ANALYSIS AND DESIGN OF HIGHWAY BRIDGES USING "VB MACROS"

A project submitted in partial fulfillment of the requirements for the award of the degree of

BACHLOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

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CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"ANALYSIS AND DESIGN OF HIGHWAY BRIDGES USING"VB MACROS""** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering and submitted to the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **RAJAT RANA(141669), ATUL KUMAR VERMA(141670) AND PRANAB CHAUHAN(141684)** under the supervision of **MR. BIBHAS PAUL** Assistant Professor, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat

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CHAPTER – 1

INTRODUCTION

In bridge engineering design firstlyRoad Project Division is needed to carry out survey for the bridge location and collect required preliminary survey data that is required for bridge planning and design. Generally 2-3 cross sections at prospective sites are taken and the bridge length is accordingly decided. Depending on site conditions, such as the foundation condition, the type of bridges i.e. reinforced, steel, prestressed bridge, high level, submersible etc. is decided.

IMPORTANT DEFINITIONS

1. **Bridge** –A structure having a total length of more than 6m between the inner faces of the abutmentsto carry traffic and other moving loads over a channel, road or railway. These bridges are classified as :-

   Small bridge –Its individual span length is less than 10m and the overall length is upto 30m

   Minor bridge – Minor bridge has length less than 60m.

   Major bridge – Bridge with length greater than 60m is considered a major bridge.

2. **Culvert** :The structure with total length 6m or less used for drainage purpose.
3. **Foot bridge :** A bridge that is mainly used to carry wayfarer, cycles and animals.
4. **High level bridge :** A bridge designed above H.F.L. of the channel taking freeboard into account.
5. **Submersible Bridge :**A Bridge which is overtopped during floods and are for some time submerged in water.
6. **Freeboard :**The difference between H.F.L. and lower level of superstructure road embankment on approaches.

7. **H.F.L. :** highest flood ever recorded on that river or the calculated level for design discharge is termed as H.F.L.

8. **L.W.L :**water level obtained in river in dry season referred as lowest water level

9. **Length of Bridge :** Overall length of the bridge measured along the center line of the bridge between the abutment.

10. **Safety Kerb :**A roadway kerb that is for occasional use for pedestrian traffic.

11. **Width of Carriageway :**Minimum clear width measured to the centerline of bridge between inside side of carriageway

12. **Abutment :**The end supports of bridge that retains earth fully or partially.

13. **Pier : piers** are the intermediate supports to support superstructure of a bridge.

14. **Foundation : t**he part of bridge which is used to transmit load to the earth strata.

SKETCH VIEW :

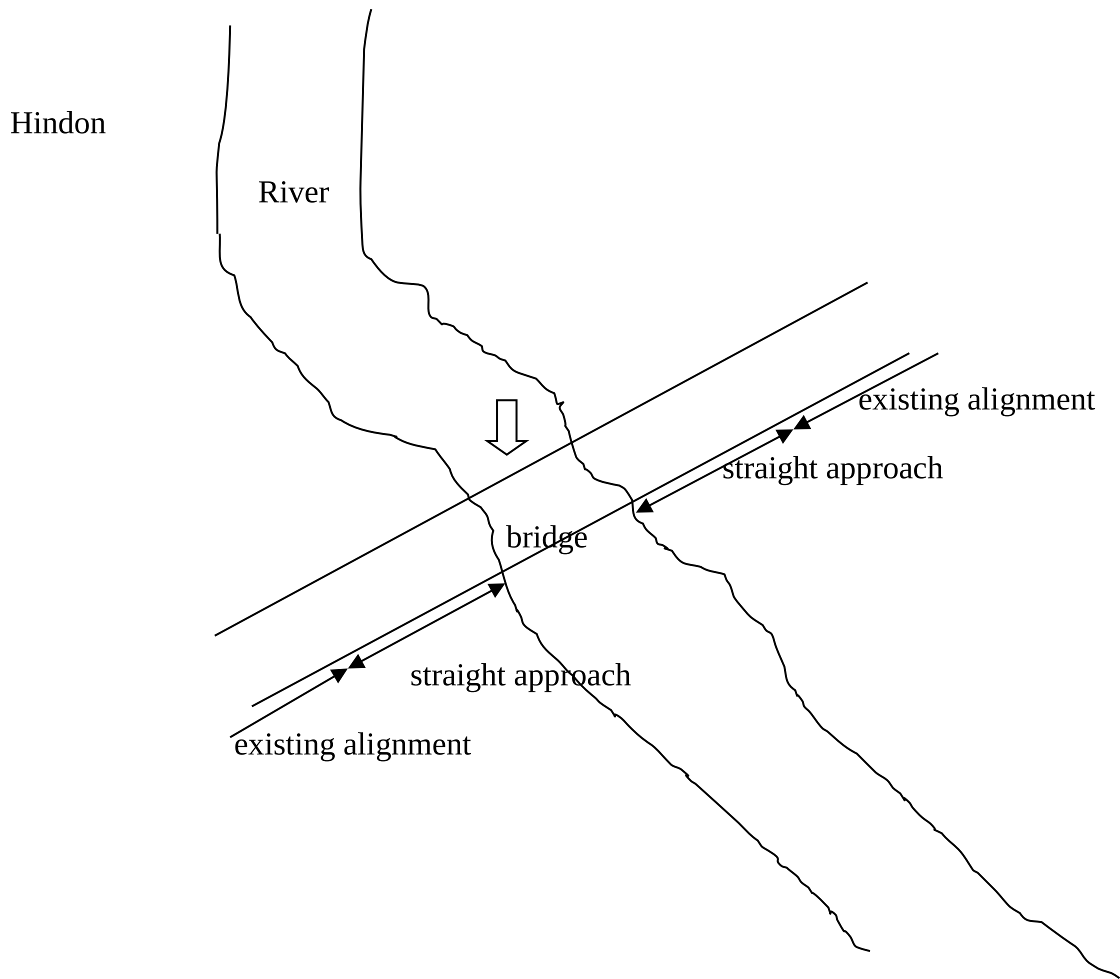


Fig. 1.1 crossing

CHAPTER - 2

TYPES OF RCC BRIDGES

1. **T- BEAM BRIDGES :**The T-beam bridges is by far the most commonly adopted type in the span range of 10m to 25m. the main longitudinal girders are designed as T-beam, which is cast monolithically with the girders. T-beam bridges are usually cast-in-situ on face work resting on ground.
   Simply supported T-beam spans of over 25m are rare as the dead load then becomes to heavy

   Typesof t-beam bridges

   a. **Girder and slab type,** in which the deck slab is casted monolithically with the longitudinal girders. In this type no cross beams are provided. In this case deck slab is designed as a one-way slab .it does not possess much torsional rigidity and the longitudinal girders can spread laterally at the bottom level.

   b. **Girder, slab and diaphragm type,** the slab is casted monolithically with the longitudinal girders, Diaphragms are provided at the support locations and at one or more intermediate locations within the span to connect longitudinalgirders .these do not extend up to the deck slab and hence slab behaves as an one-way slab .

   c. **Girder, slab and cross beam type,** the system has at least three cross beams cast monolithically with the deck slab with beams till the deck slab . The floor slab are supported by the longitudinal and cross beams. Thusdesigned as a two-way slab. This leads to more efficient use of reinforcing steel and reduces slab thickness and thus leading to the reduction of dead load on the longitudinal girders.

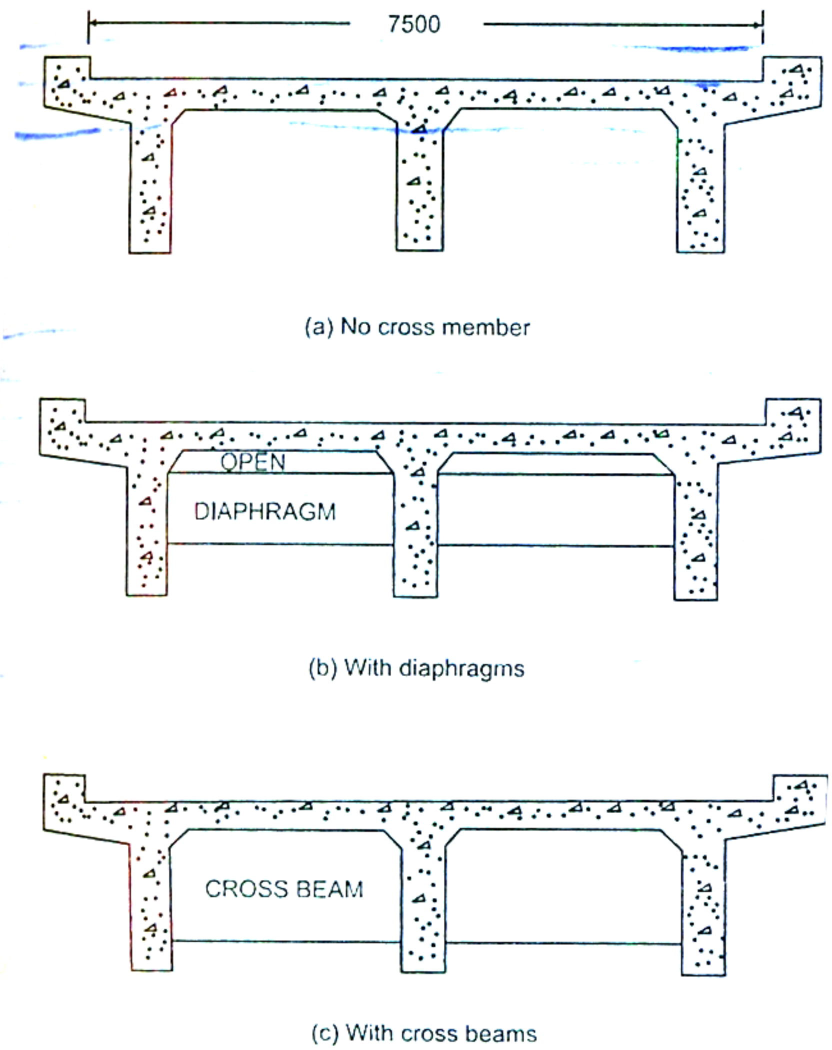


Fig. 2.1 Typical cross section of T-beam bridges

2. **HOLLOW GIRDER BRIDGES :**Reinforced concrete hollow girder bridges are economical in the span range of 25m to 30m. the closed box shape provides torsional rigidity, and the depth can be varied conveniently along the length as in continuous deck or in balanced cantilever layout. The cross-section can consist of a single cell or can be multi-cellular.

   The extra torsional stiffness of the section makes this form particularly suitable for grade separations, where the alignment is normally curved in plain. The cells can be rectangular or trapezoidal, the latter being used increasingly in prestressed concrete elevated highway structures.

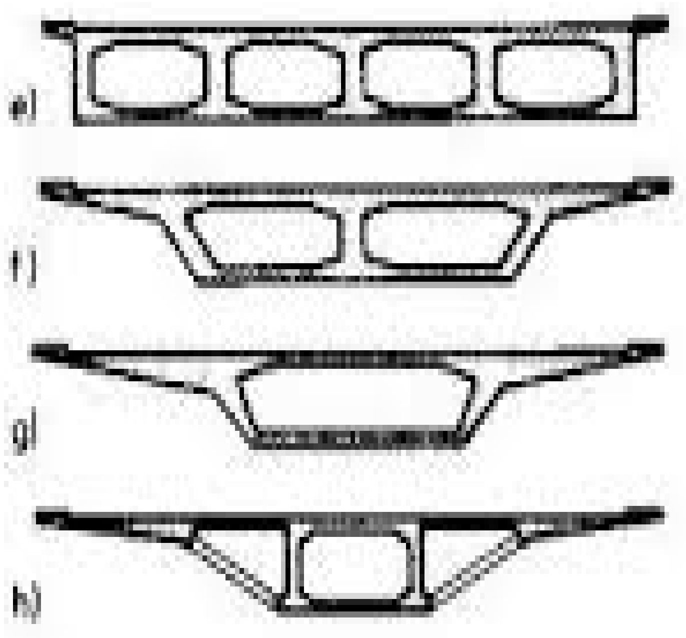


Figure 2.2 Typical Cross Section of a Hollow girder deck

3. **BALANCED CANTILEVER BRIDGES :** the governing bending moments can be reduced if continuous spans are used and thus the individual span lengths get increased. But construction is done only on unyielding support. For medium spans range is 35 to 60m, a combination of supported spans, cantilevers and suspended spans may be used as shown in Fig. 2.3. thus these bridges are commonly referred as balanced cantilever bridge.The connection between the suspended span and cantilever is called articulation. The bearings at articulations are of fixed and expansion types and are generally in the form of sliding plates, roller- rocker arrangement.

The cantilever span is generally 0.20 to 0.25 of the supported span. The simply supported design is adopted for design of suspended span. The T-beam or hollow girder type can be used for design for balanced cantilever portion.

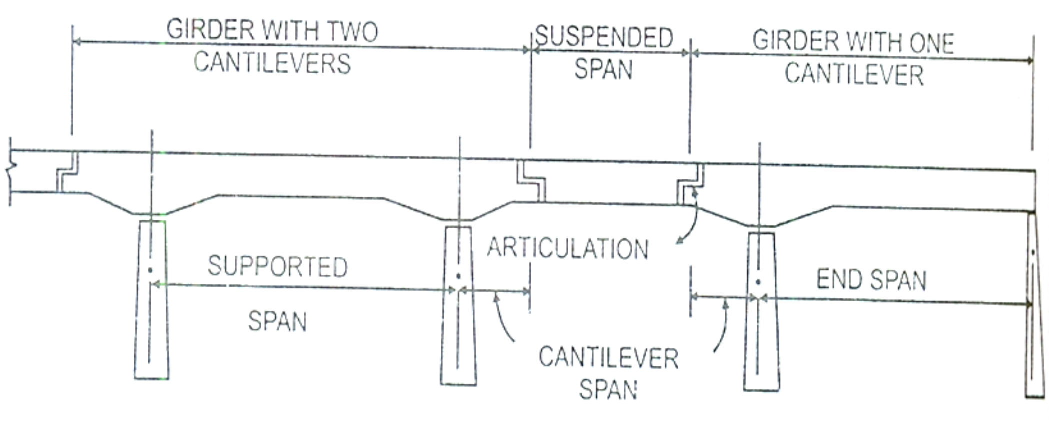


Fig. 2.3 Schematic view of balanced cantilever beam

4. **CONTINUOUS GIRDER BRIDGES :**Continuous girder bridges are suitable when unyielding supports are available. Typical shapes of these type of bridges are shown in fig. 2.4 . the spans can be equal, but usually the end spans are made atleast 16 to 20% smaller than the intermediate spans. The decking is in the form of slab.

Continuous girder bridges have the following advantages over simply supported girder bridges :

a. The depth of decking at mid span will be much smaller.
b. The quantities of steel and concrete will be less, resulting in reduced cost
c. Fewer bearing are required. For a continuous girder design, only two joints are needed at the ends, while the simply supported girder design will require joint on each abutment and pier.
d. Since the bearing are placed on the centre lines of the piers, the reactions of the continuous girder are transmitted centrally to the piers.

The disadvantages of continuous girder designs over the simply supported girder designs :

a. Uneven settlement of foundation lead to disaster
b. The detailing and placing of reinforcements need extra care.
c. Being statically indeterminate, the design is more complicated than simple beams

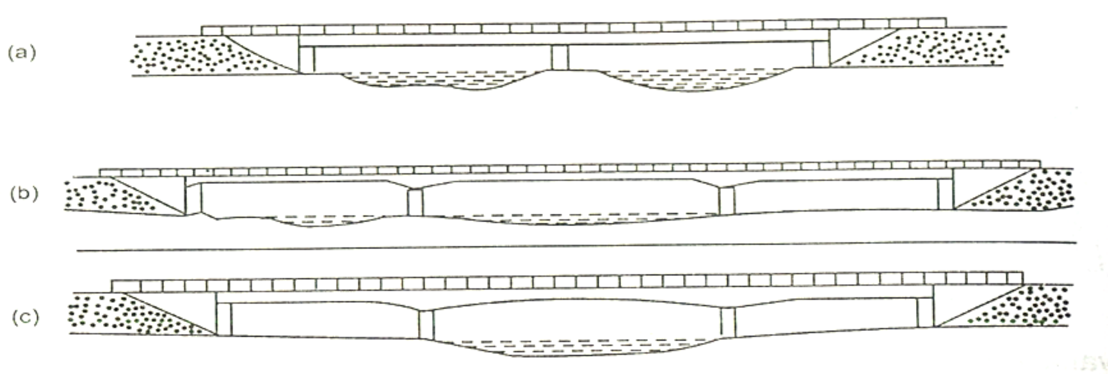


Fig. 2.4 Typical Continuous Reinforced Concrete Bridges

5. **RIGID FRAME BRIDGES:** Rigid frame bridges are structures consisting of a number of parallel girders which are rigidly connected to the supporting columns or piers. Usually the decking and substructure are casted monolithically.

In rigid frame no bearings are needed at the supports. The rigid connections result in more stable supports.

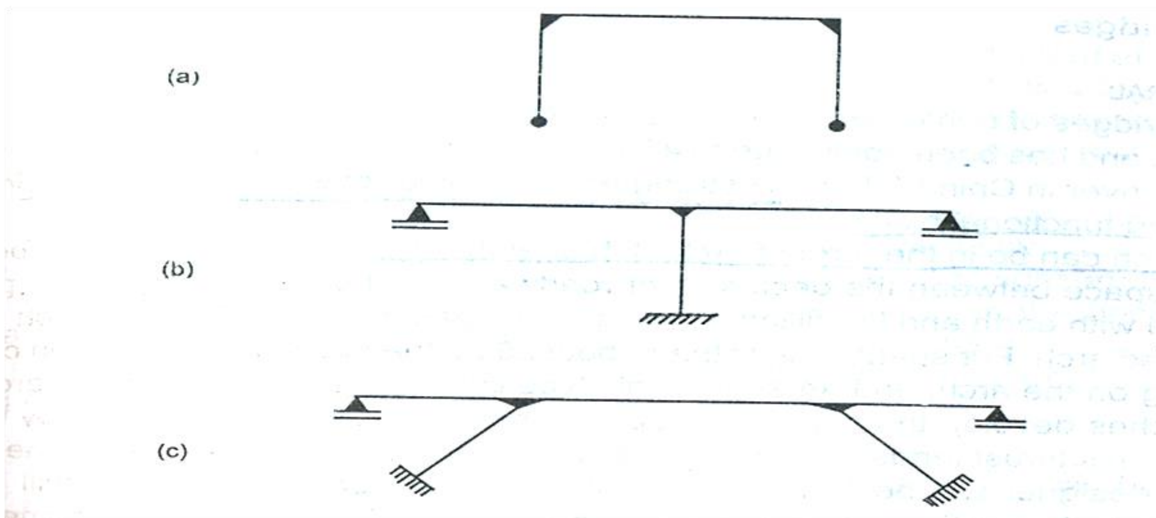


Fig. 2.5 Types of Rigid Frame Bridges

6. **ARCH BRIDGES :**Arch bridges of reinforced concrete can be used advantageously in the span range of 35 to 200m.

The arch can be in the form of arch slab or arch ribs. For span longer than 25m, the deck is supported on columns walls resting on the arch, and an arch of this type is known as open spandrel arch.

CHAPTER – 3

TYPES OF STEEL BRIDGES

1. **PLATE GIRDER BRIDGES**: Plate girder bridges are adopted for simply supported span less then 50m and for continuous spans upto 260m.

Plate girder bridges can be of two types:

1. Deck type
2. Half-through type

Deck type is normally preferred. Half-through type is adopted when the cost of additional embankment to raise the rail level is high. It consists of the deck slab and stringers running longitudinally and resting on transverse floor beams, which in turn rest on the plate girders.There is usually a choice available between

1.Using two widely spaced longitudinal girders, with the cross girder system supporting the deck

2.Providing multiple longitudinal girders with small spacing.

The two girder system necessities deeper girders and may lead to economy in certain circumstances.

1. **BOX GIRDER BRIDGES:** Developments in welding technology and precision gas cutting techniques in the post second world war period facilitated the economical fabrication of monolithic structural steel forms such as steel box girders characterized by the use of thin stiffened plates and closed form of cross section. A box girder is built up using is built up using a deck plate, vertical or inclined webs and a bottom plate. The deck plate carries the heavy traffic loads and so needs stiff stringers and transverse beams to transfer the loads to the box webs by bending. The bottom plate acts as a chord member for bending and also gets axial tension or compression.The box girder deck can have single or multiple cells, the latter being uneconomical for short spans.

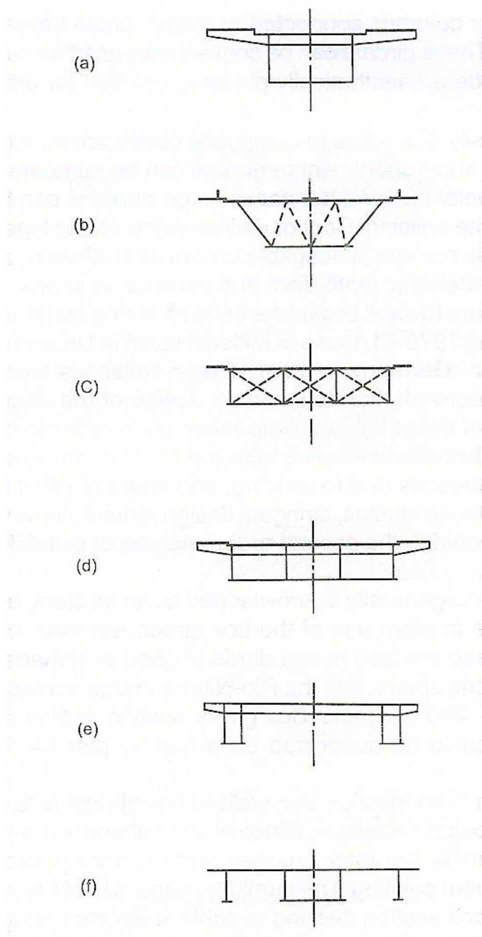


Fig. 3.1.1 Box Girder Bridges

2. TRUSS BRIDGES

Truss bridges are used in the span range of 100 to 200m. A bridge truss derives its economy from its two structural advantages

1. The primary forces in its members are axial forces

2. The erection of a truss bridge is simplified due to the relative lightness of component members.

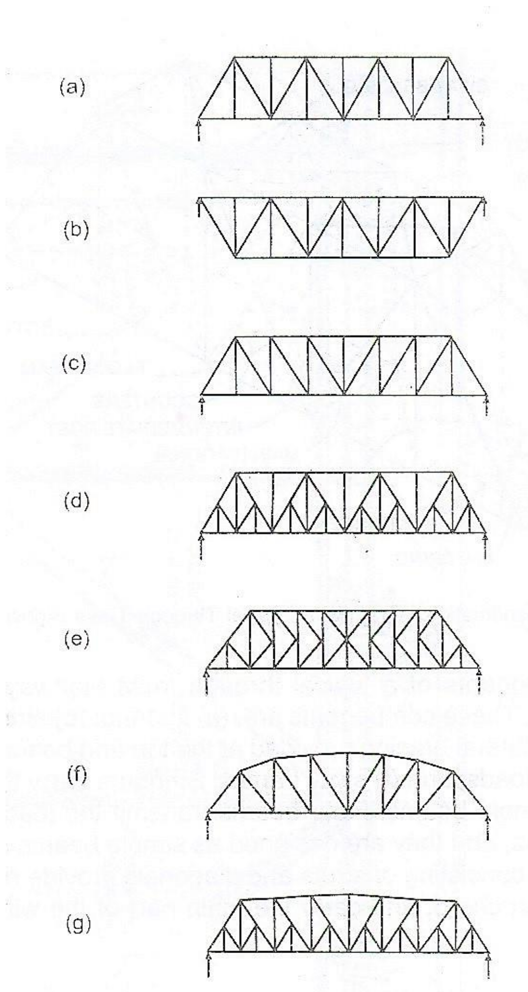


Fig. 3.2 Truss Bridges

3. ARCH BRIDGES

The arch are used in areas of deep gorges with steep rocky banks with efficient natural abutment to receive the heavy thrust exerted by the ribs. In the absence of these natural conditions, the arch usually suffers a disadvantage if natural conditions are not available the construction of suitable abutment is expensive and time consuming.It is used in the span range of 100-250m.

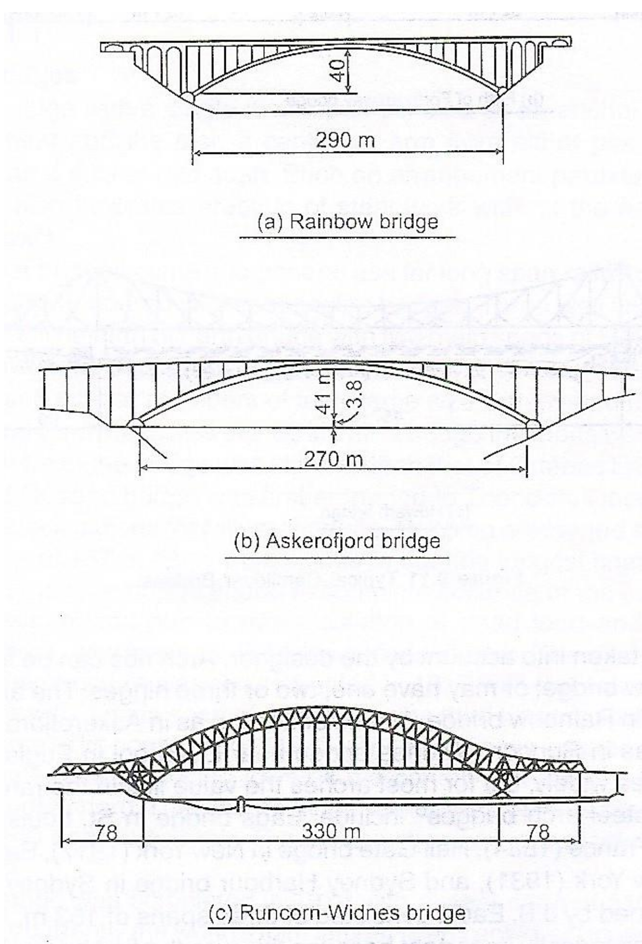


Fig. 3.3 Arch Bridges

4. CANTILEVER BRIDGES

Cantilever bridge consists of an anchor arm at either end between the abutment and pier, a cantilever arm from either pier to the end of the suspended span and a suspended span. It permits a long clear span for navigation.

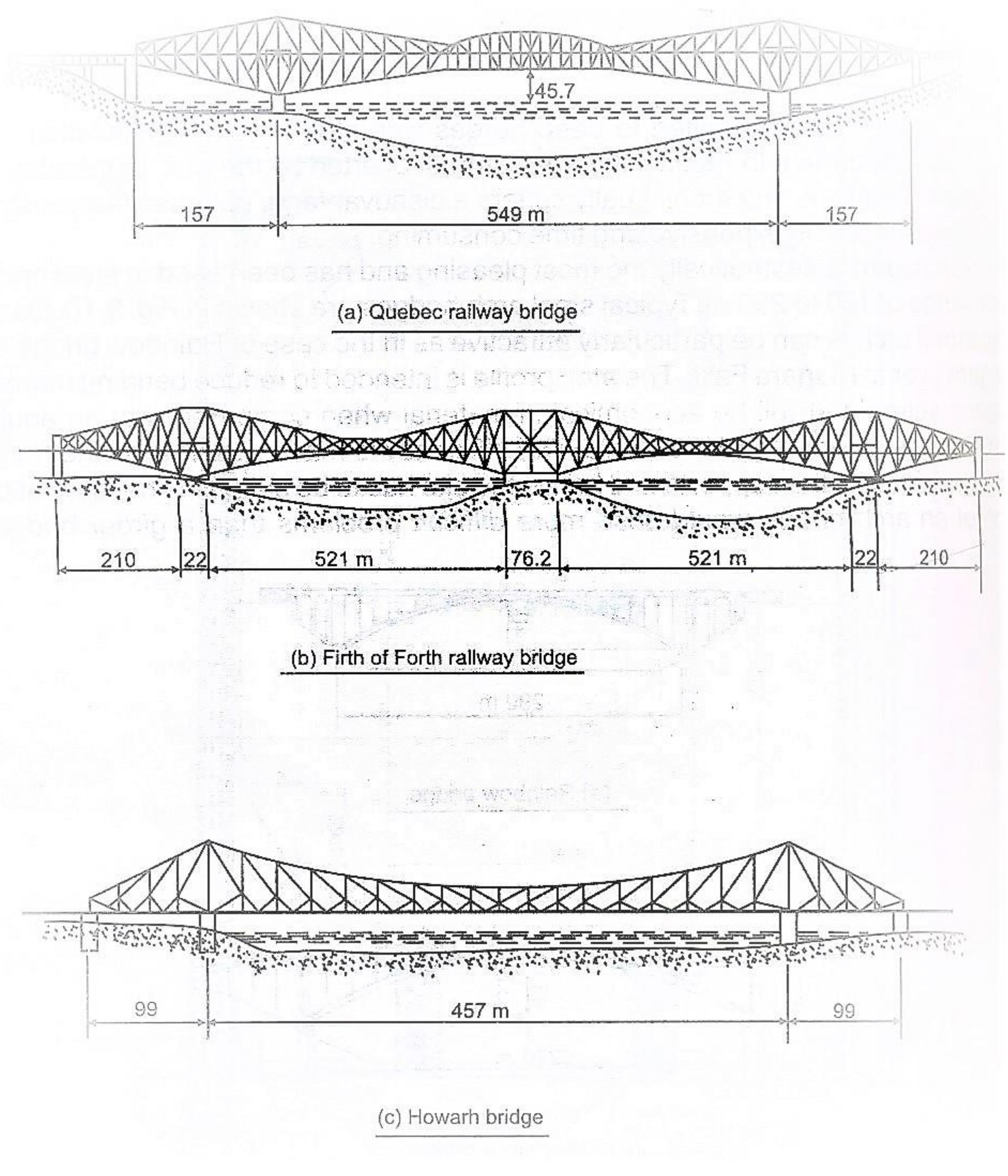


Fig. 3.4 Cantilever Bridges

5. CABLE STAYED BRIDGES

A cable stayed bridge is a bridge whose deck is suspended by multiple cables that run down to the main girder from one or more towers. The cable stayed bridge is specially suited in the span range of 200 to 900m and thus provides a transition between the continuous box girder bridge and the stiffened suspension bridge.

Components Of Cable Stayed Bridges

The main components are

1.Inclined cables

2.Towers

3.Deck

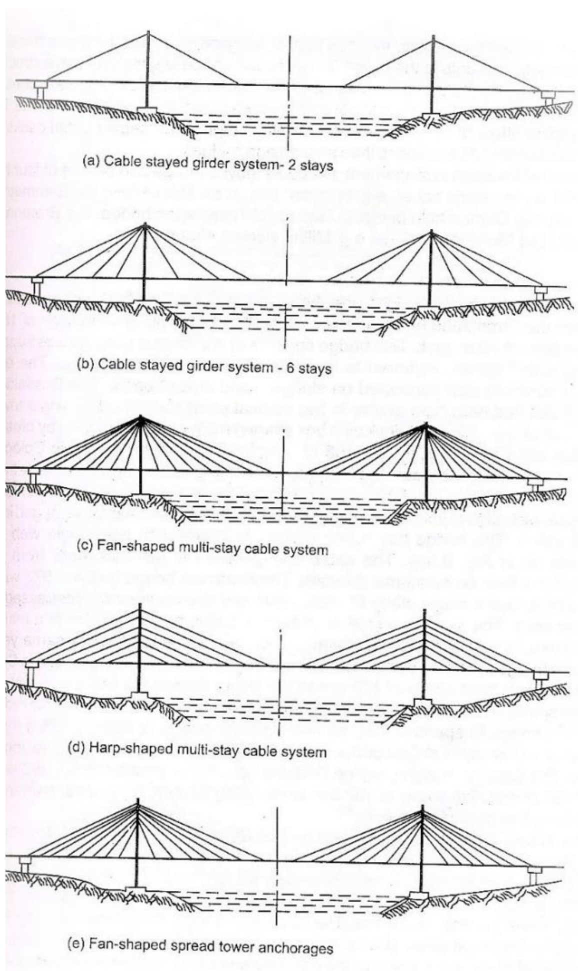


Fig. 3.5 cable stayed bridges

6. SUSPENSION BRRIDGES

The suspension bridge is currently the only solution for spans in excess of 900m and is regarded as competitive for spans down to 300m.

Components Of Suspension Bridge

Flexible main cables, towers, anchorages, hangers, deck, stiffening systems

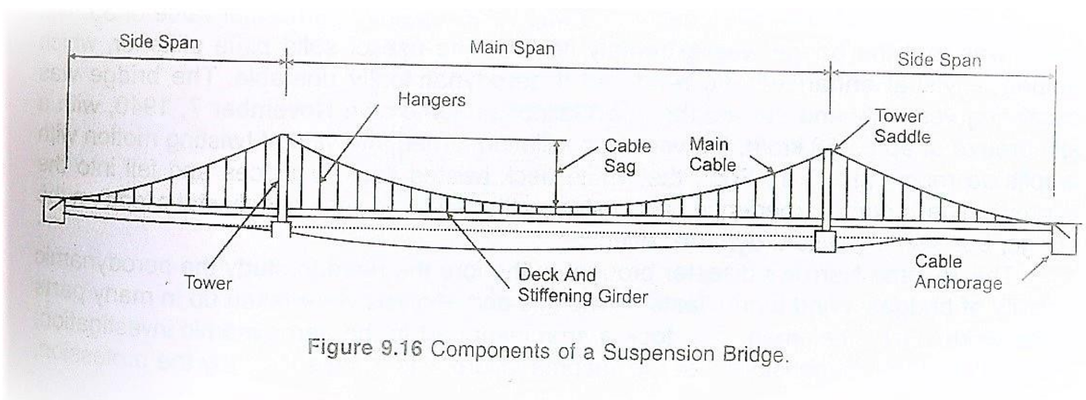


Fig.3.6suspension bridge

CHAPTER – 4

PROBLEM STATEMENTS AND ASSUMPTIONS

This report outlines the design of RCC and Steel bridge having total length of 180 m over Hindon River Ghaziabad U.P.

RCC Bridgehaving single span length of 15m, width of two lane single carriage way used for vehicle movement is 7.5m, width of footway provided for pedestrians is 1.5m each side,height of pier is 5.5m and catchment area of Hindon river is 7083.0 Km^2

- Discharge (Q) = $C \times A^{0.6}$
- C = 6.8 for flat tracts within 25 Km of the coast from IRC code
- Q = 2508.0 m^3/sec

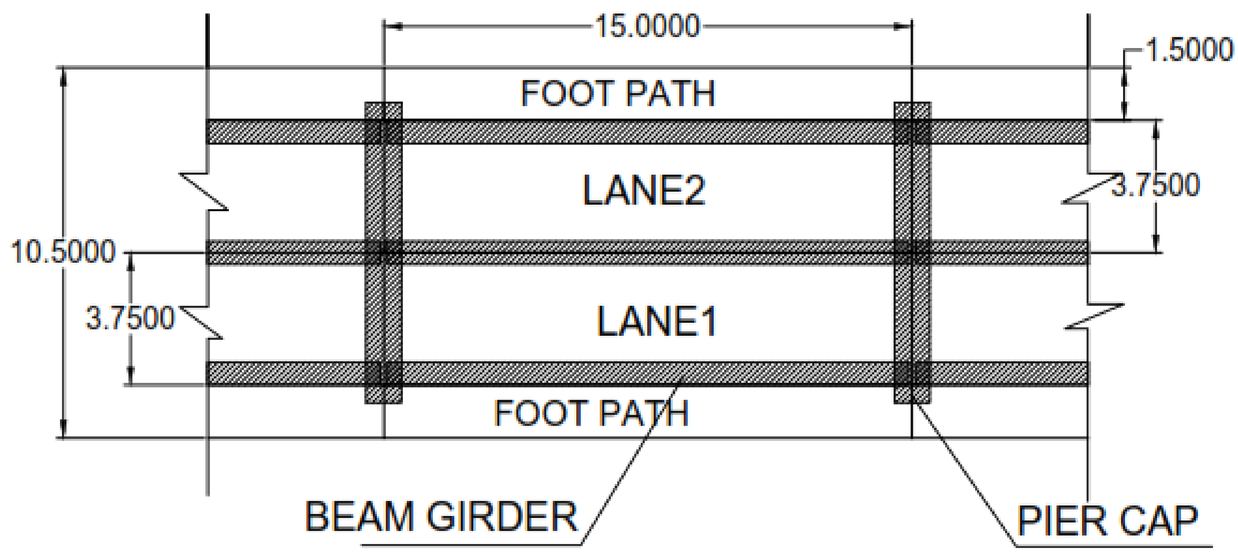


Fig. 4.1 Plan Views OfRCC Bridge

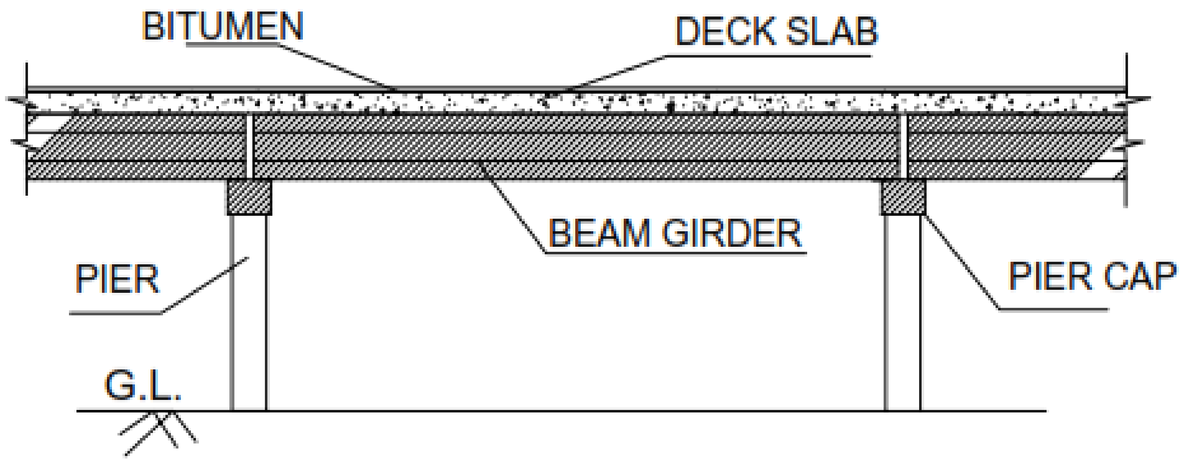


Fig. 4.2ElivationViews Of RCCBridge

Steel Truss bridge having single span length of 36 m, , width of two lane double carriage way used for vehicle moveent is 7.5m, all members of the truss are axially loaded members ,bracing members are provided to resist latera force due to wind

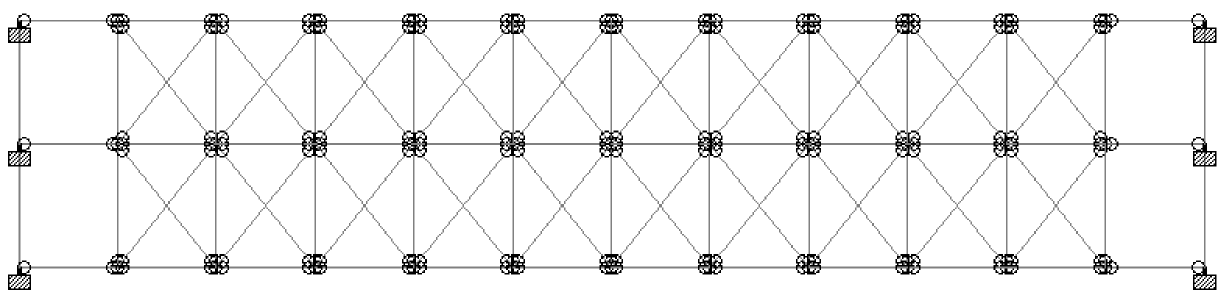


Fig. 4.3 Plan Views Of Truss Bridge

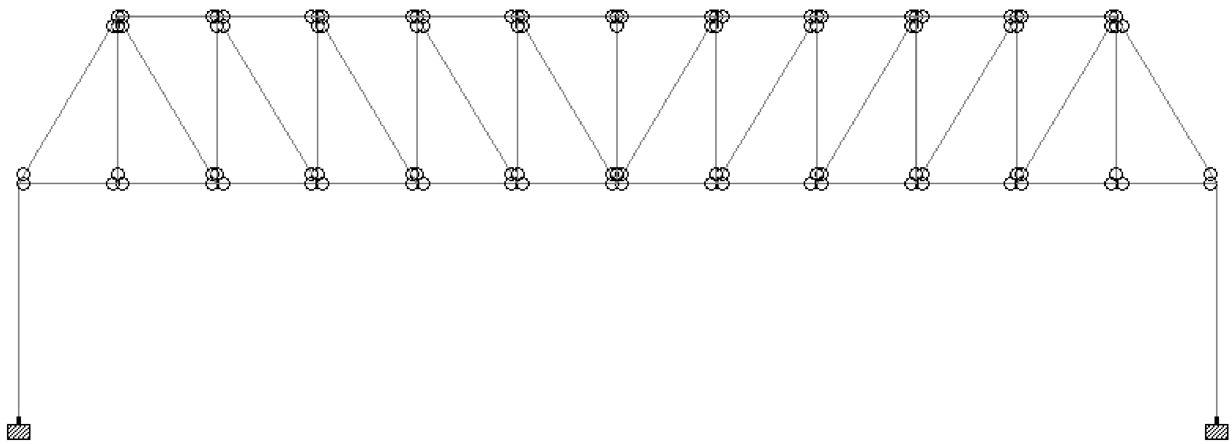
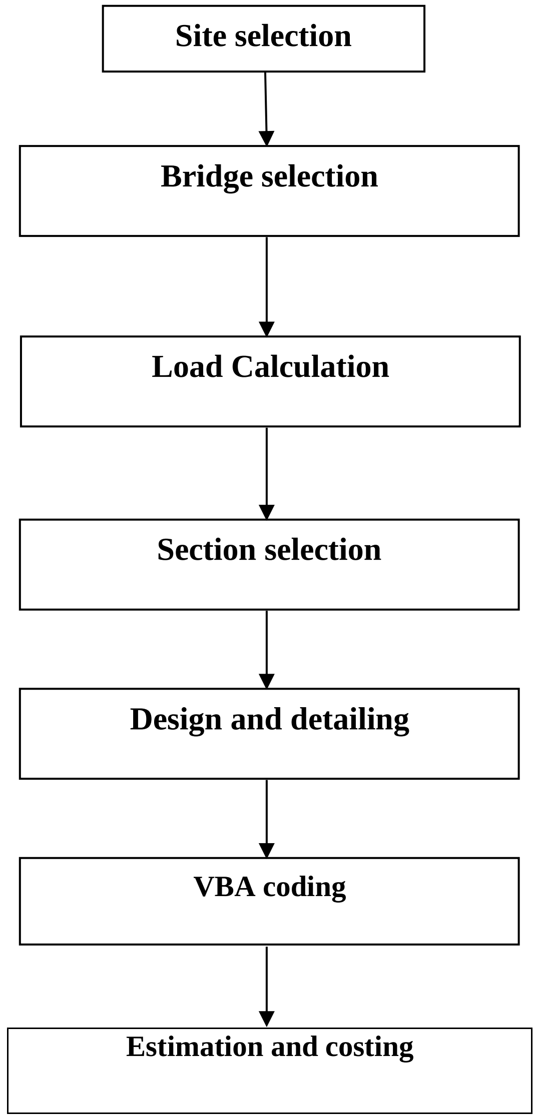


Fig. 4.4 Elevation Views Of RCC Bridge

FLOW CHART



Site selection
Bridge selection
Load Calculation
Section selection
Design and detailing
VBA coding
Estimation and costing

1. **DEAD LOAD :** The dead load carried by a girder or member include the weight of slab and superstructure which is supported by the girder or member including there own weight.

Figure showing distribution of dead load of slab on the truss members in a steel truss bridge

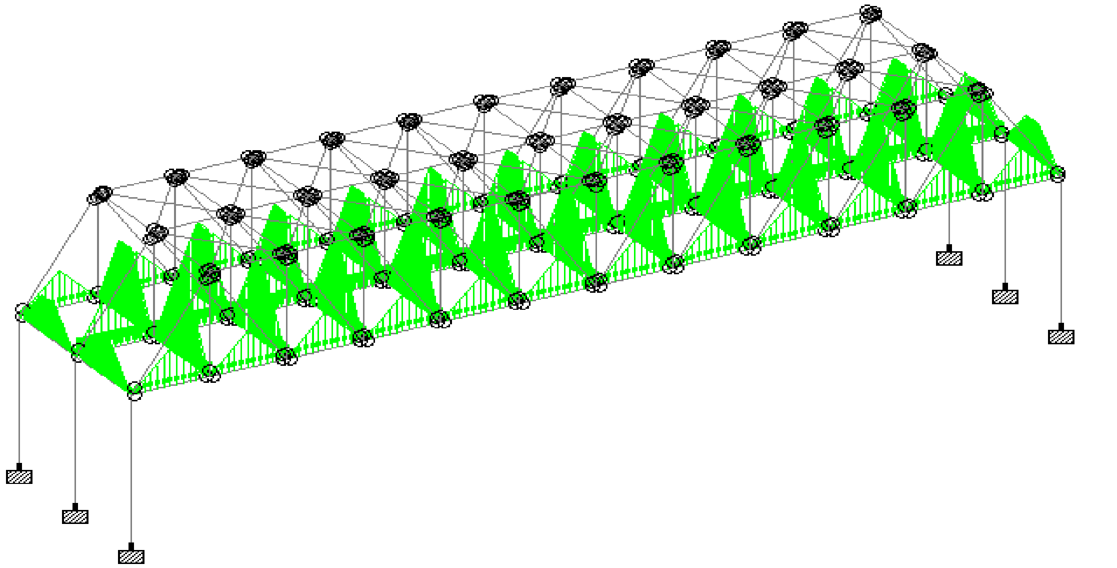


Fig. 4.5Dead Load Of Slab On Truss Member

2. LIVE LOAD : Class A train of vehicles

a. The nose to tail distance between successive trains shall not be less than 18.5m.

b. The ground contact area of the wheels shall be as under :

Table 4.1 Ground Contact area of the Wheels

Axle load (tonne)	Ground contact area	
	B mm	W mm
11.4	250	500
6.8	200	380
2.7	150	200

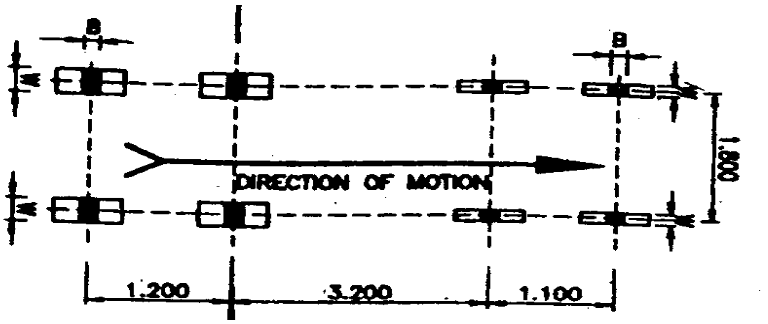


Fig.4.6 Class A train of vehicles

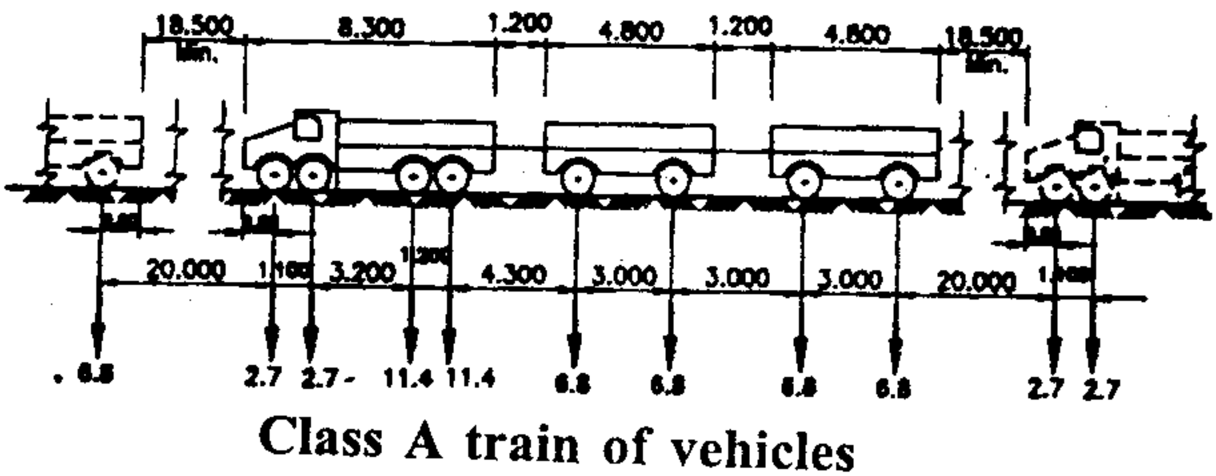


Fig. 4.7 Class A train of vehicles

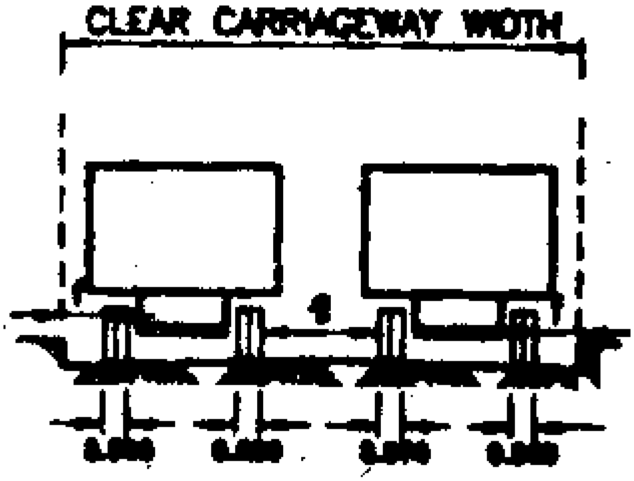


Fig.4.8 Cross section of carriageway

c. The minimum distancef of wheels outer edge and kerb, and the minimum distance g, of the outer edges of passing or crossing vehicle on multi-lane bridges shall be as given in the table:

Table 4.2 minimum clearance

Clear carria-geway width	g	f
5.5. m to 7.5 m	Uniformly increasing from 0.4 m to 1.2 m	150 mm for all carriageway widths
Above 7.5 m	1.2 m	

Table 4.3 live load combination

carriageway	Number of lanes for design purpose	Load combination
5.3 m and above but less than 9.6 m	**2**	**One lane of class 70R OR two lanes of class A**

FOOTWAY, KERB, RAILINGS AND PARAPET : While designing the main structure members of the bridge the horizontal force applied for footway, kerb, railings, parapet and crash barriers specified in this section need not be used.

For bridge floors accessible to crowed and animals and for all footways the loading shall be 400kg/m².the loading shall be increased to 500kg/m² if the crowed and animal footway loading increased like bridges in town

footways shall be designed for the following live load per square meter for footwayarea areconsidered as:

a. For effective span of 7.5m or less, 400kg/m² or 500kg/m² .
b. For effective spans more than 7.5m and less than 30m, the intensity of load shall be determined according to the equation-

$$P=P'-(40L-300)/9$$

Where p'=400kg/m² or 500kg/m².

P=live load in kg/m².

L= effective span of the main girder, truss or arch in m.

The figure given below showing the position of moving load acting on RCC bridge

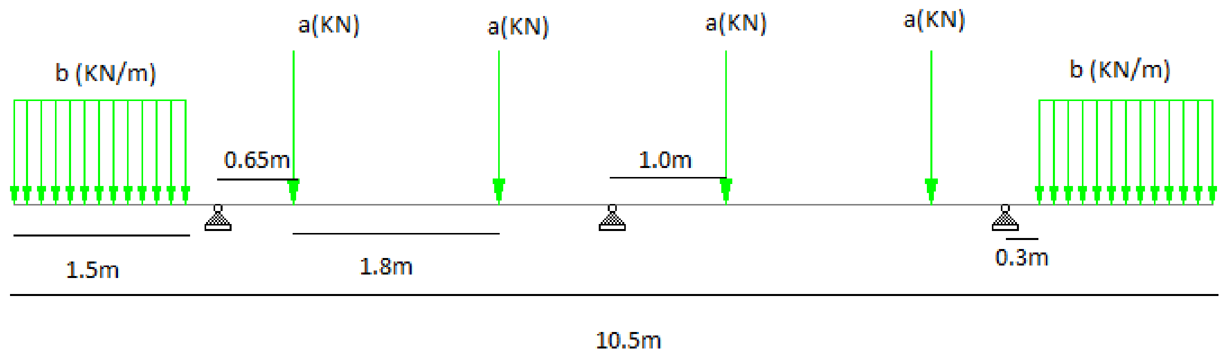


Fig. 4.8 Showing pedestrians and wheel load

For first two wheel :

a = 13.5 KN and b = 23.95 KN/m

for third and fourth wheel :

a = 57.0 KN and b = 6.0 KN/m

for the last four wheel :

a = 34.0 KN and b = 15.0 KN/m

By putting these values above we get the reaction forces in the girder beams

3. IMPACT LOAD

Provision for impact or dynamic action hall be made by increment of the live load by an impact allowance expressed as a fraction or a percentage of the applied live load..

In the members of any bridge designed for Class A loading the impact fraction shall be determined from thecurve indicated below. The impact factor shall be determined using following equations which are applicable for spans between 3m and 45m.

a. Impact factor for reinforced concrete bridges= 4.5/(6+L)
b. Impact factor for steel bridges=9/(13.5+L)

Impact factor for RCc bridge(span=15m) = 21.4%

4. WIND LOAD :

All the structures shall be designed for the following lateral wind forces. These forces shall be considered to act horizontally and in such a direction such a direction that the resultant stresses in the member under consideration are the maximum .

The wind force on a structure shall be assumed as a horizontal force of intensity as in table 4.4 and acting on area calculated as follows:

a. For a deck structure:

The area of the structure as seen in elevation including the floor system and railing, less area of perforations in the hand railing or parapet walls.

b. For a through or half-through structure:

The area of the elevation of the windward truss also adding half the area of the elevation above the deck level of all other trusses or girders.

The intensity of the wind force is based on the wind velocity and wind pressure as in table 4.4 and shall be allowed for design.

Table 4.4 wind pressure and wind velocity

H.	V.	P.	H.	V.	P.
0	80	40	30	147	141
2	91	52	40	155	157
4	100	63	50	162	171
6	107	73	60	168	183
8	113	82	70	173	193
10	118	91	80	177	202
15	128	107	90	180	210
20	136	119	100	183	217
25	142	130	110	186	224

Image given below showing wind load acting at the intersection of member in a steel truss bridge

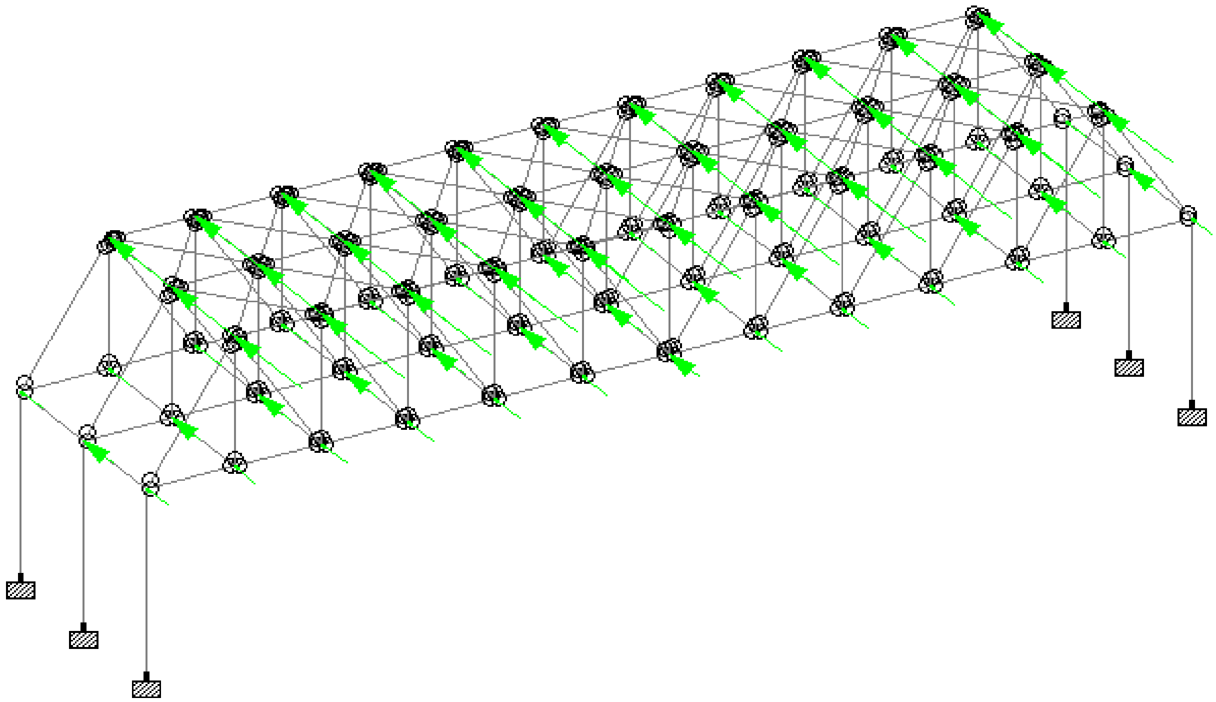


Fig. 4.4Wind Load Acting At Intersection Of Members

5. LONGITUDINAL FORCES

In all the road bridges, provision shall be made for longitudinal forces arising from any one or more of following causes:

i. Tractive effort caused through acceleration of the driving wheels,
ii. Braking effort resulted from the application of the brakes to braked wheels
iii. Frictional resistance offered to the movement of free bearings due to change in temperature or any other cause

The breaking effect on a simply supported span or a continuous unit of span or on any other type of bridge unit shall be assumed to have the following value:

i. In case singe lane or two lane bridge: 20 percent of the first train load plus 10 percent of the load of the succeeding trains or part thereof, when the entire first train is not on the full span, the braking force shall be taken equal to 20 percent of the load actually on the span.

ii. In cases of bridges having more than two lanes: as in above for two lanr plus 5 percent of the load on the lanes excess of two.

The forces due to braking effect shall be assumed to act along line parallel to the roadway and 1.2m above it. While transferring the force to the bearing, the change in the vertical reaction at the bearing should be taken into account.

6. HORIZONTAL SEISMIC FORCES

The horizontal seismic forces to be resisted shall be computed as follows except in case of long span bridges with spans greater than 150mm where special studies have to be undertaken based on site specific seismic design criteria.

$F_{eq} = A_h$ *(dead load+ appropriate live load)

Where F_{eq} = seismic force to be resisted

A_h = horizontal seismic coefficient $= \left(\frac{Z}{2}\right) \times \left(\frac{Sa}{g}\right) \times \left(\frac{R}{I}\right)$

Z = zone factor

I = importance factor for

Important bridges 1.5

Other bridges 1

T = fundamental period of bridge member (in sec.) for horizontal vibrations

For small bridges Sa/g may be taken as 2.5

R = response reduction factor (2.5)

Sa/g = average response acceleration coefficient for 5 percent damping depending upon fundamental period of vibration

Table 4.5 Zone Factor (Z)

Zone No.	Zone Factor
V	0.36
IV	0.24
III	0.16
II	0.10

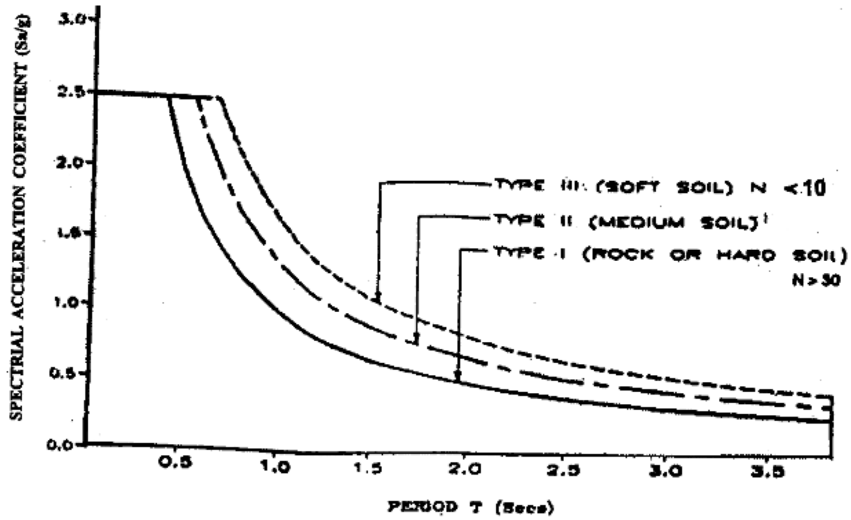


Fig. 4.9 Response Spectra

DESIGN DISCHARGE OF FOUNDATION

The design discharge should be increased over the design discharge determined as per IRC:5 to provide enough margin of safety, and foundation design for scour as given below:

Table 4.6 Design Discharge

Catchment area in km²	Increase over design discharge in per cent
0 - 3000	30
3000 - 10000	30 - 20
10000 - 40000	20 - 10
Above 40000	10

MEAN DEPTH OF SCOUR

The mean scour depth can be calculated theoretically from the following equation:

$$Dsm = 1.34 \times \left(\frac{D_b^2}{K_{sf}}\right)^{1/3}$$

where D_b = the design discharge for foundation per meter width of effective waterway

K_{sf} = silt factor

The value of $D_b$may be determined by dividing the design discharge for foundation by lower of theoretical and actual effective linear waterway as given in IRC:5.

The value of ksf for various grades of sandy bed are given below for ready reference and adoption:

Table 4.7 Silt Factor

Type of bed material	d_m	K_{sf}
Coarse silt	0.04	0.35
Silt/fine sand	0.081 to 0.158	0.5 to 0.7
Medium sand	0.233 to 0.505	0.85 to 1.25
Coarse sand	0.725	1.5
Fine bajri and sand	0.988	1.75
Heavy sand	1.29 to 2.00	2.0 to 2.42

If there is any predominant concentration of flow in any part of waterway due to bend of the stream in immediate upstream or downstream or for any other reason, like wide variation of type of bed material across the width of channel, then mean scour depth may be calculated dividing the waterway into compartments as per the concentration of flow.

CHAPTER-5

GENERAL DESIGN REQUIRNMENTS

GENERAL

Various stresses that are likely to occur in any plain and reinforced concrete structure as specified in IRC:6 should be provided for in accordance with the accepted design and construction procedure. The detailing of the reinforcement in all components shall be such as to ensure satisfactory placement .

BASIS OF DESIGN

The basis of design as employed in elastic theory are :

i. The modulus of elasticity of steel is 200GPa .
ii. The concrete tensile strength is ignored.

COVER

The minimum clear cover provided to any reinforcement bar should be 40mm.

In case concrete is exposed to severe conditions minimum cover shall be provided is 50mmand in foundations the minimum clear cover shall be 75mm.

The above cover may be reduced to 5mm forfactory made of precast products.

BAR SIZES

The maximum bar size of reinforcement shall be 40mm diameter or a section equivalent area.

The minimum diameter of any reinforcing bars(transverse ties, helicals, stirrups), should not be less than 8mm.

The longitudinal reinforcing bars diameter in columns should not be less than 12mm.

In slabs diameter of the reinforcement shall be limited to one-tenth of the depth, and the diameter of the shear reinforcement in beam-webs, including cranked bars, if any, shall be limited to one-eighth the thickness of the web.

DISTANCE BETWEEN BARS

The horizontal distance between the reinforcing bars should not be less than the greatest of the following:

i. The diameter of the bar – if the diameters are equal
ii. The diameter of the largest bar- if the diameters are unequal
iii. 10mm more than the nominal size of the coarse aggregate used in concrete.

In order to comply with the provisions of this sub- clause, the size of the coarse aggregate for the concrete around congested reinforcement may be reduced. There should be sufficient space between the bars so that the vibrator can be inserted.

The minimum vertical distance between horizontal main reinforcing bars shall be 12mm or the maximum diameter of the coarse aggregate or the maximum size of the bar, whichever is greater.

When contact of bars along the lap length cannot be avoided, such bars shall preferably be grouped in the vertical plane. In no case, however, shall there be more than three bars in contact. The vertical and horizontal distances shall be maintained between any such group and an adjacent group or bars .

Subject to satisfying crack control criteria as pitch of bares or wires of main tensile reinforcement in the slabs shall not exceed 3000mm or twice the effective depth of the slab whichever is smaller.

BOND, ANCHORAGE, SPLICE

To prevent the bond failure, design tension or compression in any reinforcing bar at any section of an element shall be developed on each size of the section by an appropriate anchorage length.

SHEAR

The design shear stress τ can be calculated by the following equation:

$$\tau = \frac{V}{(b \times d)}$$

Where V = design shear across the section
b = breadth of the member
d= the effective depth of the section

In case of beam s or slabs of varying depth:

$$\tau = \frac{\left(V \pm \left(\frac{M}{tan\beta}\right)\right)}{(b \times d)}$$

Where, M = it signifies the bending moment at the section, due to load position corresponding to shear V
β = it signifies the angle between the top and the bottom edge of the beam at that section

The negative sign indicates that bending moment M increases in the same direction as the effective depth increases, and the positive sign when the moment decreases in this direction

EFFECTIVE SPAN

In any case of free supports on line bearings, the effective span l_0 shall be l_1+d or l whichever is smaller,

Where l = distance between centre supports
l1 = clear span
d = effective depth of beams or slab.

In the case of restraints supports, the effective span can be taken as equal to the clear distance.

EFFECTIVE DEPTH

The depth from the edge of the compression section to the centroid of the tension reinforcement.

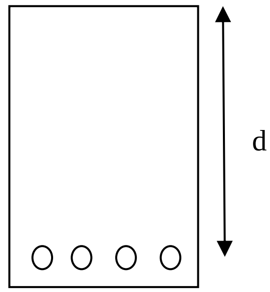


Fig 5.1Effective Depth Of Beam

EFFECT OF LIVE LOADS ON DECK SLABS

The concentrated loading on slabs spanning in one or two directions or on cantilever slabs is calculated from the influence fields of such loads or by other rational method. A value of 0.15 may be assumed for poisson’s ratio.

The bending moment per unit width of slab attained due to loading on solid slabs spanning in one direction or on cantilever slabs, may also be calculated by assessing the width of the slab that may be taken as effective in resisting the bending moment due to concentrated loads for precast slabs, the term actual width of slab shall indicate the actual width of each individual precast unit.

Solid slab spanning in one direction

i. For a single concentrated load, the effective width may be calculated in accordance with following equation :
   $b_{ef} = \alpha * a(1-a/l_0) + b_1$

where bef = the effective width of the slabs on which the load acts
l_0 = the effective span
a = the distance of centre of gravity of concentrated load from nearest support,
b_1 = the breath of concentrated area of load
α= a constant having following values depending upon the ratio b/l_0

provided that the effective width shall not exceed the actual width of the slab. And provided further that in case of a load near the unsupported edge of a slab, the effective width shall not exceed the above value nor half the above value plus the distance of the load from unsupported edge.

Table 5.1Constant (α)

$\frac{b}{l_o}$	α for simply supported slab	α for continuous slab	$\frac{b}{l_o}$	α for simply supported slab	α for continuous slab
0.1	0.40	0.40	1.1	2.60	2.28
0.2	0.80	0.80	1.2	2.64	2.36
0.3	1.16	1.16	1.3	2.72	2.40
0.4	1.48	1.44	1.4	2.80	2.48
0.5	1.72	1.68	1.5	2.84	2.48
0.6	1.96	1.84	1.6	2.88	2.52
0.7	2.12	1.96	1.7	2.92	2.56
0.8	2.24	2.08	1.8	2.96	2.60
0.9	2.36	2.16	1.9	3.00	2.60
1.0	2,48	2.24	2 & above	3.00	2.60

ii. For two or more concentrated loads in a line in the direction of the span, the bending moment per unit width of slab shall be calculated separately for each load according to its appropriate effective width of slab calculated as above.

iii. For two or more loads not in a line in the direction of the span if the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the resultant effective width for the two loads equals the sum of the respective effective widths for each loads minus the width of the overlap, provided that the designed slab is tested for the two loads acting individually.

MINIMUM REINFORCEMENT IN BEAMS AND SLABS

The area of tension reinforcement in a beam shall not be less than 0.2 percent of bt.d. when using Fe415/500 grade bars or 0.3 percent of bt.d. when using Fe240 grade where bt is width of section and d is effective depth.

In slabs in tension reinforcement shall not be less than 0.12 percent of the total cross-sectional area when using Fe415/500 grade bars and not less than 0.15 percent of the total cross-sectional area when using Fe240 grade bars.

CHAPTER-6

COLUMNS AND SUBSTRUCTURES

CLASSIFICATION

The columns are classified in three categories:

i. Pedestal columns: In pedestal columns the ratio of effective length to least radius of gyration should be less than 12.
ii. Short columns: in short columns ratio should be more than 12 and less than 50.
iii. Long columns: in case of long columns ratio lies between 50 to 150.

For calculating the radius of gyration, the cross section of the column for the column with binders and in case in the case of column with helical reinforcement, the section of core with helical reinforcement shall be considered. The effective column length are given in table 6.1

Table 6.1 Effective Column Length

Type of column	Effective column length
Properly restrained at both ends in position and direction	0.75 l
Properly restrained at both ends in position and imperfectly restrained in direction at one or both ends	A value intermediate between 0.75 l and l depending upon the efficiency of the directional restraint
Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end	A value intermediate between l and $2l$ depending upon the efficiency of the imperfect restraint

LONGITUDINAL REINFORCEMENT

In a pedestal column the cross section area should be greater than 0.15 percent of gross section area of the column and in other columns, the cross section area of longitudinal; reinforcement should lie between 0.8%to 8% of the gross section area of the column.

TRANSVERSE REINFORCEMENT

Transverse reinforcement in form of lateral ties, circular rings or a helical is desired in RCC columns.The angle should not be more than 135° and distance of the bar should not be more than150mm from such a laterally supported bar. In case the bars are located around the boundary of a circle, a circular tie may be used.

The diameter of transverse reinforcement should not be less than $1/4^{th}$ the diameter of the largest longitudinal bar in that region of the column and in no case less than 8mm.

The pitch of the transverse reinforcement should not be more than 300mm or less than the following :

i. The least lateral dimension of the column.
ii. 12 times the diameter of the least longitudinal reinforcement in column.

PILE FOUNDATION

GENERAL

Piles transfer the load of a superstructure to substrata beneath the pile. They also carry uplift and lateral loads.
The pile foundation construction requires a correct choice of piling system depending upon subsoil conditions and load characteristics of structure.For the design purpose the permissible limits of total settlement, differential settlement and unsupported length of pile should be known and design must stratify these criteria.

DESIGN AND CONSTRUCTION
For piles in streams, rivers, creeks etc., the design procedure to be checked:
1 Scour conditions are properly established
2 Permanent steel liner should be provided to the maximum scour level.The 5mmis the permissible minimum thickness of liner .

SPACING OF PILES

Spacing should be consider in relation to soil strata on which pile has to be installed,pile collectivebehavior and also the construction cost.The minimum spacing should maintained between installed piles so as to avoid damage to any adjacent construction or to the piles themselves.

The spacing of piles will be determined by:

i. The method of installation eg driven or bored;
ii. The bearing capacity of the group.

REQUIREMENT AND STEPS FOR DESIGN AND INSTALLATION

The initial design of an individual pile,group of piles and final adoption should pass through two types of major investigation and tests as follows:

i. Detailed sub surface investigation for piles to determine the end bearing capacity,friction capacity and lateral capacity of the soil where the pile is installed.
ii. Initial load test on trial piles for confirmation/modification of design and layout and routine load test on working piles for acceptance of same.

FACTOR OF SAFETY

The minimum factor of safety on ultimate axial capacity computed on the basis of static formula shall be 2.5 for piles in soil.For piles in rock,factor of safety shall be 5 on the hearing component and 10 on socket side resistance component.

MINIMUM REINFORCEMENT

The area of longitudinal reinforcement shall not be less than following percentages of the cross sectional area of piles:

i. For piles with a length less than 30 times the least width -1.25 percent;
ii. For piles with a length 30 to 40 times the least width-1.5 percent;
iii. For piles with a length greater than 40 times the least width- 2 percent.

PILE CAP

A minimum offset of 150mm shall be provided beyond the outer faces of the outermost piles in the group.If the pile cap in contact with earth at the bottom, a leveling course of minimum 80mm thick plain cement concrete shall be provided.
The top of pile shall project 50mm into the pile cap and reinforcements of pile shall be fully anchored in pile cap.
The minimum thickness of pile cap should be atleast 0.6m or 1.5 times the diameter of pile whichever is more.

PIERS

Piers in stream and channel should be located to meet navigational clearance requirements and give a minimum interference to flood flow.
Pier may be in PSC, RCC, PCC or masonry.Only solid section should be adopted for masonry piers.The thickness of the walls of hollow concrete piers should not be less than 300mm.
In case of piers consisting two or more columns, the horizontal forces at the bearing can be distributed on all columns in proportion to relative rigidities, if the thickness of the pier cap is atleast one and a half times the thickness of column.

ABUTMENTS

The abutments will carry superstructure from one side.It should be designed/dimensioned to retain earth from the approach embankment.The abutments should be designed to withstand earth pressure in normal condition in addition to load and forces transferred from superstructure.
The abutment may be plain or reinforced concrete or of masonry.The abutment may be either solid type, buttressed type, counter fort type or spill through type.
The design of Full earth retaining abutments should be done while taking the submerged/saturated unit weight of earth as appropriate during H.F.L. or L.W.L. condition.In case of abutments having counter fort, the minimum thickness of the front wall should not be less than 200mm and the thickness of the counter fort should not be less than 250mm.

CHAPTER-7

VBA PROGRAM FOR RCC BRIDGE DESIGN

INTRODUCTION

Visual Basic is a computer programming language which is used for the creation of user-defined functions and for the automation of specific computer processes and calculations. Visual Basic for Application is a standard feature for Microsoft Office products, Visual Basic for Applications (VBA) allows users to find appropriate results just by changing some input variables within a second.

SLAB DESIGN

SS SLAB

VARIABLES

PARAMETERS	VALUE	UNIT
fck	30	MPa
fy	250	MPa
A0 from table	1.68	m
concentrated force from nearest support a1 (27KN)	1.1	m
concentrated force from nearest support a2 (27KN)	2.2	m
concentrated force from nearest support a3 (114KN)	5.4	m
concentrated force from nearest support a4 (114KN)	6.6	m
concentrated force from nearest support a5 (68KN)	4.1	m
concentrated force from nearest support a6(68KN)	1.1	m
l0 effective span	15	m
width of tyre in contact with surface w1(27KN)	0.2	m
width of tyre in contact with surface w2(114KN)	0.5	m
width of tyre in contact with surface w3(68KN)	0.38	m

bitumen thickness th1	0.08	m
slab width wt	7.5	m
(ML)using staad pro we get BM due to F1,F2,F3,F4,F5,F6	277	KNm
thickness of slab th2	0.4	m
cc clear cover	40	mm
c1 dia of bar	28	mm

RESULTS

PARAMETERS moment calculation	VALUES	UNITS
bef1(27KN)	3.15248	m
bef2(27KN)	4.59392	m
bef3(114KN)	8.44608	m
bef4(114KN)	8.84928	m
bef5(68KN)	7.16528	m
bef6(68KN)	3.87248	m
load /meter width F1	17.12937116	KN/m
load /meter width F2	11.75466704	KN/m
load /meter width F3	30.4	KN/m
load /meter width F4	30.4	KN/m
load /meter width F5	18.98041668	KN/m
load /meter width F6	35.11961327	KN/m
avg live load per meter Fav	23.96401136	KN/m
Mli BM due to live load including impact	336.3571429	KNm/m
dead load per m length Ld	11.36	KN/m
BM due to DL Md	79.875	KNm/m
total BM (ll+d) is Mu	416.2321429	KNm/m
depth check		
overall depth D1	362.1796501	mm
depth check	safe	

effective thicknes of slab th3	344	mm
design		
% of steel pt	1.927643503	%
area of main steel bar Ast1	6631.093649	mm^2
spacing of main bar s1	121	mm
distribution steel		
dia od DS sd	12	mm
area of distribution steel Astd	600	mm^2
spacing sd	188	mm
shear design		
Vu	359.4601704	KN
ζv	0.998500473	MPa
100*As/bw*d	1.841970458	%
ζc	1.15	MPa
ζc max	3.2	MPa
shear reinforcement provided or not	no shear reinforcement required	

DETAILING

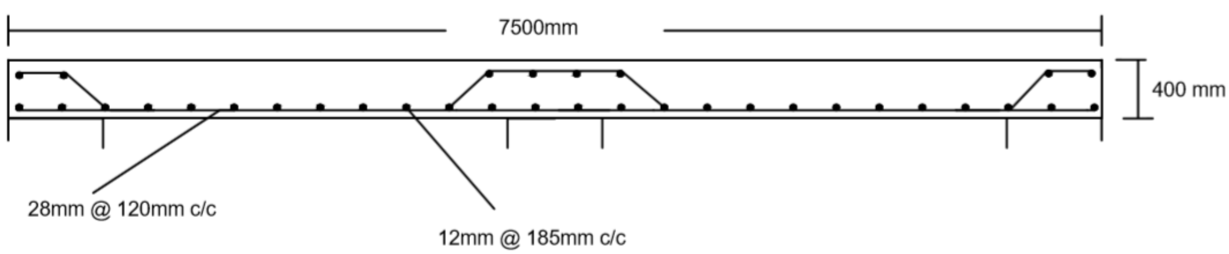


Fig 7.1 Slab Detailing

CANTILEVER SLAB

VARIABLES

PARAMETERS	VALUES	UNITS
fck	20	MPa
fy	415	MPa
Mu Bending moment	220	KNm/m
Vu shear force	480	KN
cc clear cover	40	mm
a1 dia of main bar	28	mm
tan(a) a angle of cantilever T	0.182	
width of cantilever b	1.5	m
max depth of cantilever D1	400	mm
length of member	1000	mm
total length of member Lm1	15000	mm

RESULTS

PARAMETERS	VALUES	UNITS
depth of cantilever	230.5213583	mm
overall depth	284.5213583	mm
check	safe	
effective depth de	346	mm
% of steel	0.36751855	%
area main steel Ast1	1907.421275	mm^2
no of bars n1	4	bars
shear design		
ζv	0.701883346	MPa
100*As/bw*d	0.551277825	
ζc	0.54	MPa
ζc max	3.5	MPa

shear reinforcement provided or not	shear reinforcement provided	
distribution steel		
dia od DS sd	12	mm
area of distribution steel Astd	480	mm^2
spacing sd	236	mm

BEAMDESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
fck	30	Mpa
fy	415	Mpa
bf	0.65	m
bw	0.36	m
d	1.45	m
df	0.42	m
Mu	2557.7	KNm
a(dia of steel)	0.025	m
L(member length)	15	m
wu	91.424	Kn/m
cover	40	mm

RESULTS

CONDITIONS	PARAMETERS	VALUES	UNITS
	MR1	3755.08224	KNm
	MR2	5995.485216	KNm
MR1>Mu	Xu1	0.252608311	m
MR1<Mu<MR2	Xu2	0.207080631	m
if MR1>Mu	Ast1	4911.536753	mm^2
no. of bars		10	bars
MR1<Mu<MR2	Ast3	0	mm^2
	Ast2	0	mm^2
no of bars in tension	n1	0	bars
no of bars in compression	n2	0	bars
if Mu>MR2	Ast4	0	mm^2
	Ast5	0	mm^2
no of bars in tension	n3	0	bars
no of bars in compression	n4	0	bars
	Xu	0.1004573	m
shear design			
Vu		685.68	KN
ζ_v		1.313563218	MPa
100*As/bw*d		0.940907424	
ζc		0.65	MPa
ζc max		3.5	MPa
shear reinforcement provided or not		shear reinforcement provided	
no of shear leg		2	
dia of shear leg		10	mm
Asv		157	mm^2
strength of shear stiffener		685679.6607	N
spacing			
sv1		120	mm
corresponding to min steel		394	mm

svmax		1087	mm
sv3		300	mm
minimun of above		120	mm
nominal shear		0	mm

DETAILING

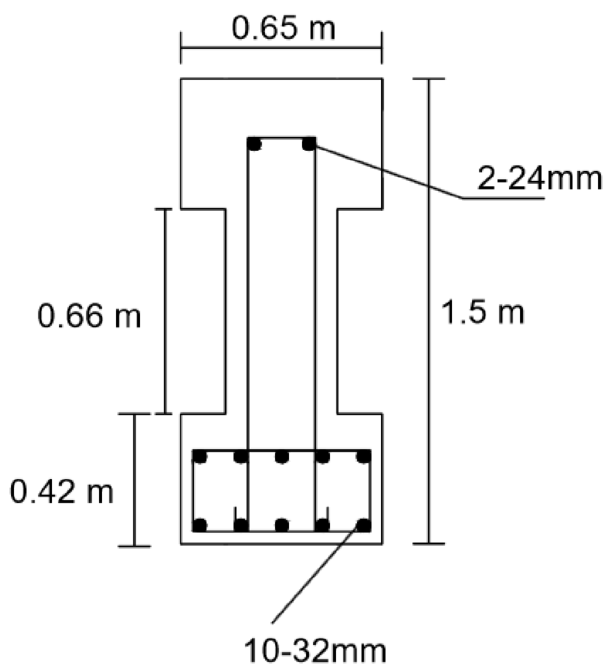


Fig 7.2 Beam Detailing

PIER CAP DESIGN

SIMPLY SUPPORTED CAP

VARIABLE

PARAMETERS	VALUES	UNITS
fck	20	MPa
fy	415	MPa
Fu	1165.5	KN
Mu	773.175	KNm
D1(depth of beam)	800	mm
b(width of beam)	1000	mm
dia of bar a1	25	mm
cc clear cover	40	mm
length of member	3500	mm

RESULTS

PARAMETERS	VALUES	UNITS
effective depth de	747.5	mm
	1542.16725	KNm
design as	under reinforced	
depth check d2	529.2786313	mm
check depth	safe	
% of steel pt	0.420060351	%
area of steel Ast	3139.951121	mm^2
no of bars n1	7	bars
shear design		

ζv	1.559197324	MPa
100*As/bw*d	0.420060351	
ζc	0.45	MPa
ζc max	3.5	MPa
shear reinforcement provided or not	shear reinforcement provided	
no of shear leg	2	mm
dia of shear leg	8	mm
Asv	100.48	mm2
strength of shear stiffener	829125	N
spacing		
sv1	33	mm
corresponding to min steel	91	mm
svmax	561	mm
sv3	300	mm
minimun of above	33	mm
nominal shear	0	mm

CANTILEVER PIER CAP

VARIABLE

PARAMETERS	VALUE	UNIT
fck	20	MPa
fy	415	MPa
Mu Bending moment	2170.8	KN/m
Vu shear force	1076	KN
cc clear cover	25	mm
a1 dia of main bar	32	mm
tan(a) a angle of cantilever T	0.182	
width of cantilever b	1	m
max depth of cantilever D1	1000	mm
length of member	2000	mm

RESULTS

PARAMETERS	VALUE	UNIT
depth of cantilever	886.8606086	mm
overall depth	927.8606086	mm
check	safe	
effective depth de	959	mm
% of steel	0.780480949	%
area main steel Ast1	7484.812298	mm^2
no of bars n1	10	bars
shear design		
ζv	0.69241226	MPa
100*As/bw*d	0.780480949	
ζc	0.8	MPa
ζc max	3.5	MPa

shear reinforcement provided or not	nominal shear provided	
no of shear leg	2	mm
dia of shear leg	10	mm
Asv	157	mm^2
strength of shear stiffener	308800	N
spacing		
sv1	0	mm
corresponding to min steel	0	mm
svmax	0	mm
sv3	0	mm
minimun of above(shear reinforcement)	0	mm
nominal shear	142	mm

PIERDESIGN

VARIABLE

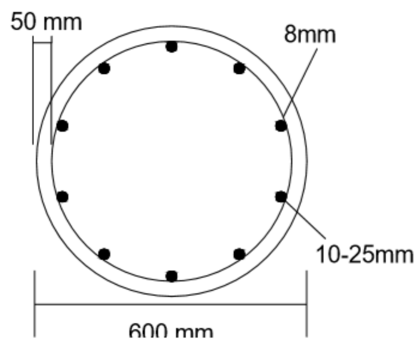
PARAMETERS	VALUES	UNITS
Length of member (L)	5.5	m
dia of member (Dm)	600	mm
factored load P	3441.3	KN
effective length fector K	1	
effective length Leff	4.5	
slenderness ratio	0.0075	
hence design as	short column	
area of concrete Ag	282600	mm^2
fck	20	MPa
fy	415	MPa
cover cc	50	mm

RESULTS

PARAMETERS	VALUES	UNITS
minimum eccentricity		
emin	20.009	mm
design as	axially loaded column	

area of steeAst	4371.412701	mm2
area of concrete Ac	278078.4	mm2
% of steel	1.546855167	%
is b/w 0.8 to 4%	hence ok	
dia of bar	25	mm
number of bars provided	10	
helical reinforcement		
core dia Dc	500	mm
area of core Acr	191728.4	mm^2
dia of spiral a2	8	mm
volume of one spiral per pitch	77614.7712	mm^3
pitch as per IS Code	1434.989256	mm
pitch provided	75	mm

DETAILING



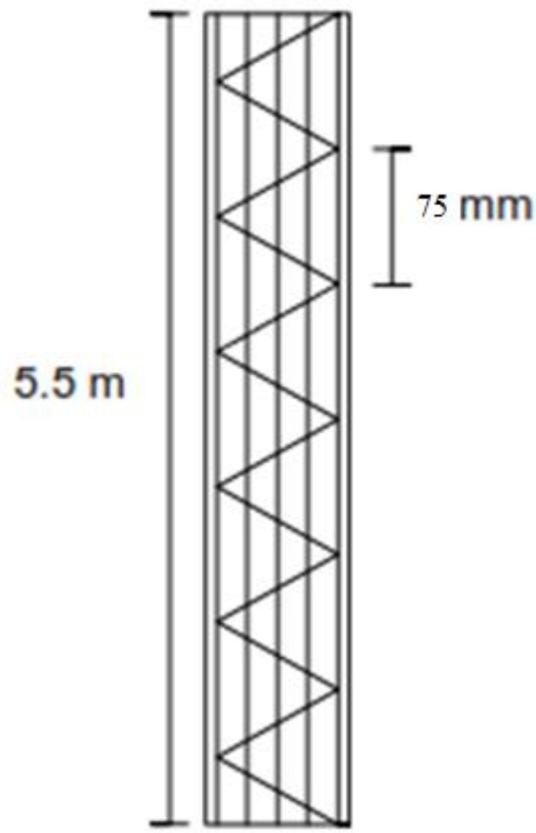


Fig 7.3 Pier Detailing

PILE FOUNDATION DESIGN

VARIABLE

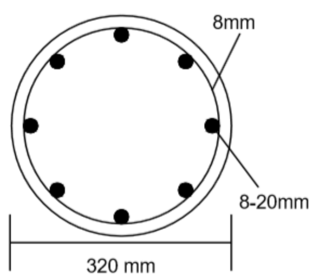
PARAMETERS	VALUE	UNIT
Pu	3500	KN
pile dia (a1)	0.32	m
no of pile (n1)	9	
no of rows (r)	3	
no of colums(c)	3	
length of pile l1	10	m
fck	20	MPa
fy	250	MPa
concrete stress (Pc)	5	MPa
steel stress (Pst)	115	MPa
dia of main bar a2	0.02	m
clear cover cc	0.04	m
dia of tie dt	0.008	m
dia of helical dc	0.008	m
weight W	610	KN
main dia bar of well cap dm	25	mm
destributiodia of well cap dd	16	mm
max S.F. Fu	305	KN
cv tau c from table	0.55	MPa
pile cap length Lp	2200	mm
pile cap width Wp	1500	mm
pile cap thickness tp	0.65	m

RESULTS

PARAMETERS	VALUE	UNIT
pile spacing s1	0.64	m
force on one pile F1	388.8888889	KN
design as long column		
reduction coeffecient (rc)	0.56640625	
corrected Pc	2.83203125	MPa
corrected Pst	65.13671875	MPa
area of main bar Ast	2587.909439	mm
if L<30*a1		
area of min main bar Astm	0	mm
if L>30*a1		
area of min main bar Astm2	1536	mm
no of main bar n2	8	
lateral reinforcement		
volume of one tievt	37860.864	mm^3
pith of pile p	235.5	mm
pith for design is p	150	mm
lateral reinforcement near pile head		
provided fo length of l2	960	mm
reinfocement is 0.6% of gross vol./mm rp	537.6	mm^3
pitch of spiral(helical) p1	59	mm
lateral reinforcement near pile bottom		

provided fo length of l3	960	mm
reinfocement is 0.6% of gross vol./mm rp	537.6	mm^2
volume of one tievt	37860.864	mm^3
pitch of tie p2	70	mm
design of pile cap		
moment due to W is Mz	228.75	KNm
effective depth dcoresponding to fy	277.9935348	mm
id d<600mm		
take d as	600	mm
overall depth D1	650	mm
Ast1 area of man bars of pier cap	2255.91716	mm^2/m
spacing of main bar s2	217	mm
area of distribution bars of pier cap Ast2	780	mm^2/m
spacing of distribution bar s3	258	mm
shear check		
shear stress sv	0.508333333	MPa
shear provided or not	no shear reinforcement provided(sv<cv)	

DETAILING



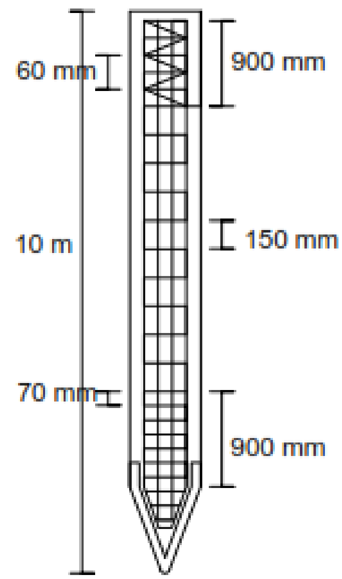


Fig 7.4 Pile Detailing

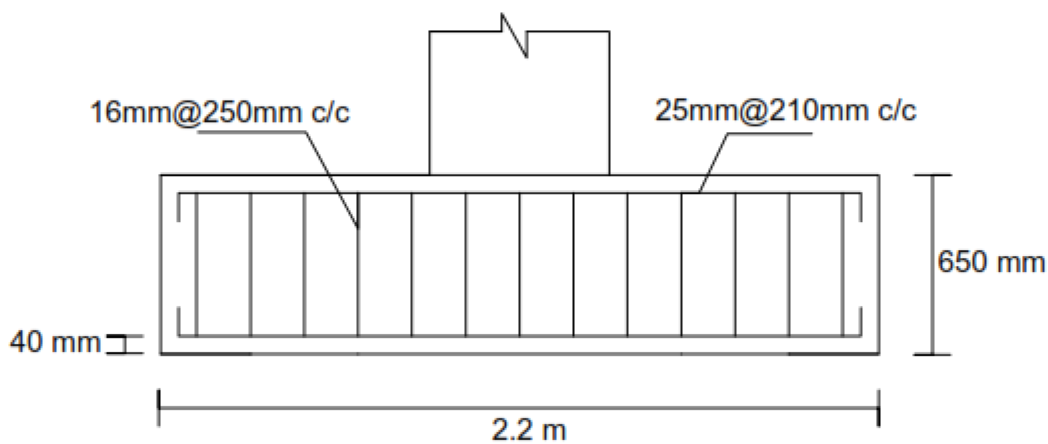


Fig 7.5 Pile Cap Detailing

ABUTMENT DESIGN

VARIABLE

PARAMETERS	VALUES	UNITS
backfill height h1	7	m
ht above water table h2	3.5	m
submerged height of soil h3	4	m
active earth pressure Ka	0.33	
superstructure moment M11	153.4	KNm
bulk unit wt of soil gb	18	KN/ m^3
submerged unit wt of soil gs	10	KN/ m^3
unit wt of water gw	9.81	KN m^3
fck	20	MPa
fy	250	MPa
main bardia d1	32	mm
clear cover cc	50	mm
over all depth Da	800	mm
Pu load on abutment	1150.5	KN
Dt toelength	1.2	m
Dh heel length	2	m
h4	6.5	m
th1 thickness	0.5	m
total width L	4	m
heel with dia of bar a2	16	mm
transvers heeldia a3	8	mm
coefficient of friction u	0.5	
Kp	3	
ds depth of shear key	400	mm
total length Lm	7500	mm

RESULTS

	PARAMETERS	VALUES	UNITS
	moment due to earth pressure M22	349.319055	KNm
	design moment Mu	754.0785825	KNm
	effective depth de	421	mm
	check	safe	
	actual effective depth dea	734	mm
	Mu/(fck*b*d^2)	0.058912389	
	Pu/(fck*b*D)	0.07190625	
	p/fck(from table)	0.04	
	% of steel pt	0.8	%
	area of longitudinal steel Ast1	6400	mm^2
	spacing of bar s1	126	mm
moment from toe			
parameter	weight(KN)	distance(m)	moment(KNm)
concrete abutment(w1)	124.8	1.6	199.68
heel(w2)	24	3	72
toe(w3)	14.4	0.6	8.64
soil(w4)	252	3	3
sum	415.2	8.2	1036.32
	e eccentricity	0.345373447	
	check	safe	
	FOS against overturning	2.966686143	
		safe against overturning	
heel design			
	Mh BM of heel	140.4580251	KNm/m
	main Ast2 of heel	1403.878312	mm^2
	spacing of bar s2	143	mm

	transvers Ast3 of heel	552	mm^2
	spacing of bar s3	91	mm
Toe design			
	Mt BM of toe	60.99641142	KNm/m
	main Ast4 of heel	609.6592846	mm^2
	spacing of bar s4	330	mm
	transvers Ast5 of heel	552	mm^2
	spacing of bar s5	91	mm
check saftey against sliding			
	sliding force Pa	141.2625	KN
	resisting force Fs	207.6	KN
	FOS2 against sliding	1.46960446	
	check	safe against sliding	
	provide	no shear key required	
	Fos after providing shear key if required	`	
	check	safe against sliding	
distribution steel			
	dia od DS sd	12	mm
	area of distribution steel Astd	9000	mm^2
	spacing sd	94	mm

DETAILING

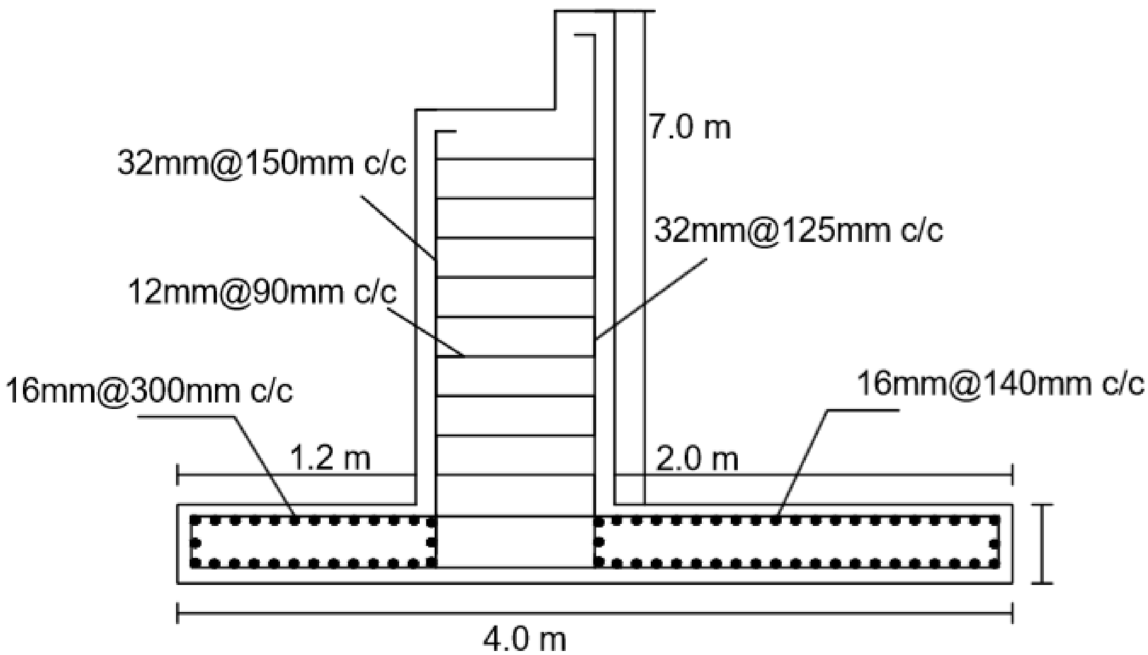


Fig 7.6 Abutment Detailing

CHAPTER – 8

BRIDGE DESIGN IN STAAD PRO

BRIDGE MODEL

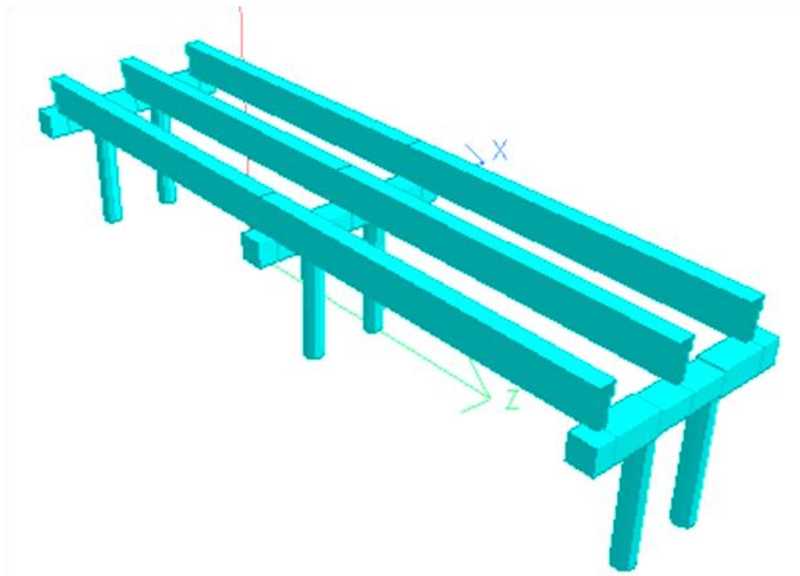


Fig. 8.1 Bridge Model

LOADING

Moving load for class A Vehicle loading andself weight of members

CONCRETE DESIGN

Concrete design using IS456 :2000

RESULTS

BEAM GIRDER

B E A M   G I R D E R D E S I G N R E S U L T S

M20   Fe415 (Main)   Fe250 (Sec.)

LENGTH: 15000.0 mm   SIZE: 400.0 mm X 1500.0 mm   COVER: 25.0 mm

FLANGE WIDTH: 650.0 mm   FLANGE DEPTH: 500.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	3750.0 mm	7500.0 mm	11250.0 mm	15000.0 mm
TOP REINF.	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	2684.86 (Sq. mm)	4552.85 (Sq. mm)	3273.63 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	3750.0 mm	7500.0 mm	11250.0 mm	15000.0 mm
TOP REINF.	3-25í 1 layer(s)	3-25í 1 layer(s)	3-25í 1 layer(s)	3-25í 1 layer(s)	3-25í 1 layer(s)
BOTTOM REINF.	3-25í 1 layer(s)	6-25í 1 layer(s)	10-25í 2 layer(s)	7-25í 1 layer(s)	3-25í 1 layer(s)
SHEAR REINF.	2 legged 10í @ 210 mm c/c	2 legged 10í @ 210 mm c/c	2 legged 10í @ 210 mm c/c	2 legged 10í @ 210 mm c/c	2 legged 10í @ 210 mm c/c

PIER CAP

1. CANTILEVER PIER CAP

CANTILEVER B E A M D E S I G N R E S U L T S

M20 Fe415 (Main) Fe250 (Sec.)

LENGTH: 2750.0 mm SIZE: 1000.0 mm X 1000.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	687.5 mm	1375.0 mm	2062.5 mm	2750.0 mm
TOP REINF.	2048.19 (Sq. mm)	2048.19 (Sq. mm)	2048.19 (Sq. mm)	4513.20 (Sq. mm)	7565.27 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	687.5 mm	1375.0 mm	2062.5 mm	2750.0 mm
TOP REINF.	6-32í 1 layer(s)	6-32í 1 layer(s)	6-32í 1 layer(s)	6-32í 1 layer(s)	10-32í 1 layer(s)
BOTTOM REINF.	3-32í 1 layer(s)	3-32í 1 layer(s)	3-32í 1 layer(s)	3-32í 1 layer(s)	3-32í 1 layer(s)
SHEAR REINF.	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c	2 legged 12í @ 100 mm c/c	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c

2. SIMPLY SUPPORTED PIER CAP

B E A M D E S I G N R E S U L T S

M20 Fe415 (Main) Fe250 (Sec.)

LENGTH: 3000.0 mm SIZE: 1000.0 mm X 1000.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	750.0 mm	1500.0 mm	2250.0 mm	3000.0 mm
TOP REINF.	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	1971.39 (Sq. mm)	3038.09 (Sq. mm)	1971.39 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	750.0 mm	1500.0 mm	2250.0 mm	3000.0 mm
TOP REINF.	5-25í 1 layer(s)	5-25í5-25í5-25í5-25í 1 layer(s)	1 layer(s)	1 layer(s)	1 layer(s)
BOTTOM REINF.	5-25í 1 layer(s)	6-25í 1 layer(s)	7-25í 1 layer(s)	6-25í 1 layer(s)	5-25í 1 layer(s)
SHEAR REINF.	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c	2 legged 12í @ 120 mm c/c

PIER

C O L U M N   D E S I G N   R E S U L T S

M20 Fe415 (Main) Fe250 (Sec.)

LENGTH: 5500.0 mm CROSS SECTION: 800.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 1 END JOINT: 5 SHORT COLUMN

REQD. STEEL AREA : 4148.26 Sq.mm.

REQD. CONCRETE AREA: 498506.59 Sq.mm.

MAIN REINFORCEMENT : Provide 9 - 25 dia. (0.88%, 4417.86 Sq.mm.)

(Equally distributed)

TIE REINFORCEMENT : Provide 10 mm dia. helical ties @ 50 mm c/c

COMPARISON OF STAAD RESULTS WITH VBA RESULTS

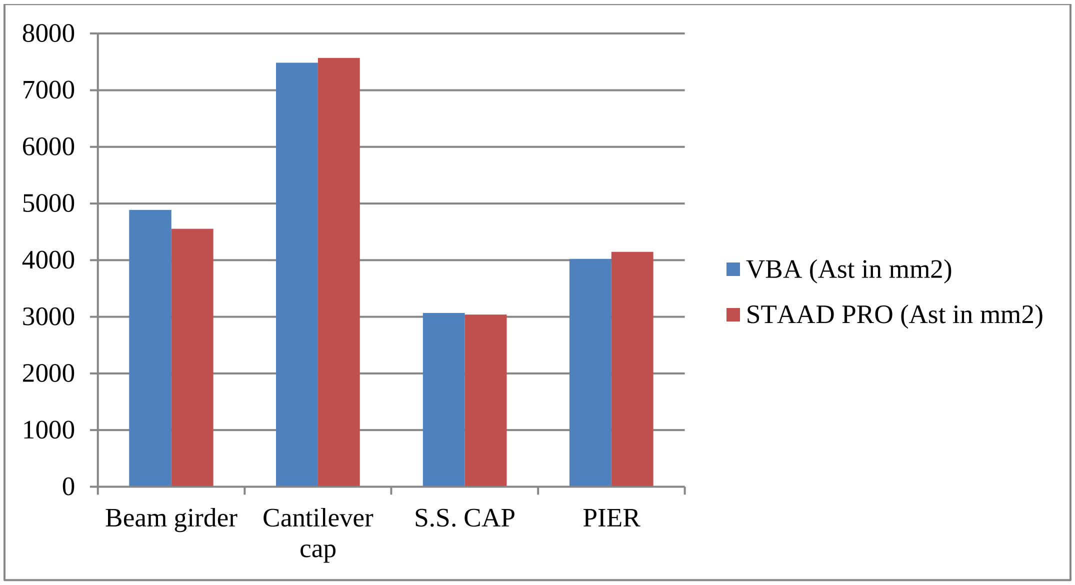


Fig. 8.2 Bar Chart

CHAPTER – 9

VBA PROGRAM FOR STEELBRIDGE DESIGN

1. BEAM MEMBER DESIGN

VARIABLE

PARAMETERS	VALUES	UNITS
UDL w(KN/m)	150.75	KN/m
length of member	3.75	m
bearing width b1	100	mm
Steel Grade Fe	410	
section selection		
fy (yield strength)	250	MPa
fyw(yield strength of web)	250	MPa
maximum bending moment (M)	397.4853516	KNm
maximum shear force (F)	423.984375	KN
plastic section modulus Zpz	2273616.211	mm^3
assumed section	ISMB550	
properties of section		
depth of section h	550	mm
width of flange bf	190	mm
thickness of flange tf	19.3	mm
thickness of web tw	11.2	mm
radius at root R1	18	mm
depth of web d	475.4	mm

moment of inertia Iz	648936000	mm4
plastic section modulus Zpzz	2359800	mm3
elastic section modulus Zez	2359800	mm3

RESULTS

PARAMETERS	VALUES	UNITS
section classification		
b/tf	5	
d/tw	42.44642857	
section	section is plastic	
shear buckling check	S.B. check of web will not be required	
check for shear capacity		
Fd design shear strength of section	808.2903769	KN
	safe in shear	
βb	1	
Md	536.3181818	KNm
moment check	safe in moment capacity	
Mfd	0	KNm
Mdv	0	KNm
safe or not	safe in moment capacity	
deflection check		
deflection limit	9.375	mm
maximum deflection	2.990813336	mm
safe or not	safe in deflection	

check for web bearing		
bearing strength Fw	491.9090909	KN
safe or not	safe in bearing	

2. BOTTOM CHORD MEMBER DESIGN

VARIABLE

PARAMETERS	VALUES	UNITS
member load P	2385	KN
length of member Lm	3000	mm
section ISMB	550	
cross section details		
section depth h	550	mm
gross area Ag	13211	mm^2
Net area An	13211	mm^2
width of outstanding leg w(flange)	190	mm
thickness of web tw	11.2	mm
gross area of outstanding leg Ago	7334	mm^2
net area of connected leg Anc	6160	mm^2
thickness of flange tf	19.3	mm
shear leg width bs=w	190	mm
length of end connection Lc	550	mm
material property		
fy	250	MPa
fu	410	MPa
saftey factor		
f0	1.1	
f1	1.25	
fmw	1.25	

RESULTS

PARAMETERS	VALUES	UNITS
design strength due to gross section yielding		
Tdg	3002.5	KN
design strength due to net section rupture		
β<=(fu/fy)*(f0/f1)	1.128420969	
β>=0.7	1.128420969	
Tdn	3699.304588	KN
weld design		
gusset plat thikness	12	mm
size of weld	8	mm
total weld length	2248.922038	mm
weld length in tension Lwt	550	mm
weld length in shear Lws	1698.922038	mm
design strength due to block shear		
Avg=Avn	20387.06446	mm^2
Atg=Atn	6600	mm^2
Tdb1	4623.506917	KN
Tdb2	4894.011634	KN
Tdb	4623.506917	KN
check		
safe or not	section is safe in tension	

3. VERTICAL COMPRESSION MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	1020	KN
section type ISMC	350	sheet 2
sectional area Ae	5366	mm^2
radius of gyration r	136.6	mm
buckling class	c	sheet 3
α imperfection fector (x)	0.49	
effective length factor of member	1	SHEET 4
effective length of member	5000	mm
material property		
fy	250	MPa
f0	1.1	
elastic modulus E	200000	MPa

RESULTS

PARAMETERS compression design	VALUES	UNITS
fcc(euler bulking stress)	1471.806382	MPa
λ (effective slenderness ratio)	0.412139902	
fcd	202.4731512	MPa
Pd(design compressive strength)	1086.47093	KN
design is safe or not	Design is safe	

4. CORNER INCLINED MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	1682.25	KN
section type ISMB	550	sheet 2
sectional area Ae	13211	mm^2
radius of gyration r	221.6	mm
buckling class	a	sheet 3
α imperfection fector (x)	0.21	
effective length factor of member	1	SHEET 4

effective length of member	5831	mm
material property		
fy	250	MPa
f0	1.1	
elastic modulus E	200000	MPa

RESULTS

PARAMETERS compression design	VALUES	UNITS
fcc(euler bulking stress)	2848.017943	MPa
λ (effective slenderness ratio)	0.296277481	
fcd	222.3547378	MPa
Pd(design compressive strength)	2937.528441	KN
design is safe or not	Design is safe	

5. INCLINED TENSION MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	1326	KN
length of member Lm	5831	mm
section ISMC	400	
cross section details		
section depth h	400	mm
gross area Ag	6293	mm^2
Net area An	6293	mm^2
width of outstanding leg w(flange)	100	mm
thickness of web tw	8.6	mm
gross area of outstanding leg Ago	3060	m mm^2
net area of connected leg Anc	3440	mm^2
thickness of flange tf	15.3	mm
shear leg width bs=w	100	mm
length of end connection Lc	400	mm
material property		
fy	250	MPa
fu	410	MPa
		MPa
safety factor		
f0	1.1	
f1	1.25	
fmw	1.25	

RESULTS

PARAMETERS	VALUES	UNITS
design strength dur to gross section yielding		
Tdg	1430.227273	KN
design strength due to net section rupture		
β<=(fu/fy)*(f0/f1)	1.265286444	
β>=0.7	1.265286444	
Tdn	1895.437208	KN
weld design		
gusset plat thickness	12	mm
size of weld	8	mm
total weld length	1250.344077	mm
weld length in tension Lwt	400	mm
weld length in shear Lws	850.3440767	mm
design strength due to block shear		
Avg=Avn	10204.12892	mm^2
Atg=Atn	4800	mm^2
Tdb1	2755.943954	KN
Tdb2	2789.679031	KN
Tdb	2755.943954	KN
check		
safe or not	section is safe in tension	

6. TOP CHORD COMPRESSION MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	2425.5	KN
section type ISMB	550	sheet 2
sectional area Ae	13211	mm^2
radius of gyration r	221.6	mm
buckling class	a	sheet 3
α imperfection factor (x)	0.21	
effective length factor of member	1	SHEET 4
effective length of member	3000	mm
material property		
fy	250	MPa
f0	1.1	
elastic modulus E	200000	MPa

RESULTS

PARAMETERS compression design	VALUES	UNITS
fcc(Euler bulking stress)	10759.35642	MPa
λ (effective slenderness ratio)	0.152432249	
fcd	227.2727273	MPa

Pd(design compressive strength)	3002.5	KN
design is safe or not	Design is safe	

7. TOP HORIZONTAL MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	265.65	KN
length of member Lm	3750	mm
section ISMC	150	
cross section details		
section depth h	150	mm
gross area Ag	2088	mm^2
Net area An	2088	mm^2
width of outstanding leg w(flange)	75	mm
thickness of web tw	5.4	mm
gross area of outstanding leg Ago	1350	mm^2
net area of connected leg Anc	810	mm^2
thickness of flange tf	9	mm
shear leg width bs=w	75	mm
length of end connection Lc	150	mm
material property		
fy	250	MPa
fu	410	MPa
safety factor		
f0	1.1	
f1	1.25	
fmw	1.25	

RESULTS

PARAMETERS	VALUES	UNITS
design strength dur to gross section yielding		
Tdg	474.5454545	KN
design strength due to net section rupture		
β<=(fu/fy)*(f0/f1)	1.078184282	
β>=0.7	1.078184282	
Tdn	569.918541	KN
weld design		
gusset plat thikness	12	mm
size of weld	8	mm
total weld length	250.4931402	mm
weld length in tension Lwt	150	mm
weld length in shear Lws	100.4931402	mm
design strength due to block shear		
Avg=Avn	1205.917683	mm^2
Atg=Atn	1800	mm^2
Tdb1	689.6003006	KN
Tdb2	609.8504963	KN
Tdb	609.8504963	KN
check		
safe or not	section is safe in tension	

8. TOP BRACING MEMBER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
member load P	178.2	KN
section ISMC	150	
cross section details		
section depth h	150	mm
gross area Ag	2088	mm^2
Net area An	2088	mm^2
width of outstanding leg w(flange)	75	mm
thickness of web tw	5.4	mm
gross area of outstanding leg Ago	1350	mm^2
net area of connected leg Anc	810	mm^2
thickness of flange tf	9	mm
shear leg width bs=w	75	mm
length of end connection Lc	150	mm
material property		
fy	250	MPa
fu	410	MPa
saftey factor		
f0	1.1	
f1	1.25	
fmw	1.25	
sectional area Ae	2088	mm^2
radius of gyration r	61.1	mm
buckling class	c	sheet 3
α imperfection fector (x)	0.49	

effective length factor of member	1	SHEET 4
effective length of member	4802	mm
elastic modulus E	20000000	MPa

RESULTS

PARAMETERS	VALUES	UNITS
TENSION MEMBER DESIGN		
design strength dur to gross section yielding		
Tdg	474.5454545	KN
design strength due to net section rupture		
β<=(fu/fy)*(f0/f1)	1.078184282	
β>=0.7	1.078184282	
Tdn	569.918541	KN
weld design		
gusset plat thikness	10	mm
size of weld	6	mm
total weld length	224.043554	mm
weld length in tension Lwt	150	mm
weld length in shear Lws	74.04355401	mm
design strength due to block shear		
Avg=Avn	740.4355401	mm^2
Atg=Atn	1500	mm^2
Tdb1	539.9598179	KN
Tdb2	464.1758242	KN
Tdb	464.1758242	KN
safe or not	section is safe in tension	
COMPRESSION MEMBER DESIGN		

fcc(euler bulking stress)	319.2474234	MPa
λ (effective slenderness ratio)	0.884924673	
fcd	138.4369188	MPa
Pd(design compressive strength)	289.0562864	KN
design is safe or not	Design is safe	
safe or not in both tension and compression	member is safe in C and T	

9. BOTTOM BRACING MEMBER DESIGN

VARIABLES

member load P	176.75	KN
section ISMC	150	
cross section details		
section depth h	150	mm
gross area Ag	2088	mm^2
Net area An	2088	mm^2
width of outstanding leg w(flange)	75	mm
thickness of web tw	5.4	mm
gross area of outstanding leg Ago	1350	mm^2
net area of connected leg Anc	810	mm^2
thickness of flange tf	9	mm
shear leg width bs=w	75	mm
length of end connection Lc	150	mm
material property		

fy	250	MPa
fu	410	MPa
saftey factor		
f0	1.1	
f1	1.25	
fmw	1.25	
sectional area Ae	2088	mm^2
radius of gyration r	61.1	mm
buckling class	a	sheet 3
α imperfection fector (x)	0.21	
effective length factor of member	1	SHEET 4
effective length of member	4802	mm
elastic modulus E	20000000	MPa

RESULTS

TENSION MEMBER DESIGN design strength dur to gross section yielding		
Tdg	474.5454545	KN
design strength due to net section rupture		
β<=(fu/fy)*(f0/f1)	1.078184282	
β>=0.7	1.078184282	
Tdn	569.918541	KN
weld design		
gusset plat thikness	10	mm
size of weld	6	mm
total weld length	222.2205285	mm
weld length in tension Lwt	150	mm

weld length in shear Lws	72.22052846	mm
design strength due to block shear		
Avg=Avn	722.2052846	mm^2
Atg=Atn	1500	mm^2
Tdb1	537.567647	KN
Tdb2	461.1408753	KN
Tdb	461.1408753	KN
safe or not	section is safe in tension	
COMPRESSION MEMBER DESIGN		
fcc(euler bulking stress)	319.2474234	MPa
λ (effective slenderness ratio)	0.884924673	
fcd	169.0421632	MPa
Pd(design compressive strength)	352.9600367	KN
design is safe or not	Design is safe	
safe or not in both tension and compression	member is safe in C and T	

10. SLAB DESIGN

VARIABLES

PARAMETERS	VALUE	UNIT
fck	30	MPa
fy	250	MPa
A0 from table	1.68	m
concentrated force from nearest support a1 (27KN)	1.1	m
concentrated force from nearest support a2 (27KN)	2.2	m
concentrated force from nearest support a3 (114KN)	5.4	m
concentrated force from nearest support a4 (114KN)	6.6	m
concentrated force from nearest support a5 (68KN)	1.1	m
l0 effective span	12	m
width of tyre in contact with surface w1(27KN)	0.2	m
width of tyre in contact with surface w2(114KN)	0.5	m
width of tyre in contact with surface w3(68KN)	0.38	m
bitumen thickness th1	0.08	m
slab width wt	3.75	m
(ML)using staad pro we get BM due to F1,F2,F3,F4,F5,F6	277	KNm
thickness of slab th2	0.5	m
cc clear cover	40	mm
c1 dia of bar	32	mm

RESULTS

PARAMETERS moment calculation	VALUE	UNIT
bef1(27KN)	3.1186	m
bef2(27KN)	4.4584	m
bef3(114KN)	7.6296	m
bef4(114KN)	7.6296	m

bef5(68KN)	3.8386	m
load /meter width F1	17.31546207	KN/m
load /meter width F2	14.4	KN/m
load /meter width F3	60.8	KN/m
load /meter width F4	60.8	KN/m
load /meter width F5	36.26666667	KN/m
avg live load per meter Fav	37.91642575	KN/m
Mli BM due to live load including impact	346.25	KNm/m
dead load per m length Ld	13.76	KN/m
BM due to DL Md	24.1875	KNm/m
total BM (ll+d) is Mu	370.4375	KNm/m
depth check		
overall depth D1	344.8457536	mm
depth check	safe	
effective thicknes of slab th3	444	mm
design		
% of steel pt	0.937649833	%
area of main steel bar Ast1	4163.165258	mm^2
spacing of main bar s1	193	mm
distribution steel		
dia od DS sd	12	mm
area of distribution steel Astd	750	mm^2
spacing sd	151	mm
shear design		
Vu	568.7463862	KN
ζv	1.236405187	MPa
100*As/bw*d	0.905035926	%
ζc	1.15	MPa
ζc max	3.2	MPa
shear reinforcement provided or not	shear reinforcement required	

11. SS PIER CAP DESIGN

VARIABLES

PARAMETERS	VALUE	UNIT
Fck	20	MPa
Fy	415	MPa
Fu	1672.5	KN
Mu	1463.45	KNm
D1(depth of beam)	1000	mm
b(width of beam)	1000	mm
dia of bar a1	28	mm
cc clear cover	40	mm
length of member	3500	mm

RESULTS

PARAMETERS	VALUE	UNIT
effective depth de	946	mm
limiting BM	2469.96816	KNm
design as	under reinforced	
depth check d2	728.172718	mm
check depth	safe	
% of steel pt	0.506355715	%
area of steel Ast	4790.125063	mm^2
no of bars n1	9	bars
shear design		
ζ_v	1.767970402	MPa
100*As/bw*d	0.506355715	
ζc	0.8	MPa

ζc max	3.5	MPa
shear reinforcement provided or not	shear reinforcement provided	
no of shear leg	2	mm
dia of shear leg	12	mm
Asv	226.08	mm^2
strength of shear stiffener	915700	N
spacing		
sv1	84	mm
corresponding to min steel	204	mm
svmax	710	mm
sv3	300	mm
minimum of above	84	mm
nominal shear	0	mm

12. CANTILEVER PIER CAP DESIGN

VARIABLES

PARAMETERS	VALUE	UNIT
Fck	20	MPa
Fy	415	MPa
Mu Bending moment	2131.8	KNm
Vu shear force	1254	KN
cc clear cover	40	mm
a1 dia of main bar	28	mm
tan(a) a angle of cantilever T	0.182	
width of cantilever b	1	m
max depth of cantilever D1	1000	mm
length of member	2000	mm

RESULTS

PARAMETERS	VALUE	UNIT
depth of cantilever	878.8579546	mm
overall depth	932.8579546	mm
check	safe	
effective depth de	946	mm
% of steel	0.789415604	%
area main steel Ast1	7467.871617	mm^2
no of bars n1	13	bars
shear design		
ζv	0.892035007	MPa
100*As/bw*d	0.789415604	
ζc	0.8	MPa
ζc max	3.5	MPa
shear reinforcement provided or not	shear reinforcement provided	
no of shear leg	2	mm
dia of shear leg	10	mm
Asv	157	mm^2
strength of shear stiffener	497200	N
spacing		
sv1	108	mm
corresponding to min steel	142	mm
svmax	710	mm
sv3	300	mm
minimum of above(shear reinforcement)	108	mm
nominal shear	0	mm

13. PIER DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
Length of member (L)	5.5	m
dia of member (Dm)	750	mm
factored load P	4985.65	KN
effective length factor K	1	
effective length Leff	5.5	
slenderness ratio	0.007333333	
hence design as	short column	
area of concrete Ag	441562.5	mm^2
fck	20	MPa
fy	415	MPa
cover cc	50	mm

RESULTS

PARAMETERS	VALUES	UNITS
minimum eccentricity		
emin	25.011	mm
design as	axially loaded column	
area of steeAst	5381.040548	mm^2
area of concrete Ac	435684.42	mm^2
% of steel	1.218636218	%
is b/w 0.8 to 4%	hence ok	
dia of bar	25	mm
number of bars provided	12	
helical reinforcement		
core dia Dc	650	mm
area of core Acr	325784.42	mm^2
dia of spiral a2	8	mm
volume of one spiral per pitch	101277.8112	mm^3
pitch as per IS Code	1328.116381	mm
pitch provided	75	mm

DETAILING

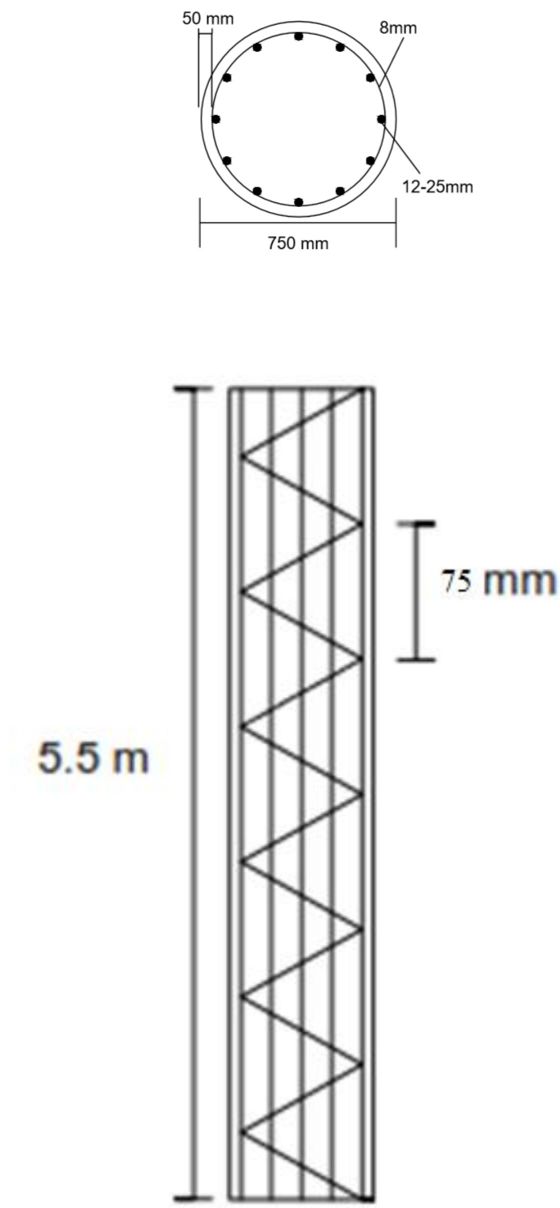


Fig. 9.1 Detailing Of Pier

14. PILE FOUNDATION DESIGN

VARIABLES

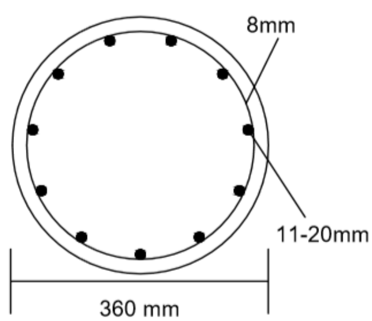
PARAMETERS	VALUE	UNIT
Pu	5200	KN
pile dia (a1)	0.36	m
no of pile (n1)	9	
no of rows (r)	3	
no of columns(c)	3	
length of pile l1	10	m
Fck	20	MPa
Fy	250	MPa
concrete stress (Pc)	5	MPa
steel stress (Pst)	115	MPa
dia of main bar a2	0.02	m
clear cover cc	0.04	m
dia of tie dt	0.008	m
dia of helical dc	0.008	m
weight W	610	KN
main dia bar of well cap dm	25	mm
Distributiondia of well cap dd	16	mm
max S.F. Fu	305	KN
cv tau c from table	0.55	MPa
pile cap length Lp	2520	mm
pile cap width Wp	2520	mm
pile cap thickness tp	0.65	m

RESULTS

PARAMETERS	VALUE	UNIT
pile spacing s1	0.72	m
force on one pile F1	577.7777778	KN
design as long column		
reduction coefficient (rc)	0.642361111	
corrected Pc	3.211805556	MPa
corrected Pst	73.87152778	MPa
area of main bar Ast	3552.540541	mm
if L<30*a1		
area of min main bar Astm	1296	mm
if L>30*a1		
area of min main bar Astm2	0	mm
no of main bar n2	11	
lateral reinforcement		
volume of one tievt	44171.008	mm^3
pith of pile p	217.0864198	mm
pith for design is p	150	mm
lateral reinforcement near pile head		
provided fo length of l2	1080	mm
reinforcement is 0.6% of gross vol./mm rp	680.4	mm^3
pitch of spiral(helical) p1	56	mm
lateral reinforcement near pile bottom		
provided fo length of l3	1080	mm

reinforcement is 0.6% of gross vol./mm rp	680.4	mm^3
volume of one tievt	44171.008	mm^3
pitch of tie p2	65	mm
design of pile cap		
moment due to W is Mz	228.75	KNm
effective depth dcorresponding to fy	277.9935348	mm
id d<600mm		
take d as	600	mm
overall depth D1	650	mm
Ast1 area of man bars of pier cap	2255.91716	mm^2/m
spacing of main bar s2	200	mm
area of distribution bars of pier cap Ast2	780	mm^2/m
spacing of distribution bar s3	258	mm
shear check		
shear stress sv	0.508333333	MPa
shear provided or not	no shear reinforcement provided(sv<cv)	

DETAILING



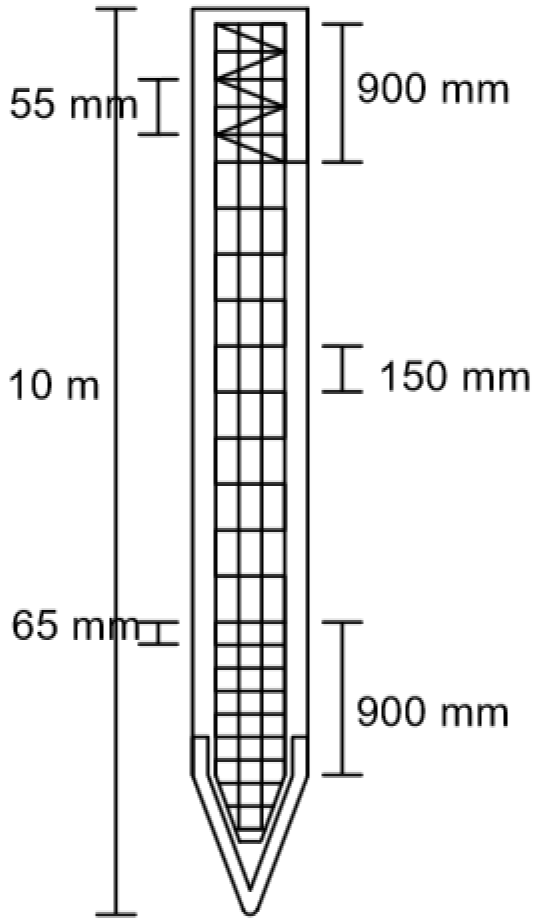


Fig. 9.2 Detailing Of Pile

15. ABUTMENT DESIGN

VARIABLES

PARAMETERS	VALUES	UNITS
backfill height h1	7	m
ht above water table h2	3.5	m
submerged height of soil h3	4	m
active earth pressure Ka	0.33	
superstructure moment M11	334.5	KNm
bulk unit wt of soil gb	18	KN/ m^3
submerged unit wt of soil gs	10	KN/ m^3
unit wt of water gw	9.81	KN/ m^3
Fck	20	MPa
Fy	250	MPa
main bar dia d1	32	mm
clear cover cc	50	mm
over all thickness Da	800	mm
Pu load on abutment	1672.5	KN
eccentricity of load	0.2	m
Dt totel lengh	1.2	m
Dh heel length	2	m
h4	6.5	m
th1 thickness	0.5	m
total width L	4	m
heel with dia of bar a2	16	mm
transvers heeldia a3	8	mm
coefficient of friction u	0.5	
Kp	3	
ds depth of shear key	400	mm
total length Lm	7500	mm

RESULTS

	PARAMETERS	VALUES	UNITS
	moment due to earh pressure M22	349.319055	KNm
	design moment Mu	1025.728583	KNm
	effective depth de	421	mm
	Check	safe	
	actual effective depth dea	734	mm
	Mu/(fck*b*d^2)	0.080135046	
	Pu/(fck*b*D)	0.10453125	
input	p/fck(from table)	0.045	
	% of steel pt	0.9	%
	area of longitudinal steel Ast1	7200	mm^2
	spacing of bar s1	112	mm
moment from toe			
parameter	weight(KN)	distance(m)	moment(KNm)
concrete abutment(w1)	124.8	1.6	199.68
heel(w2)	24	3	72
toe(w3)	14.4	0.6	8.64
soil(w4)	252	3	3
sum	415.2	8.2	1036.32
	e eccentricity	0.345373447	
	Check	safe	
	FOS against overturning	2.966686143	
		safe against overturning	
heel design			

	Mh BM of heel	140.4580251	KNm/m
	main Ast2 of heel	1403.878312	mm^2
	spacing of bar s2	143	mm
	transvers Ast3 of heel	552	mm^2
	spacing of bar s3	91	mm
Toe design			
	Mt BM of toe	60.99641142	KNm/m
	main Ast4 of heel	609.6592846	mm^2
	spacing of bar s4	330	mm
	transvers Ast5 of heel	552	mm^2
	spacing of bar s5	91	mm
check saftey against sliding			
	sliding force Pa	141.2625	KN
	resisting force Fs	207.6	KN
	FOS2 against sliding	1.46960446	
	Chek	safe against sliding	
	Provide	no shear key required	
	Fos after providing shear key if required	1.46960446	
	Check	safe against sliding	
distribution steel			
	dia od DS sd	12	mm
	area of distribution steel Astd	9000	mm^2
	spacing sd	94	mm

DETAILING

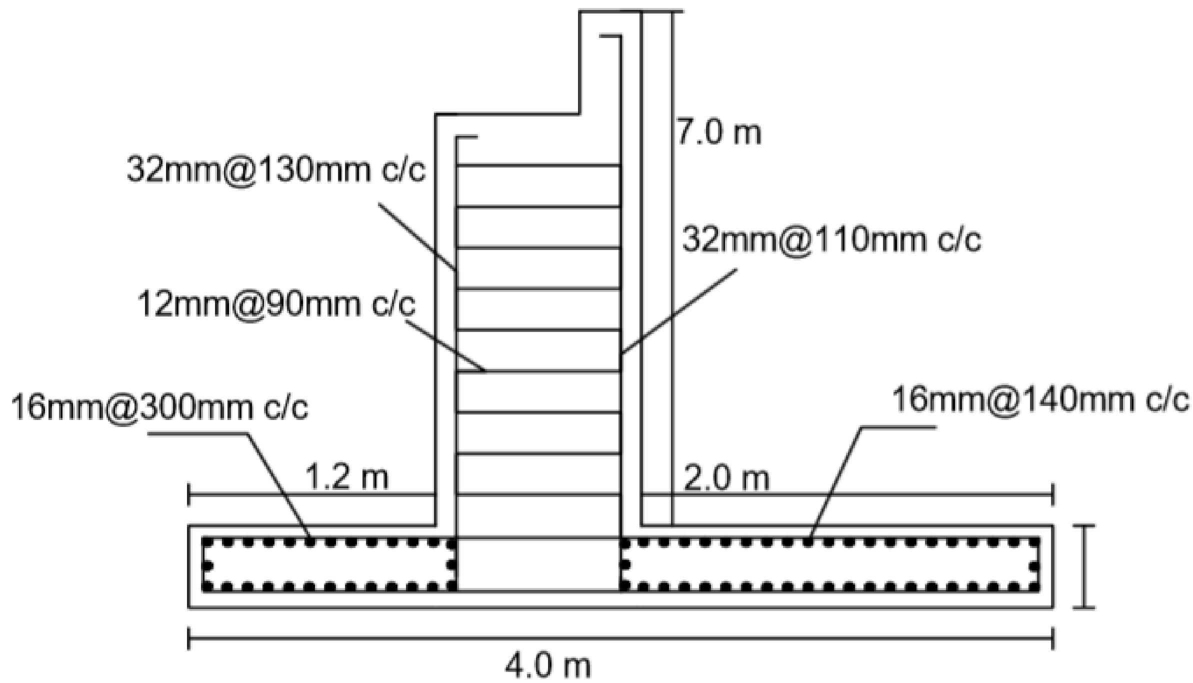


Fig. 9.3 Detailing Of Abutment

CONECTIONS

BOTTOM CORNER CONNECTION

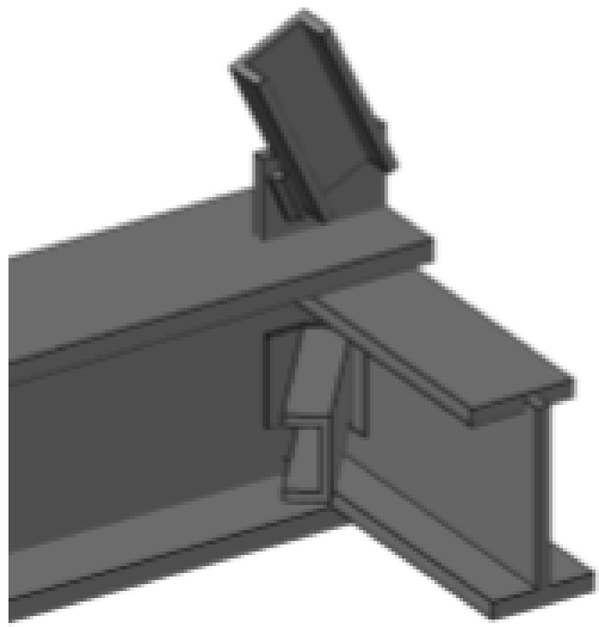


Fig. 9.4. Connection of truss member at corner edge

BOTTOM MIDDLE CONNECTION

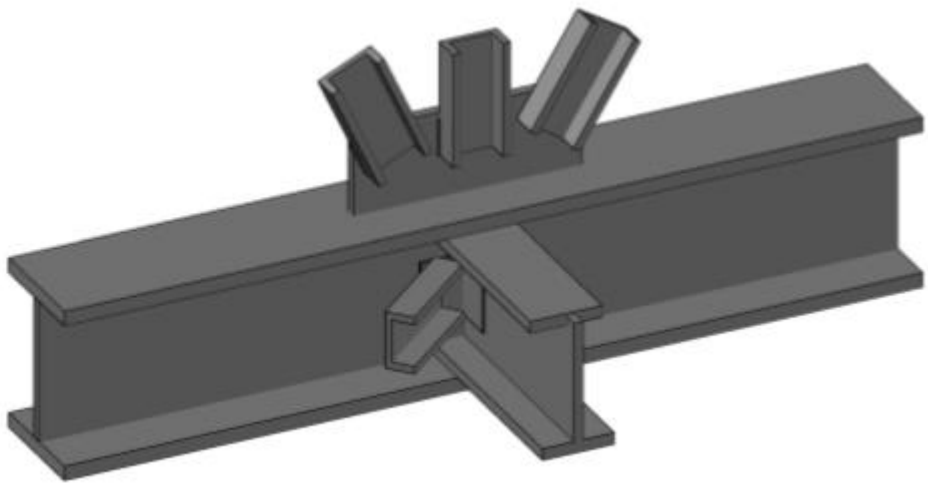


Fig. 9.5. Connection of truss member at middle edge

BOTTOM MIDDLE SIDE CONNECTION

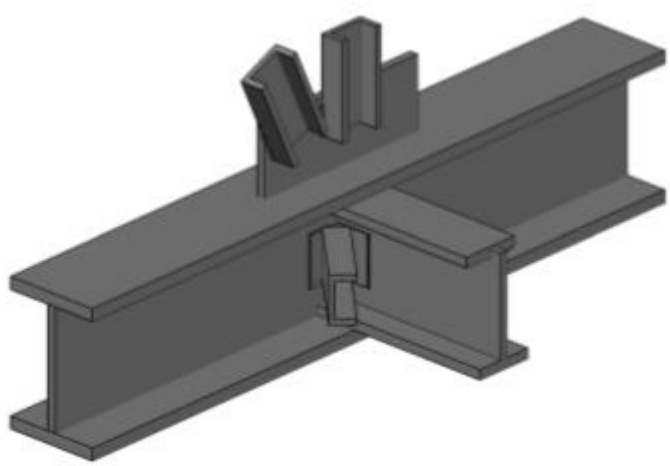


Fig. 9.6. Connection of truss member at middle side edge

CONCLUSIONS

In this project, the RCC bridge over Hindon river in Ghaziabad, Uttar Pardesh has been analyzed and then designed. After analysis, the results were found to be in limits and reliable. The report involves different management aspects too which are taken into consideration at the time of designing the bridges as per IRC

The bridge designing is done with the help of VBA Excel programming. These programming cover all the design consideration of bridge and also facilitate any future changes in data without any change in programming. This is easy to use and give reliable results.

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D. Johanson Victor, Essentials of Bridge Engineering

B.C. Punmia, Ashok Kumar Jain and Arun Kumar Jain, limit state design of reinforced concrete

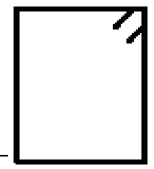
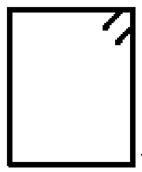
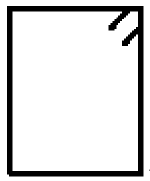
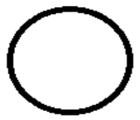
S.K. Duggal, limit state design of steel structures, Green park Extension, New Delhi

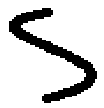
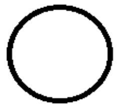
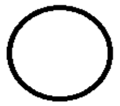
http://www.excel-easy.com/vba.html

APPENDIX A :

ESTIMATION OF MATERIAL OF RCC BRIDGE

BAR BENDING SCHEDULE

sr.no	member name	bar mark	bar shape	bar dia (mm)	No. of member	No. of bars	bar length (mm)	subtracted length (mm)	overlap length (mm)	actual bar length(mm)	total bar length(mm)	total bar weight(Kg)
1	slab	longitudinal bar		32	12	62	14920	64	1280	16136	12005184	75271.92743
		distribution bar		12	12	80	7420	48	0	7372	7077120	6239.981629
2	beam	main bar		25	33	10	14920	100	1000	15820	5220600	19978.58363
		shear bar		10	33	100	3620	100	80	3600	11880000	7274.124
3	ss pear cap	main bar		25	11	7	3420	0	0	3420	263340	1007.769263
		shear bar		8	11	106	3280	96	64	3248	3787168	1484.085098
4	cantilever P cap	main bar		32	22	10	1950	128	0	1822	400840	2319.920824
		shear bar		10	22	14	3800	120	80	3760	1158080	709.092384
5	pier	longitudinal bar		25	22	9	4500	0	0	4500	891000	50278.74509
		hoop bar		8	22	12	25.12	0	64	89.12	23527.68	9.219839017

		helical bar		10	22	1	4500.11	0	0	4500.11	99002.4101	60.6191757
6	abutment	longitudinal bar		32	2	119	6900	128	256	7028	1672664	10487.52299
		distribution bar		12	2	138	7400	48	0	7352	2029152	1789.127668
		heel main bar		16	2	105	1950	64	128	2014	422940	662.9533747
		heel transverse bar		8	2	44	7400	32	0	7368	648384	254.0835348
		toe main bar		16	2	45	1150	64	128	1214	109260	171.2637389
		toe transverse bar		8	2	26	7400	32	0	7368	383136	150.1402706
7	foundation	longitudinal bar		20	132	7	9960	0	0	9960	9203040	22540.08557
		distribution bar		8	132	55	152	0	0	152	1103520	432.4385894
		pile head lateral bar		8	132	15	690.8	16	64	738.8	1462824	573.2397665
		pile bottom lateral bar		8	132	12	690.8	16	64	738.8	1170259.2	458.5918132
		pier cap main bar		25	22	14	2120	100	200	2220	683760	2616.66405
		pier cap transverse bar		16	22	17	1420	64	0	1356	507144	794.9421343
8		total										183025.0363
9		increase by 10%										18302.5036
10		final total										201327.54

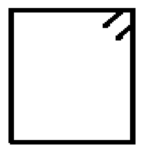
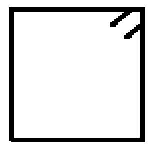
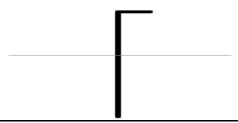
RCC WORK

sr. No	particulars of items and and details	number of member	length (m)	breadth/diameter (m)	height or depth (m)	quantity(cum)	explanatory notes
	R.C.C work 1:2:4 excluding steel and its bending						
1	slab	12	15	7.5	0.4	540	no deduction for volume of steel
2	beam	66	15	0.65	0.42	270.27	no deduction for volume of steel
		33	15	0.36	1.49	265.518	
3	pier cap	11	3.5	1	0.8	30.8	no deduction for volume of steel
4	cantilever cap	22	2	1	1	44	no deduction for volume of steel
5	pier	22		0.6	4.5	27.9774	no deduction for volume of steel
6	abutment	2	7.5	0.8	6.5	78	no deduction for volume of steel
		2	7.5	2	0.5	15	
		2	7.5	1.2	0.5	9	
7	pile	132		0.32	10	106.10688	no deduction for volume of steel
8	pile cap	22	2.2	1.5	0.65	47.19	
9	total					1433.86228	
10	increase by 15%					215.079342	
11	final total					1648.941622	

APPENDIX B :

ESTIMATION OF MATERIAL OF STEEL BRIDGE

BAR BENDING SCHEDULE

sr.no	member name	bar mark	bar shape	bar dia (mm)	number of member	number of bars	bar length (mm)	subtracted bar length (mm)	overlap length (40*dia) (mm)	actual bar length required(mm)	total bar length(mm)	total bar weight(Kg)
1	slab	longitudinal bar		32	30	19	11920	64	1792	13648	7779360	48776.21379
		distribution bar		12	30	79	3670	48	0	3622	8584140	7568.739248
2	ss pear cap	main bar		28	4	9	3420	112	0	3308	119088	571.673846
		shear bar		12	4	42	3680	144	96	3632	610176	537.9995013
3	cantilever P cap	main bar		28	8	13	1920	112	0	1808	188032	833.2013814
		shear bar		10	8	19	3680	120	80	3640	553280	338.773344
4	pier	longitudinal bar		28	8	18	5500	0	0	5500	792000	60831.0743
		hoop bar		8	8	14	25.12	0	64	89.12	9981.44	3.911446856
		helical bar		10	8	1	5500.089632	0	0	5500.089632	44000.71706	26.94163905
5	abutment	longitudinal bar		20	48	11	9960	0	0	9960	5258880	12880.0489

		distribution bar		8	48	52	152	0	0	152	379392	148.6731018
		heel main bar		8	48	19	879.2	16	64	927.2	845606.4	331.3694712
		heel transverse bar		8	48	17	879.2	16	64	927.2	756595.2	296.4884742
		toe main bar		25	8	25	2440	192	0	2248	449600	1585.670861
		toe transverse bar		16	8	20	2440	128	0	2312	369920	579.845161
6	foundation	longitudinal bar		32	2	134	6900	128	0	6772	1814896	11379.3108
		distribution bar		12	2	138	7400	48	0	7352	2029152	1789.127668
		pile head lateral bar		16	2	105	1950	64	0	1886	396060	620.8192973
		pile bottom lateral bar		8	2	44	7400	32	0	7368	648384	254.0835348
		pier cap main bar		16	2	45	1150	64	0	1086	97740	153.2062771
		pier cap transverse bar		8	2	26	7400	32	0	7368	383136	150.1402706
7	total											149657.2668
8	increase by 10%											14965.7267
9	final total											164622.9935

STEEL WORK

sr no.	number of span	member name	member classification	weight in Kg/m	number of member	member length (m)	total member length(m)	total member weight(Kg)
1	5	beam	ISMB550	103.7	26	3.75	487.5	50553.75
2	5	Bottom chord member	ISMB550	103.7	36	3	540	55998
3	5	vertical member	ISMC350	42.1	33	5	825	34732.5
4	5	corner inclined member	ISMB550	103.7	6	5.831	174.93	18140.241

5	5	inclined tension member	ISMC400	49.4	30	5.831	874.65	43207.71	
6	5	top chord tension member	ISMB550	103.7	30	3	450	46665	
7	5	top horizontal member	ISMC150	16.4	22	3.75	412.5	6765	
8	5	top bracing member	ISMC150	16.4	48	4.802	1152.48	18900.672	
9	5	Top beam	ISMC150	16.4	20	4.802	1152.48	7875.28	
10	total							282838.153	
11	increase by 10%							28283.8153	
12	final total							311121.9683	

RCC WORK

sr. No	particulars of items and and details	number of member	length (m)	breadth/diameter (m)	height or depth (m)	quantity(cum)	explanatory notes
	R.C.C work 1:2:4 excluding steel and its bending						
1	slab	12	15	7.5	0.4	540	no deduction for volume of steel
3	pier cap	11	3.5	1	0.8	30.8	no deduction for volume of steel
4	cantilever cap	22	2	1	1	44	no deduction for volume of steel
5	pier	22		0.8	4.5	49.7376	no deduction for volume of steel
6	abutment	2	7.5	0.8	6.5	78	no deduction for volume of steel
		2	7.5	2	0.5	15	
		2	7.5	1.2	0.5	9	
7	pile	132		0.36	10	134.29152	no deduction for volume of steel
8	pile cap	22	2.2	1.5	0.65	47.19	
9	total					948.01912	
10	increase by 15%					142.202868	
11	final total					1090.221988	