacking. Therefore in practice while making selection of site the site having least objections should be selected. Therefore to select such a site, investigations for a number of probable alternative sites should be carried out.

CLASSIFICATION OF BRIDGES

Bridges can be classified as follows:

Sl.No.	According to	Type of bridge
1.	life	a) Temporary bridges b) Permanent bridges
2.	loadings	a) Class 'AA' bridges b) Class 'A' bridges c) Class 'B' bridges
3.	span length	a) Culverts (span less than 8 m) b) Minor bridges (span between 8 to 30 m) c) Major bridges (span above 30 m) d) Long span bridges (span above 120 m)
4.	purpose	a) Aqueducts b) Viaducts c) Grade separations d) Foot bridges e) Highway bridges f) Railway bridges
5.	materials used for construction	a) Timber bridges b) Masonry bridges c) Iron and steel bridges d) Reinforced cement concrete bridges e) Prestressed concrete bridges
6.	structural form	a) Beam type bridges: R.C.C. Tee beam, balanced cantilever, steel girder, plate girder, box girder, truss and portal frame bridges. b) Arch type bridges: Open spandrel, filled spandrel barrel and rib type bridges. c) Suspension type : ramp bridges, trestle bridges.
7.	alignment	a) Straight bridges b) Skew bridges
8.	level of bridge floor	a) Deck bridges b) Semi-through bridges c) Through bridges
9.	position of high flood level	a) Submersible bridges b) Non-submersible bridges
10.	the flexibility of superstructure	a) Fixed-span bridges b) Movable bridges

WATERWAY Linear

The most important factor to be decided in the design of bridges is the determination of the waterway required for the bridge or culvert.

The area under a bridge through which the water flows is called the natural waterway of a bridge. For important and big bridges, it should be designed to carry water at the time of maximum flood discharge. The natural waterway is the unobstructed area of the river or stream through which the water flows at the bridge site.

The area through which the water flows under a bridge superstructure is known as the waterway of the bridge. The linear measurement of this area along the bridge is known as the linear waterway. This linear waterway is equal to the sum of all the clear spans. This may be called as artificial linear waterway.

For determining the water way followings should be known:

- Maximum expected flood discharge which will pass under the bridge.
- Maximum permissible velocity.

Generally maximum velocity should not be allowed more than 3 m/sec.

While fixing the waterway of a bridge, the following guiding principles must be kept in mind to ensure safety of the bridge structure :

- The increased velocity due to obstructed waterway should not exceed the permissible velocity under the bridge.
- The free board for high level bridges should not be less than 600 mm.
- Sufficient clearance should be allowed according to the navigation requirements.

AFFLUX or BACKWATER

When a bridge is constructed, the structures such as abutment and piers cause the reduction of the natural waterway area. Due to this reduction in natural waterway, the velocity under the bridge increases so as to carry the maximum flood discharge. This increased velocity gives rise to a sudden heading up of water on the upstream side of the stream or river. The phenomenon of this heading up of water is known as Afflux. Afflux is taken as the difference of levels of the downstream and upstream water surface of the bridge.

Greater the afflux greater will be the velocity under the downstream side of the bridge and greater will be the depth of scour and consequently greater will be the depth of foundations required.

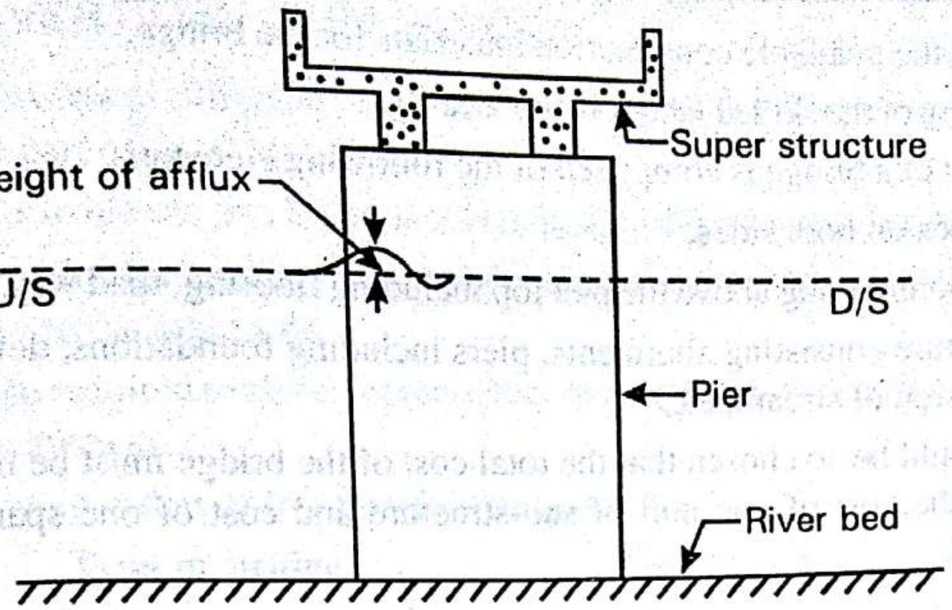

Fig. 9.8 : Afflux

The afflux should be kept as low as possible due to the following reasons:

- Lower the afflux, lower will be the velocity under the down stream side of the bridge and lower will be depth of scour and thus shallow will be depth of foundation required.
- It helps in deciding the top levels and lengths of guide banks and flood protection bunds conveniently and economically.
- It facilitates the provision of the bridge at lower level with sufficient free board.

Afflux may be taken 600mm in alluvial and deltaic regions, 900 to 1200mm in trough regions and higher in steep reaches of rivers with boulders and rocky beds for rough calculations.

ECONOMIC SPAN OF A BRIDGE

The span for which the total cost of the bridge will be minimum, is known as the economic span of the bridge.

The length of each span will have to be decided keeping in view the depth of the foundation, height of pier and the nature of the river bed at the site of the bridge.

The length of each span depends on :

- the nature of river bed,
- the depth and type of foundations required, and
- the height of piers required.

In navigation canals and rivers, the span, height of piers and clearance depends on the size of biggest vessels passing under the bridge.

Actually the cost of a bridge is affected by the following factors:

i) The length of span.
ii) Nature of water way or river to be bridged or crossed.
iii) The conditions under which the structure is to be constructed.

iv) Nature of the available construction materials for the bridge.

v) Availability of the skilled labour in the locality.

The total cost of a bridge is comprised of the following elements:

- Approaches on both sides.
- Super structure lying above the pier top including flooring, road way, parapet or railing.
- Sub-structure consisting abutments, piers including foundations, dewatering, excavation and diversion of stream, etc.

The span should be so chosen that the total cost of the bridge must be minimum. For this, a relation between the cost of one unit of sub-structure and cost of one span of super structure should be obtained.

SCOURING

The process of cutting or deepening of river bed due to action of water is called scouring.

When the velocity of stream water exceeds the limiting velocity, it causes vertical cutting of the river bed, which is known as scouring. Scouring should not be understood as horizontal widening of river, because it is then known as 'Erosion'.

If no steps are taken to prevent scouring, the soil below the bridge and around the piers will be washed and it will undermine the foundations. When all the soil around the piers will be washed scouring will be started in the foundation. The bearing capacity of the soil will be reduced and the pier will sink down, causing the failure of the bridge. Therefore, for safe and sound design of a bridge, it is important to estimate the correct scour depth.

The scour pattern at a bridge depends upon factors like flood discharge, bed slope, direction of flow, bed material, alignment of pier, pier geometry i.e. its shape and size, etc.

For the prevention of scour, the following measures should be taken :

- The site of the bridge should have stream-line flow.
- The slope of the pier should be designed in such a way that it may not cause eddies and currents in the water.
- Sufficient water-way should be provided below the bridge, so that velocity of water may not exceed the limit after which scouring starts.
- At the site of the bridge the particles of the bed should have such a shape, weight and inter cohesion property which can stand the high velocity of water.
- The river bed on upstream side, downstream side and portion below the bridge should be properly pitched with heavy and long stones.
- To prevent scouring piles can be driven in the river bed.

Problems On Design Discharge.

1) Explain the methods for the calculation of design discharge with formula

It is the maximum discharge in the stream during the lifespan of the bridge.

IRC has recommended the following methods to determine the design discharge.

* Using Empirical Formula
* Using Rational Formula
* Area - Velocity Method
* Past Records
* Hydrograph method.

①

1) Emperical Formula Method

a) Dicken's Formula. $\boxed{Q = CA^{3/4}}$

where, Q = Discharge in m^3/se

C = Constant depending upon region

11.37 for North - India

13.77 for Central - India

22.04 for Western - India.

A = Catchment area in m^2

b) Ryves Formula, $\boxed{Q = CA^{2/3}}$

where, C = 6.74 for area 24 km from coast.

= 8.45 for area b/w 24 - 161 km from coast

= 10.1 for hilly areas.

2) Rational Method.

$$\boxed{Q = A\, I_0\, \lambda}$$

$$I_0 = \frac{F(T+1)}{2T}\ ;\quad \lambda = \frac{0.56\, P_f}{t_c + 1}\ ;\quad t_c = \left(0.89\ \frac{L^3}{H}\right)^{0.385}$$

where, I_0 = Characteristics of catchment
F = Total Rainfall
T = Duration of Rainfall
P = Co-efficient of Runoff varies varies from 0.3 - 0.9
f = a factor to consider variation of intensity of rainfall in catchment area
t_c = Concentration Time in hours
L = Distance of critical point in km.
H = Difference in elevation b/w critical point & bridge

3) Area - Velocity Method

$$\boxed{Q = AV}$$

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

This method is based on characteristic (hydraulic characteristics) of streams i.e., velocity, slope of the stream, c/s of stream, shape & roughness of stream.

where, Q = discharge in m^3/se
A = Wetted area m^2
V = Velocity of flow m/sec

As per Manning's formula, $V = \frac{1}{N} R^{2/3} S^{1/2}$

where N = Manning's Coefficient varies from 0.01 - 0.03
S = Slope of the stream.
R = Hydraulic mean depth, $R = \frac{\text{Wetted Area}}{\text{Wetted Perimeter}}$

Problems

1) Determine the design discharge @ a bridge site using area velocity - method. c/s area of the stream, MFL = $125 m^2$ in wetted perimeter 90 m, bed slope of the stream 1 in 500. stream has straight & clean fair condition.

Sol: Given:-
Area = $125 m^2$
Perimeter = 90 m
Slope = 1 in 500
n = 0.03

$V = \frac{1}{n} R^{2/3} S^{1/2}$ $\qquad R = \frac{125}{90} = 1.389$

$V = \frac{1}{0.03} \times (1.38)^{2/3} (1/500)^{1/2}$

$\boxed{V = 1.85 \text{ m/sec}}$

$Q = A \times V \Rightarrow Q = 125 \times 1.85 \Rightarrow \boxed{Q = 231.76 \text{ m}^3/\text{sec}}$

Q) Determine the design discharge @ a bridge site using i) emperical formulae ii) Area velocity
catchment area = $160 m^2$; distance of site from coast = 11km
c/s area of stream @ MWL = $120 m^2$, wetted perimeter of stream = 90m ; stream condition : straight bank.
slope of the stream = 1 in 500.

Sol: i) Area velocity.

$V = \frac{1}{n} R^{2/3} S^{1/2}$; $R = \frac{\text{Area}}{\text{Perimeter}} = \frac{120 m^2}{90} = 1.33.$

$= \frac{1}{0.03} \times (1.33)^{2/3} * (1/500)^{1/2}$

$\boxed{V = 1.802 \text{ m/sec}}$

$Q = A \times V = 120 \times 1.802 \Rightarrow \boxed{Q = 216.34 \text{ m}^3/\text{sec}}$

ii) Emperical Method

Ryve's Formula. $Q = CA^{2/3}$

$= 6.74 * (160)^{2/3}$

$\boxed{Q = 198.64 \text{ m}^3/\text{sec.}}$

---

2) Problems of scour Depth

scouring depth, $\boxed{d = \frac{1.21 * Q^{0.63}}{f^{0.33} W^{0.60}}}$

maximum scour depth $(d_{max}) = 1.5 * d$.

---

1) A stream with hard banks has a width of 80m, a bed is aluvial ($f = 1.1$) & discharged through section $500 m^3$/sec. Calculate the max scour depth under the bridge having a single span of 50m.

Sol: $W = 80m$; $f = 1.1$; $Q = 500 m^3/sec.$, $L = 50m$

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$$d = \frac{1.21 * (500)^{0.63}}{(1.1)^{0.33} * (80)^{0.60}} \Rightarrow \boxed{d = 4.24m}$$

$$(d)_{max} = 1.5 * d = 1.5 * 4.24 \Rightarrow \boxed{(d_{max}) = 6.36\,m}$$

24/09/18

Problems on Waterway & Afflux

1) Determine the waterway for a bridge across a stream with a flood discharge of $225\,m^3/sec$, velocity $= 1.5\,m/sec$ & width of flow @ HFL 60m & allowable velocity under the bridge is $1.8\,m/sec$.

sol^n. Given. $Q = 225\,m^3/sec$; $V = 1.5\,m/sec$; $V_a = 1.8\,m/sec$.

$L = 60\,m$.

To findout linear waterway i.e., $L_1 = \frac{a}{d_b + x}$

where, x = afflux, a = area of the artificial waterway

d = mean depth of flow

$\Rightarrow \quad Q = A \times V \Rightarrow 225 = A \times 1.5 \Rightarrow \boxed{Area = 150\,m^2}$

$\Rightarrow$ Mean depth of flow, $d = \frac{A}{L} = \frac{150\,m^2}{60}$

$$\boxed{d = 2.5\,m}$$

Assume safe velocity as 90% of the allowable velocity

$$= \frac{90}{100} \times 1.8$$

$$\boxed{\text{Safe Velocity} = 1.62\,m/sec}$$

$\Rightarrow$ Area of artificial waterway, $a = \frac{\text{Discharge}}{\text{safe Velocity}}$

$$a = \frac{225}{1.62} \Rightarrow \boxed{a = 138.9\,m^2}$$

$\Rightarrow$ From molesworth formula,

$$\text{Afflux, } x = \left[\frac{V^2}{17.9} + 0.015\right]\left[\frac{A^2}{a^2} - 1\right]$$

$$= \left[\frac{(1.8)^2}{17.9} + 0.015\right]\left[\frac{(150)^2}{(138.9)^2} - 1\right]$$

$$\boxed{x = 0.0325\,m}$$

$\Rightarrow$ linear waterway, $L_1 = \frac{a}{d+x} = \frac{138.9}{2.5 + 0.0325}$

$\boxed{L_1 = 54.84\,m}$

---

2) Determine the waterway for a bridge across a stream with a flood discharge of 235 m³/sec, the velocity of the flow in the stream 1.8 m/sec with a max permissible velocity of 2.2 m/sec. Width of flow @ a HFL 55 m; Ans: 49.2m

Sol^n: $Q = 235\,m^3/sec$, $V = 1.8\,m/se$; $Va = 2.2\,m/sec$, $L = 55m$

$\Rightarrow Q = A \times V \Rightarrow 235 = A \times 1.8 \Rightarrow$ Area $= 130.56\,m^2$.

$\Rightarrow$ Mean depth of flow, $d = \frac{A}{L} = \frac{130.56}{55} \Rightarrow d = 2.37\,m$.

Assume safe velocity as 90% of allowable velocity. $= \frac{90}{100} \times 2.2$

Safe velocity $= 1.98\,m/se$.

Area of artificial waterway, $a = \frac{Q}{V_s} = \frac{235}{1.98} \Rightarrow a = 118.68\,m^2$

From molesworth formula, $x = \left[\frac{V^2}{17.9} + 0.015\right]\left[\frac{A^2}{a^2} - 1\right]$

$= \left[\frac{(2.2)^2}{17.9} + 0.015\right]\left[\frac{(130.56)^2}{(118.68)^2} - 1\right] \Rightarrow x = \cancel{0.478\,m}\ 0.059$.

$\Rightarrow$ Linear Waterway, $L_1 = \frac{a}{d+x} = \frac{118.68}{2.37 + \cancel{0.478}\ 0.059} \Rightarrow L_1 = 48.85\,m$

---

3) Design waterway for a bridge across the stream having flood discharge 28 m³/sec over a trapezoidal channel having side slope 1:1 bed slope 1:800 & bed width & depth ratio 6:1. The bed material is sand with safe velocity of flow of sand is 2.5 m/sec. The afflux should not be more than 60mm. Manning's constant is 0.025

Sol^n: Given Data.

$Q = 28\,m^3/sec$; side slope 1:1; Bed slope 1:800

$x = 60\,mm$; $N = 0.025$

$b/d = 6/1 = 6d$; $V = 2.5\,m/sec$

$n = 1:1$

i) Area of natural waterway,

Area, $A = d[B + nd]$

$= d[6d + 1d]$

$A = 7d^2$

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ii) Velocity using Manning's Formula

$$V = \frac{1}{n}\left[R^{2/3} S^{1/2}\right] \qquad R = \frac{Area}{Perimeter}$$

Perimeter, $P = b + d\sqrt{2} + d\sqrt{2}$

$$R = \frac{7d^2}{b + d\sqrt{2} + d\sqrt{2}} = \frac{7d^2}{6d + d\sqrt{2} + d\sqrt{2}}$$

$$\boxed{R = 0.792d}$$

$$V = \frac{1}{0.025} * (0.792)^{2/3} d^{2/3} * \left[1/800\right]^{1/2}$$

$$\boxed{V = 1.21 d^{2/3}}$$

$$Q = A * V = 7d^2 \times 1.21 d^{2/3}$$

$$Q = 28 = 7d^2 * 1.21 d^{2/3}$$

$$\boxed{d = 1.56 m}$$

Velocity, $V = 1.2 * (1.56)^{2/3} \Rightarrow \boxed{V = 1.62 \ m/sec}$

Area, $A = 7d^2 \Rightarrow 7 * (1.56)^2 \Rightarrow \boxed{A = 17.0 \ m^2}$

$$x = \left[\frac{V^2}{17.9} + 0.015\right]\left[\frac{A^2}{a^2} - 1\right]$$

$$0.06 = \left[\frac{(1.62)^2}{17.9} + 0.015\right]\left[\frac{(17)^2}{a^2} - 1\right]$$

$$\boxed{a = 14.54 \ m^2}$$

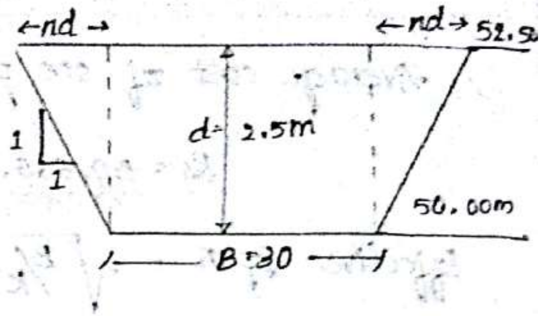
4) A 2 span plate girder bridge is to be provided across a river having following data.

Flood discharge = 100 m³/sec ; Bed width = 30m

side slope = 1:1 ; Bed level = 50.00 mts

HFL = 52.50 m ; Maximum afflux = 15cm

Calculate the span of the bridge.



Given Data:

$Q = 100 m^3/sec$;

$B = 30m$

$d = 2.5 m$

$x = 0.15m$

→ Area of natural water,

$$Area = d[b+nd] = 2.5[30+1(2.5)]$$

$$\boxed{Area, A = 81.25m^2}$$

→ Velocity of flow,

$$Q = A \times V \Rightarrow 100 = 81.25 \times V$$

$$\boxed{V = 1.23 m/sec}$$

→ Using Afflux,

$$x = \left[\frac{V^2}{17.9} + 0.015\right]\left[\frac{A^2}{a^2} - 1\right]$$

$$0.15 = \left[\frac{(1.23)^2}{17.9} + 0.015\right]\left[\frac{(81.25)^2}{a^2} - 1\right]$$

$$\boxed{a = 51.31 m^2}$$

→ Linear water way,

$$L = \frac{a}{d+x} = \left[\frac{51.31}{2.5+0.15}\right] \Rightarrow \boxed{L = 19.36 m}$$

---

5) The approximate cost of 1 superstructure & pier for a meter span bridge are given below. Estimate the economic span of the bridge

Span (m)	12	18	21
Super structure Cost (Rs)	34,000	80,000	1,50,000
Sub structure Cost (Rs)	50,000	54,000	48,000

Soln The average cost co-efficient is calculated as shown in the following table. The calculation based on assumption that cost of super st. is based on the span.

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Span (m)	Cost Effective	Average Cost Efficiency
12	$34000/12^2 = 236.11$	$\frac{236.11 + 246.11 + 340.13}{3}$
18	$80000/18^2 = 246.91$	$= 274.38$
21.	$150.000/21^2 = 340.13$	

$\therefore$ Average cost of one pier. (Rs) $= \frac{50,000 + 54,000 + 48,000}{3}$

$$\boxed{Rs = 50,666.6}$$

Effective span $= \sqrt{B/K} = \sqrt{\frac{50,666.6}{274.38}}$

$$\boxed{\text{Effective span} = 13.58\,m}$$

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3. List the forces to be considered for the design of bridges & explain any four.

1. Dead Load
2. Live Load
3. Snow Load
4. Impact factor on vehicular live load
5. Impact due to floating bodies or vessels as the above may be
6. Vehicle collision load
7. Wind load
8. Water Current
9. Longitudinal forces caused by tractive effort of vehicles or by braking of vehicles and are those caused by restraint of movement of free bearings by friction or deformation
10. Centrifugal force
11. Buoyancy
12. Earth pressure including liveload surcharge if any
13. Temperature effects
14. Deformation Effects
15. Secondary effects
16. Erection effects
17. Seismic force
18. Wave Pressure
19. Grade Effect.

1) Dead Load : The dead Load indicates the load of stru itself depends on various factors such as live load to be arrived, length of span, working stresses adopted in the design etc. It has to be initially assumed for the design purpose

Following 2 rules are to be followed

a) The D.L of the str is assumed by reference to suitable empirical formulae or by comparision to similar existing str.
b) After the design is finalized, the actual weight of the str

is worked out. If there is appreciable difference b/w the actual & assumed D.L, the design is revised.

iii) Deformation stresses

Any bending stresses which is developed in a steel member either due to vertical deformation or rigidity of the joints is termed as deformation stress. The deformation stresses are to be taken into consideration for steel bridges only. The steel bridges are to be designed, manufactured & erected in such a way that deformation stresses are brought down to a min possible level. For the purpose of assumption only, deformation stresses may be taken as not < 16% of the L.L & D.L stresses. The deformation stresses are to be ignored in case of pre-stressed girders of steel.

(5)

iv) Earth Pressure

The components of bridge which are required to retain earth should be designed for suitable earth pressure. The position of L.L on earth causes pressure. The position of L.L on earth surcharge & it should be properly considered in the design of bridge.

iv) Centrifugual forces

When a road or a railway bridge is situated on a curve, when a the effect due to centrifugal force is to be considered in the bridge design. Following formulae are adopted for road & railway bridges

Road Bridges

$$\boxed{C = \frac{WV^2}{127R}}$$

C = Centrifugal force
V = Designed vehicle speed in kmph
W = Live Load
R = Radius of curvature in m.

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APPLICATION OF IRC LOADING IN DESIGN OF BRIDGES

Highway bridge decks have to be designed to withstand the live loads specified by the Indian Roads Congress.

The standard IRC loads specified in IRC: 6-2000 are grouped under four categories as detailed below:

1. IRC Class AA Loading

Two different types of vehicles are specified under this category grouped as tracked and wheeled vehicles. The IRC Class AA tracked vehicle (simulating an army tank) of 700 kN and a wheeled vehicle (heavy duty army truck) of 400 kN are shown in Fig. 11.19.

All the bridges located on National Highways and State Highways have to be designed for this heavy loading. These loadings are also adopted for bridges located within certain specified municipal localities and along specified highways. Alternatively, another type of loading designated as Class 70 R is specified instead of Class AA loading.

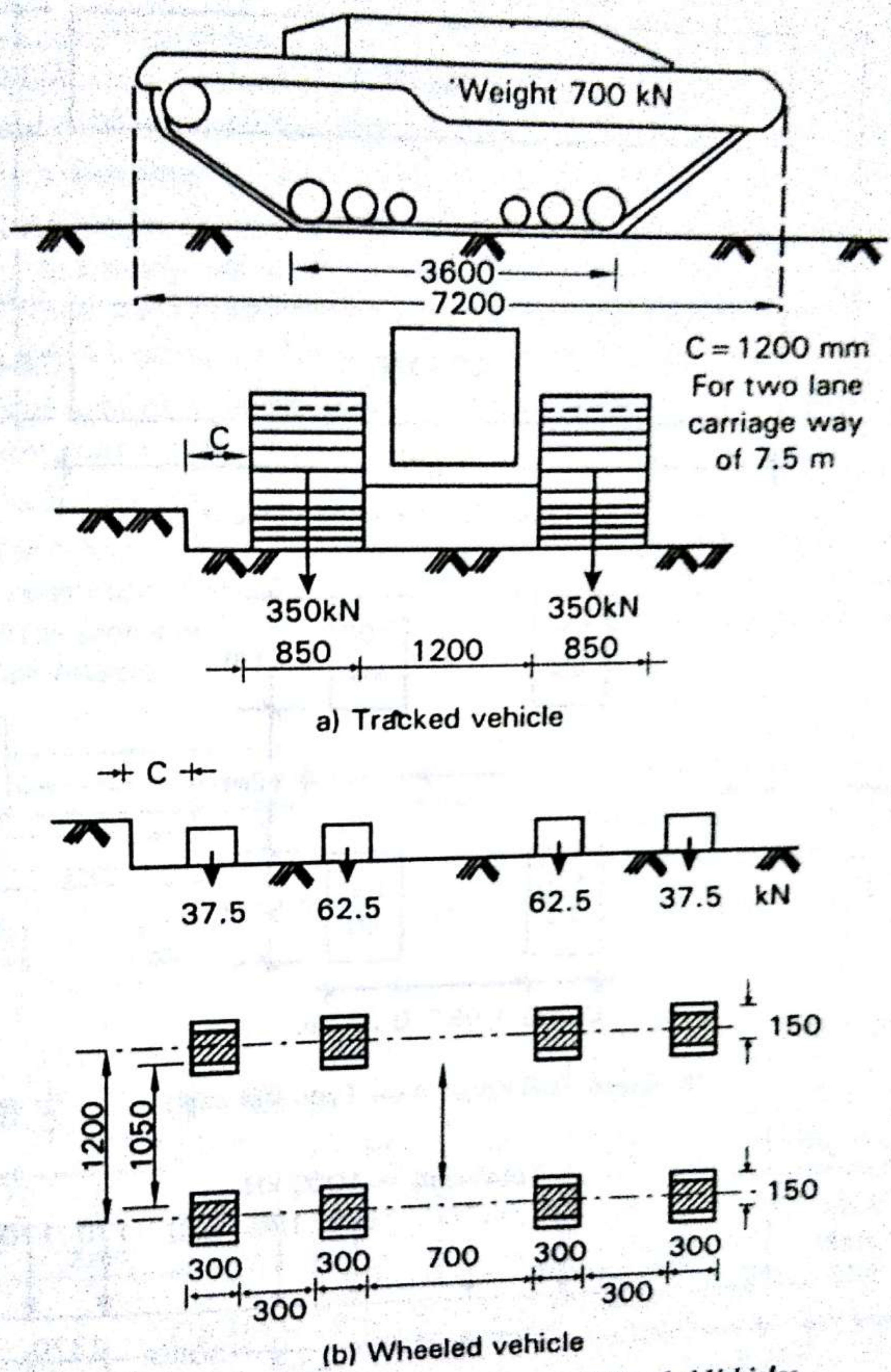


Fig. 11.19 : I.R.C. Class AA Tracked and Wheeled Vehicles

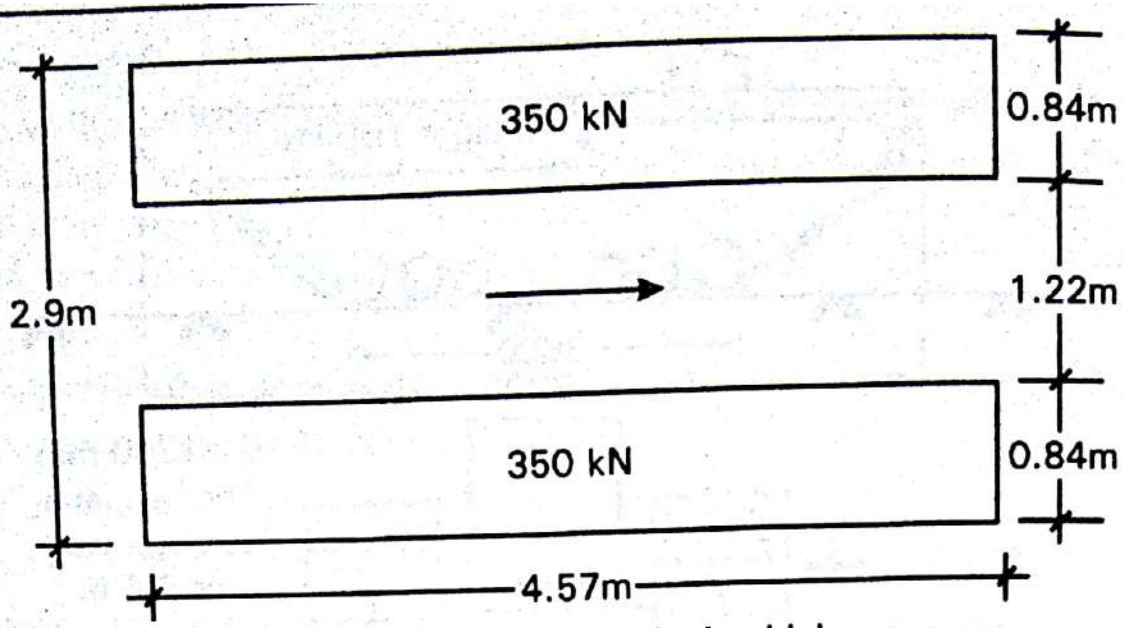
2. IRC Class 70 R Loading

IRC 70 R loading consists of the following three types of vehicles.

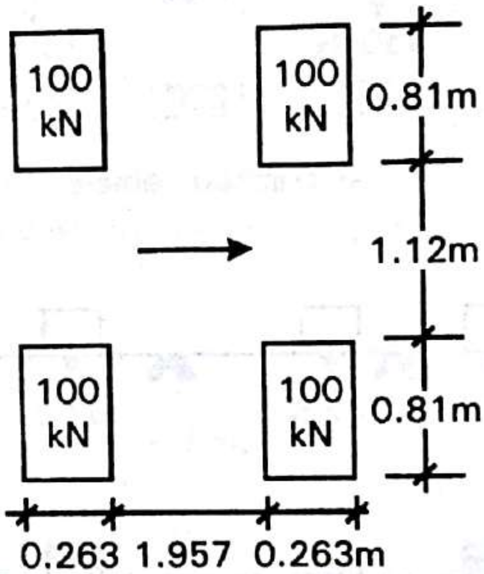
i) Tracked vehicle of total load 700 kN with two tracks each weighing 350 kN.

ii) Wheeled vehicle comprising 4 wheels, each with a load of 100 kN totaling 400 kN.

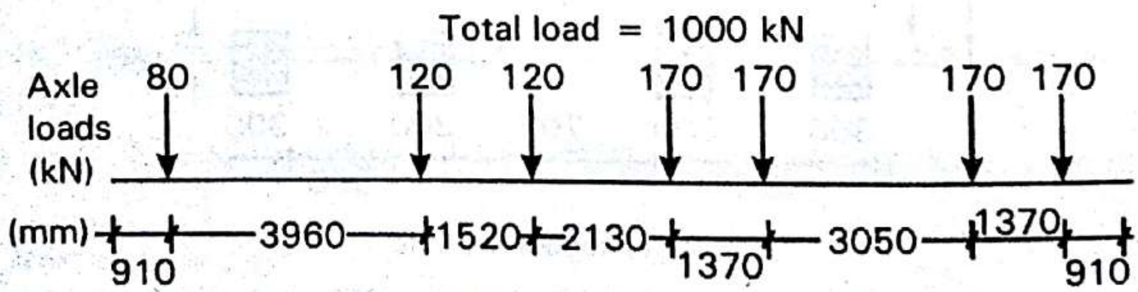
iii) Wheeled vehicle with a train of vehicles on seven axles with a total load of 1000 kN.



(a) Class 70R Tracked vehicle



(b) Class 70R Bogie Axle Type Vehicle



(c) Class 70R Wheeled Vehicle Loading

Fig. 11.20 : I.R.C. Class 70 Tracked and Wheeled Vehicles

The tracked vehicle is somewhat similar to that of Class AA, except that the contact length of the track is 4.87 m, the nose to tail length of the vehicle is 7.92 m and the specified minimum spacing between successive vehicles is 30 m. The wheeled vehicle is 15.22 m long and has seven axles with the loads totaling to 1000 kN. The bogie axle type loading with 4 wheels totaling 400 kN is also specified.

The various categories of loads are to be separately considered and the worst effect has to be considered in design. Only one lane of Class 70 R or Class AA load is considered whereas both the lanes are assumed to be occupied by Class A loading if that gives the worst effect.

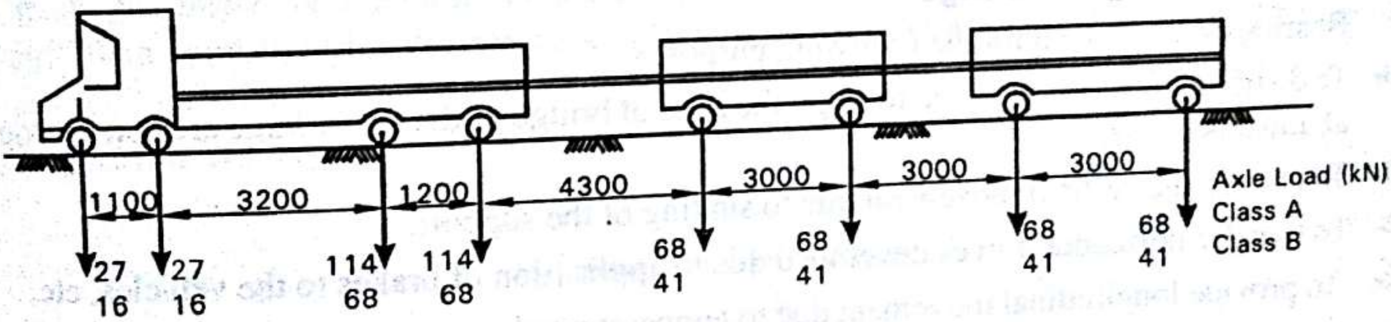
3. IRC Class A Loading

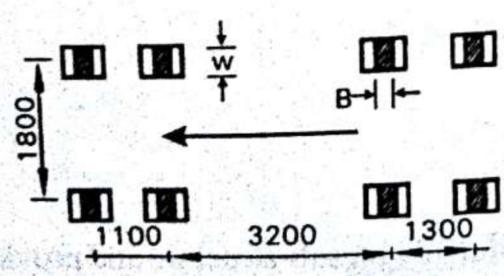
IRC Class A type loading consists of a wheel load train comprising a truck with trailers of specified axle spacing and loads as shown in Fig. 11.21. The heavy duty truck with two trailers transmits loads from 8 axles varying from a minimum of 27 kN to a maximum of 114 kN. The Class A loading is a 554 kN train of wheeled vehicles on eight axles.

This type of loading is recommended for all roads on which permanent bridges and culverts are constructed.

4. IRC Class B Loading

Class B type of loading is similar to Class A loading except that the axle loads are comparatively of lesser magnitude. The axle loads of Class B are a 332 kN train of wheeled vehicles on eight axles as shown in Fig. 11.21. *This type of loading is adopted for temporary structures and timber bridges.*





Axle Load kN	Ground contact area B(mm)	W(mm)
114	250	500
68	200	380
41	150	300
27	150	200
16	125	175

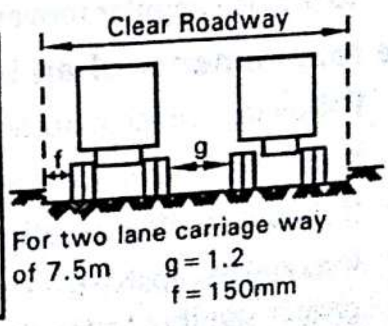


Fig. 11.21 : I.R.C. Class A and B Loading vehicle

MODULE - 2 DESGIN OF SLAB BRIDGES [SLAB CULVERTS]

21/08/18

Steps

1. Calculation of permissible stresses [constants]
2. Calculation of Depth of slab.
3. Calculation of Effective span of slab.
4. Load Calculation.
   a) Dead Load
   b) Live load
5. Moment Calculation
   a) Live load moment
   b) Dead load moment
6. Design of slab
   a) Ast along shorter direction
   b) Ast along longer direction
7. Necessary Checks
8. Detail

Problems

1) Design a reinforced slab culvert for a NH crossing the following data.

Carriage Way - 2 lane (7.5m)
Foot path - 1m on either side
Clear span - 6m
Wearing coat - 80mm
Width of bearing 400mm
Use M-25 concrete & 415 steel. Adopt IRC AA tracked wheeled Vehicles.

Data:

Clear span = 6m
2-lane carriage way
Foot Path - 1m.
Bearing - 400mm
Wearing Coat - 80mm
M25 Concrete & Fe 415 steel
IRC AA tracked wheel Vehicle.

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1. Calculation of permissible stress (Constants)

$m = \frac{280}{3\sigma_{cb}}$ $K = \frac{m\sigma_{cb}}{m\sigma_{cb}+\sigma_{st}}$ $j = 1 - K/3$ $Q = \frac{\sigma_{cb} j k}{2}$

(IRC 21 - 2000) page no 18 Properties

For M-25 ⇒ $\sigma_{cb} = 8.33 \ N/mm^2$

Table 10, For Fe 415 steel ⇒ $\sigma_{st} = 200 \ N/mm^2$

$m = \frac{280}{3 \times 8.33} = 11.204 \Rightarrow \boxed{m = 11.20}$

$K = \frac{m\sigma_{cb}}{m\sigma_{cb}+\sigma_{st}} = \frac{11.204 * 8.33}{11.204 \times 8.33 + 200} \Rightarrow \boxed{K = 0.3181}$

$j = 1 - \frac{k}{3} = 1 - \frac{0.318}{3} \Rightarrow \boxed{j = 0.894}$

$Q = \frac{\sigma_{cb} j k}{2} = \frac{8.33 \times 0.894 \times 0.318}{2} \Rightarrow \boxed{Q = 1.18}$

2) D Calculation of Depth of slab.

Adopt,

Overall depth, D - Wearing Coat × Clear Span

$= \cancel{80 \times 6000}$ 80×6

$\cancel{= 48,0000}$ $\boxed{D = 480mm} \simeq 500 \ mm$

Adopt 20mm dia bars & 30mm cover

Effective depth, $d = D - \text{Cover} - d/2$

$= \overset{500}{\cancel{480}} - 30 - 20/2$

$\boxed{d = 440 \ mm \simeq 460 \ mm} = d$

3) Effective length of slab

$L_{effective}$ = Clear span (L_c) + Effective depth

$= 6 + 0.46$

$= 6.46m$

$L_{effective}$ = Clear span (L_c) + Bearing

$= 6 + 0.40$

$= 6.4m$

∴ Take $\boxed{l_{eff} = 6.4m}$

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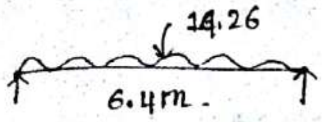
4) Load Calculation. a) Dead load:

→ Self Weight of slab = $D \times \gamma = 0.5 \times 25 = 12.5\ KN/m^2$

→ Self Weight of Wearing coat = $0.08 \times 22 = 1.76\ KN/m^2$

Total load = $14.26\ KN/m^2$



Dead load Bending Moment $= \frac{WL^2}{8} = \frac{14.26 \times (6.4)^2}{8}$

$\boxed{= 73.01\ KN\text{-}m}$

Deadload Shear Force $= \frac{WL}{2} = \frac{14.26 \times (6.4)}{2}$

$\boxed{= 45.632\ KN}$

b) Live load :

For IRC Class AA tracked vehicle Live load is 350 KN

Since we have considered 2 wheels the Live load = 2×350

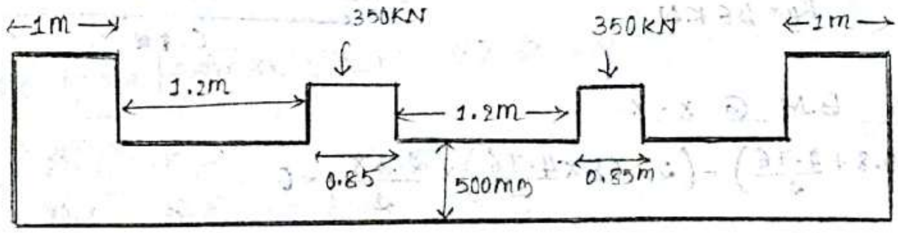
$= 700\ KN$

Total intensity of Live load (UDL) $= \frac{W \times IF}{L_d \times B_d}$

where W = Live load , IF = Impact factor = 1.2

L_d = Effective length of Load

B_d = Net Effective Width of Dispersion.



Effective Width of Slab.

$b_e = kx\left|1 - x/L\right| + b_w$

L = Effective span = 6.4m

$x = \frac{L}{2} = \frac{6.4}{2} = 3.2m$

$b_w = 0.85 + 2(0.08) = 1.01m$ (W.C)

By interpolation. (Pg. 53)

1.4	2.80
1.5	2.84
1.48	?

To find out K, Calculate

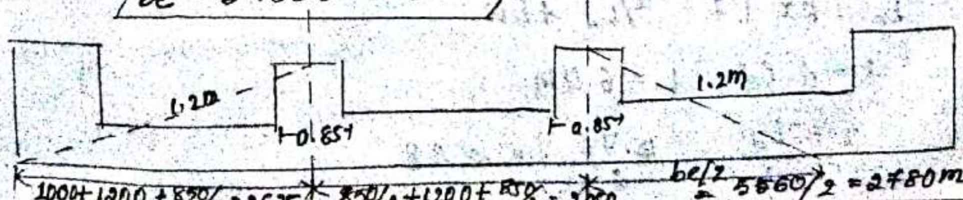
$b/L_0 = \frac{7.5+2}{6.4}$ (2 lane + carriage way)

$b/L_0 = 1.48$

∴ $b/L_0 = 1.48$

$b_e = 1.48 \times 3.2\left|1 + \frac{3.2}{6.4}\right| + 1.01$

$\boxed{b_e = 5.538 \simeq 5.56m}$



Net Effective Depth of Dispersion (b_b)

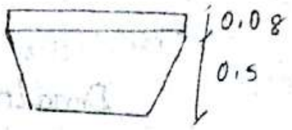
$b_b = 2625 + 2050 + 2780$

$\boxed{b_b = 7455\,mm}$

Total live load intensity $= \dfrac{W \times IF}{L_d \times b_b}$ $L_d = 3.6 + [D + \text{W.C}]\,2$

$L_d = 3.6 + [0.5 + 2(0.08)]\,2$

$\boxed{L_d = 4.76}$



$= \dfrac{700 * 1.2}{7.45 \times 4.76}$

$\boxed{\text{Live load (UDL)} = 23.68\,KN/m^2}$

Lo

Length of load (L_d) $= 4.76m$.

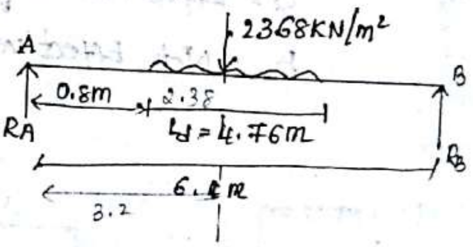
Effective length of slab (L_{eff}) $= 6.4m$

⇒ Calculation of Live load Bending Moment

$\Sigma M_A = 0;$

$0 = (23.68 \times 4.76) \times \left(\dfrac{4.76}{2} + 0.8\right) - 6.4\,R_B$

$\boxed{R_B = 56\,KN}$



Maximum B.M @ x-x

$R_A \times \left(0.8 + \dfrac{4.76}{2}\right) - \left(23.68 \times \dfrac{4.76}{2}\right) \times \dfrac{2.38}{2} = 0$

$\boxed{(R_A)\ LL\ @\ B.M = 111.01\,KN\text{-}m.}$

Calculation of Liveload Shearforce

W.K.T for IRC Class a-a tracked vehicle

Live load $= 700\,KN$

Intensity of load (UDL) $= \dfrac{W \times I_f}{L_d \times B_d}$

where $W = 700KN$

$I_F = 1.2$

$L_d = 4.76m$

$b_e = Kx\,[1 - x/L] + bw$

$K = 2.82;\ L = 6.4m$

$x = \dfrac{L_d}{2} = \dfrac{4.76}{2} = 2.38$

$b_w = 1.01$

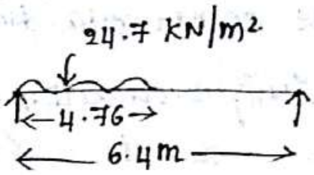
$b_e = (2.83 \times 2.38)\left[1 - 2.38/6.4\right] + 1.01$

$b_e = 5.24 m \simeq 5.256 m.$

$B_b = 2625 + 2050 + 2628$

$\boxed{B_b = 7303 mm = 7.3 m}$

$L.L.I = \dfrac{700 \times 1.2}{4.76 \times 7.3} = 24.17 \ KN/m^2$



$\Sigma M_A = 0;$

$24.07 \times 4.76 \times \dfrac{4.76}{2} - R_B \times 6.4 = 0$

$\boxed{R_B = 43.78 \ KN}$

$\Sigma V = 0;$

$R_A + R_B = 24.7 \times 4.76$

$\boxed{R_A = 73.85 \ KN}$

Live load shear force = 73.85 KN

D.L	B.M	=	73.01 KN-m
D.L	S.F	=	45.63 KN
L.L	B.M	=	111.01 KN-m
L.L	S.F	=	73.85 KN

Total B.M = L.L BM + D.L BM

$= 73.01 + 111.01$

$\boxed{= 184.02 \times 10^6 \ N\text{-}mm}$

Total shear force = D.L SF + L.L SF

$= 45.63 + 73.85$

$= 119.53 \ KN$

$\boxed{= 119.53 \times 10^3 \ N}$

Design of slab

→ Along shorter direction

Required depth $= \sqrt{\dfrac{M}{Q b}} = \sqrt{\dfrac{184.02 \times 10^6}{1.18 \times 1000}}$

$d = 394.23 mm < 460 mm$ Hence O.K

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Calculation of Ast

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{184.02}{200 \times 0.894 \times 460}$$

$$\boxed{A_{st} = 2247\ mm^2}$$

We have assumed diameter of bars 20mm

$$\text{Spacing} = \frac{1000 a_{st}}{A_{st}}$$

$$a_{st} = \frac{\pi d^2}{4} = \frac{\pi \times (20)^2}{4} = a_{st} = 314.15 mm^2$$

$$\text{Spacing} = \frac{1000 \times 314.15}{2247}$$

$$\boxed{= 139.81 \simeq 140mm}$$

Provide 20mm dia bars @ 140mm c/c in S.D

* Along longer direction :- Assume the dia of bars as 12mm

$$d = d_x - \frac{\phi_x}{2} - \frac{\phi_y}{2} = 460 - \frac{20}{2} - \frac{12}{2}$$

$$\boxed{d = 444mm}$$

Total B.M, $M = 0.2 M_d + 0.3 M_L = (0.2 \times 73.01 + 0.3 \times 111.01) \times 10^6$

$$\boxed{M = 47.905\ KN\text{-}m}$$

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{47.905}{200 \times 0.894 \times 444} \quad \boxed{\Rightarrow A_{st} = 603.37 mm}$$

$$\text{Spacing} = \frac{100 \frac{\pi \times (12)^2}{4}}{603.37} = 187.44m \simeq 185mm$$

Providing 12mm dia bars @ 185 mm c/c along longer direction

Check for shear

Total shear, $V_u = DL\ SF + L.L\ SF = 45.63 + 73.85$

$$\boxed{V_u = 119.48 \times 10^3 N}$$

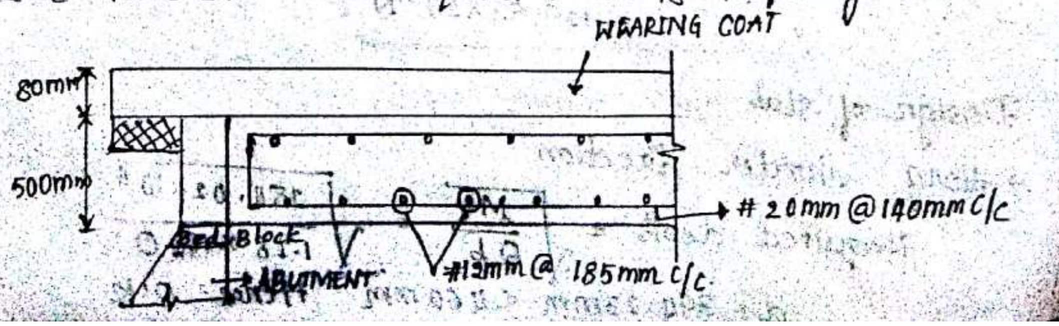
$$\tau_v = \frac{V_u}{bd} = \frac{119.13 \times 10^3}{1000 \times 460} = 0.258\ N/mm^2$$

To calculate τ_c, Calculate % of steel

$$P_t = \frac{100 A_{st}}{b.d} = \frac{100 \times 2247}{1000 \times 460} = 0.488 \approx 0.49$$

IRC 21, P-36 From table, For M25 Concrete, $\boxed{\tau_c = 0.3\ N/mm^2}$

$\tau_v < \tau_c$ Hence SF is not required or it is safe against shear



2) A reinforced concrete slab culvert has a clear span of 5.5m & the following data.

i) Width of bearing on either side = 500mm

ii) Clear width of carriage way - 7.5m

iii) Width of footpath on either side = 1m

iv) Wearing coat thickness = 80mm

v) Live load expected - Class A-A tracked vehicle

vi) Grade of concrete - M30

vii) Grade of steel - Fe 415

Design & detail the slab culvert for flexure only.

Sol^n: Step 1: Calculation of Permissible stresses

$\sigma_{cb} = 10$

$\sigma_{st} = 200 \, N/mm^2$

$m = \frac{280}{3\sigma_{cb}} = \frac{280}{3 \times 10} \Rightarrow \boxed{m = 9.33}$

$k = \frac{m\sigma_{cb}}{m\sigma_{cb} + \sigma_{st}} = \frac{9.33 \times 10}{9.33 \times 10 + 200} \Rightarrow \boxed{k = 0.318}$

$j = 1 - k/3 = 1 - \frac{0.318}{3} \Rightarrow \boxed{j = 0.894}$

$Q = \frac{\sigma_{cb} j k}{2} = \frac{10 \times 0.894 \times 0.318}{2} \Rightarrow \boxed{Q = 1.421}$

Step 2: Depth of slab.

D = Wearing Coat × Clear span = $80 \times 5.5 \Rightarrow \boxed{D = 440}$

$\boxed{\therefore D \simeq 500mm}$

Adopt 20mm dia bar.

$d = D - \phi/2 - d' = 500 - 20/2 - 30$

$\boxed{d = 460mm}$

Step 3: Effective length of slab.

i) L_{eff} = Clear span + d = 5.5 + 0.41 = 5.96m

ii) L_{eff} = Clear span + Bearing = 5.5 + 0.5 = 6m

$\therefore$ Take $L_{eff} = 5.96m$.

Step 4: Load Calculation

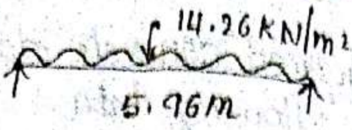
i) Dead Load

Self Weight of slab = $D \times \gamma$ = 0.45×25 = $12.5 \, KN/m^2$

Self Weight of Wearing Coat = $D \times \gamma$ = $0.08 \times 22 = 1.76 \, KN/m^2$

Total load = $14.26 \, KN/m^2$

Bending Moment, B.M $= \frac{wL^2}{8}$



$= \frac{14.26 (5.96)^2}{8}$

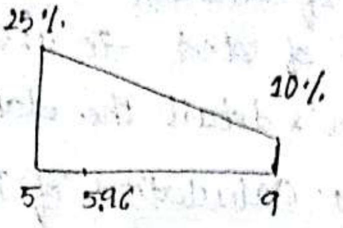
$\boxed{B.M = 63.31\ kN\text{-}m}$

Shear force, S.F $= \frac{wL}{2} = \frac{14.26 (5.96)}{2} \Rightarrow \boxed{S.F = 42.49 kN}$

ii) Live load

350 KN for 2 wheels, $2 \times 350 = 700 KN = W$

$UDL = \frac{W \times IF}{l_d \times B_d}$



$b_e = Kx\,|1 - x/l| + b_w$

$x = \frac{\text{effective span}}{2} = 3.2m$

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

For 5.96 $= 21.4\%$

To find out K calculate b/L_0

$\frac{21.4}{100} + 1$

$= \frac{7.5 \times (1+1)}{5.96} = 2.51$

$I_F = 0.214 + 1 = 1.214 = I_F$

Pg no 53 $K = 3$

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

$be = 3 \times 3.2\,|1 - 3.2/5.92| + 1.01$

$l_d = 3.6 + (0.5 + 0.08) \times 2$

$\boxed{be = 5.45m}$

$\boxed{L_d = 4.76m}$

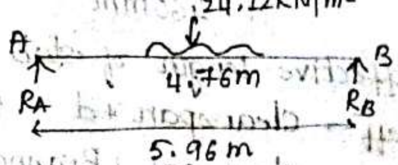
$B_b = 2625 + 2050 + \frac{5450}{2} = 7400mm$

$UDL = \frac{W \times IF}{l_d \times Bd} = \frac{700 \times 1.214}{4.76 \times 7.400 m}$

$\boxed{UDL = 24.12 KN/m^2}$

Calculation of LLBM

$\Sigma M_A = 0;$



$0 = 24.12 \times 4.76 \times \left[\frac{4.76}{2} + 0.8\right] - 5.96 R_B$

$\boxed{R_B = R_A = 61.25 KN}$

Max BM @ x-x.

$R_A \times (0.8 + \frac{4.76}{2}) - (24.12 \times 4.76/2) \times 2.38/2$

$\boxed{M_x = 126.46\ KN\text{-}m}$

Calculation of S.F

For IRC class A-A tracked vehicle Liveload = 700 KN

$W = 700 KN$

$I_F = 1.214$

$l_d = 4.76m$

$$v = \frac{W \times I_F}{L_f \times Bd}$$

$$x = \frac{4.76}{2}$$

$$\boxed{x = 2.38\,m}$$

$$b_e = kx\left|1 - \frac{x}{L}\right| + b_w$$

$k = 3$; $L = 5.96\,m$; $b_w = 1.01\,m$

$$b_e = 3 \times 2.38\left|1 - \frac{2.38}{5.96}\right| + 1.01$$

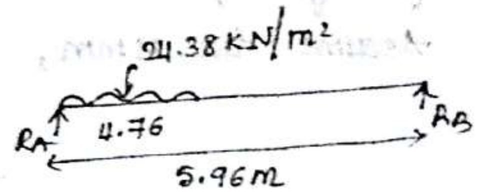
$$\boxed{b_e = 5.29\,m}$$

$$B_b = 2625 + 2050 + \frac{5290}{2} = 7320\,mm$$

$$\boxed{B_b = 7.32\,m}$$

$$UDL = \frac{700 \times 1.214}{4.76 \times 7.32}$$

$$\boxed{UDL = 24.38\,KN/m^2}$$



$$-R_B \times 5.96 + 24.38 \times 4.76 \times \frac{4.76}{2} = 0$$

$$\boxed{R_B = 46.34\,KN}$$

$\Sigma V = 0$;

$$R_A + R_B - 24.38 \times 4.76 = 0$$

$$\boxed{R_A = 69.7\,KN}$$

Live load S.F $= R_A = 69.7\,KN$

D.D SF	=	42.49 KN
D D B.M	=	63.31 KN-m
L.L S.F	=	69.7 KN
L.L B.M	=	126.46 KN-m

Calculate total Bending moment

$= LLBM + DLBM = 189.77\,KN\text{-}m$

Total SF = LLSF + DLSF = 63112.2 KN

Step 6: Design of slab

Along shorter direction

$$\text{Required depth} = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{189.77 \times 10^6}{1.184 \times 1000}}$$

$$\boxed{d_{req} = 40034}$$

$d_{prov} = 460mm > d_{req}$ hence Okay

Calculation of Ast

$$A_{st} = \frac{M}{\sigma_{st} J d} = \frac{189.79 \times 10^6}{200 \times 0.894 \times 460}$$

$$\boxed{A_{st} = 2307.53 mm^2}$$

Provide 20mm dia bars. spacing $= \frac{a_{st}}{A_{st}} \times 1000$

$$= \frac{\pi d^2}{4 \times 2307.53} \times 1000$$

$$= 136.14 mm$$

$$\boxed{Spacing = 140mm \text{ or } 135 mm}$$

Along longer span Direction

Assume dia 12mm, $d = D - \frac{\phi_x}{2}$

$$= 460 - 20/2 - 12/2$$

$$\boxed{d = 444 mm}$$

$M = 0.2M_d + 0.3M_L$

$= 0.2 \times 63.31 + 0.3 \times 126.46$

$$\boxed{M = 50.6 kN\text{-}m}$$

$$A_{st} = \frac{M}{\sigma_{st} J d} = \frac{50.6 \times 10^6}{200 \times 0.894 \times 444}$$

$$\boxed{A_{st} = 637.38 mm^2}$$

$$Spacing, = \frac{a_{st}}{A_{st}} = \frac{\pi d^2}{4 A_{st}} \times 1000 = \frac{\pi (12)^2}{4 \times 637.38} \times 1000$$

$$\boxed{Spacing = 177.4 mm \simeq 175mm}$$

step 7: Check for shear

Total shear $= V_d + V_L = 112.2 kN$

$$\tau = \frac{V_u}{bd} = \frac{112.2 \times 10^3}{1000 \times 460} \Rightarrow \boxed{\tau = 0.243 N/mm^2}$$

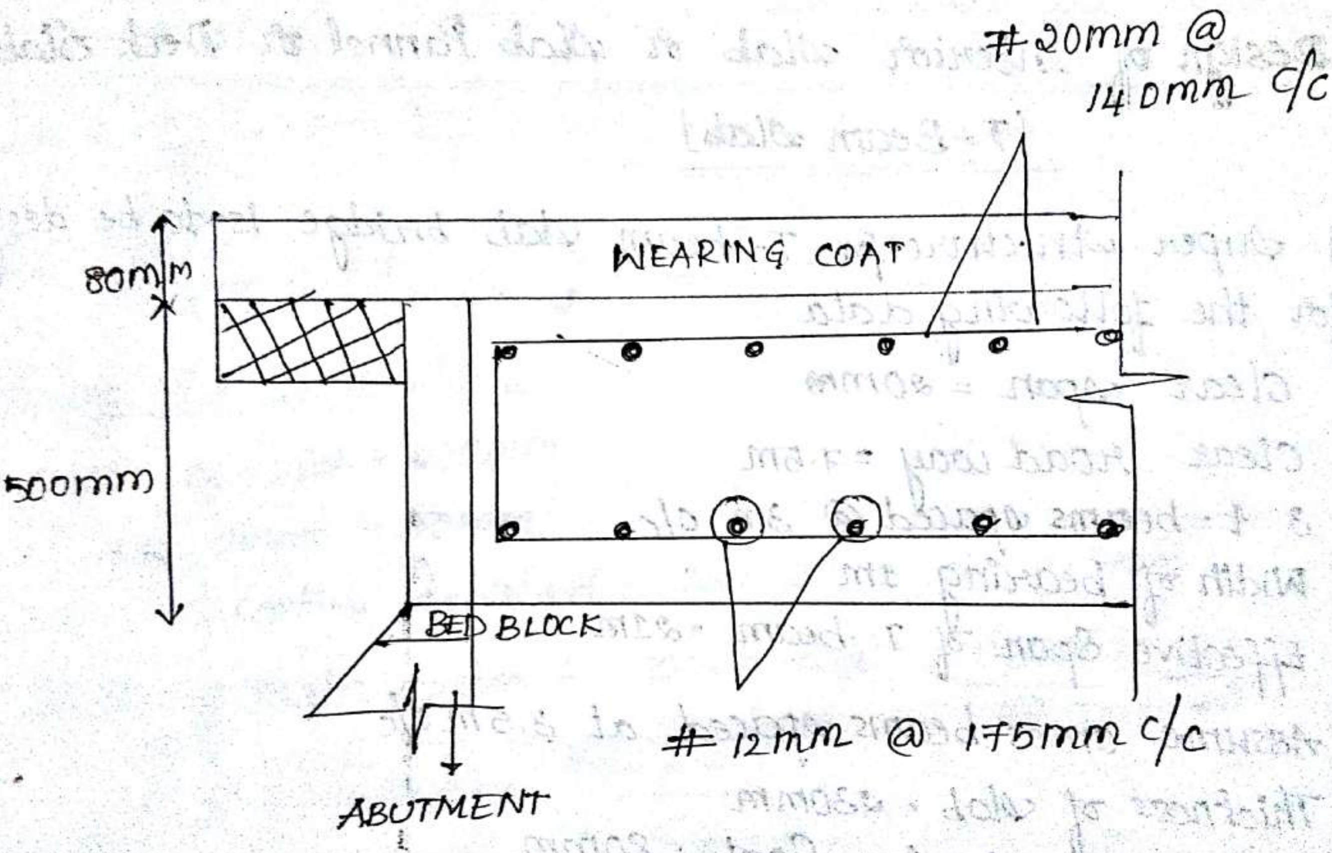
To calculate τ_c, calculate % of steel, $P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 2307.53}{1000 \times 460}$

$$\boxed{P_t = 0.501}$$

From Eq 36 of IRC 21 $\tau_c = 0.5 N/mm^2$

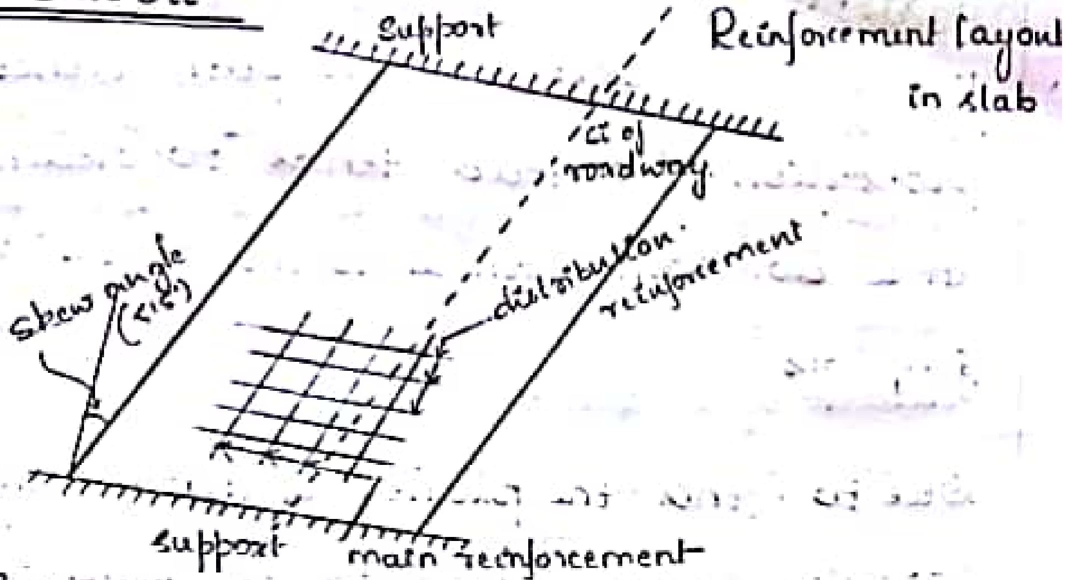
$\tau < \tau_c$ hence safe

step 6:


#20mm @
140mm c/c
WEARING COAT
80mm
500mm
BED BLOCK
ABUTMENT
#12mm @ 175mm c/c

5	15	465	20 ϕ 130	10 ϕ 90	20 ϕ 140	10 ϕ 300	16 ϕ 300
	30	465	20 ϕ 130	10 ϕ 90	20 ϕ 130	10 ϕ 300	16 ϕ 300
	45	465	20 ϕ 130	10 ϕ 90	20 ϕ 120	10 ϕ 200	16 ϕ 190
	60	465	20 ϕ 120	10 ϕ 110	20 ϕ 120	10 ϕ 180	16 ϕ 110
6	15	540	20 ϕ 115	10 ϕ 100	20 ϕ 110	10 ϕ 300	16 ϕ 300
	30	540	20 ϕ 115	10 ϕ 100	20 ϕ 110	10 ϕ 260	16 ϕ 300
	45	540	20 ϕ 115	10 ϕ 100	20 ϕ 110	10 ϕ 200	16 ϕ 170
	60	540	20 ϕ 95	10 ϕ 160	20 ϕ 110	10 ϕ 160	16 ϕ 90
8	15	700	25 ϕ 150	10 ϕ 90	20 ϕ 150	10 ϕ 260	20 ϕ 300
	30	700	25 ϕ 150	10 ϕ 110	20 ϕ 140	10 ϕ 230	20 ϕ 300
	45	700	25 ϕ 150	10 ϕ 150	20 ϕ 130	10 ϕ 160	20 ϕ 180
	60	700	25 ϕ 95	10 ϕ 180	20 ϕ 120	10 ϕ 90	20 ϕ 90

Skew slab culvert



when the alignment of a road crosses a stream at any angle other than 90° the crossings of the road is skew the road bridge crossing the stream is classified as skew slab culvert.

present day traffic Requirements demand straight alignment of the road in view of the fast traffic & this intern necessitate..

The analysis and design of skew bridge deck is more complicated then they adopt a straight bridge crossing at right angle to the stream.

The span length, deck area and the length of abutment supporting a bridge deck increases in proportion to the skew angle of the bridge.

with the increase in the skew angle the stress distribution in the skew slab differ's significantly in comparision with the straight slab.

A load applied on the deckslab is transmitted to support ...

various support possible path Hence a major proportion of a load tends to reach the support in a direction normal to the face's of the abutment & pier's.

Due to this the planes of maximum stresses are not parallel to the centreline of roadway. & the slab Exhibits for warped or twisted deformation characteristics due to passage of wheel loads on deck.

The reaction at the obtuse angle of slab support are larger than the other end with its magnitude ranging from 0 to 50% for skew angles of 20° to 50°

The bearing reaction developed are such that the acute angle supports lift up with a increase in the skew angle.

Centre of support & parallel to the centreline of roadway.

Distribution Reinforcement comprising 0.2% of the effective cross-section of slab is placed parallel to the support as shown in above fig.

Design steps.

1. Given data
2. permissible stress
3. depth of slab & effective span
4. dead load B.M
5. Live load B.M
6. Design BM DL & LL.
7. check for depth of slab.
8. Reinforcement details.
    a) main reinforcement
    b) Distribution reinforcement
9. Reinforcement details.

Problem :-

Design a skew slab culvert for a national highway crossing. of a stream to suit the following data.

clear span = 6m

width of bearing = 370mm

width of carriage way = 7.5m

overall depth of slab = 540mm .

wearing Coat = 80mm .

Skew angle = 30°

Type of loading = IRC class AA tracked vehicle.

material - M_{20} Concrete & Fe415 HYSD bars.

Solution:

Step1 : Given data .

Step2 : Depth of slab and . Effective span .

Referring to the design table 5.1 for the given clear span grade of concrete & slab the min. depth of slab

Effective span:-

i] [clear span + effective depth] $\Rightarrow$ $6+0.49 = \underline{6.49m}$

ii] [clear span + Bearing] $\Rightarrow$ $6+0.37 = \underline{6.37m}$

$\therefore$ Effective span = l = $\underline{6.37m}$

step3:- Bending moment due to dead load.

i] Self weight of the slab $\Rightarrow$ $D \cdot \gamma_{cc}$

$= 0.54 \times 25$

$= \underline{13.50} \; kN/m^2$

ii] self weight of wearing coat $\Rightarrow$ $t_w \cdot \gamma_{cc}$

$= 0.08 \times 22 = \underline{1.76} \; kN/$

$\therefore$ total dead load $\Rightarrow$ $\underline{15.26} \; kN/m^2$

$\therefore$ Load/m length: $15.\underline{26} \; kN/m$

Dead load Bending moment = $BM = \frac{wl^2}{8}$

$= \frac{15.26 \times 6.37^2}{8}$

$= 77.400 \; kN$

Generally the B.M due to live load will be Max at IRC class AA tracked vehicle.

Impact factor for IRC class AA vehicle is 25% for 5m span decreasing linearly to 10% for 9m span.

Therefore for 6.37m span = 19.86%.

The tracked vehicle is placed ~~sym~~ Symmetrically

The Effective length of load:-

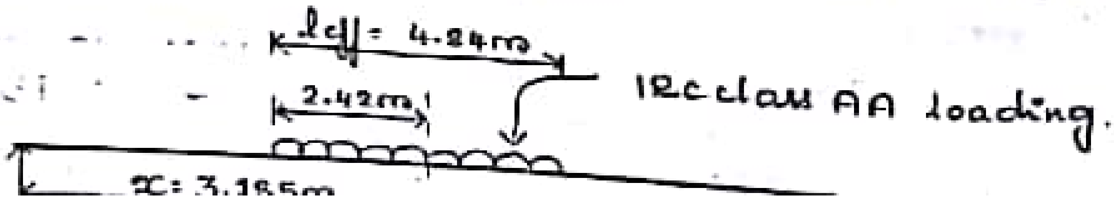
$$l_{eff} = x \cdot l' + 2(t_w + t_s)$$

$$= 3.6 + 2(0.54 + 0.08)$$

$$= \underline{4.84m}$$

The Effective width of slab perpendicular to span

$$b_{eff} = Kx\left[1 - \frac{x}{l}\right] b_w \, .$$



$\therefore\ K = B/L = \dfrac{9.5}{6.37} = \underline{1.491}$.

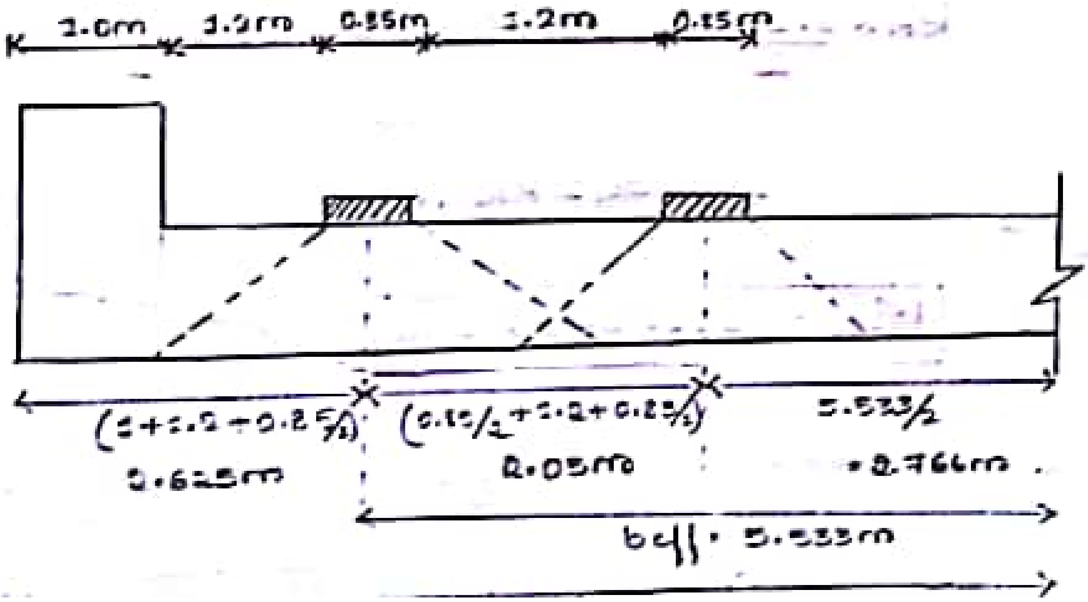
from table 4.6 $(B/L) = \underline{1.491}$ & simply supported slab the value of $K = \underline{2.84}$.

$b_w = (0.25 + 2t_w) = 1.01\,m$.

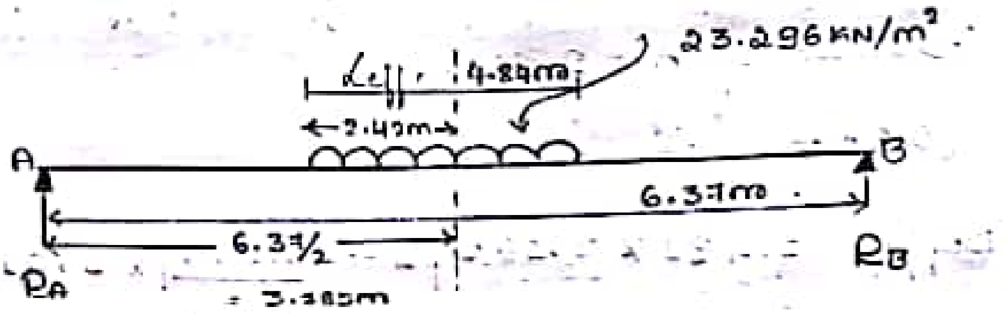
$$\therefore\ b_{eff} = 1.491 \times 3.185 \left[1 - \frac{3.185}{6.37}\right] \times 1.01$$

$b_{eff} = \underline{5.533m}$

Tracked vehicle is placed closed to the Kerb with minimum clearance. As shown in figure.



maximum B.M due to live load is given by



Support reaction $R_A = R_B = \frac{wl}{2} = \frac{23.296 \times 4.84}{2}$

$= 56.376$ KN

The max. B.M at midspan

$$R_A \times \frac{6.37}{2} - 23.296 \times 2.42 \times \frac{2.42}{2}$$

$= 111.342$ KN-m

$$\boxed{M_{LL} = 111.342 \text{ KN-m}}$$

Step 5:

Calculation of total B.M.

∴ the total B.M = dead load + live load

$= (1.35 \times 77.400) + (111.342 \times 1.5)$

$= \cancel{186.44}$ 271.503 KN-m

Step 6:

check for depth & reinforcement

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

$$d = \sqrt{\frac{M_{ulim}}{0.138 f_{ck} b}}$$

$$d = \sqrt{\frac{271.503 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d = 313.644mm < d_{provided}$ (490mm)

Hence safe.

Typical reinforcement skew slab culvert:

from. table 5.1

Bottom reinforcement:-

main reinforcement of 20mm diameter bars are spaced at 115mm c/c.

Distribution reinforcement of 10mm dia@100mm c/c

The free edges of the skewslab are strengthened by usi 20mm diameter bar at 110mm c/c.

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} . b . d}\right]$$

$$271.503 \times 10^6 = 0.87 \times 415 \times A_{st} \times 490 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 490}\right]$$

$\therefore A_{st} = \underline{1649.78 mm^2}$

Assume 20mm dia bars

$$\therefore \text{spacing} = \frac{a_{st}}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 20^2}{1649.78} \times 1000$$

$= 180.41 mm$

∴ provide # $20mm\phi$ @ $180mm$ c/c

from table 5.1 # $20mm\phi$ @ $115mm$ c/c.

Hence consider the # $20mm\phi$ @ $115mm$ c/c

Distribution reinforcement:

$A_{st} = 0.12\%$ Gross area.

$A_{st} = 0.12\% \times D \times b$

$= \frac{0.12}{100} \times 540 \times 1000$

$A_{st} = 648 mm^2$

∴ spacing $= \frac{a_{st}}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 78.54}{648} \times 1000$

$= 121.20 mm$

∴ provide # $10mm$ @ $120mm$ c/c.

table 5.1 # $10mm$ @ $100mm$ c/c

Hence consider the provide # $10mm$ dia @ $120mm$ c/c.

Step 7 :- check for strength & serviceability limit state

ultimate flexural strength using the following parameters of reinforced slab

$d = 490mm$

$f_y = 415 \ N/mm^2$

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

$A_{st} = 2700\ mm^2$

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} b d}\right]$$

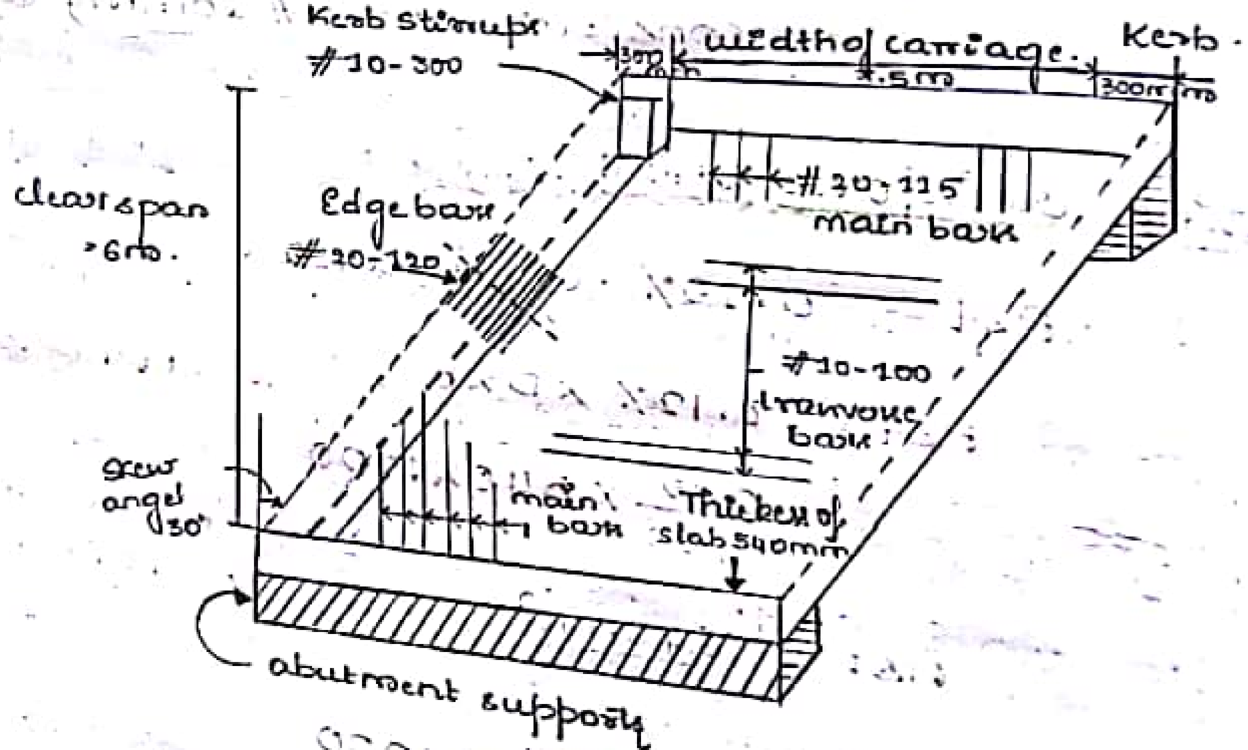
$$M_u = 0.87 \times 415 \times 2700 \times 490 \left[1 - \frac{490 \times 415}{20 \times 1000 \times 490}\right]$$

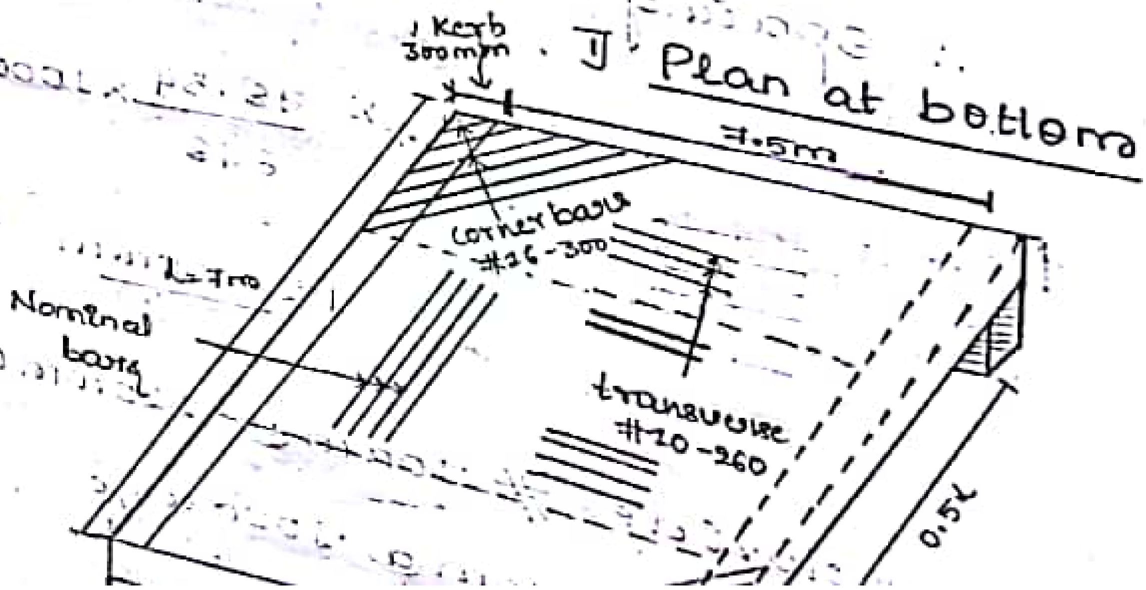
$M_u = 423.\ KN\text{-}m > 271.503\ KN\text{-}m$

Hence safe.

Scanned by CamScanner

Reinforcement details:-





27/03/17

MODULE - 03

Design of Interior Slab or Slab Pannel or Deck Slab

[T-Beam Slab]

1) A Super structure for T-beam slab bridge is to be des. for the following data

Clear span = 20mm

Clear road way = 7.5m

3 T-beams spaced @ 3m c/c

Width of bearing 1m

Effective Span of T-beam = 21m

Assume cross-beams spaced at 3.5m c/c

Thickness of slab = 220mm

Thickness of Wearing Coat = 80mm

Considering 1 pannel of deck slab & compute dead load mom-nt & live load moment due to IRC Class A-A tracked loading including impact factor & design the slab panel using M-20 concrete & Fe-415 steel.

Soln:- Clear span = 20m

Effective span = 21m

slab depth = 220mm

road way = 7.5m

Bearing width = 1m

Wearing Coat = 80m

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

$f_y = 415\ N/mm^2$

IRC class A-A tracked loading

a) Girder Details

3 main girders spaced at 3m c/c

Cross-beams are provided spaced at 3.5 c/c

1) Calculation of Permissible stresses

For M-20 ⇒ $\sigma_{cb} = 6.67\ N/mm^2$

For Fe-415 ⇒ $\sigma_{st} = 200\ N/mm^2$

$$m = \frac{280}{3\sigma_{cb}} = \frac{280}{3 \times 6.67} \Rightarrow \boxed{m = 13.99}$$

$K = \frac{m\sigma_{cb}}{m\sigma_{cb}+\sigma_{st}} = \frac{13.99 \times 6.67}{13.99 \times 6.67 + 200}$ $\Rightarrow \boxed{K = 0.318}$

$j = 1 - \frac{K}{3} = 1 - \frac{0.318}{3}$ $\Rightarrow \boxed{j = 0.894}$

$Q = \frac{\sigma_{cb} j k}{2} = \frac{6.67 \times 0.894 \times 0.318}{2}$ $\Rightarrow \boxed{Q = 0.943}$

2) Fixing the dimensions.

Depth of slab = 230mm

Take cover = 20mm

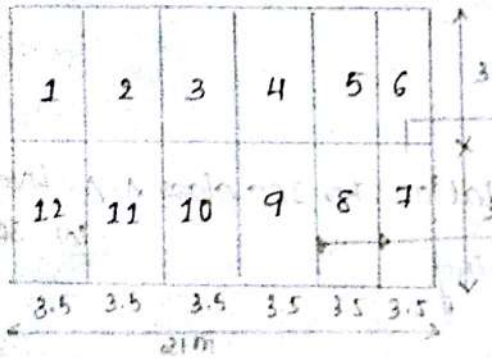
Effective depth = d = 230 − 20 ⇒ d = 210mm

Width of main girder & cross girder = $\frac{Span}{40} = \frac{21}{40}$

$\boxed{B = 0.525m.}$

Depth of main girder & c/s girder

$D = \frac{Span}{10} = \frac{21}{10} = 2.1m$



3) Load Calculation

a) Dead Load

Self Weight of slab = 0.23 × 25 = 5.75 KN/mm²

Self Weight of Wearing Coat = 0.08 × 22 = 1.76 KN/m²

$\boxed{\text{Total Load} = 7.51 KN/m^2}$

Total load on each panel = 3 × 3.5 × 7.51

$\boxed{W_1 = 78.85 KN}$

Calculation of U/B & V/L

For dead load calculation, U/B = V/L = 1

Calculate the value of, K = B/L

B = 3m ; L = 3.5m

K = 3/3.5

$\boxed{K = 0.857}$

Using Pigeaud's Curve Calculate moment co-efficients - [Pg. 99]

$m_1 = 0.039$ $m_2 = 0.035$ $\left(m_1 = \frac{4.16}{100} = 0.0416 \right.$

moment along shorter span [Refer $K = 0.9$ curve]

$M_B = W[M_1 + 0.15 m_2]$

$= 78.85[0.039 + 0.15 \times 0.035]$

$\boxed{M_B = 3.48 \text{ KN-m}}$

moment along longer span

$M_L = W[m_2 + 0.15 m_1]$

$= 78.85[0.035 + 0.15 \times 0.039]$

$\boxed{M_L = 3.22 \text{ KN-m}}$

Since the slab is continuous, consider continuity factor 0.8

Calculate $M_B = C.F \times 3.48$

$= 0.8 \times 3.48$

$\boxed{M_B = 2.784 \text{ KN-m}}$

$M_L = C.F \times 3.22$

$= 0.8 \times 3.22$

$\boxed{M_L = 2.576 \text{ KN-m}}$

b) Live load Bending moment Calculation: For IRC Class A-A Live load, $\boxed{W = 350 \text{ KN}}$

By considering Dispersion of wheel load & wearing coat,

$u = 0.85 + 2(W.C)$

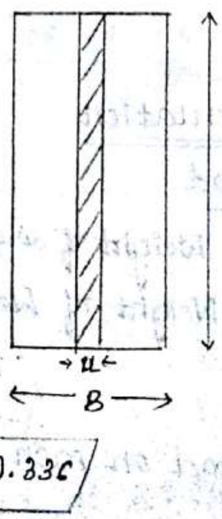
$= 0.85 + 2(80)$

$u = 160.85 \Rightarrow \boxed{1.01 m = u}$

$v = 3.6 + 2(W.C)$

$= 3.6 + 2(0.08)$

$\boxed{v = 3.76 \text{ m}}$



$\frac{u}{v} = \frac{1.01}{3.76} \Rightarrow \frac{u}{B} = \frac{1.01}{3} \Rightarrow \boxed{\frac{u}{B} = 0.336}$

$\frac{v}{L} = \frac{3.76}{3.5} \Rightarrow \boxed{\frac{v}{B} = 1.07}$

$K = \frac{B}{L} = \frac{3}{3.5} \Rightarrow \boxed{K = 0.856}$

Using Pigeaud's Curve, $m_1 = 0.08$, $m_2 = 0.0852$

where $m_1 = \frac{8}{100} = 0.08$, $m_2 = \frac{5.2}{100} = 0.052$

Calculate moments, consider impact factor as 25% [For only live load] (1.25)

$M_B = W[m_1 + 0.15 m_2]$

$= 380[0.08 + 0.15(0.052)] \times 1$

$\boxed{M_B = 38.412 \text{ KN-m}}$

$M_L = I_F \times W[m_2 + 0.15 m_1]$

$= 1.25 \times 350[0.052 + 0.15(0.08)]$

$\boxed{M_L = 28 \text{ KN}\cdot\text{m}}$

Considering continuity factor as 0.8

Ultimate moment

$M_B = 0.8 \times 38.412 \Rightarrow \boxed{M_B = 30.728 \text{ KN}\cdot\text{m}}$

$M_L = 0.8 \times 28 \Rightarrow \boxed{M_L = 22.4 \text{ KN}\cdot\text{m}}$

Total design moment, $\boxed{M_B = 30.728 \text{ KN-m}}$

$\boxed{M_L = 22.4 \text{ KN-m}}$

Design moments

Design of Slab.: Required Depth

$d_{req} = \sqrt{\dfrac{m}{Qb}} = \sqrt{\dfrac{30.72 \times 10^6}{0.948 \times 1000}} \Rightarrow d_{req} = 180.18 \text{ mm}$

d_{pro} (210mm) > d_{req} (180.18mm) Hence ok.

Calculation of Ast, Along shorter span.

$Ast = \dfrac{M}{\sigma_{st} j d} = \dfrac{30.72 \times 10^6}{200 \times 0.894 \times 210}$ $\boxed{Ast = 819.75 \text{ mm}^2}$

Assume 16mm ϕ

$\text{Spacing} = \dfrac{ast}{Ast} \times 100 = \dfrac{\pi(16)^2/4 \times 100}{819.75} = 245.27 \simeq 245 \text{mm c/c}$

$\boxed{\text{Provide 16mm diameter bars @ 245mm c/c}}$

Ast along longer span

$M = M_L = 22.4 \times 10^6 \text{ N}\cdot\text{mm}$

$Ast = \dfrac{M}{\sigma_{st} j d} = \dfrac{22.4 \times 10^6}{200 \times 0.894 \times 210} \Rightarrow Ast = 596.56 \text{mm}^2$

Assume 10 mm dia bars along longer span.

$\text{Spacing} = \dfrac{1000 ast}{Ast} = \dfrac{1000 \times \pi(10)^2/4}{596.56} \Rightarrow 131.65 \text{ mm c/c}$

$\boxed{\text{Provide 10 mm dia bars @ 131.65 mm c/c}}$

Check for Shear : Maximum design shear

$$V_u = \left[DL + IF \times \frac{\text{Wheel load}}{\text{Area of panel}}\right] 0.5$$

$$= \left[7.51 + 1.25 \times \frac{350}{3 \times 3.5}\right] \times 0.5$$

$$\boxed{V_u = 24.58\ KN}$$

Calculate, $\tau_v = \frac{V_u}{b \times d} = \frac{24.58 \times 10^3}{1000 \times 210} \Rightarrow \boxed{\tau_v = 0.117\ N/mm^2}$

τ_c = To calculate τ_c value, calculate % of steel

$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 819.75}{1000 \times 210}$

0.25 0.22
0.50 0.30

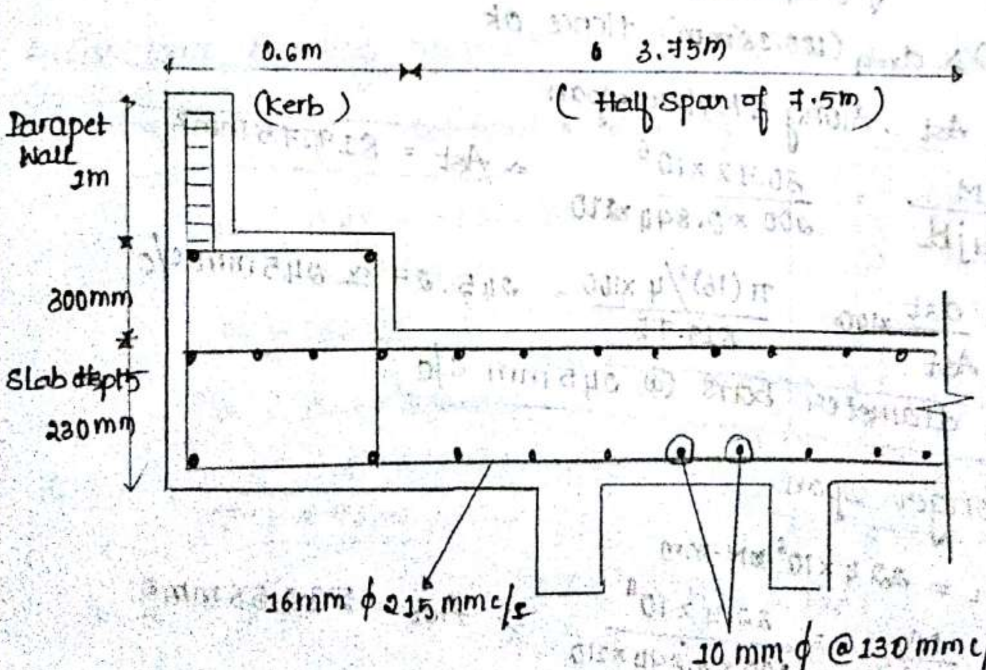
$\boxed{P_t = 0.39}$

The value of τ_c is IRC 21 Pg 36 ; $\tau_c = 0.26$.

$\tau_v < \tau_c$

Shear reinforcement is not required.

Detailing



Design of Main Girder

06/09/2018

Q) Design a RCC T-beam girder to suit the following details
Width of road way = 7.5m
Effective span = 16m
Thickness of W.C = 80mm
Use M-20.25 concrete & Fe 415 steel. Adopt IRC class A-A tracked live load condition.
Design the long girder & sketch the details

Sol^n: Given
Carriage way = 7.5m
Effective span = 16m
Thickness of W.C = 80mm
$f_{ck} = 25\ N/mm^2$; $f_y = 415$
IRC Class A-A tracked vehicle.

⇒ Assume 3 no's of long girders at 2.5m c/c
⇒ Assume 4 no's of cross girders at 4m c/c

i) Calculation of permissible stresses.

$\sigma_{cb} = 8.33$

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

$$m = \frac{280}{3\sigma_{cb}} = \frac{280}{3 \times 8.33} \Rightarrow \boxed{m = 11.20}$$

$$k = \frac{m\sigma_{cb}}{m\sigma_{cb} + \sigma_{st}} = \frac{11.20 \times 8.33}{11.2 \times 8.33 + 200} = k = 0.318$$

$$j = 1 - k/3 = 1 - 0.318/3 = \boxed{0.8954}$$

$$Q = \frac{kj\sigma_{cb}}{2} = \frac{0.318 \times 0.954 \times 8.33}{2} \quad \boxed{Q = 1.184}$$

2) Fixing of Dimensions

" (Depth of slab =

Clear cover =)"

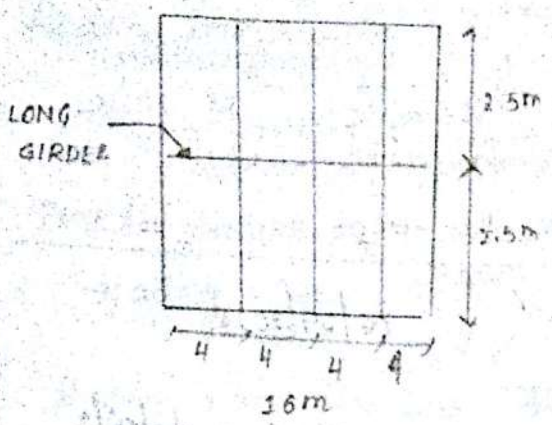
Width of Girder = $\frac{Span}{40} = \frac{16m}{40}$

$\boxed{\text{Width of Girder} = 0.4m}$

Depth of Girder = $\frac{Span}{10} = \frac{16m}{10}$

$\boxed{\text{Depth of Girder} = 1.6m}$

Assume thickness of slab = 200mm



3) Load Calculation

Dead Load,

Dead load of slab, $= 0.2 \times 25 \times 2.5 = 12.5$ KN/m

Dead load of W.C $= 0.08 \times 22 \times 2.5 = 4.4$ KN/m

Self Weight of long girder $= 0.4 \times 1.6 \times 25 = 16$ KN/m

Self Weight of c/s girder $= 0.4 \times 1.6 \times 25 = 16$ KN/m

$\boxed{\text{Total load} = 48.9 \text{ KN/m}}$

Dead load Bending Moment $= \frac{WL^2}{8} \Rightarrow \frac{48.9 \times (16)^2}{8}$

$\boxed{D.L \ B.M = 1564.8 \text{ KN-m}}$

Dead load shear Force $= \frac{WL}{2} = \frac{48.9 \times (16)}{2}$

$\boxed{D.L \text{ of } S.F = 391.2 \text{ KN}}$

4) Calculation of Live load Bending Moment & L.L Shear force

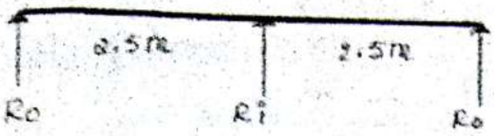
For IRC class A-A tracked vehicle, Live load $\boxed{W = 350 \text{KN}}$

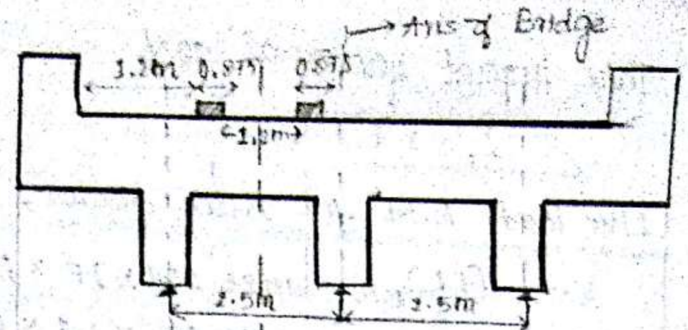
To calculate L.L Bending moment & S.F it is necessary to find out reaction factors for both outer girder & inner girder

The general formula for calculating reaction factor

$$R_x = \frac{\Sigma W}{n}\left[1 + \frac{\Sigma I d_x e}{\Sigma (d_x^2 I)}\right]$$

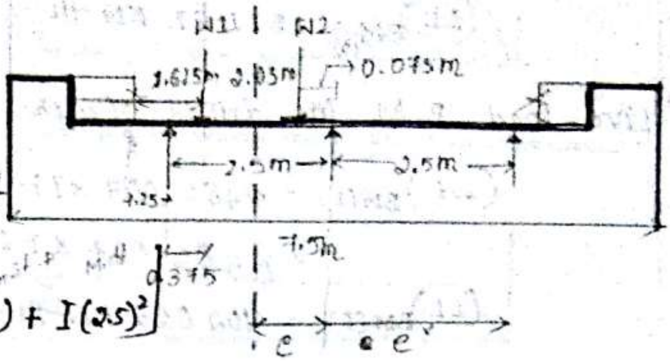
where e = eccentricity, calculated based on Courbon's





$$e = \frac{2.05}{2} + [0.075]$$

$$\boxed{e = 1.1m}$$



⇒ Reaction factor for outer girder

$$R_o = \frac{\Sigma W}{n}\left[1 + \frac{3I(2.5)1.1}{I(2.5)^2 + I(0) + I(2.5)^2}\right]$$

$$= \frac{\Sigma W}{3}\left[1 + \frac{3(2.5)(1.1)}{I[(2.5)^2 + (2.5)^2]}\right]$$

no of girders

$$= \frac{1}{3}\left[1 + 0.66\right]$$

$$\boxed{R_o = 0.553\,\Sigma W}$$

⇒ Reaction factor for inner girder

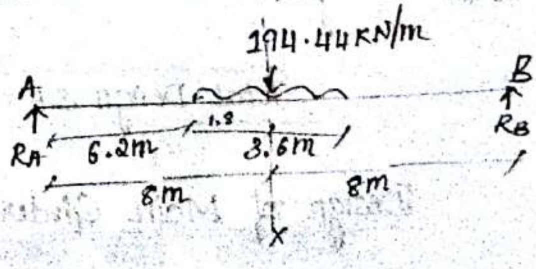
$$R_i = \frac{\Sigma W}{n}\left[1 + \frac{3(I(0)(1.1)}{I(2.5)^2 + I(0) + I(2.5)^2}\right]$$

$$= \frac{\Sigma W}{3}\left[1 + \frac{I(3 \times 1.1)}{I[(2.5)^2 + (2.5)^2]}\right]$$

$$\boxed{R_i = 0.33\,\Sigma W}$$

⇒ Calculation of Live Load & Bending Moment

Intensity of UDL $= \frac{350 \times 2}{3.6}$ ⇒ $\boxed{UDL = 194.44\ KN/m}$



$\Sigma M_A = 0;$

$$-16R_B + 194.44 \times 3.6 \times \left(\tfrac{3.6}{2} + 6.2\right) = 0$$

$$\boxed{R_A = R_B = 350KN}$$

$\Sigma M_{x-x} = 0;$

$$350 \times 8 - \left[194.44 \times \frac{3.6}{2} \times \frac{3.6}{4}\right] = 0$$

$$\boxed{M_{x-x} = 2485.007\ KN\text{-}m}$$

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Take Impact factor as 10%

$\boxed{I_F = 1.1}$

Live load B.M for outer girder,

$(LL)_{BM} = 2485.007 \times IF \times$ Reaction factor for outer girder

$= 2485.007 \times 1.1 \times 0.553$

$\boxed{(LL)_{BM_{(o)}} = 1511.62 \text{ KN-m}}$

Live load B.M for inner girder

$(LL)_{BM(I)} = 2485.007 \times I_F \times$ Reaction factor for inner girder

$= 2485.007 \times 1.1 \times 0.33$

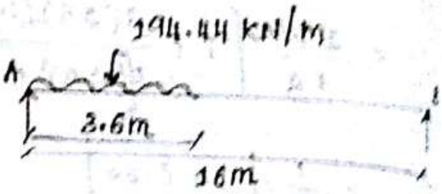
$\boxed{(LL)_{BM(I)} = 902.057 \text{ KN-m}}$

Live load Shear Force

$\Sigma M_A = 0$;

$-16R_B + (194.44 \times 2.6 \times 2.6/2) = 0$

$\boxed{R_B = 78.74 \text{ KN-m}}$



$\Sigma V = 0$;

$R_A + 78.74 - (194.44 \times 2.6) = 0$

$\boxed{R_A = 621.24 \text{ KN}}$

Maximum shear force with impact factor

$= 621.24 \times 1.1 \Rightarrow \boxed{(S.F)_{Max} = 683.36 \text{ KN}}$

Total Design Bending Moment $=$ DLBM + LLBM

$= 1564.8 + 1511.62$

$\boxed{\text{Total Design B.M} = 3076.42 \text{ KN-m}}$

Total Design shear force $=$ DLSF + LLSF

$= 391.2 + 621.24$

$\boxed{\text{Total Design S.F} = 1012.44 \text{ KN}}$

Design of Main Girder.

$$Ast = \frac{M}{\sigma_{st} j d} = \frac{3076.42 \times 10^6}{200 \times 0.8954 \times 1500} \Rightarrow \boxed{Ast = 11470.61 \text{ mm}^2}$$

Approximate Lever-Arm Distance

$= 1600 - 200/2$

$\boxed{d = 1500 \text{mm}}$

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Assume dia 32mm bars & calculate no of bars

$$\text{No of bars} = \frac{Ast}{ast} = \frac{11470.61}{\pi(32)^2/4} \Rightarrow 14.26 \simeq 16$$

$\therefore$ $\boxed{\text{No of bars} = 16 \text{ no's}}$

$\therefore$ Provide 16 no's of 32mm dia rods as main reinforcement

Check for shear.

$$V_u = 1012.44 \text{ KN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{1012.44 \times 10^3}{1500 \times 400} \Rightarrow \boxed{\tau_v = 1.68\ N/mm^2}$$

$$\tau_c = P_t = \frac{100 Ast}{bd} = \frac{100 \times 12067}{1500 \times 200} \Rightarrow P_t =$$

$$\boxed{\tau_t = 2.01\ N/mm^2}$$

$$\boxed{\tau_c = 0.5\ N/mm^2}$$

$\therefore \tau_v > \tau_c$.

Hence it is unsafe, shear reinforcement is required.

Shear Reinforcement Design.

Assume 4 Legged 10mm ϕ stirrups.

$$\text{Spacing} = \frac{\sigma_{st} Ast\, d}{V_u}$$

$$Ast = 4 \times \frac{\pi (10)^2}{4} \Rightarrow \boxed{Ast = 314.15 mm^2}$$

$$S_v = \frac{200 \times 314.15 \times 1500}{1012.43 \times 10^3}$$

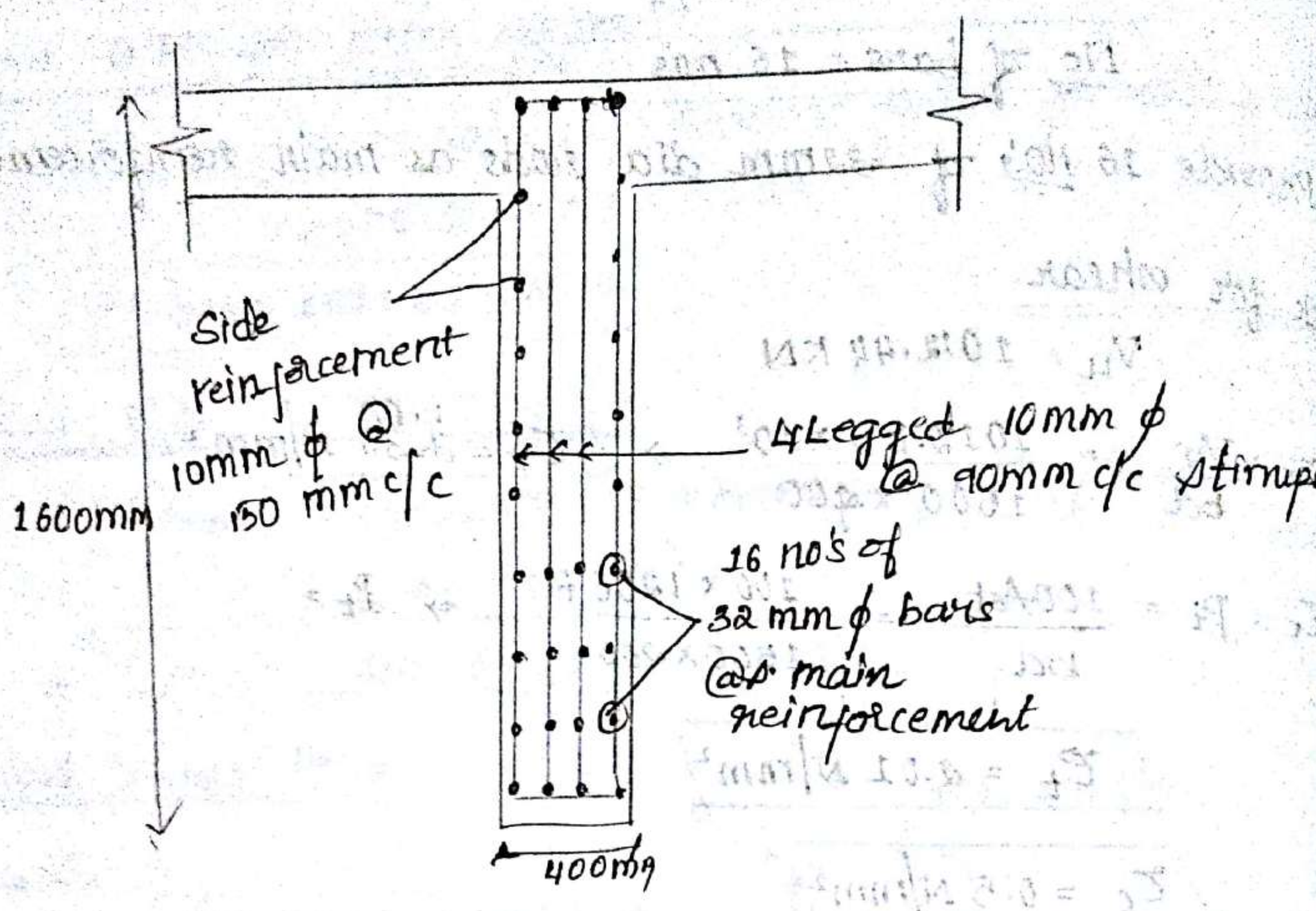
$$\boxed{S_v = 93 mm \simeq 90 mm}$$

$\boxed{\therefore \text{Provide } 10mm\ \phi \text{ stirrups @ 90mm c/c}}$

If the depth of the girder is more than 750 mm, side reinforcement should be provided.

Provide 10mm ϕ bars @ 150 mm c/c as side reinforcement

Detailing.



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MODULE - 04

DESIGN OF BOX CULVERT

1) Design a box culvert having dimension 3m * 3m. The culvert is subjected to a dead load 14 kN/m² & Live load IRC class A-A tracked vehicle. Assume unit weight of soil to be 18 kN/m³. Angle of repose 30°. Use M-25 concrete & Fe 415 steel. Road width = 7.5m. Show the reinforcement details.

Given Data -

Dimension, = $3m \times 3m$.

Dead load = $14 \, kN/m^2$

Unit Weight / Density = $18 \, kN/m^3$

$\phi = 30°$

Clear span of box = 3m

Height of vent = 3m

M25 Concrete, Fe 415 steel

Class A-A tracked vehicle.

a) Calculation of permissible stresses

$m = 280/\sigma_{cbc} 3 \Rightarrow \boxed{m = 11.20}$

$k = \dfrac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} \Rightarrow \boxed{k = 0.318}$

$j = 1 - k/3 \Rightarrow \boxed{j = 0.8954}$

$Q = \dfrac{\sigma_{cbc} * k * j}{2} \Rightarrow \boxed{Q = 1.184}$

b) Dimension of box culvert.

The given dimension of the box = $3 * 3m$

Adopt. thickness of the slab is 100mm/m for span

thickness of the slab = thickness of the wall

$t_{slab} = t_{wall} = 100 * 3$

$\boxed{= 300 \, mm}$

Effective span., $l_{eff} = 3 + 0.3/2 + 0.3/2 = \boxed{3.3 \, m}$

Calculation of Live load intensity of UDL by considering with impact factor

$$\text{Intensity of UDL} = \frac{\text{Impact factor} * \text{Load}}{B_d * b}$$

where ; impact factor = 25%

L_d = length of dispersion ; B_d = Width of dispersion.

$L_d = 3.6 + 2$ [Deck slab thickness + Wearing Coat]

$= 3.6 + 2(0.3 + 0.08)$

$\boxed{L_d = 4.36\,m}$

To calculate width of dispersion, first calculate effective length on which wheel load acts.

$$b_{eff} = Kx\left[1 - \frac{x}{L_{eff}}\right] + B_W$$

where $L_{eff} = 3.3\,m$; $x = \frac{3.3}{2} = 1.65$; $B_W = 0.85 + 2(0.08) = 1.01\,m$.

To calculate the value of $K = B/L = \dfrac{\text{Cariage Way Distance + 600 Kerbs Both side}}{3.3}$

$$= \frac{7.5 + 0.6 + 0.6}{3.3} \Rightarrow \boxed{B/L = 2.636}$$

From the value of $B/L = 2.636$, reffering IRC 21 the value of K is 2.6 for continuous slab.

$$b_{eff} = 2.6 * 1.65\left[1 - \frac{1.65}{3.3}\right] + 1.01$$

$$\boxed{b_{eff} = 3.155\,m}$$

Width of dispersion $= \dfrac{b_{eff}}{2} = \dfrac{3.155}{2} \Rightarrow 1.577\,m$

Calculate ~~width of dispe~~

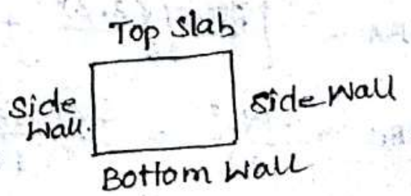
$b_d = 1.577 + 1.577 + 1.2 + 0.85 = \dfrac{b_{eff}}{2} + \dfrac{b_{eff}}{2} + \left(\dfrac{1.2}{1} + \dfrac{0.85}{2} + \dfrac{0.85}{2}\right)$

$\boxed{b_d = 5.204}$

UDL Intensity, $W = \dfrac{I_F \times load}{L_d \times b_d} = \dfrac{1.25 * 700}{4.36 * 5.204}$

$\boxed{W = 38.56\ KN/m^2}$

c) Load Calculation.



i) Top slab load = Self Wt + D.L + L.L

$= 9.26 + 14 + 38.56$

$= 61.82\ KN/m^2$

ii) Side Wall on top = $\frac{1}{3}$ * Total Load of top slab.

$= \frac{1}{3} * 61.82$

$= 20.61\ KN/m^2$

iii) Bottom Wall load = Top load + soil pressure.

$= 20.61 + K_a \gamma h \Rightarrow 20.61 + 0.33 * 18 * 3.3$

$= 40.41\ KN/m^2$

Bottom slab load = load from top slab + SW of side Wall

$= (61.82 * 3.3 * 1) + (2 * 3.3 * 0.3 * 25)$

$= \underline{253.506\ KN/m^2}$

Self Weight of slab = Thickness $\times \gamma_c$

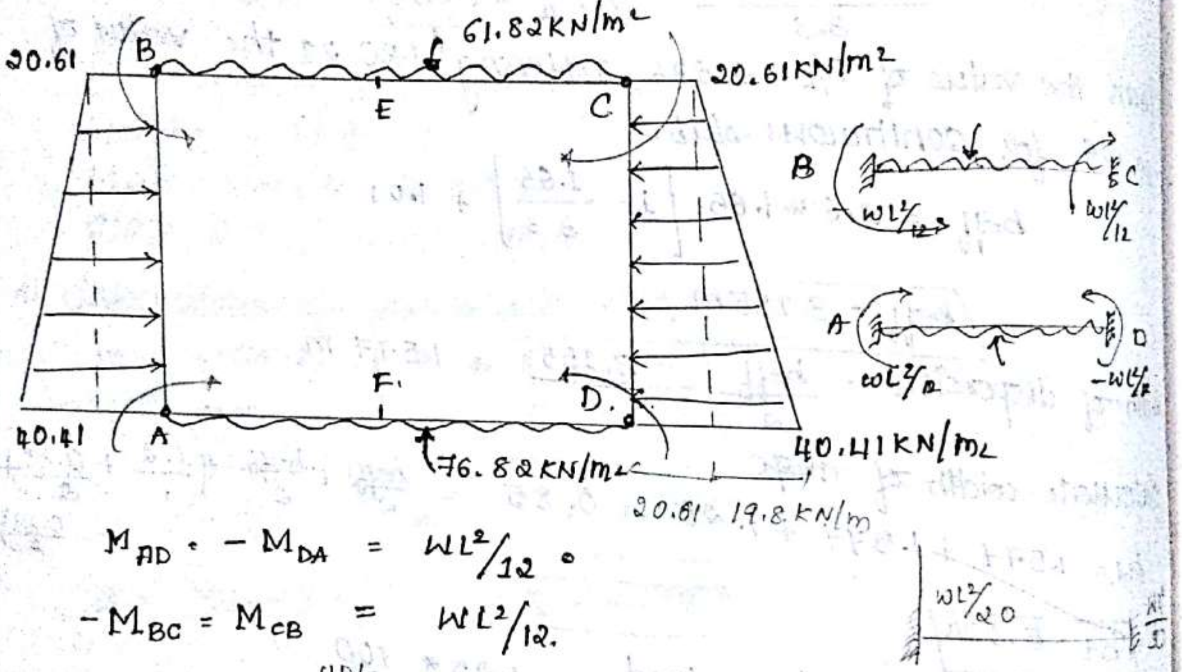
$= 0.3 * 25$

$= \underline{7.5 KN/m^2}$

Self Weight of Wearing Coat $= 0.08 \times 22 = \underline{1.76\ KN/m^2}$

Total load $= 9.26\ KN/m^2$

Intensity $= \dfrac{253.506}{1 * 3.3} \Rightarrow \boxed{\text{Intensity} = 76.82\ KN/m^2}$



$M_{FAD} = -M_{DA} = WL^2/12$

$-M_{BC} = M_{CB} = WL^2/12$

$M_{FAB} = -\left[\underbrace{\frac{WL^2}{12}}_{UDL} + \left(\underbrace{\frac{WL^2}{20}}_{UVL}\right)\right] \qquad M_{FBA} = \left[\frac{WL^2}{12} + \left(\frac{WL^2}{20}\right)\right]$ UVL

$M_{FAB} = -\left[\frac{20.61(3.3)^2}{12} + \frac{19.8(3.3)^2}{20}\right] \Rightarrow \boxed{M_{FAB} = -29.48\ KN\text{-}m}$

$M_{FBA} = \left[\frac{20.61(3.3)^2}{12} + \frac{19.8(3.3)^2}{20}\right] \Rightarrow \boxed{M_{FBA} = 25.85\ KN\text{-}m}$

$M_{FBC} = \frac{-WL^2}{12} = -\frac{61.82(3.3)^2}{12} \Rightarrow \boxed{M_{FBC} = -56.10\ KN\text{-}m}$

$M_{FCB} = \frac{WL^2}{12} = \frac{61.82(3.3)^2}{12} \Rightarrow \boxed{M_{FCB} = 56.10\ KN\text{-}m}$

$M_{FAD} = \frac{WL^2}{12} = \frac{76.82(3.3)^2}{12} \Rightarrow \boxed{M_{FAD} = 69.714\ KN\text{-}m}$

$M_{FDA} = \frac{-WL^2}{12} = \frac{76.82(3.3)^2}{12} \Rightarrow \boxed{M_{FDA} = -69.714\ KN\text{-}m}$

Balancing of moments is done by moment distribution Method.

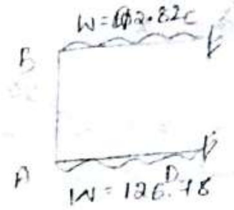
Distribution factors

$$\boxed{\begin{aligned} D_{AB} &= D_{BA} = 2/3 \\ D_{BE} &= D_{AF} = 1/3 \end{aligned}}$$

JOINT	A		B	
MEMBER	AD	AB	BA	BC
DIST. FACTOR	1/3	2/3	2/3	1/3
Fixed E. M.	69.714	-29.48	25.89	-56.12
Balance	-13.41	-26.8	20.14	10.09
carry over.	0	10.07	-13.4	0
Balance	-3.35	-6.71	+8.94	4.47
Carry Over.	0	+4.47	-3.355	0
Balance.	-1.49	-2.98	2.23	1.167
carry Over.	0	1.11	-1.49	0.
Balance	-0.37	-0.74	0.99	0.49
Carry over	0	0.495	-0.37	0.
Balance		~~0.185~~ -0.33	0.246	0.128
Σ.	51.09	-50.89	39.62.	-39.7

Moments

$$\boxed{\begin{aligned} M_{AD} &= 51.09\ KN\text{-}m \\ M_{AB} &= -50.89\ KN\text{-}m \\ M_{BA} &= 39.62\ KN\text{-}m \\ M_{BC} &= -39.7\ KN\text{-}m \end{aligned}}$$



W = Top slab load

Calculation of reactions

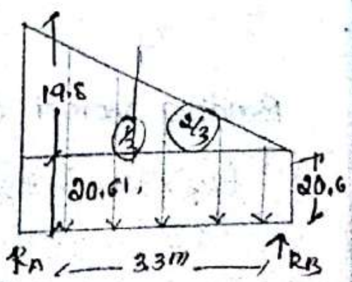
(i) Span BC

Vertical Reactions $= R_B = R_c = WL/2 = \frac{61.82 \times (3.3)}{2}$

$$\boxed{R_B = R_c = 102.05 KN}$$

ii) Span AD $= R_A = R_D = \frac{WL}{2} = \frac{76.82 \times 3.3}{2}$

$$\boxed{R_A = R_D = 126.78 KN}$$



iii) Span AB $= \Sigma M_B = 0;$

$$3.3 R_A - \left(20.6 \times 3.3 \times \frac{3.3}{2}\right) - \left[\tfrac{1}{2} \times 3.3 \times 19.8\right] \times \left(\tfrac{2}{3} \times 3.3\right) = 0$$

$\boxed{R_A = 55.70\,kN}$

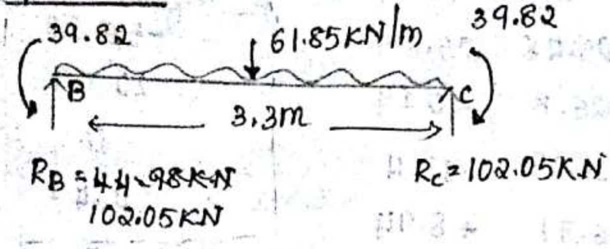
$\Sigma V = 0;$

$$R_A + R_B = (20.61 \times 3.3) + (\tfrac{1}{2} \times 3.3 \times 19.8)$$

$$\boxed{R_B = 44.9\,KN}$$

Calculation of Moments for the different spans.

Span BC



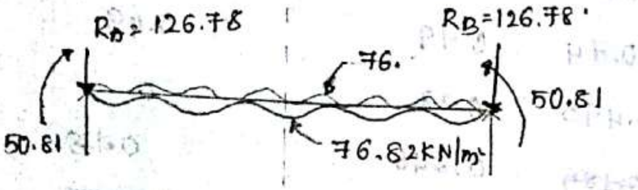
Bending moment @ centre

$$= \left(-61.85 \times 1.65 \times \frac{1.65}{2}\right) + (102.05 \times 1.65) - 39.82$$

$$\boxed{= 44.369\,KN\text{-}m}$$

Bending moment @ support $\boxed{= 39.82\,KN\text{-}m}$

Span AD



Bending moment @ centre.

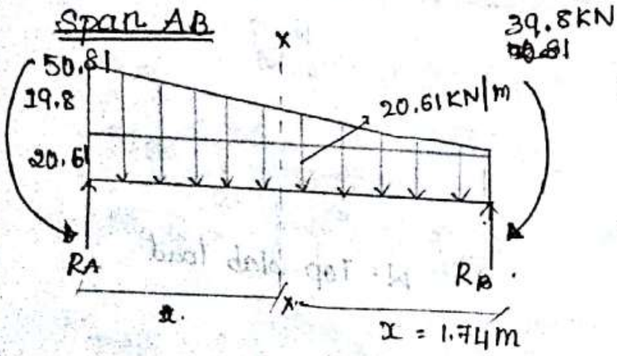
$$= \left(76.82 \times \frac{3.3}{2} \times \frac{3.3}{4}\right) - (126.75 \times 1.65)$$

$$+ 50.81$$

$$\boxed{= -53.80\,KN\text{-}m}$$

Bending Moment @ support $= \boxed{50.81\,KN-m}$

Span AB



$\Sigma V = 0;$

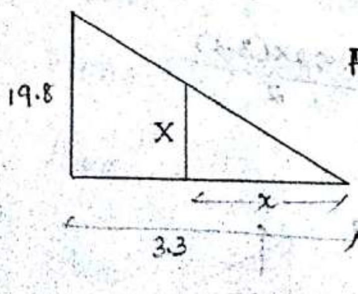
$$R_B = (20.61 \times x) + \tfrac{1}{2} \times x \times X$$

$$= 20.61x + \tfrac{1}{2} \times x \times 6x$$

$$R_B = 3x^2 + 20.61x$$

$$3x^2 + 20.61x - 449 = 0$$

$$x = 1.74\,m$$



From similar Δ^{le}

$$\frac{19.8}{3.3} = \frac{X}{x}$$

$$X = 6x$$

$$X = 6(1.74)$$

$$\boxed{X = 10.44}$$

Bending Momet @ X-X

$$(R_B \times 1.74) - \left(20.61 \times 1.74 \times \frac{1.74}{2}\right) - \left(\tfrac{1}{2} \times 1.74 \times 10.44\right) \times \left(\tfrac{2}{3}\,1.74\right)$$

$$\cancel{-50.81} - 39.8$$

$$\cancel{= 44.01\,KNm} \quad \boxed{= +3.4\,KN\text{-}m}$$

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Bending moment @ support = 50.81 kN·m & 39.8 kN

Design of top slab.

Span BC

Bending moment @ centre = 44.369 kN·m

Bending moment @ support = 39.82 kN·m

$d = 300 - 50 = 250$

$$A_{st} \text{ @ mid span} = \frac{M}{\sigma_{st}\, j \cdot d} = \frac{44.369 \times 10^6}{200 \times 0.894 \times 250}$$

$$\boxed{A_{st} = 992.59\ mm^2}$$

Assume 16mm ϕ bars

$$\text{spacing} = \frac{\pi(16)^2/4}{992.59} \times 1000 \Rightarrow S = 202.6 \simeq 200mm$$

∴ Provide # 16mm ϕ bars @ 200mm c/c for mid span & bent at support

$$A_{st} \text{ @ support} = \frac{M}{\sigma_{st}\, j \cdot d} = \frac{39.82 \times 10^6}{200 \times 0.894 \times 250}$$

$$\boxed{A_{st} = 890.82\ mm^2}$$

Assume 10 mm ϕ bars

$$\text{Actual } A_{st} \text{ @ support} = 890.82 - \frac{992.39}{2}$$

$$\cancel{\text{Spacing} = \frac{\pi(10)^2/4}{890.82}}$$

$$\boxed{(A_{st})_{act} = 394.57 mm^2}$$

$$\text{spacing} = \frac{\pi(10)^2/4}{394.57} \times 1000 \Rightarrow S = 199.05 \simeq \overset{195}{\cancel{200}} mm$$

∴ Provide # 10mm ϕ bars @ $\overset{195}{\cancel{200}}$ mm c/c @ support

Distribution Steel.

$$A_{st} = 0.2\%\ bD$$

$$= \frac{0.2}{100} \times 1000 \times 300$$

$$\boxed{A_{st} = 600\ mm^2}$$

$$\text{Area of each face} = \frac{600}{2} = 300 mm^2$$

Assume 8 mm ϕ bars.

$$\text{Spacing} = \frac{\pi(8)^2/4}{300} \times 1000 \Rightarrow S = 167.55 mm$$

∴ Provide # 8mm ϕ bars @ 160mm c/c as distribution steel

Design of Bottom slab (span AD)

Bending Moment @ centre = 53.80 KN-m

Bending Moment @ support = 50.81 KN-m

Ast at midspan = $\frac{53.80}{200 \times 0.894 \times 250}$ $\Rightarrow$ $\boxed{Ast = 1203.579\ mm^2}$

Provide # 16mm ϕ bars.

Spacing = $\frac{\pi (16)^2/4}{1203.57} \times 1000$ $\Rightarrow$ $\boxed{S = 167.054\ mm}$

$\boxed{\therefore \text{Provide \# 16mm } \phi \text{ bars at } 165 \text{ mm c/c}}$

Ast at support $\Rightarrow$ Ast = $\frac{50.81}{200 \times 0.894 \times 250}$ $\Rightarrow$ $\boxed{Ast = 1136.689\ mm^2}$

$M = 50.81 - \frac{53.80}{2}$ Actual Ast = $1136.68 - \frac{1203.579}{2}$

=

$\boxed{Ast = 534.89\ mm^2}$

Spacing = $\frac{\pi(10)^2/4}{534.89} \times 1000 = 146.833\ mm^2$

$\boxed{\therefore \text{Provide \# 10mm } \phi \text{ bars @ } 145 \text{ mm c/c}}$

Distribution Bars. : It is same as span BC

Design of side Wall (span AB & BC)

NOTE : To design side wall limit state design method is preferred

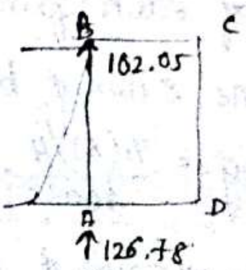
B.M @ centre = 3.42 KN-m

B.M @ support = 50.81 KN-m

Direct force (P_u) = 55.7 KN

$M_u = 50.81$ KN-m; $P_u = 55.7$ KN

$\frac{M_u}{f_{ck} b D^2} = \frac{50.81 \times 10^6}{25 \times 1000 \times (300)^2} = 0.022$



$\frac{P_u}{f_{ck} b D^2} = \frac{126.78 \times 10^3 \ \cancel{55.7 \times 10}}{25 \times 1000 \times (300)^2} = 0.056$

Referring to SP 16 curves by keeping the values $\frac{M_u}{f_{ck} b D^2}$ & $\frac{P_u}{f_{ck} b D}$

The value of $\boxed{P/f_{ck} = 0.02}$

% of steel = $f_{ck} \times 0.02$ $\langle d'/d = \frac{50}{300} = 0.167 \rangle$

$= 25 \times 0.02$

$= 0.5\%$

As per limit state, $A_{st} = \frac{PbD}{100} = \frac{0.5 \times 1000 \times 300}{100}$

$A_{st} = 1500\,mm^2$

Minimum steel, $= 0.8\%\ bD$

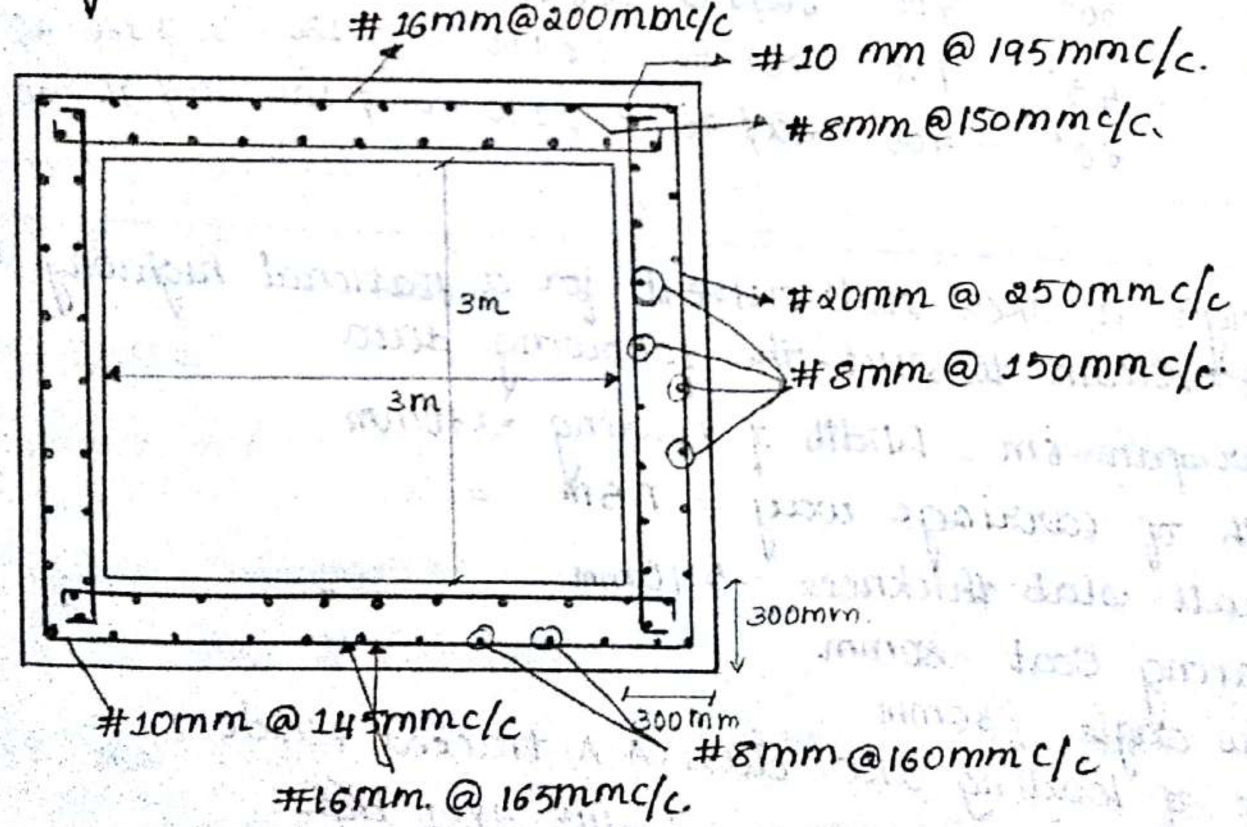
$= \frac{0.8}{100} \times 1000 \times 300$

$(A_{st})_{min} = 2400\,mm^2$

For one face $= \frac{2400}{2} = 1200\,mm^2$

Provide 20mm ϕ bars @ 250mm c/c b main bars.
Distribution bars of 8mm ϕ @ 150mm c/c

Detailing.



DESIGN OF PIPE CULVERT

1) Design a pipe culvert for the following data

Road-embankment height - 6m

Width of road - ~~8m~~ 7.5m

Formation width - 10m

Side slope of embankment - 1.5 : 1

Discharge - 5 m^3/sec

Safe velocity of flow - 3m/sec

Loading IRC class A-A - tracked vehicle.

Entry condition - Bell mounted

Edge bearing strength - 72 KN/m

Spiral Reinforcement - 215.2 N/m

Longitudinal Reinforcement - 26.6 N/m

Unit Weight of embankment - 20 KN/m^3

$C_e = 1.5$; $C_s = 0.015$; $d_i = 1m$; $d_o = 1.23m = D$

Given Data:

$Q = 5 m^3/sec$; $v = 3m/sec$; Loading = IRC class A-A tracked

$d_i = 1m$; $D = 1.2m$; $\gamma_{emb} = 20 KN/m^3$; side slope = 1.5 : 1

Sol:

1) Hydraulic Design.

W.K.T, $Q = KAV$ where, K = constant (convergence factor) depends on entry cond^n, roughness of pipe & length of pipe & it is calculated by

$$K = \frac{1}{\sqrt{1 + K_e + K_f}}$$

K_e = co-efficient of head loss @ entry.

= 0.08 for bell-mounted entry

= 0.51 for sharp edge entry

K_f = co-efficient of head loss due to friction

i.e., calculated by. $K_f = 0.0033 * \frac{L}{(R)^{1/3}}$

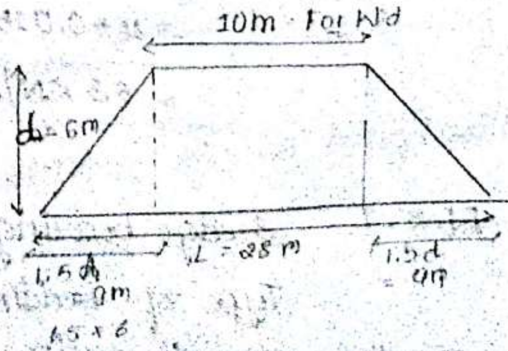
where, L = Length of pipe in m

R = Hydraulic mean depth.

$K_e = 0.08$;

$$R = \frac{Area}{Perimeter} = \frac{\pi d_i^2/4}{\pi d_i}$$

$$R = \frac{\pi (1)/4}{\pi (1)} \Rightarrow R = 0.25$$



$$K_f = 0.0033 * \frac{l}{R^{1/3}} = 0.0033 * \frac{28}{(0.25)^{1/3}}$$

$$K_f = 0.146$$

$$\Rightarrow K = \frac{1}{\sqrt{1+K_e+K_f}} = \frac{1}{\sqrt{1+0.08+0.146}} \Rightarrow \boxed{K = 0.903}$$

$\Rightarrow$ $Q = KAV \Rightarrow 5 = 0.903 * A * V^3$

$$\boxed{\text{Area}, A = 1.845 m^2}$$

$\Rightarrow$ But area of single pipe is, $a = \frac{\pi (d_i)^2}{4}$

$$= \frac{\pi (1)^2}{4}$$

$$\boxed{a = 0.785 m^2}$$

$\Rightarrow$ No of pipes $= \frac{Ast}{a} = \frac{1.845}{0.785} = 2.835$

$$\boxed{\text{no of pipes} \simeq 3 \text{ no's}}$$

Provide 3 no's of pipe with internal diameter 1m

2) Structural Design.

A] Type of Loading

(i) In structural design

(a) Load on the pipe due to earthfill in KN/m

$$= C_e W D^2$$

where, C_e = Co-efficient ; W = unit Wt of embankment

D = External diameter of the pipe

$$= 1.5 * 20 * (1.23)^2$$

$$\boxed{= 45.38 \text{ KN/m}}$$

(b) Load of pipe due to IRC. Class A-A tracked vehicle (KN/m)

$= 4 C_s I P$, where P = Live load due to Class A-A tracked vehicle

$= 4 * 0.015 * 1.5 * 700$ $P = 700$ KN

$\boxed{= 63 \text{ KN/m}}$ I = constant $= 1.5$

$C_s = 0.015$

W.K.T 3 edge bearing strength $= 72$ KN/m

Type of bedding under the pipe is selected based on the strength factor due to embankment load in KN/m & this is calculated using following eqn

$$\frac{\text{3 Edge bearing strength}}{1.5} = \frac{\text{Load on pipe due to earthfill (KN/m)}}{\text{strength factor}} + \frac{\text{load on pipe due to IRC class A-A tracked vehicle (KN/m)}}{1.5}$$

$$\frac{72}{1.5} = \frac{45.38}{S.F} + \frac{6.3}{1.5}$$

$\boxed{\text{strength factor} = 7.56}$

∴ The strength factor is greater than 3.7, hence concrete craddel bedding may be provide

* Sprial Reinforcement
Minimum sprial reinforcement to be provided = 215.2 N/m

Using 12mm ϕ bars @ spacing 60mm c/c

weight of 1 sprial = circumference * Area of steel * Density of steel (N)

$= \pi D * \pi d^2/4 * 78.000\ (N)$

$= \pi(1.115) * \pi \frac{(\overset{0.012}{\cancel{0.05}})^2}{4} * 78{,}000$

$\boxed{= 30.9\ N}$

$D = \frac{d_i + d_o}{2} = \frac{1 + 1.23}{2}$

$D = 1.115\,m$

No of spiral in 1m length $= \frac{1000}{60} = 16.67$.

• Weight of spiral reinforce = Wt of 1 spiral * no of spiral

$= 30.9 * 16.67$

$\boxed{= 515.10\,N > 215.2\ N/m}$

Hence it is safe.

* Minimum Longitudinal Reinforcement in 1m length = 26.6 N/m

Using ~~26.6N/m~~ 6mm ϕ bars as longitudinal bars.

Wt of 1 longitudinal bar for 1m length = Area × 1m × Density of steel

$= \frac{\pi d^2}{4} * 1m * 78000$

$= \frac{\pi(0.006)^2}{4} * 1 * 78000$

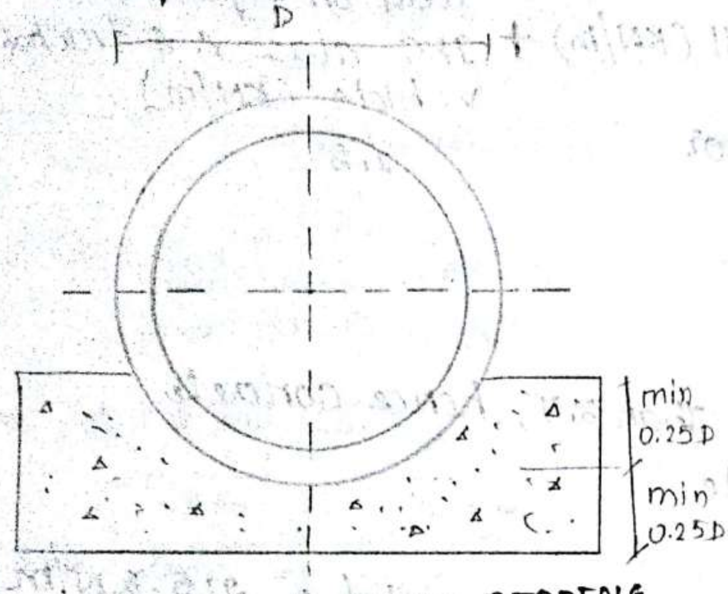
$\boxed{= 2.2\ N}$

No of bars required $= \frac{26.6}{2.2} = 12.09 \simeq 12$ no's

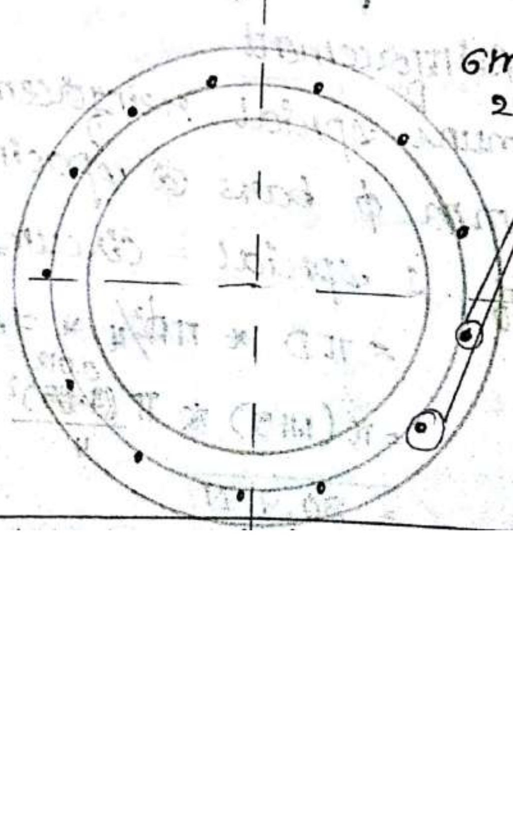
spacing of longitudinal bars $= \frac{\pi D}{\text{no of bars}} = \frac{\pi(1.115) * 10^3}{12}$

$= 290\,mm$

Provide 6mm dia bars @ spacing 290mm c/c as longitudinal reinforcement.



CONCRETE CRADDLE BEDDING



Design a suitable RCC pipe culvert to suit following data

Discharge through pipe culvert = 1.57 m³/se

velocity of the pipe = 2m/sec.

Width of road (2 lane) = 7.5m

Top width of embankment = 1.5 : 1

Bed level of stream = 100.00

Top of embankment = 103.00

Loading IRC Class A-A wheeled vehicle.

Given Data:

$Q = 1.57\ m^3/sec$; $V = 2m/sec$; Top Width of embankment = 1.5 : 1

a) Diameter of pipe culvert

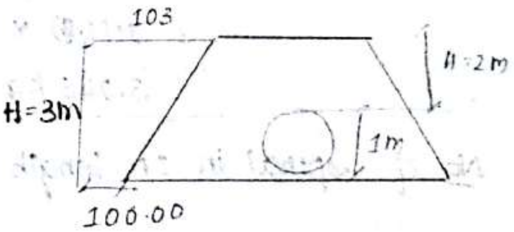
Discharge = Area × Velocity Area = $\frac{\pi d^2}{4}$

Q 1.57 = A × 2 $0.785 = \frac{\pi (d)^2}{4}$

Area, A = 0.785 $d = 0.996m \simeq 1m$

Adopting RCC heavy duty non pressure pipe for carrying heavy road traffic, referring to IS 458, 1971 for internal dia 1m the external ϕ = 1.2m

b) Load Due to Earthfill



But we are considering height of the embankment over the pipe = 2m

From table 5.3 for d = 1000mm & h = 2m, Load due to earthfill = 59.5 KN/m

Load due to IRC class AA Wheeled load

Load due to IRC class AA wheel- load vehicle = 62.5 KN

For IRC class A-A wheel- load vehicle ⇒ I = 1.5 ; the value of C_s is taken by table 5.4,

$C_s = 0.032$

Loading on pipe = 4 C_s I P

= 4 × 0.032 × 1.5 × 62.5

Loading on pipe = 12 KN/m

c) Check for strength factor

$$\frac{\text{3 edge bearing strength}}{1.5} = \frac{\text{load on pipe due to earthfill (KN/m)}}{\text{strength factor}} + \frac{\text{load on pipe due to IRC Class AA wheeled load}}{1.5}$$

Refferring to IS 458, 1971 code, 3 edge bearing strength for the selected pipe, 100mm ϕ is 1111 KN/m

$$\frac{111}{1.5} = \frac{59.5}{S.F} + \frac{12}{1.5}$$

Strength factor = 0.90

The strength factor for the 1st class bending is 0.3 & for concrete craddel bedding. out of these two any one can be provided.

d] Design of reinforcement

From IS 485 1971 the min reinforcement of steel wire with ~~a min steel reinforcement~~ a permissible stress of $140 N/mm^2$ is 44 kg/m. The

The min longitudinal reinforcement of mild with a permissi. stress of $126.5 mm^2$ is 5.80 kg/m

i) Design of spiral reinforcement.

Assume 12 mm ϕ bars @ 60mm c/c spacing as spiral reinforcement

Wt of 1 spiral = Circumference * Area of steel * Density of steel

$= \pi D * \pi d^2/4 * 7800$

$= \pi(1.1) * \pi (0.012)^2/4 * 7800$

$D = \frac{1+1.2}{2} = 1.1m$

$d = 0.012m$

$\boxed{= 3.046\ kg}$

No of spiral in 1m length $= \frac{1000}{spacing}$

$= \frac{1000}{60}$

$\boxed{= 16.67}$

Weight of spiral reinforcement for 1m length = Wt of 1 spiral * no of s.

$= 3.046 * 16.67$

$\boxed{= 50.776 N > 44 kg \text{ (min)}}$

Hence it is safe. Provide 12mm ϕ bars @ 60 mm c/c spacing as spiral reinf.

ii] Design of longitudinal reinforcement

The min longitud^n reinforcement is 5.80 kg/m

Providing 6mm ϕ mild steel bars as longitudinal reinforcement

Wt of each bars = Area * 1m * Density of steel

$= \pi d^2/4 * 1m * 7800$

$= \frac{\pi (0.006)^2}{4} * 1 * 7800$

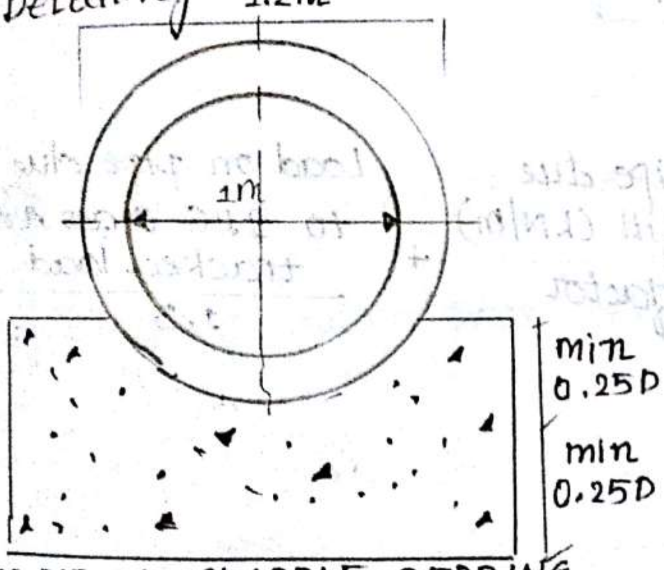
$\boxed{= 0.2205 kg/m}$

No of bars required $= \dfrac{5.80}{600.22} = 26.36$

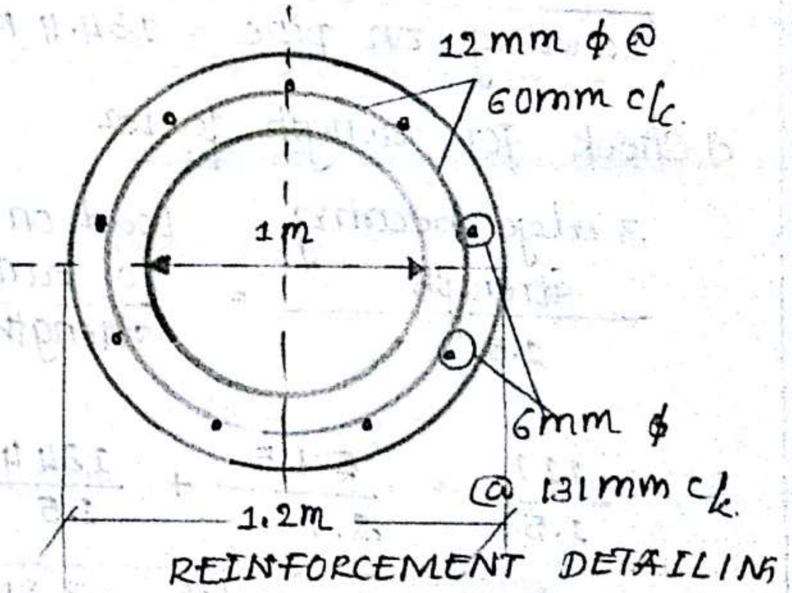
Spacing of bars $= \dfrac{\pi D}{\text{No of bars}} = \dfrac{\pi(1.1)}{26.36} = 131\ mm$

Provide 6mm dia bars @ spacing 131mm c/c as longitudinal reinforcement.

7) Detailing



CONCRETE CRADDLE BEDDING



REINFORCEMENT DETAILING

MODULE - 5 01/11/2018

DESIGN OF PIERS AND ABUTMENTS

PIERS :

Forces to be considered for the design of piers

i) Dead load of superstructure & pier
ii) Live load of vehicle.
iii) Effect of eccentric liveloads
iv) Impact effect for different classes of loads
v) Effect of bouancy of submerged part of pier
vi) Pier.
vii) Forces due to water current
viii) Forces due to wave action
viii) Longitudinal forces due to tractive effort of vehicle
ix) Longitudinal forces due to braking of vehicles
xi) Earthquake forces

Design standards of piers

1) Height

Top level of the pier is fixed at 1 - 1.5m above the high flood level, depending upon the depth of water on the upstream side.

2) Pier Width

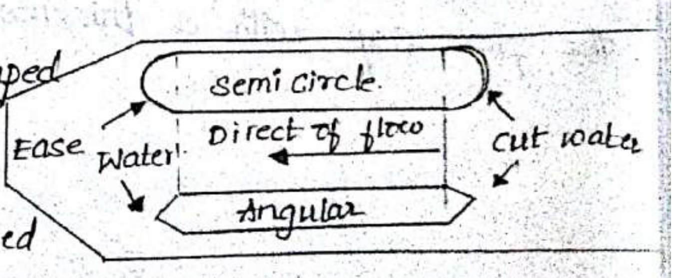
Top width of the pier should be sufficient to accamodate the 2 bearings. It is usually kept at a min of 600mm

3) Pier Batter

Generaly the sides are provided with a batter of 1 in 12 to 1 in 24, Short piers have vertical sides.

Cut and Ease Water

The pier ends are shaped for stream lining the passage of water. Normally cut & ease water are either shaped circular of triangular as shown in figure



ABUTMENTS

Loads to be considered for the design of

i) Dead load due to super str
ii) Live load on super str
iii) Self weight of abutments
iv) Longitudinal forces due to tractive effect & braking
v) Forces due to temperature variation
vi) Earth pressure due to backfill

Design Standards

1) Height

The height of the abutment is kept equal to the pier

2) Abutment Batter

The water face is kept vertical or a small batter of 1 in 24 to 1 in 12. The earth face is provided with a batter of 1 in 3 to 1 in 6 or it may be stepped down

3) Abutment Width

The top width should provide enough space for bearings & bottom width is taken as 0.4 to 0.5 times the height of the abutments

4) Length of Abutment

The Length of abutments must be atleast equal to width of the bridge

5) Abutment Cap

The bed block over the abutment is similar to the pier cap with a thickness of 450 - 600 mm

The portion of the bridge structure below the level of the bearing and above the foundation is generally referred to as substructure.

The bridge substructure consists of the following elements:

- Piers
- Abutments
- Wing walls
- Approaches

Piers are the intermediate supports for super structure. Abutments are the end supports for the superstructure and wing walls are the walls constructed on both sides of the abutments to retain the embankment of approaches and protect them from the wave action of water.

Piers and abutments are generally constructed with masonry, mass concrete or reinforced concrete.

Approaches are provided to connect the bridge proper to the road or railway track on either ends of a bridge.

PIERS

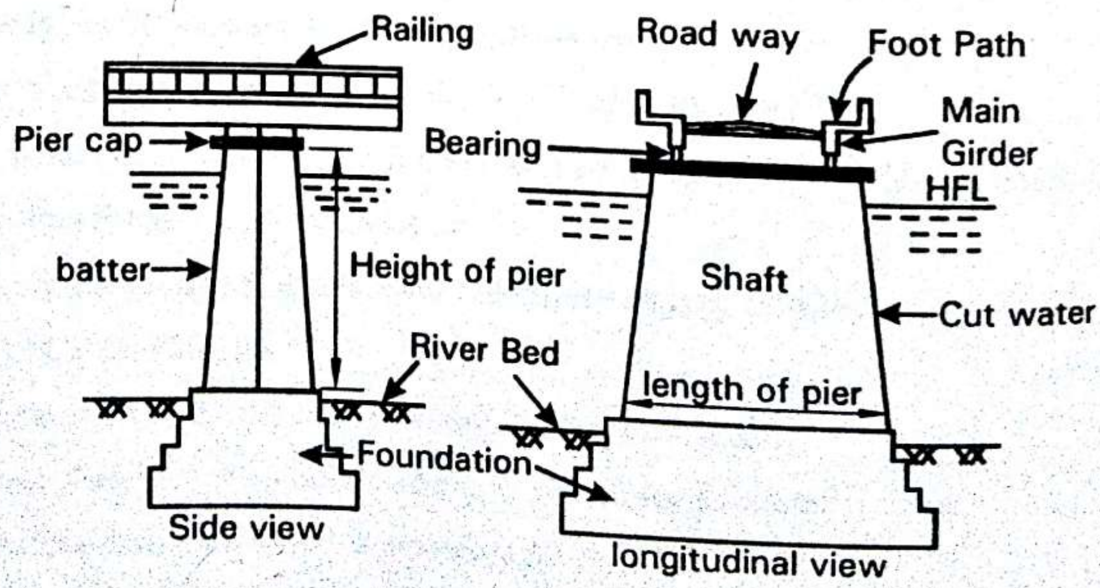


Fig. 11.1 : Details of pier

Piers are the intermediate supports of the superstructure in case of multi-span bridge. A pier essentially consists of two parts- A column or shaft and the foundation. It is sometimes provided with projections, called cut water and ease water for easy passage of water.

The function of a pier is

1. To divide the total length of bridge into suitable spans with minimum obstruction to the stream or river.
2. To transmit the load from the bridge superstructure to the sub soil lying underneath.

The salient features of pier are as follows:

vi) It should be quite stable against lateral and longitudinal thrusts of water.

vii) It should involve less maintenance cost.

Types of piers

The type of pier to be adopted depend on the type of super-structure, sub-soil conditions and the construction procedure of the bridge.

Bridge piers can be broadly classified into the following two types:

1. Solid piers
2. Open piers

1. Solid piers

The piers which have a solid section in elevation, plan and end views are known as solid piers. These piers can be made of brick or stone masonry, mass concrete or R.C.C. These piers present less friction to flow of stream or river water. Solid piers are suitable in rapid running water or in waters subjected to ice or debris. Such type of construction of pies very popular in the bridge construction due to following reasons:

➢ It can be used for any type of superstructure of the bridge.

➢ It provides excellent resistance to the actions of floating bodies.

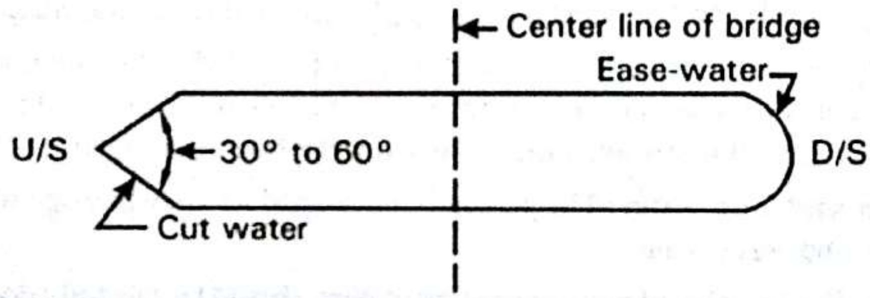


Fig. 11.2 : Plan of solid pier

The ends of solid piers may be rectangular. But they may be given any suitable shape to make the entry and passage of water easy and smooth. The end or nose of pier on upstream side is known as the cut-water and that on downstream side is known as the ease-water as shown in fig. 11.2

The cut-waters need not be very long and they should be carried down to the base. The ease-waters are usually semi-circular or they may consist of two parabolic arcs.

The provision of cut-water and ease-water in the structure of solid piers results in the following advantages:

➢ The eddying effect of current of water is minimized.

➢ The effect of horizontal pressure due to the current of water is reduced.

➢ The natural movement of the water is disturbed to the minimum possible extent.

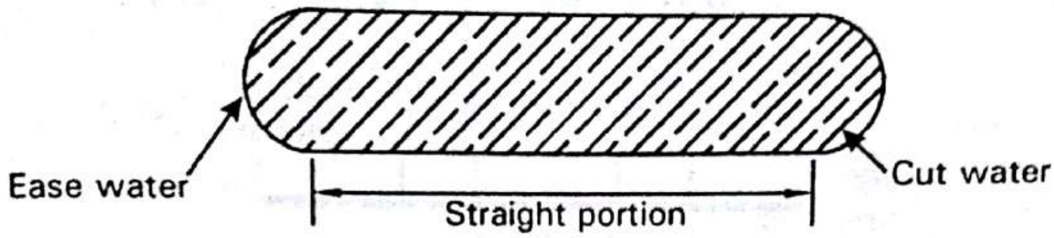
Solid piers are further classified into the following two types :

i) Solid masonry piers

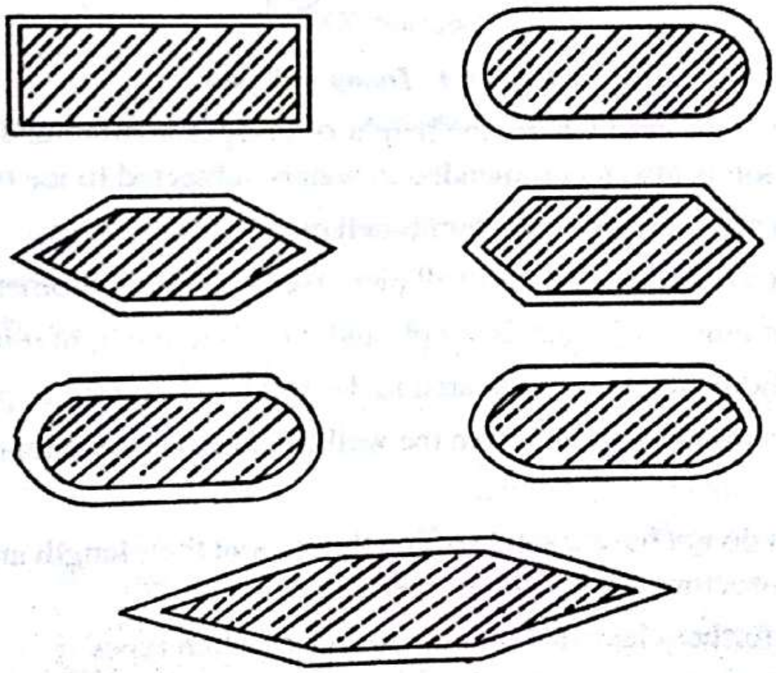
ii) Solid R.C.C. piers

i) Solid masonry piers

The sold piers constructed of brick masonry, stone masonry, plain cement concrete, etc. are known as solid masonry piers. Plain cement concrete may be used cast-in-situ or in the form of precast blocks for constructing these piers. The modern trend for constructing solid masonry pier is to use stone masonry for the exposed portions and to fill the interior with mass concrete. This method saves expenses on shuttering and provides better appearance. The stone layers must be properly bonded with the interior by means of bond stones. These piers are used for bridges of moderate heights.



a) Plan of solid masonry pier



b) Shapes of Bridge-piers

Fig. 11.3 : Shapes of Bridge-piers

ii) Solid R.C.C. piers

The solid piers constructed of reinforced cement concrete are called solid R.C.C. piers.

These piers are generally rectangular in cross-section with usual cut and ease waters.

A special type of solid R.C.C. pier is Dumb-well pier. It consists of two end columns connected together by means of a thin reinforced concrete web, provided all along their height, in a direction transverse to the direction of the bridge. Such a type of concrete web, provided in between the columns of a pier, is known as diaphragm wall.

But if this type of piers in the full height are not sufficiently strong to offer resistance to the impact of floating bodies. The wall type pier is taken upto the H.F.L. and above this level twin piers are constructed.

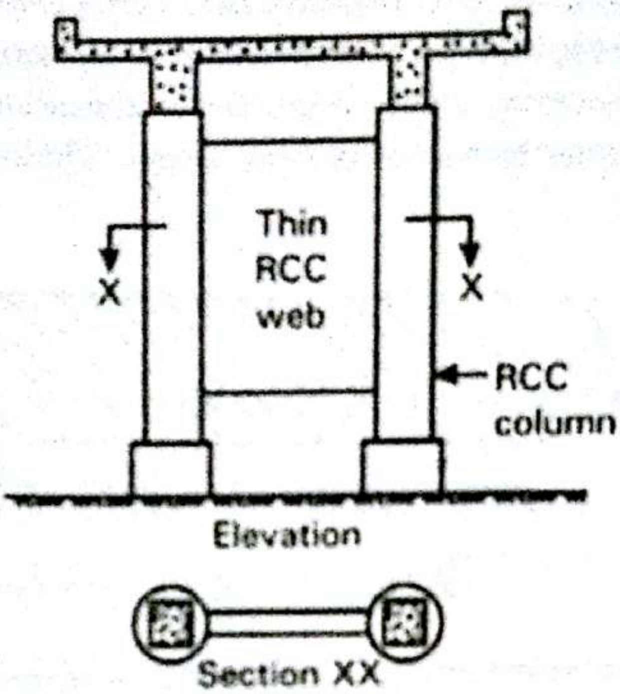


Fig. 11.4 : Dump well pier

Solid R C.C. piers are used where, the height of bridges is more and the current of water is swift. Their construction is also recommended in waters subjected to ice or debris.

Following are the advantages of the dumb-bell piers:

- As compared to its mass, a dumb-bell pier gives maximum moment of inertia.
- The design of dumb-bell piers is simple and it leads to the light reinforcement.
- They are light in weight as compared to the solid mass concrete piers.
- They are very much suitable when the well foundations are adopted.

2. Open piers

The piers which do not have a solid section throughout their length and allow free passage of water through their structure are known as open piers.

Open piers are further classified into the following main types :

i) Cylindrical piers
ii) Column bents
iii) pile bents
iv) Trestle piers or trestle bents

i) Cylindrical piers

The open piers constructed of mild steel or cast iron cylinders filled with concrete are known as cylindrical piers. The cylinders are filled with concrete and the main girders of the bridge superstructure are supported on their top ends. The cylinders are then braced together by steel frame work for additional stability as shown in fig. 11.5. Cylindrical piers are used for bridges of moderate heights.

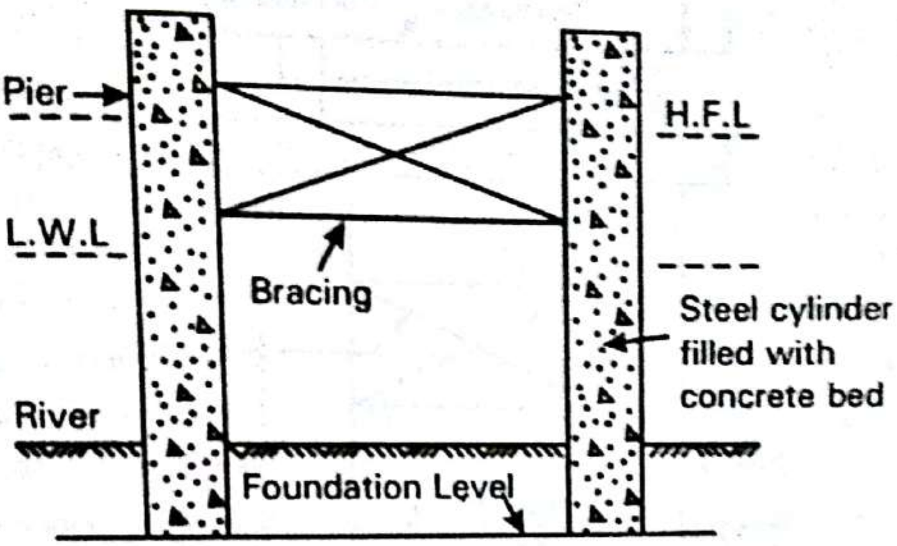


Fig. 11.5 : Cylindrical pier

ii) Column bents

The open piers constructed of two or more rolled steel or R.C.C. columns, built in a row and connected together by beams and braces or short diaphragms, are known as column bents or multiple bents. The columns of such piers may be uniform in section or of varying section The main girders of the bridge super-structure are laid over the beams connecting the top ends of column as shown in fig. 11.6. Column bents are used for high bridges as over-pass purpose.

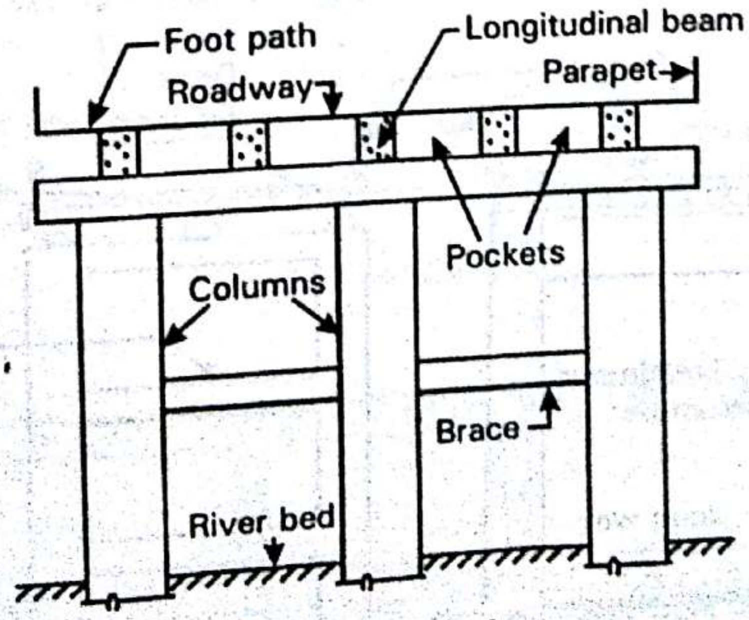


Fig. 11.6 : Column bents

iii) Pile bents : The pile bent is used for low piers over unstable ground or muddy ground. The open piers constructed of a number of R.C.C or steel piles driven deep into the ground, provided with a cap at their top to support the main girders are known as pile bents.

The piles of these piers are laterally connected by R.C.C. or steel braces to make them more stable.

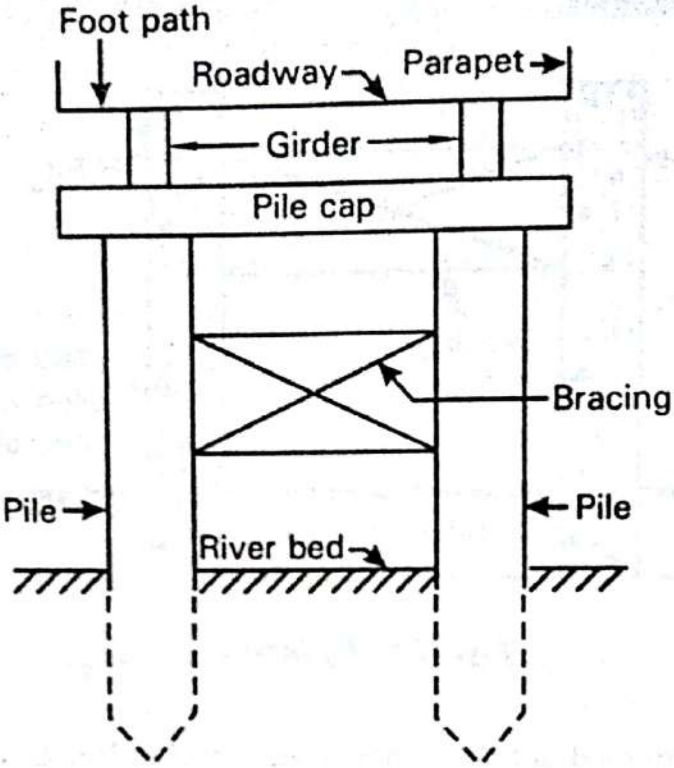


Fig. 11.7 : Pile bents

iv) Trestle bents : The open piers constructed of frame work of R.C.C. steel or timber members are known as trestle bents or trestle piers. Each trestle consists of two or more vertical posts, braced horizontally and diagonally, to support the bridge superstructure. The trestle bents are useful for constructing piers for a bridge along a viaduct or in case of flyovers and elevated roads.

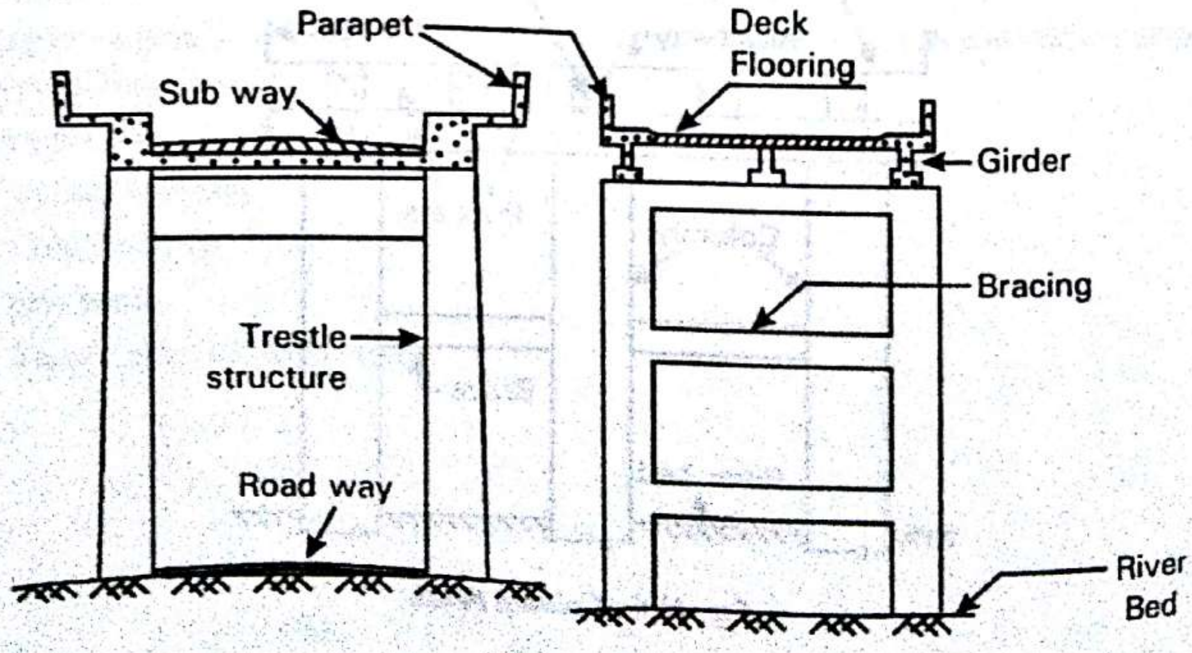


Fig. 11.8 : Testle-piers

R.C.C. and steel trestles are used in case of permanent bridges, whereas timber trestles are used for temporary bridges. They are less stable than the solid piers.

Apart from the different types of piers discussed above, there are some special types of piers such as :

a) Abutment piers

b) Cellular piers

c) Framed Piers

a) Abutment piers

In case of multi-span arch bridges, every third or forth pier is designed as an abutment to receive the thrust from either side. Such a pier which is made thicker in section than the adjacent piers is known as abutment pier. These piers are splayed at their top to receive the arch rings from either side.

The following are the functions of providing abutment piers in a multi-span arch bridge

- To facilitate the construction of an arch bridge in steps, thus expenditure in centring is much saved.
- To provide safety to the remaining part of the bridge in case damage occurs to any portion of the bridge due to flood, etc.

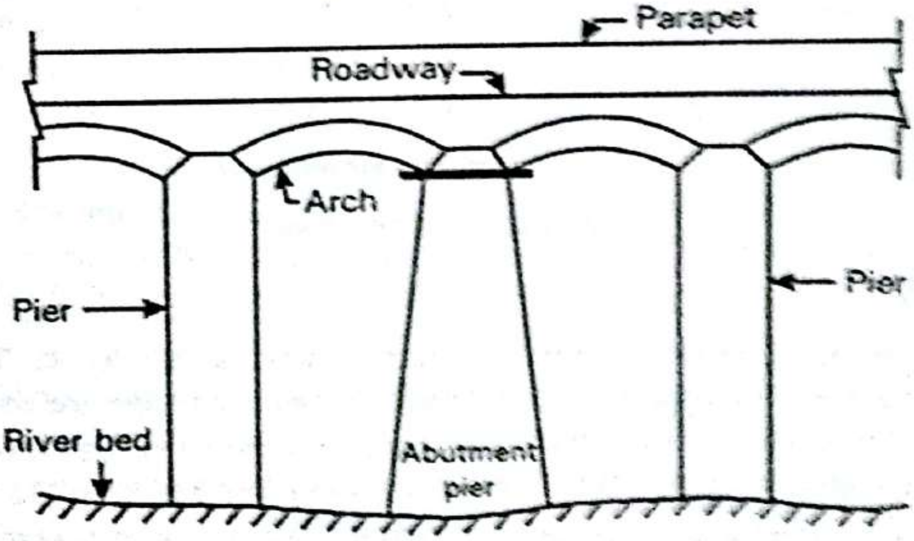


Fig. 11.9 : Abutment pier

b) Cellular piers

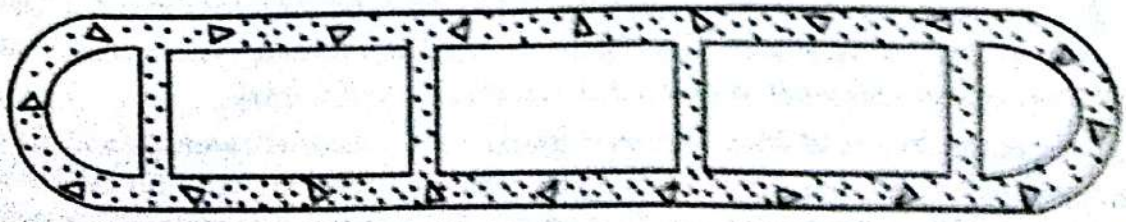


Fig. 11.10 : Plan of cellular pier

These piers consists of two concentric R.C.C. shells connected by radial ribs and horizontal bands at suitable intervals. The intermediate space between the shells is filled with some cheap filler material like sand. This will result in saving of cement concrete which is a costly material. But they involve difficult shuttering and additional labour in placing reinforcement. These piers provide more top width.

R.C.C. or prestressed concrete cellular piers are suitable for major bridges where both the span and the depth are considerable.

c) Framed Piers

This type of piers result in reduced effective span lengths for girders on either side of the centre line of the pier leading to economy in the cost of superstructure. This type of piers if used in rivers subjected to sudden floods near hills and forests would be subjected to floating debris.

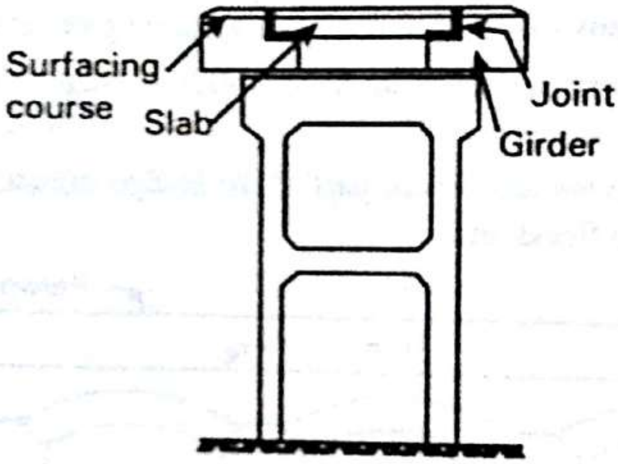


Fig. 11.11 : Framed pier

ABUTMENTS

The end supports of a bridge superstructure are known as abutments. They retain earth on their back which serves as an approach to the bridge. For a river bridge, the abutment also protects the embankment from scour of the stream. Generally all types of bridges have abutments except cause way without vent or a pipe drain. It serves both as a pier and retaining wall.

They are built either with brick masonry, stone masonry, plain concrete, precast concrete blocks or reinforced concrete. The top surface of the abutment is made flat for girder bridges or semi circular arch bridges, but provided with skewbacks if the bridge arches are segmental or Elliptical.

The top surface of the abutment is kept flat when the superstructure of the bridge consists of girders, trusses or semi-circular arch. But if the superstructure consists of segmental or other types of arches the top of the abutment is given a slope to receive the arch.

It is important in abutment construction to place the fill material carefully and to arrange for its proper drainage. A good drainage system may be secured by placing rock fill immediately behind the abutment and proper drain pipes at the bottom. Weep holes are provided at different levels through the body of the abutment to drain of the retained earth.

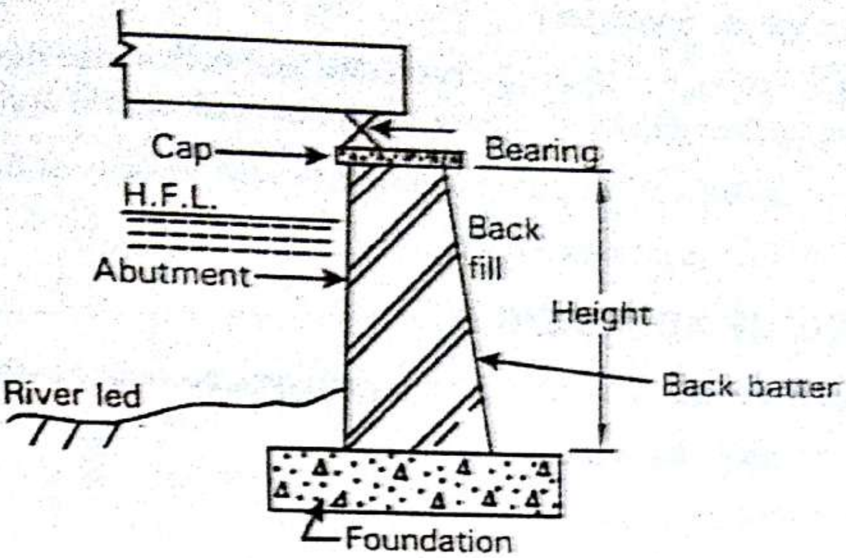


Fig. 11.12 : Parts of an Abutment

The salient features of abutments are as follows:

- **Height :** The height of an abutment is fixed up by the difference between the bed level of river banks and the formation level of the road or railway line.
- **Abutment batter :** The water face of the abutment is usually kept vertical but batter of 1 in 12 to 1 in 24 can be provided as for piers. The face retaining earth is given a batter of 1 in 6 or may be stepped down.
- **Abutment width :** The top width of the abutment should be sufficient to accommodate the ends of the bridge superstructure.
- **Length of abutment :** The length of an abutment is represented by the overall width of the bridge including footpaths, if any. The top width of an abutment depends on the span of the bridge. But it should not be less than 500 mm in any case. It also depends on the use of bridge.
- **Abutment cap :** The design of abutment cap is similar to that of the pier cap.

Functions of abutments

Following are the functions of abutments:

1. To provide final formation level to the bridge superstructure.
2. To retain the approach road embankment.
3. To transmit the load from the super structure of the bridge to the foundation.
4. To connect the approach road to the bridge deck.

Requirements of a good abutments

i) It should be easily and cheaply constructed.

ii) It should be constructed of durable material.

iii) It should have sufficient bearing area at its top to receive bearing which supports the bridge girder.

iv) It should have pleasing appearance.

v) It should be strong enough to take and transmit the load from bridge superstructure to the sub-soil lying underneath and to retain the pressure of embankment of approaches.

vi) It should be quite stable against side erosion due to more velocity of flow of water.

vii) It should involve less maintenance cost.

CLASSIFICATION OF ABUTMENTS

Bridge abutments are classified into the following types, according to their layout in plan:

1. Abutments without wing walls,
2. Abutments with wing walls.

1. Abutments without wing walls

These abutments are suitable in the case when river banks at bridge site are sufficiently firm, the velocity of flow of river water is less and there is no danger of side erosion.But in floods there is possibility that flow of water may wash the earth-fill around the abutments and damage it.

Abutments without wing walls can be further classified into the following types :

i) Straight abutments

ii) Tee abutments

iii) Hollow or arch abutments

iv) Burried abutments

i) Straight abutments

The abutments usually rectangular in plan are known as straight abutments. This type of abutment is suitable when the banks are firm and height of the approach embankment is small. These abutments are also used when velocity of flow of water is less and there is no danger of side erosion.

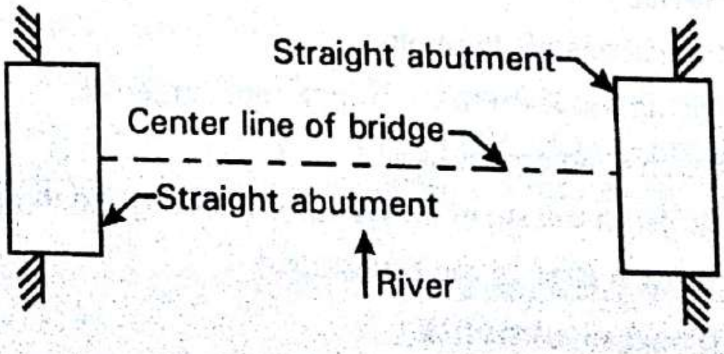


Fig. 11.13 : Straight abutment

ii) Tee-abutments

The abutments of the T-shape in plan are known as Tee abutments. This type of abutment is an improved form of straight abutment. Its projected leg provides more stability to the abutment. It also provides some support to a poor earthen approach of the bridge.

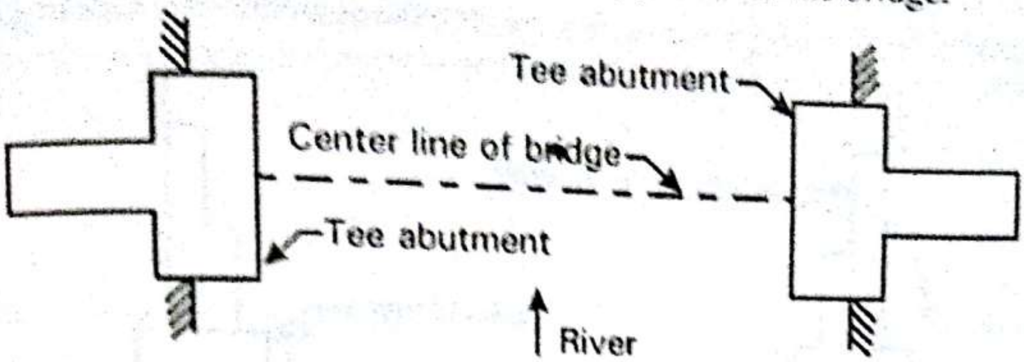


Fig. 11.14 : Tee abutment

iii) Hollow or arched abutments

The abutments curved in plan are known as hollow or arched abutments. These abutments are suitable at rail and road crossings on land. i.e., in the case of an under bridge or grade separation. Sometimes these abutments are provided with curved wing-walls.

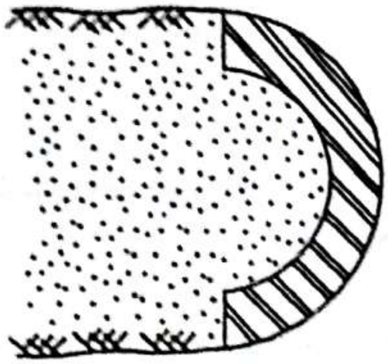


Fig. 11.14 : Hollow abutment

iv) Burried abutments

At certain places when the channels are narrow, the abutments are constructed inside the banks and do not cause any obstruction to the flow of water. Such abutments are called buried abutments. In this case the earth pressure in the back of abutment is balanced by the fill in the front. Since earth is filled on both the sides of these abutments, the resultant earth pressure on the abutment is low and hence cheaper abutments will serve the purpose.

2. Abutments with wing walls

These abutments are suitable in case when height of the approach embankment is more, the velocity of flow of river water is high and there is danger of side erosion.

Abutments with wing walls are further classified into the following types :

i) Abutments with straight wing walls

ii) Abutments with splayed wing walls

iii) Abutments with return wing walls

i) Abutments with straight wing walls

Fig. 11.15 shows an abutment with straight wing walls. These abutments are so called as these are provided with straight wing walls. These abutment is unsuitable for bridge where flowing water is likely to damage the embankment behind the wing wall. This type of abutment is suitable for a bridge crossing roadway i.e., in the case of an under bridge or grade separation.

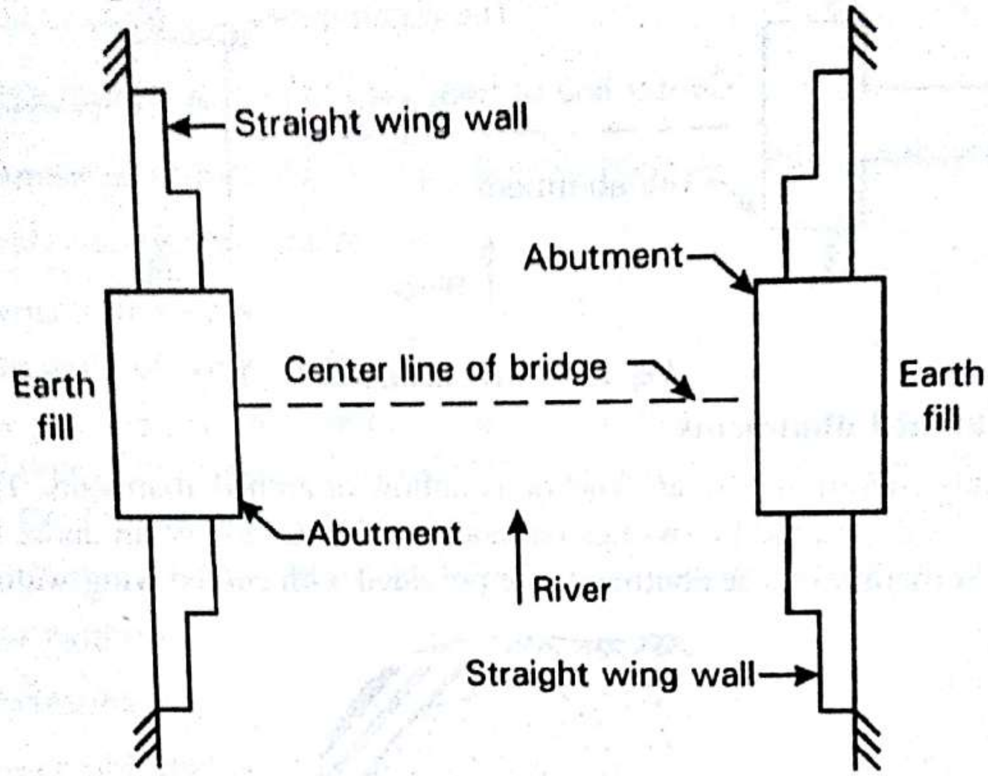


Fig. 11.15 : Abutment with straight wing wall

ii) Abutments with splayed wing walls

Fig. 11.16 shows an abutment with splayed wing wall. These abutments are provided with splayed wing walls. This type of abutment is suitable for the bridge with waterway as it permits smooth entry and exit of water under the bridge. These abutments are used, where the approaches are higher than the banks in order to prevent damage to the embankment of approaches and also used for bridges on streams or rivers.

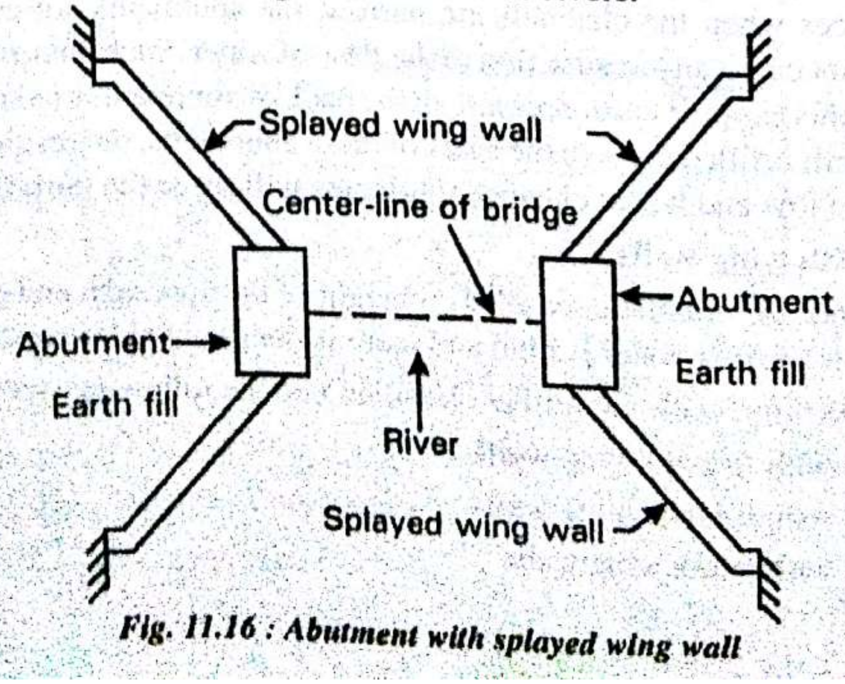


Fig. 11.16 : Abutment with splayed wing wall

(iii) Abutments with return wing walls

Fig. 11.17(a) shows an abutment with return wing wall. The abutments with return wing walls are known as U-abutments or box abutment, since it forms a box for filling the earth for making the embankment. These abutments are suitable in the case when the height of the embankment is more and there is danger of side erosion of the embankment of approaches. In this type abutment is extended at right angles on both the ends to some distance to protect the earth work.

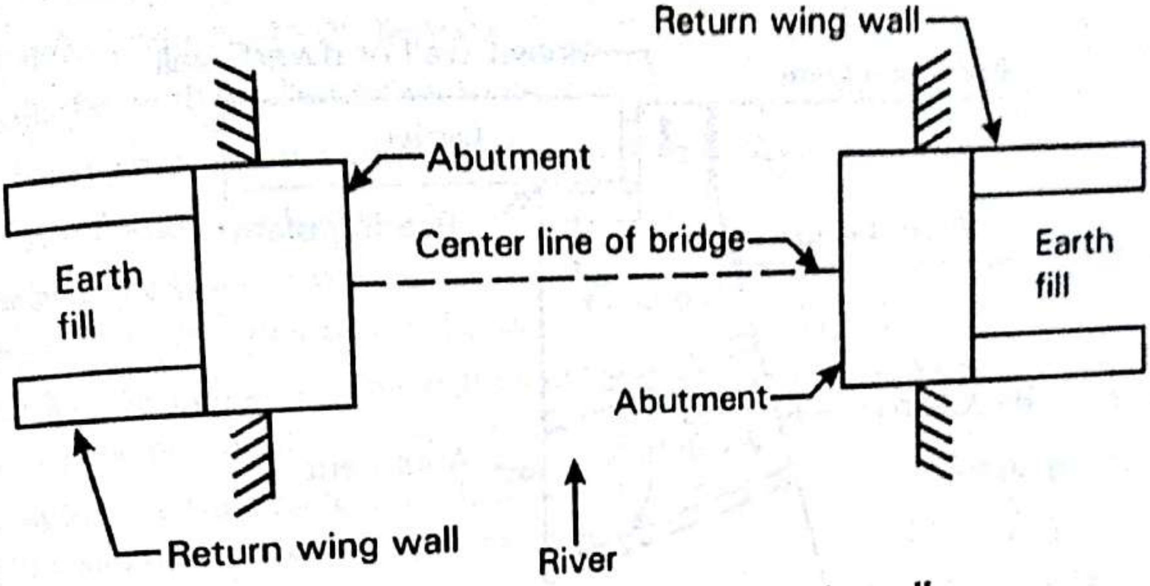


Fig. 11.17(a) : Abutment with return wing wall

It is however not suitable for rivers or streams subjected to heavy floods as considerable portion of embankment outside the wing walls remains unprotected from the scouring action of water.

This type of abutments wing walls can be cast monolithically with the abutment back and cantilevered both vertically and horizontally as shown in fig. 11.17(b).

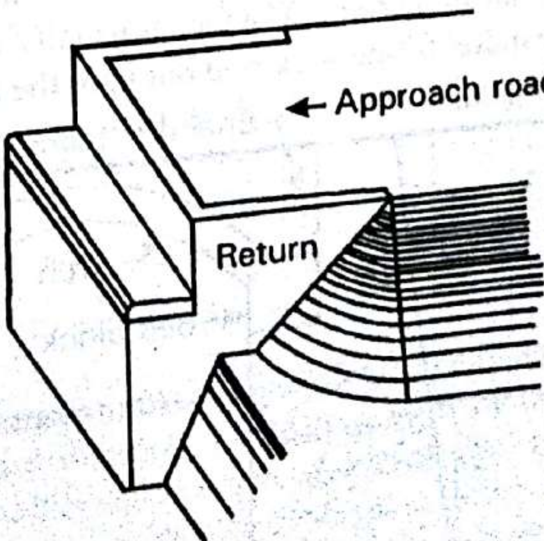


Fig. 11.17(b) : U - Abutment

BRIDGE BEARINGS

The devices fixed on abutments and piers to allow for free expansion, contraction and deflection of the bridge superstructure are known as bridge bearings. The loads coming from the superstructure of a bridge are transmitted to the substructure through the bearings.

Importance of bearings

It is observed that faulty design or improper working of the bearings is the main cause of failure of many bridges that have collapsed. The design of bearing to be adopted for a particular bridge will mainly depend on the type of supports, length of the span and the type of superstructure.

The bearings form an important component of a bridge and hence, extreme care and skill should be exercised in its design, execution and maintenance.

Bearings are provided in bridges to transmit the load from the superstructure to the substructure in such a manner that the bearing stresses induced in the substructure are within permissible limits and also to allow for certain movements of the superstructure.

In all steel bridges, the girders are liable to expand and contract considerably due to variations in temperature. If one end of the simply supported girder is not kept free to move in the longitudinal direction, internal stresses will develop in the bridge superstructure

In R.C.C. and prestressed concrete bridges, where rich mixes are adopted shrinkage starts from the moment the concrete is poured and may continue for two or three months or more. In order to ensure that no internal stresses result in the bridge super-structure due to its expansion or contraction, end of the super-structure must rest on bearings which will allow freely such longitudinal movement to take place.

In case of major bridges, the cost of bearings accounts for as much as 10 to 15% of the total cost of the bridge.

Functions of bridge bearings

Bearings are provided for the following purposes:

- To distribute the load received through the ends of bridge girders over large area on the top of abutments or piers.
- To take up the vertical movement due to sinking of the support.
- To transfer horizontal forces developed due to application of brakes to the vehicles, etc.
- To provide longitudinal movement due to temperature changes.
- To provide angular movement due to deflection.

The requirements of an ideal bridge bearing

Following are the requirements of an ideal bearing:

- It should be compact in size and easy to install.
- It should be offer excellent resistance to weathering.
- It should be capable to distribute the superimposed load uniformly on sub-structure and provide greater stability to the structure.
- Its maintenance cost should be minimum.
- Its initial cost should not be very high.
- It should be capable to accommodate maximum expected deck movement and rotation with least possible resisting force.

Types of bearings

Free and Fixed bearings

A free bearing is free to slide or move or roll and it thus allows longitudinal movement of the girder. A fixed bearing is fixed in position, but it rotates according to the deflection of the structure which is being supported by it. Thus a fixed bearing allows free angular movement and it does not permit any longitudinal movement of the girder. The design of fixed bearing depends on the length of span, type of supports and type of superstructure.

Different designs are possible for each of these two categories. The particular design of bearing to be adopted for any given bridge depends upon the type of superstructure, type of supports and also on the length of the span.

Following are the commonly adopted bearings for the bridges:

1. Cement mortar pad
2. Expansion bearing
3. Knuckle bearing
4. Rocker and roller bearing
5. Rocker bearing
6. Rubber bearing
7. Neoprene bearing pads
8. Sliding bearing
9. Sole plate on curved bed plate

1. Cement mortar pad

This is the cheapest type of fixed bearing which is adopted for road girder bridges of small spans. It consists of a 30 mm thick cement grout pad of proportion 1:1. The dowel bars in sufficient number and of diameter about 25 mm are provided to connect the superstructure to the bed block. The dowel bars are designed to take up the longitudinal forces in shear.

2. Expansion bearing

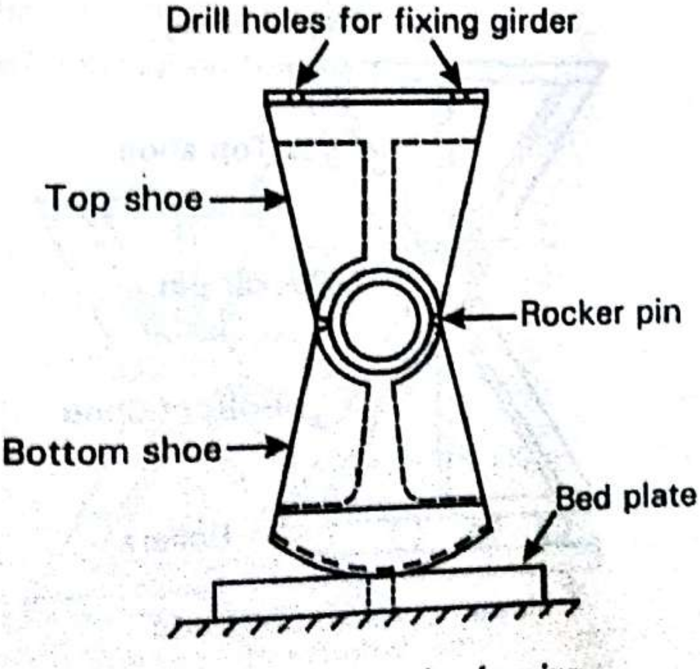


Fig. 11.22 : Expansion bearing

In case of expansion bearing, the bottom shoe is given a circular shape. The centre of circular surface coincides with the centre of rocker pin. The bottom shoe rests on the bed plate. The top shoe is provided with drill holes for fixing of the girder. Fig. 11.22 shows an expansion bearing. It allows free angular as well as longitudinal movement of the girder and it is useful for girders having small spans.

3. Knuckle bearing

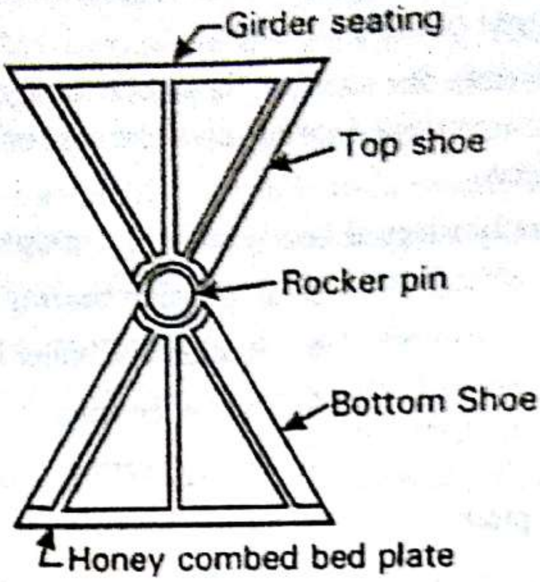


Fig. 11.23 : Knuckle bearing

Fig. 11.23 shows a typical knuckle bearing. The bearing has a semi-circular concave surface of the top shoe. The bottom shoe has a semi-circular surface of convex type. It has the same radius as that of the top shoe. The bearing allows only angular movement of the girder fixed to the top-shoe. This type of bearing is suitable for long spans (over 20 m) of girder bridges.

4. Rocker and roller bearing

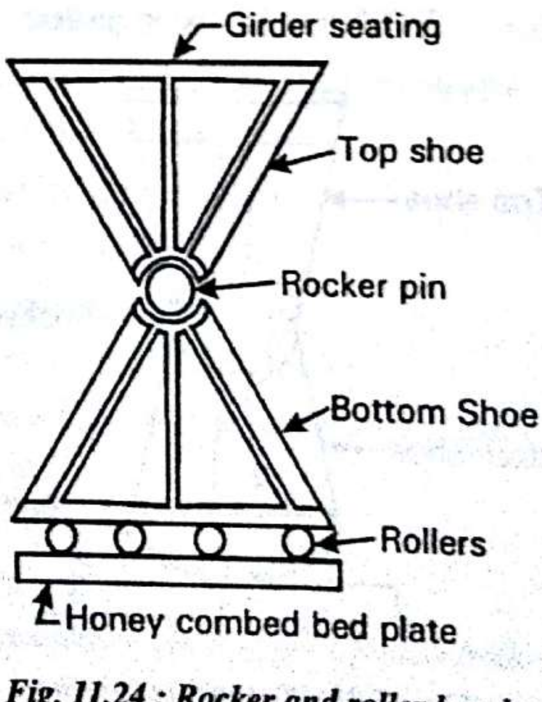


Fig. 11.24 : Rocker and roller bearing

It allows for free longitudinal as well as angular movements. It is used for heavy loads. In case of rocker and roller bearing, a rocker pin is provided between the top shoe and the bottom shoe and it is so arranged that the bottom shoe rests on rollers as shown in fig. 11.24 The rollers are cylindrical in shape and they are free to roll on steel bed plate. A rocker and roller bearing is therefore a free bearing which does not slide, but which rolls as well as rocks over a smooth bed plate.

Generally, for spans over 20 m, a rocker bearing is provided on one end and a rocker and roller bearing on the other end of the bridge girder.

5. Rocker bearing

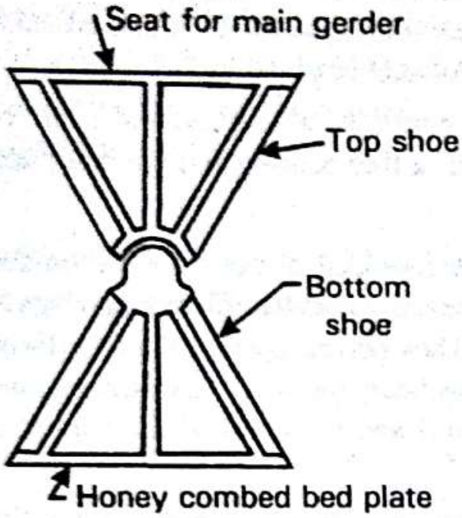


Fig. 11.25 : Rocker - Bearing

A fixed bearing which rocks about a pin like a hinge is known as a rocker bearing. In case of a rocker bearing, a rocker pin is provided between the top shoe and the bottom shoe as shown in fig. 11.25. A rocker bearing allows only free angular movement of the main girder and at the same time, it transmits the pressure centrally to the bed plate.

For spans greater than 20 m or so, a rocker bearing is provided at one end and at the other end, a rocker and roller bearing is provided.

6. Rocker bearings with curved bottom base

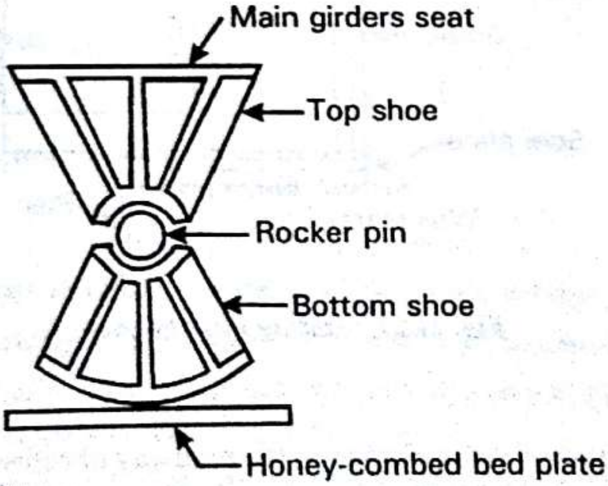


Fig. 11.26 : Rocker bearing with curved plate

This is a type of rocker bearing but in this case, the bottom shoe is provided with a curved bottom which offers minimum resistance to the longitudinal movement of the bridge girder. Thus, this type of bearing allows for the deflection and expansion of the bridge girder with a reduced horizontal force. This type of bearing is suitable for spans 12 to 20 m.

7. Rubber bearing

A rubber bearing consists of layers of rubber plates. The steel plates or wire-meshes are introduced between the successive layers and a minimum cover of about 5 mm of rubber is maintained along the edges. The desired thickness of rubber bearing can be obtained by selecting a proper number of rubber plates.

If a thick rubber bearing is provided at one end and a thin rubber bearing is provided at the other end, the former acts as a free bearing and the latter acts as a fixed bearing.

8. Neoprene bearing pads

Neoprene bearing pads are moulded or cut from a moulded sheet of high-grade synthetic rubber compounds. They are economically efficient bearings for pre-cast, pre-stressed concrete or steel beams in bridges. They permit a smooth and uniform transfer of load from the beam to the substructure and allow beam rotation at the bearing due to deflection of the beam under load. They also allow lateral and longitudinal movement of the beam caused by thermal forces.

Neoprene Pads have no movable parts and thermal expansion and contraction are absorbed to give and take in shear. Correctly designed and suitably compounded neoprene bearings can be confidently expected to function efficiently for at least a hundred years.

9. Sliding-bearing

In this type of bearing the sole plate is fixed to the bed plate and bolted to it. Slotted types of holes are provided in the sole plate, which allow longitudinal movements of the girder. The bed plate is fixed by anchor bolts to the masonry.

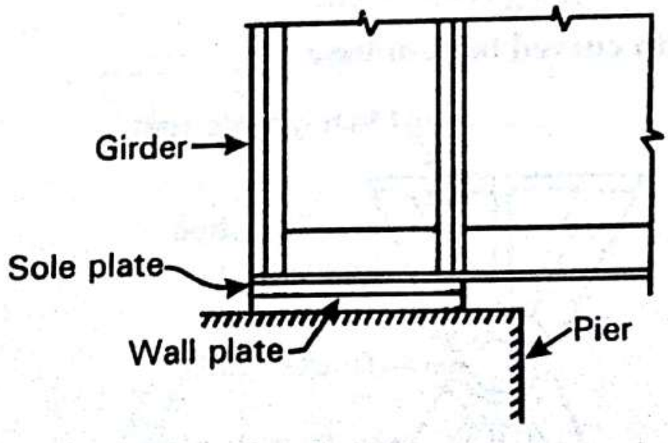


Fig. 11.27 : Sliding plate bearing

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What is expansion joints in bridge and explain types of E J

Types of Expansion Joints.

Expansion joints are designed to take up longitudinal & transverse movements of a bridge caused by thermal expansion, contraction & certain loading condition. The type of joint selected for a deck is generally dependent on the joint type & magnitude of movements that a joint has to accomadate. An expansion joint in a bridge has to satisfy the following requirements

* It should allow the movement of the bridge. so that the stresses caused by temperature, shrinkage & loading are relieved.
* It should not allow percolation of water.
* It should be durable & structurally strong.
* It should be accessible for easy inspection & maintaince.

Deck joints

These are 2 types
① Open & ② Closed

An open joints is nothing more than an opening b/w the concrete deck & an adjacent structural element (deck/deck, deck/abutment)

In a closed joint. the gap b/w the adjacent elements of the deck is covered by a sealant. A mechanical system is provided to take up the movement of the bridge. Open joints are prone to leakage & detoriation & can handle small longitudinal movements only.

Closed joints

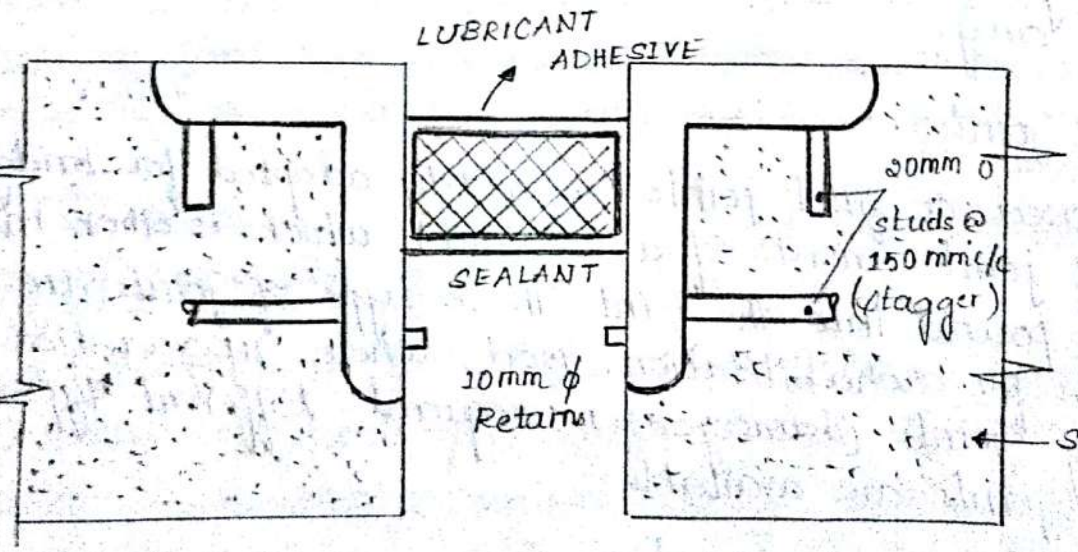
Closed or fillet joints are widely adopted for bridges. This type of joint consists of a sealant, which is either inserted or hot poured into the joint. These type of joints are also suitable for rehabilitation. Work, where upgradation of existing joints (damages) is required. Different types of closed joints are available.

Compressive seal joint.

This joint is made by squeezing a sealant material into an open joint. An adhesive lubricant is also provided along with the sealant. The common material is used in the extrude neoprone. This material takes up the movement of the bridge by getting itself compressed for added dura-bility, compression seals are combined with steel angles @ the deck slab edge to form an armoured joint. This type of joints is used for decks, which are expected to sustain movements ranging from 12mm to 60mm. However, a common problem with the type of joints is the loading of bond b/w the seal & the concrete surface. This loosening inturn gives rise to loss of compression.

Strip Seal Joint.

This type of joint concrete of an elastomeric material, which is placed b/w the dual rails that are anchored to the face placed of the joint opening. The most commonly used material is neoprene rubbers, Here, the material is mecha-nically fitted into the steel Rail. assembles. These joints can accomodate a larger movement than that by compressive seals (upto 100 mm)



Modular Joints.

A modular joints uses multiple strip seals to accom-odate very large deck movement. The seals are fitted b/w rolled beams which run along the length of the joint. This type of joint can accomadate movement ranging from

900mm to 1200mm. It is used for skewed & curved decks