DESIGN OF RCC BRIDGE AGAINST FLASH FLOODS

A

PROJECT REPORT

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

IN

CIVIL ENGINEERING

With specialization in

STRUCTURAL ENGINEERING

Under the supervision

of

Chandra Pal Gautam

(Assistant Professor)

by

PRIYANSHA GUPTA (192657)

to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

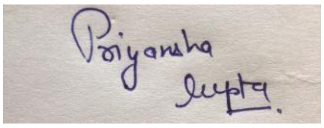
WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

May – 2021

STUDENT'S DECLARATION

I here by declare that the work presented in the Project report entitled **"Design Of RCC Bridge Against Flash Flood"** submitted for partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering, with specialization in Structural Engineering at **Jaypee University of Information Technology**, **Waknaghat**, is an authentic record of my work carried out under the supervision of **Chandra Pal Gautam, Assistant Professor**. This work has not been submitted elsewhere for the reward of any other degree/diploma. I am fully responsible for the contents of my project report**.**



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CERTIFICATE

This is to certify that the work which is being presented in the thesis titled **"Design Of RCC Bridge Against Flash Flood "** in partial fulfillment of the requirements for the award of the degree of Master of Technology in Civil Engineering with specialization in "Structural Engineering" and submitted to the Department of Civil Engineering, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by **Priyansha Gupta (192657)** during a period from July 2020 to November 2021 under the supervision of **Chandra Pal Gautam**, Assistant Professor, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

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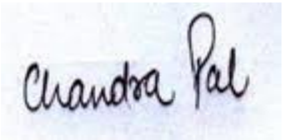

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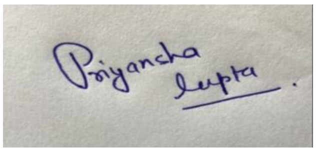

Priyansha Gupta

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ABSTRACT

A flood is defined as when land is dry the water which is flowing on land inundates the land. Flood also arises when water overflows from adjacent land, including a stream, river and ocean. Floods are catastropical events that are arised from sudden increase in discharge due to heavy intensity of rainfall for longer duration, Slope of ground. Human changes to the environment often increase the intensity and frequency of flooding, Scouring is major causes of bridge failure which is resultant of flash floods. Scouring mainly happens due to accumulation of water around bridge pier Methods have been developed to reduce scouring. The local scour is one of the reasons for bridge failure. Failure because of scouring is one of the predominant motives in bridge scour round bridge pier, float prediction and it interplay are broadly attracted with the aid of using the civil, mechanical and Marin.

CHAPTER 1

INTRODUCTION

GENERAL

Bridge is structure built to cross over the obstructions that are encountered in the middle of roadways, railways. Bridge is constructed for provision of passage for trade and communication purposes. Bridge design varies depending upon for the function for which it is served.

Types of bridges:

Bridge may categorised as numerous exceptional ways. Similar classes encompass the kind of uses of structure use, via way of means of what they carry, whether or not they may be constant or movable, and via way of means of the substances used. Bridges are constructed to withstand moves of Tension, Compression, Shear, Torsion maximum of the bridges are constructed to withstand those tiers of freedom.

Beam bridge:

Beam bridges structs which are supported horizontally at every stop via way of means of substructure gadgets and may be both virtually supported while the beams simplest join throughout a span, or non-stop while the beams are related throughout or greater lengths. When the length is more the intermediate supports are termed as piers.

Fig.1 Showing beam Bridge

TRUSS BRIDGE:

Bridge whose load bearing structure consists of a truss. This truss is a amalgamation of linked factors forming triangular units. The linked factors (usually straight) can be supports tension, compression, or every so often each in against response to dynamic loads.

Fig.2 Truss Bridge

Cantilever bridge

Cantilever bridges are constructed the usage of cantilevers — horizontal structure which are propoed on one end. Most of the cantilever bridges are centrally suspended on one span and are supported by pin arrangement in the middle.

Fig.3 Cantilever bridge

Arch bridge:

In arch bridge the weight of bridge is carried and supported on abutments which are present on either side of river banks . These are also one of the oldest form of ancient bridges.

Fig.4 Arch Bridge

Suspension bridge:

Suspension bridges are the bridges which are supported in the form of cables. These cables run along the height of the bridge and are attached to caissons. Which are drilled to deep into the bed under sea or a river or an obstructioon. The main load carrying members are suspensiin cables.

Fig. 5 Suspension Bridge

Cable-stayed bridge:

Cable-stayed bridge, like suspension bridge are supported by cable.

Fig.6 Cable Stayed Bridge

Classification according to span

3 types of bridges are classified are as follows

Culverts - 0m to 6m

Minor bridges - greater than 6m less than 60m

Major bridges - greater than 60m .

Culvert or minor bridge:

The culvert is typically a tunnel-like shape that lets in water to byskip below a roadway or railway. The bridge is a passage of transportation over a massive frame of water or bodily obstruction.

Fig.7 Culvert Bridge

COMPONENTS OF BRIDGE

Components of bridge consists of mainly superstructure and substructure

Superstructure:

Superstructure consists of

- Deck slab
- Girders
- Footpath
- Kerbs and Railings

Substructure:

Substructure consists of

- Bearings

Piers

- Pier cap
- Pedestal
- Foundation
- Abutments

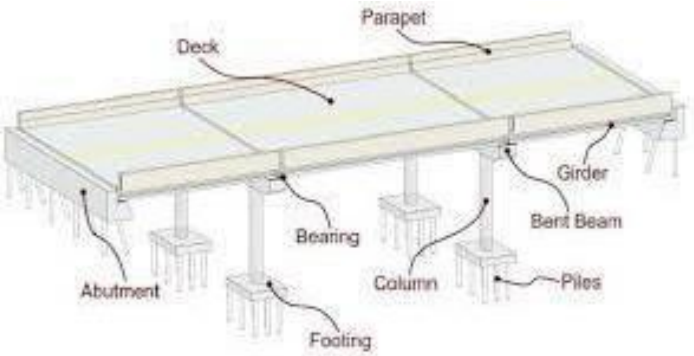

Fig.8 COMPONENTS OF BRIDGE

TYPES OF FLOODS

Fluvial floods (river floods):

Fluvial floods are result of heavy torrential rainfall for short periods which results in rise of waterlevel in streams, rivers and thereby resulting in overland flow and submerging the lands near by,

The damage resulting from the river flood can be very dangerous as it submerges entire land because of the discharge it encompasses.which could influence water level present in dams and dikes. The severity of river flood is largely influenced by rainfall for how much time it is taking place and It is mainly dependent on catchment area .

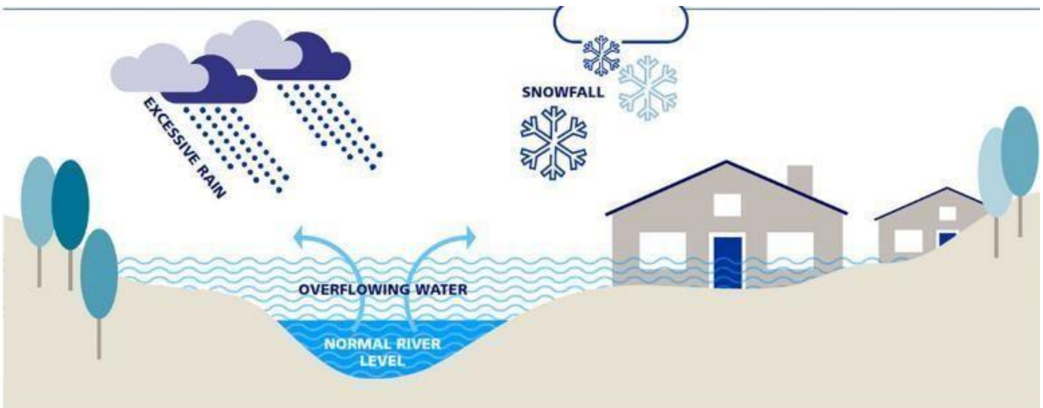

Fig.9 Fluvial Floods

Surface water floods or pluvial floods:

A pluvial floods are combination of both surface water floods and flash flood

Surfae water floods*:*
Occurs mainly when city drainage are not properly designed due to which water which is result in rainfall will crush on to city roads and streets And structures which are in close proximity . It takes place over a period of time and possess less danger and Usually occurs in shallow depth and not possess instant damage but can damage financially.

Flash floods :
These are mainly categorised according to nature which are very intenseand excessive in nature and can be arised from increase in level of water from a river or a dam upstream and these can be resulted from high terrains They also can arise through surprising launch of water from an upstream levee or a dam. Flash floods are very risky and unfavourable now no longer simplest due to the pressure of the water, however additionally the hurtling particles this is frequently swept up withinside the flow.

Fig.10 Showing Pluvial And Surface Water Flooding

Coastal flood:

Coastal floods are result of floods resulting from rise in level of water from. Sea.. Casual reasons for coastal floods are windactivities taking place on the equal time as excessive tide (hurricane surge) and tsunamis.

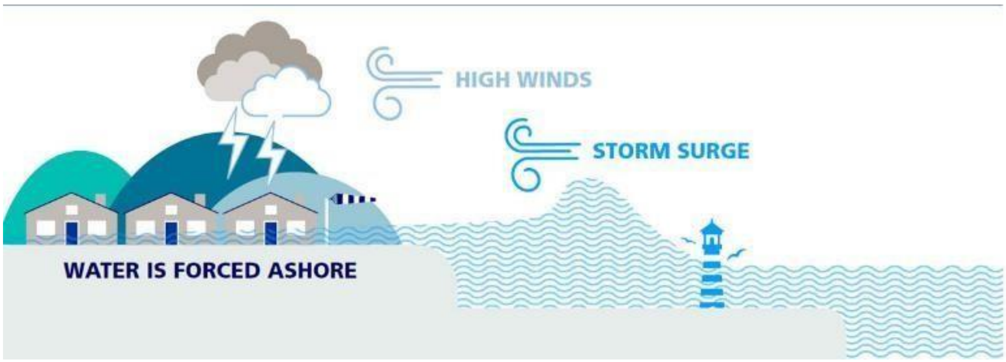

Fig.11 Coastal Floods

Storm surge is created whilst excessive winds from windstorm pressure water onshore — that is the main motive of coastal flooding and regularly the finest risk related to a windstorm. The outcomes growth relying at the tide - windstorms that arise at some stage in excessive tide bring about devastating hurricanes

SCOURING :

Scouring is primary purpose for bridge failure which can be lateral,vertical and torsional in nature. Absolutely 90%of bridges are failed due to scouring resulting from vertical and lateral directions. Scouring is removal of sand and soil which is accumulated due to floods around bridge piers and abutments.and these are unexpectedly transported from flood waters.

Water typically flows quicker round piers and abutments making them at risk of neighborhood scour. At bridge openings the contraction scour can happen in narrow areas and water flows through upstream of the bridge . And degradation scour can happen in huge areas of bridge. Over lengthy intervals of time, this may bring about reducing of the flow mattress Scouring is measured through intensity of scour hole.

Fig.12 Scour Around Pier

CAUSES OF SCOUR

The maximum often encountered bridge scour issues typically contain free alluvial fabric that may beeffortlessly eroded. However, one need to now no longer count on that overall scour in cohesive or cemented soils will now no longer be as huge as in non-cohesive soils; the scour honestly takes longer to increase in bridge piers inclination perspective contributes to outcomesat the scour hollow, eraround a unique unmarried cylinder piers.

Thus, the multiple with the drift depth and pier length contributes to an multiplied scour depth, a scour quantity floor location and a hollow. In addition, all of the neighborhood scours close to the piers are nicely asymptotically evolvedbelow the steadygo with the drift conditions. Despite the truth that when a bridge is willing closer to upstream, the maximum downstream pier offers illustration of a better expectation of scour depth values round it. Generally, the improvement of the scour hollow across the bridge piers is the principal purpose of failure.

TYPES OF SCOUR

Scouring types are as follows ;

- Live bed scour
- Contraction scour
- Degradational scour

Live bed scour:

Live bed scour is result of removal from local sediment material around piers and abutments. (Piers and abutments are structures supporting bridge). Live bed scour is result of water flowing through abutments and piers which may scoop in holes called scour holes which are formed around bridge.

Contraction scour:

Contraction scour is the removal of soil from the top and bottom and sides of the bridge. Contraction scour is a result of increasing speed of water as it flows through bridge openings that is narrow than river channel stream.

Degradation scour:

Degradation scour is result of elimination of soil or small sediments backside through upstream of river.

Degradation scour is natural process taking place in upstream side of river. However can deposit sediments gradually over a period of time.

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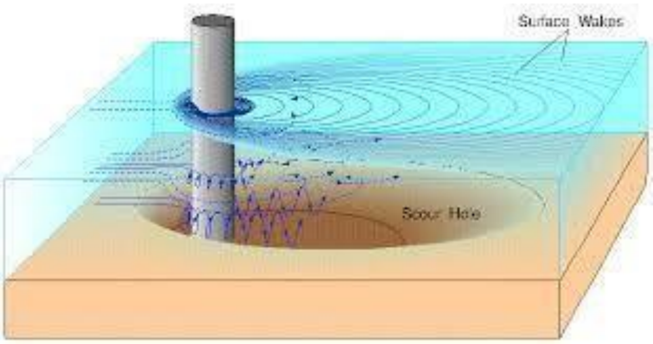


Fig.13 Local Scour

SCOUR PROTECTION MEASURES ;

The nearby scour obtrusive at bridge piers is taken into consideration as a nice protection risk for public use. In addition, it's miles a situation for vehicles. Again, if it's miles via way of means of any danger decided that the scour at a bridge pier has a ability ofadversely affecting the steadiness of the bridge, then numerous countermeasures to be used in protective the pier ought to be taken into consideration. According to [1] the countermeasures hired to be used included; the approach channel manipulate, drainage manipulate, higher bridge modifications, the downstream- channel manipulate and a higher armoring of bridge opening.

Effect of pier shape on scour geometry

Experiments have been conducted for various shapes such as Trapezoidal, Rectangular, oblong which are having similar float conditions and similar scour goemetry and are having common aspect ratio to observe flow conditions from their respective scour geometries.

Pier models

5 pier models are been tested as shown in fig below Rectangular (P1), Circular (P2), Traingular nosed (P3), oblong (P4), Elliptical(P5)

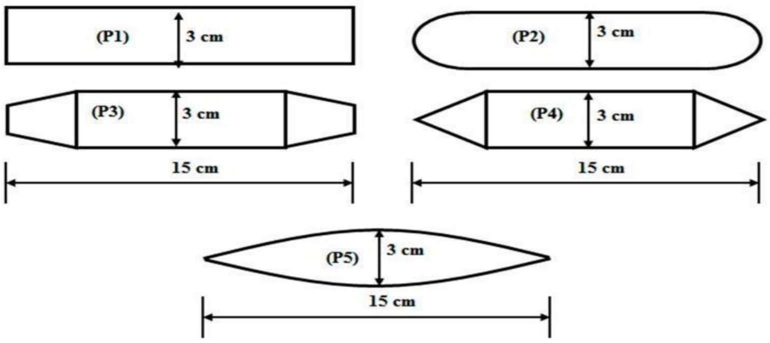


Fig.14 Pier Models

The blockage ratio withinside the float became described because the ratio among the flume width and pier width, which became approximately 10 instances of the pier width for the clean water float.

CHAPTER 2

LITERATURE REVIEW

General:

In this chapter the literature study was conducted about influence of piers shape on flooding and selection of appropriate piers shape numerous number of papers were studied on papers which briefly elaborate the influence of flash flood on different types of bridge and finding were published. Work has also been done on how to reduse scouring around bridge pier and pier modification measures are been identified in order to reduced scouring rate aroud different shapes of piers.

Adnan Ismael Mustafa Gunal had conducted the study on latest method that are used to reduse the scouring and redusing the intensity of scouring rate infont of bridge pier among many piers that are been tested the pier with downstream down round nosed pier had shown possible result in which downflow detected or directed away from bridge piers, experiments are been conducted on 3 piers which are having dimensions mainly round pier having dimension of 10 m, upstream round nosed pier (10m – 4m) and downstream round nosed pier having dimension of (4m – 10 m) are been tested for flow conditions having discharge of 581s of period of 3 hours amoung these downstream round nosed pier had showned some possible results .

And by using these type of pier the intensity of vortex formation infront of bridge piers has been reduced. Also by using this type of piers there is no need for pier modification measure Which involves using of rip rap stones around piers.

Mayam Nasim, HessamMohseni, SujeevaSetunge, Shiwei Zhou, Amila Dissanayake hHad conducted study and found that the debris loading and resistance to flood are essential parameters effecting the stability of bridge beneath underwater. In their experiment they have assumed the overall flood force as a static force which can have overall impact of floods and debris loading on specially constructed piers It is crucial to find out the weightage of flood loading on different piers So they have simulated field conditions on software known as ansys fluent in which they have simulated flood loading on different piers To determine flood loading on piers 2 equation are been produced As5100 equation which gives strain on rectangular piers but it is insufficient to find out strain on Round piers.

Mubeen Beg1, Salman Beg[2] Had said that for safe and low budjet design, scrub across the bridge support is required to be controlled and managed. Concluding the performance of scor or scrub safety tool round bridge piers depend upon how the tool counter the scrubing. Attempts were made to reduce the intensity of scour through various means of putting the riprap over the pier, transmitting an array of piles in front of piers, a collar across the support or piers, overflowled or drowned vanes, a delta – wing like fin in front of support, a slot through the pier and partial pier corporation and tetrahrdron frame placed over the piers,

In this conclusion depth evaluation of renouated pointing on scour discount round bridge piers is parameters influencing scour intensity, time – variant of scour. Flow changing gadgets may seen very low budget, in specific whilst the riprap fabric is required amounts is not very modrate to close to bridge webpage or is expensive. Still there were certainly many barriers on using these flow changing gadgets to reduse the scour, intensity at piers a slot can be blocked through way of means of floating debries, futhermore its manufacture is difficut sacrificial piles can be seem as useless through the gride drawing close the piers changes its direction underneath matters which diver direction effects is compelety safe and powerful against scour.

CHAPTER 3

INTRODUCTION

Main motto of this project is to design a bridge which can resist flash floods for this purpose optimum pier shape needs to be selected which can resist floods and designing substructure and superstructure of bridge manually According to IRC: 5-1998, A bridge is described as a shape having a complete period of above 6 m for sporting visitors or different shifting hundreds over a melancholy or an obstruction which includes a channel, avenue or railway. These bridges are categorised as givenbelow.

Minor Bridge: Bridge having total span of 60m or less is called minor bridge.

Major Bridge: Bridge having total span of 60m or above 60m

Components of bridge -

Components of superstructure of bridge are as follows:

- Deck arrangement consisting of deck slab,trusses,girders.
- Bearings under superstructure

Parts of bridge substructure are:

- Abutments and piers
- Foundations for substructure.

The Deck is the part of the superstructure which carries the moving load. This load is then transferred to the substructure by the following:
Longitudinal and Cross Girders (as in
beam bridge). Trusses/ Frames.
Box Girders.
Balanced Cantilevers.

Forces acting on bridge deck:

The above parts of structure resist and transform the load in the form of shear force and bending moment and transform load to piers and abutments.

Bridge load standards:

Bridges are designed according to IRC 6: 2000 which are defined by Indian roads congress. The loads are grouped under four divisions

IRC Class AA Loading:

Bridges those located on State and national highway are designed for heavy loading. The Ircclas AA vehicle similar to army tank which has wheel load of 700 KN and wheeled vehicle of 400kn are assumed.

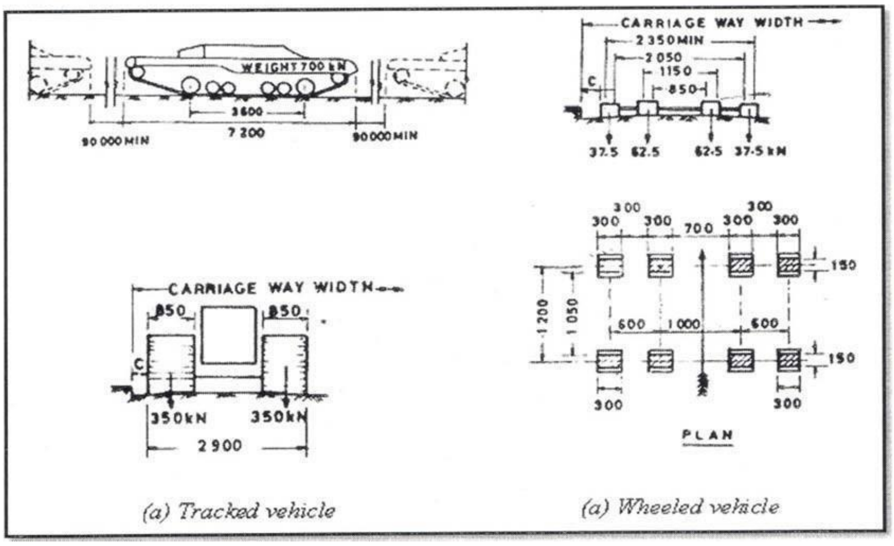


Fig.15 showing Grouping of vehicle loads under IRC class AA

IRC Class A Loading:

Wheel load consisting of truck with specified axle spacing and loads are assumed in this type of loading.

IRC class B loading:

class A and class B loading are both defined for supporting similar type of structures except for some axle loading conditions.

IRC class 70r loading:

This type of loading is grouped under 3 vehicles

Impact factor

Impact factor is different for type of material that are used in bridges i.e., different for reinforced and steel bridges and also it is inversely proportional to length of span. Impact factor is applicable to distributed loads to include effects of loads on to the bridges.

Impact factor for bridges are assumed as following,

A. RC Bridges- 10% up to a span of 40m.
B. Steel Bridges- 25% up to a span of 23m .

Materials used in bridge:

As per IRC:18-2000, for concrete that is to be used in bridge should be of grade not less than 35N/mm^2. For prestressed bridges the mix to be followed should be Design mix only and mix that is designed should follow code of IS10262:1982 which layouts the procedures for proper design mix.

Properties/Permissible Stress	**M20**	**M40**	**M60**
Modulus of Elasticity(Gpa)	29	32.5	37
Permissible Direct Compressive Stress ((N/mm^2)	6.25	10.0	15
Permissible Direct flexural stress(N/mm^2)	2.33	13.33	20

Table showing properties of concrete In order to safeguard cracks and shear

torsion reinforcement should be used In addition to higher grade of concrete.

Composite bridge:

3.4.1 Introduction:

A composite bridge is made up of concrete and steel which are connected by bearing. Earlier bridges are used to be made of only one material But Now a days to resist higher loads composite bridges are used The following figure showing the components of bridge.

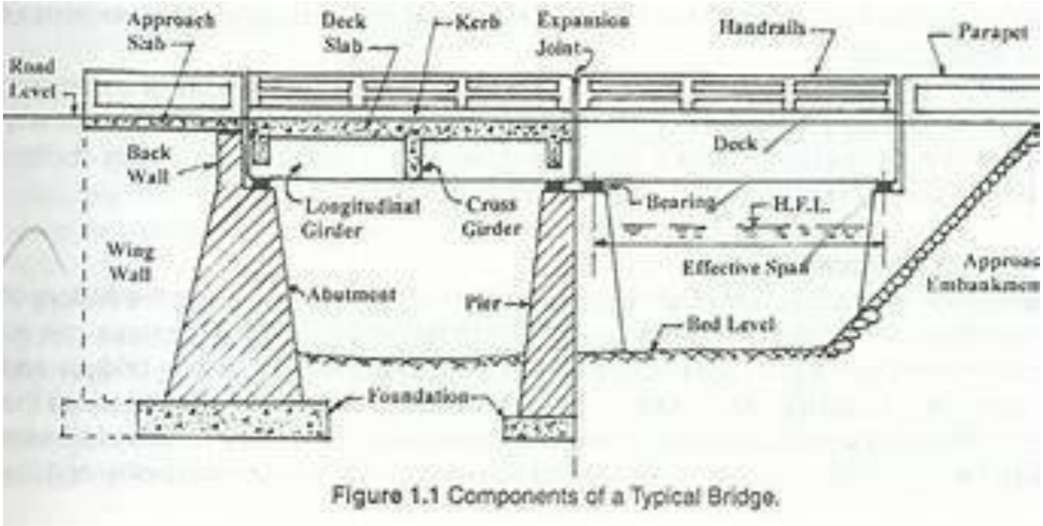


Fig.16 showing components of bridge

Deck:

The deck is the road or walkway surface. In avenue packages it's miles generally a poured constructed up stable section, but can likewise be metal framework or wooden board. The deck contains any avenue paths, medians, walkways, railings or railings, and various things like seepage and lighting.

Supporting structure:

Comprises of the metal or strong framework assisting the deck. This includes the real helps, sor cross-helps, and (the bracket or curve framework. In a assist join this will include simply the braces and the propping framework. The braces are the critical burden assist, whilst the propping framework each lets in the helps to behave all together, and maintains the bars from overturning.

Bearing pads:

It is to allow the superstructure to transport pretty freely of the base. All substances generally develop and settlement with temperature - if a scaffold had been completely inflexible, this will purpose superfluous weight on the development and will spark off unhappiness or harm. By solving the superstructure towards one side, the longitudinal way, heat burdens are mitigated and the lifestyles expectancy of the scaffold expanded**.**

Girders:

Parts of grider:

A - Web:

Web may be same height through out or may be height can be varied. The dimensions of girder which is varied is called haunched girder. For designing girder we will assume depth of girder and also its thickness. The depth of weis computed from following formula.

M= Design bending moment.

$$D = 5^3 \sqrt{\frac{M}{\sigma_b}}$$

σ_b=permissible bending stress in steel which is taken as 0.66fy (fy is the yield stress of steel) According to IRC 24, a minimum thickness of 8 mm is adopted to provide for wear causedbycorrosion.Inadequatedimensioningofawebmayleadtowebbuckling.

B -Flanges:

Flange comprises of single plate. The width of flange ranges from ratio of 40 to 45.The flanges are connected to web y means of welding to transmit shear force in addition to vertical loads. The thickness of flange area may be calculated according to site conditions.

The area of flange is given by

$$A_f = \frac{M}{\sigma_b d} - \frac{A_w}{6}$$

Where;

d = depth of the web

Aw = area of the web

The area of flange must not be more than 20 times of its thickness. and the section must be designed for critical stresses as given by IS code:800-2007

C - Stiffeners:

The web is supported by stiffeners to avoid buckling. It is also should be complied with web thickness. There ae two types of stiffeners

a.Vertical stiffeners:

The vertical stiffeners are at a spacing of 1.5d Where d is depth of web, The web panel dimensions between two stiffeners must not be more than 270 times the thickness of the web . The length of overhanging leg of the vertical stiffener shall be taken as 12 times the thickness of the web .

$$I = \underline{1.5d^3\ e^3}$$

Where,

I= moment of inertia of the pair about the face of the web

t = thickness of theee web

c = clear distance between vertical stiffeners

d = depth of the web

Bracings:

Bracings are Frames which are held cross in the plane which is horizonal for connecting flanges to resist lateral deformation. The deformation is induced by loads that are caused by wind which act normal to web. When length of bridge exceeds 30m lateral bracings are required.

Connectors :

Connectors are provided to fill gap between concrete slab and steel girder by transferring shear force along surface without slip.

The types of connector used are:

Rigid connectors consisting of bars that are welded on flange as well as welded to shear connectors.

The flexible connectors consisting of studs or tees welded on the flange plates of prefabricatedunits.

Anchorage type shear connectors are provided for sections consisting of precast concrete girders and cast *in-situ* reinforced concreteslab.

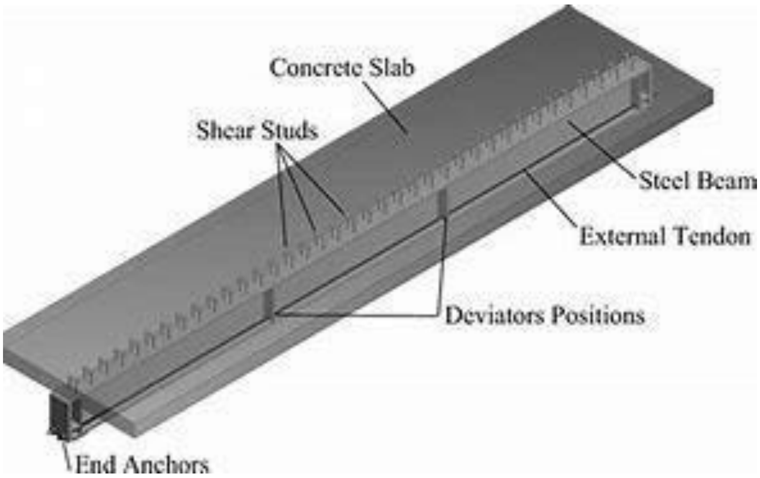


Fig.17 Image showing connectors

CHAPTER 4

PROJECT SPECIFICATION

Objective:

To design the substructure and superstructure of bridge and determine best pier shape which can withstand flooding.

Specifications:

1. To design a composite bridge with against flood with following specifications:

- Clear Width Of roadway = 7.5m
- Span = 108 m
- No. Of Traffic Lanes = 2m
- IRC Class AA Wheeled Live Load.
- Wearing Coat thickness = 75 mm
- M-35 Grade Of Concrete And Fe – 415 Grade OfSteel

2. To Design I section Steel Plate Girders For Above Bridge Deck With the Following Specifications.

- Span = 15m
- Simply Supported.
- To Check The Effect of pier shape on scouring.

CHAPTER 5

Design procedure

IRC limit state design procedure for bridges:

Design philosophy:

Limit state design is based on statistical and probabilistic approach of design which considers the working loads multiplied with probable factor of safety to evaluate design load. Limit state should provide suitable serviceability condition that a structure should satisfy.

5.1.2 Design coefficients:

Based on stresses the constants are used for calculation of depth (d) and area of steel(A_{st}) in tension zone with n and lever arm factor(j) and permissible stresses in concrete

Design of deck slab:

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

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

a. Dissipation of load along the length:

The span of deck slab is calculated as addition of tires contact area on the top coat of slab in along the span and twice the overall depth of span including the thickness of wearing coat

D=overall depth of slab

H=Height of slab

X = wheel load dispersion along the length

V=Length of dispersion;

$V = x + 2(D+H);$

b. Dispersion of slabs running in both directions:

Moment of wheel loads will be dispersed along transverse and longitudinal directions. These moments are computed by using pie guards method which is applicable to slabs supported on both ends and loading condition should be symmetrical. In calculation of dispersion width following denotations are used.

L = Long spanlength

B = Short span length

u &v = Dimensions of the load spread after allowing for dispersion through the wearing coat and structural slab

K = Ratio of short to long span of slab (B/L)

$M1$ = Moment in the short span direction

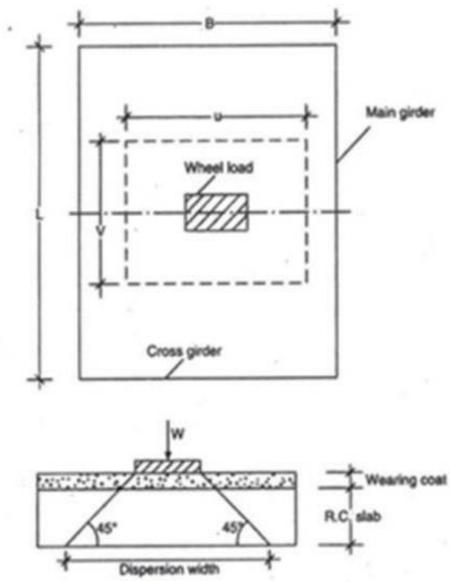
$M 2$ = Moment in the long span direction

$m 1$ &$m2$ = Coefficients for moments along the short and long spans

p = Poisson 's ratio for concrete generally assumed as 0.15 as per IRC : 21-2000

W = Wheel load under consideration

wheel load is assumed to be 45 degrees to wearing surface as per IRC code.



<u>**Fig.18** showing dispersion of wheel load through at 45 degrees</u>

The bending moment along short and long lengths are given as follows.

m1= (mi + pm2)

m2= (m1+ pm1)

Moment coefficients m1, and m2 corresponding to *K* and *(1/K)* for slabs supporting uniformly distributed load (dead load of the slab) are also obtained from these curves.

Design of slab for end span:

Bending moments:

Dead load of slab= (1*1*0.25*24) =6.00kN/m^2
Dead load of wearing coat= (0.0075*22) = 0.165KN/m^2
Total dead load=6.165KN/m^2

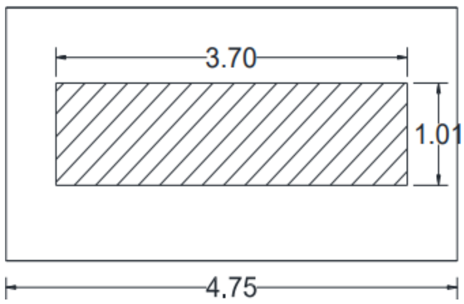


FIG.19 showing placing of wheel load for max bm

U=B+2t

V=L+2t

U=0.85+2x0.075=1

V=3.6+2x0.075=3.75

u/B=3.26

V/L =4.16

From Pigeaud‟s curve,

u/B=1.01/2.5 = 0.404

V/L = 3.76/4.75 = 0.792

From Pigeaud‟scurve,

K= B/L =0.53

m_1=0.095 and m_2=0.02

M_B=w(m1+0.15m2) =350(0.095+0.15*0.02) =34.30kN-m

As the slab is continuous, Design BM= 0.8M_B

Design BM including impact and continuity factor=1.25*0.8*34.3=34.30kN-m

M_L=w(m_2+0.15m_1)=350(0.02+0.15*0.095) =11.98kN-m

Shear force:

Dispersion in direction of length: 0.85+2(0.075 +0.25) = 1.5m load is kept such that the whole dispersion is within fullspan.The load is kept at 1.51/2=0.755m from the edge of the beam.

Clear length of panel=4.75-0.2=4.55m

B/L= 4.55/2.3=1.978
From table, (IRC 21-2000) value of k for continuous slab is = 2.6

Width of slab = $k_x[1-(x/L)]+b_w$

= 2.6*0.755[1-(0.755/2.3)]+[3.6+(2*0.075)]=5.06

Load per meter width = 350/5.06 = 69.16KN

Shear force per meter width = 70(2.3-0.755)/2.3 = 47.00kN

shear force = 1.25*47.0=58.75kN

Dead Load BM And Shear Force:

Total dead weight = 6.165KN/m^2

Total dead load on panel = 2.5*4.75*6.165=73.20KN

For UDL U/B=1 and V/L= 1;K=B/L= 2.5/4.75=0.526; 1/K=1.9
From Pigeaud"s curve,

m_1=0.049 and m_2=0.01

M_B=73.20(0.049+0.15*.01)= 3.7 Kn-m

M_L= 73.20(0.01+0.15*0.049)= 1.27 Kn-m

Design BM including continuity factor; M_B=0.8*4.65=3.72kN-m M_L=0.8*160=1.28kN-m

Dead load shear force=w_l/2 =(6.16*2.3)/2=7.0KN

Design moments and shear forces:

Total MB=34.3+3.72=38.02kN-m

ML=11.98+1.28=13.26kN-m

Design Of Slab Section And Reinforcement:

Effective depth , d=√(M/Qb) =√(38.02*106)/(0.762*1000) =223.37mm; $d_{provided}$> $d_{efective}$.

A_{st}= M/(σ_{st}*j*d)=(38.02*10^6)/(200*0.96*250)=792.08mm^2

Use 12mm dia. Bars @150mm c/c Center to centre spacing

=[1000x(πx122)/4]/792=142.80mm=150mm

Ast = [1000*(*122)/4]/150 =755mm

Effective depth for long span using 10mm dia.,=250-6-5=239mm

Ast= (13.26*106)/(200*0.96*239)=288.96mm^2= 300mm^2

But minimum reinforcement using HYSD bars according o IRC 18-2000 is 0.15% ofcross section area.

Hence,A_{st} = 0.0015*1000*250 =375mm2

Check For Shear Stress:

Nominal shear stress = τ_v=V/bd =(58.75*103)/(1000*250) =0.235N/mm2

Hence, (100Ast)/bd= (100*755)/(1000*250) = 0.302

For M35 concrete, τc=0.37N/mm2

(From IS456:2000)

For overall depth 300mm, K=1.01(From table 12C IRC: 21-2000)

Permissible shear stress in concrete slab= Kxτc =1.10*0.3=0.43N/mm2>τv = 0.19N/mm2

Hence the shear stresses are within the safe permissible limits.

Design principles of Longitudinal girder:

Design of girder in bridge involves determining reaction factors coming from superstructure by considering the load that is given by IRC and locating transverse disposition of vehicle.It involves calculating dead load from deck slab and imposing load on to the each girder and determining dead load moment and dead load shear and designing section and determining no of prestressing cables against load.

It also various steps as follows:

1. Self weight of girder is given by *(0.2L* + I) kN/m, where *L* = span of thegirder.
2. Design moments and shear are calculated by considering impact factors
3. Approximate Depth Of Girder=1/8 to 1/10 ofspan

$$\text{Economical Depth , } D=5\sqrt[3]{\frac{M}{\sigma}}$$

Where; M=design bending moment;

Sigma(b)=permissible bending stress as specified in IRC:24:2001

For girders permissible stresses are given by 141 N/mm^2 and 150 N/mm^2

Assuming the thickness of web as *'t'*(not less than 8mm), the depth of the web is obtained as,

$$d = \frac{V}{\tau_v t}$$

Where*V* = Shear force

t = thickness of web

Tv = Average shear stress specified as 85 N/mm^2 for mild steel

A suitable web depth is proportioned based on flexure and shear computations.

4. Flange area is calculated as given below

$$A_f = \frac{M}{\sigma_b d} - \frac{A_w}{6}$$

Flange width ranges from L/40 to L/60; outstand flange width should not exceed 16 times thickness for mild steel

5. The section is checked for permissible stresses as per specifications of IRC 24:2001

6. Connections between flange and web are designed to resist maximum shear force.

$$\tau = \frac{Va\bar{y}}{I}$$

V= shear force at section;

Y= distance from centroid to neutral axis

I=second moment of inertia.

.Design and reinforcement detailing of I section girder for given load:

Reaction Factors:

Using Courbon‟s theory, the IRC class AA loads are arranged for maximum eccentricity as shown in fig.

Reaction factor for outer girder A is,

R_A=(Σw1/n)*[1+(nex1)/(Σx2)]

=(2w1/4)*[1+(4*1.1*3.75)/{(2*3.752)+(2*1.252)}] =0.764 w1

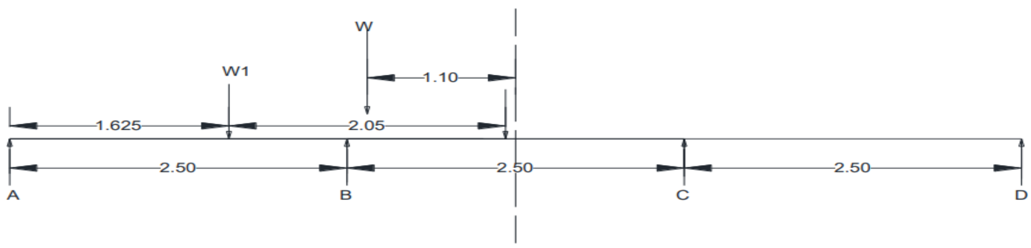


Fig 20. I section Girder
Transverse Disposition of IRC Class AA Tracke Vehicle

Reaction factorS for inner girder B is ,

RB =2w1/4)*[1+(4*1.25*1.1)/{(2*3.752)+(2*1.252)}] =0.588 w1
If w_1= 0.5w

RA=0.764*0.5w= 0.382w

RB=0.588*0.5w =0.294w

Dead Load From Slab Per Girder:

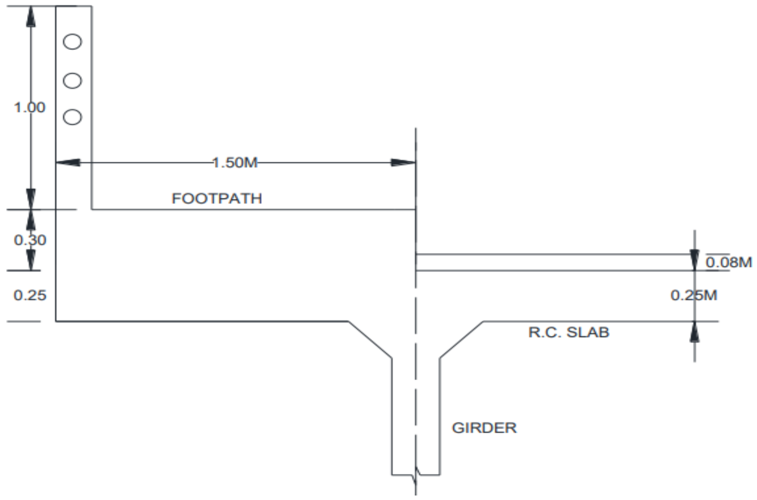


Fig.21 Showing structure of girder

Weight of ,

Parapet railing (lump sum) = 0.92kN/m

Foot path and kerb = 0.3*1.5*24 = 10.08kN/m

Weight of deck slab=0.25*1.5*24 = 9kN/m

Total load on deck slab = [(2*20)+(6.125*7.5)] = 85.93KN/m

It is assumed that the deck load is shared equally by all the four girders.

Dead load per girder = 85.93/4 = 21.48 kN/m

Dead Load On Main Girder:

Overall depth of girder = 1800mm

Dead weight of rib = 1.15*0.3*24 = 8.28kN/m

Dead weight of bottom flange = (0.5*0.4*24) = 4.80kN/m

Total load = 13.08kN/m

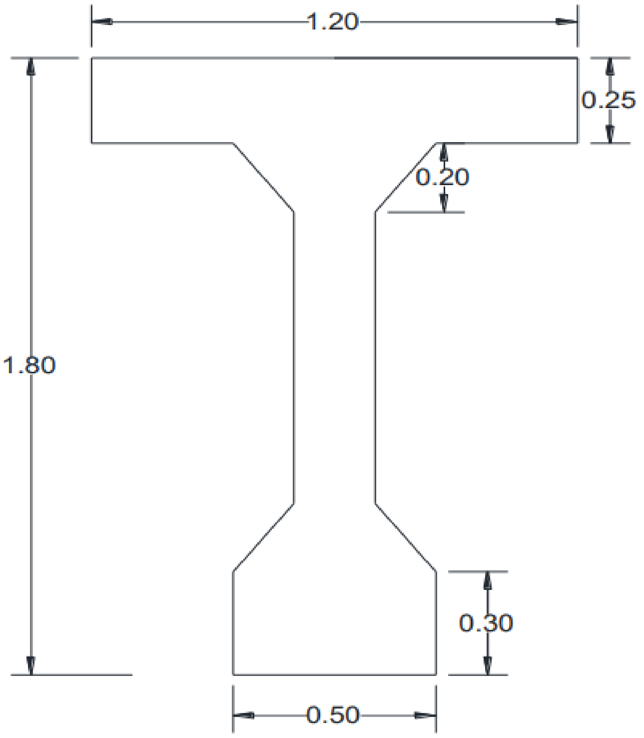


Fig.22 Showing Cross section of precast concrete girder

Dead Load Moment and Shear in Main Girder:

Self-weight of main girder = 13.08kN/m

Total dead load on girder = (21.48+13.08) = 34.56 kN/m

M_{max} = [(0.125*34.56*19^2)+(0.25*15*19)+(15*9.5)+(15*4.75) =1844.52 kN-m

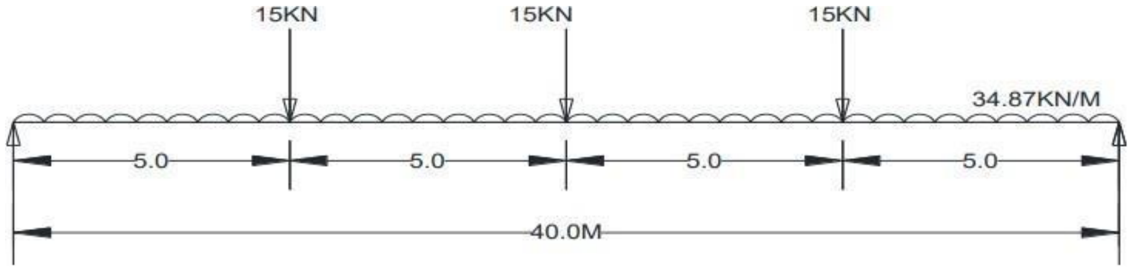


Fig.23 Dead load and live load on main girder

Dead load shear at support

V_{max} = [(0.5*34.87*19) + (0.5*15*3) = 353.76kN

Live Load Bending Moment In Girder:

Span of the girder =40m

Impact factor (class AA) = 10%

Live load is placed centrally on the span

Bending moment at centre of span= 0.5*(9.1+10)*700=6685kN-m

Bending moment including impact and reaction factor for outer girder is,

Live load bending moment = 6685*1.1*0.382 = 2809.04kN-m

For inner girder, BM = 6685*1.1*0.294 = 2161.93kN-m

Live Load Shear Forces In Girder:

For maximum live load shear in the girder, the IRC class AA loads are placed as,

Reaction of w2 on girder B = (350*0.45)/2.5 = 63kN

Reaction of w2 on girder A = (350*2.05)/2.5 = 287kN

Total load on girder B = 350+63 = 413kN

Maximum reaction in girder B = (413*38.2)/40 = 394.41kN

Max reaction in girder A = (287*38.2)/40 = 274.08kN

Max live load shear with impact factor in inner girder = 394.41*1.1= 433.95kN

Outer girder = 274.08*1.1 = 301.488kN

Properties Of Main Girder Section:

Cross sectional area = (1200*250)+(1150*300)+(400*500) = 8.45*10^5mm

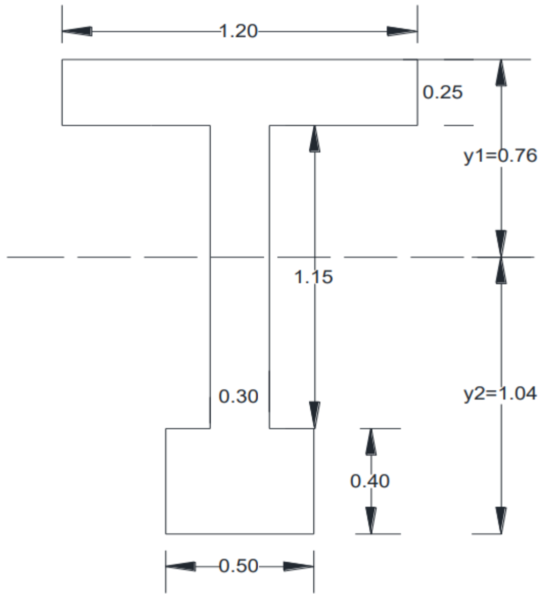


Fig.24 showing cross section of girder

Moment of inertia about axis of bending,

I = [{(1200*250^3)/12}+(1200*250)(760-125)^2] +[{(1150*300^3)/12}+(1150*300)(760-825$^{)2}$] +[{(500*400^3)/12}+(500*400)(760-1600)^2]=3.57x10^{11}mm^4

Z_t = I/y_1 = 4.02*108mm3

Z_b = I/y_2 = 2.94*108mm^3

Check for Minimum Section Modulus:

f_{ck}= 50N/mm2; f_{ct} = 18N/mm2

f_{ci} = 40N/mm2 ;f_{tt}= f_{tw}= 0 * M_D= 0

ɳ = 80%

M_L = 2809.04kN-m; M $_G$= 8126kN-m

M $_D$= 10935.04kN-m

F_{br} = ɳ f_{ct} - f_{tw} =0.80*18 – 0 = 15.30N/mm

Ftr= fcw - ɳ ftt = 16N/mm2

Inferior stress = finf=(ftw/ ɳ) + (M D/ ɳZb) = 0 + (10935.04*106)/(0.80*2.94*108) =43.75N/mm2

Minimum section modulus required,

$Z_{required}$ = [ML + (1- ɳ)MG]/ Fbr = [2809.04*10^6+ (1- 0.80)8126*10^6]/ 15.30

= 2.63*108mm^3< 2.94*108 mm^3

Hence the section provided is adequate.

Pre Stressing Force:

Assume a cover of 200mm

Maximum possible eccentricity e = (1040.08 – 200) = 840.08mm

Prestressing force is obtained as,

P = (A.f_{inf}.Z_b.)/(Z_b+A.e)= [(0.845*106*43.75*2.94*108)/(2.94*108)+(0.845*106*840.08)]

= 10826.94*10^3N
= 10826.94kN

Using Freyssinet system, anchorage type 7K-15 (7 strands of 15.2mm diameter) in 65mm cables duct

Force in each cable = (7 * 0.8 * 280.7) = 1459kN

No. of cables = 10826.94/1459 = 7.41 =8 no.s

Area of each strand=140mm^2

Area of 7 strands in each cable=780mm^2

Area of each strand in 3 cables = Ap = 8*980 = 7840mm2

The cables are arranged at centre of span as shown;

Permissible Tendon Zone:

At Support section,

e=< (Z_b.f_{ct}/P) – (Z_b/A)

=< [(2.94*10^8*18)/(10826.94*10)] – (2.94*10)/(0.845*10^6)

=< 140.85mm and e=> (Z_b.f_{tw}/ɳP) – (Zb/A)=> 0 - (2.94*10)/(0.845*10^6)=> -347.928mm

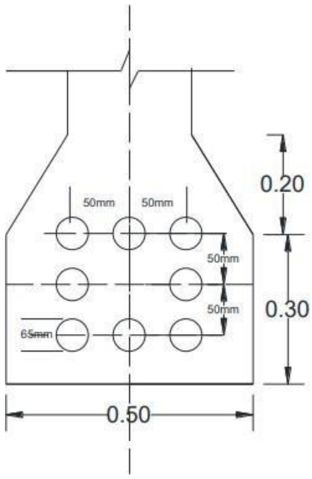


Fig.25 Cables arrangement at centre of span

Check For Stresses:

For the centre of the span section, we have

P = 10826.94kN; e = 840mm

A = 0.845*106mm2Zt =4.02*108mm^3; Z_b =2.9*108mm3

M_L =2809.04kN-m; M_G =8126kN-m

At transfer stage,

σ_t = [(P/A) – (Pe/Z_t)+(M_L/Z_t)]= [(10826.94*10^3)/(84.5*10^4) – {(10826.94*10^3*840)/(4.02*10^8)}+(2809.04*10^6)/(4.02*10^8)]=10.4N/mm^2

σ_b = [(P/A) + (Pe/Z_b) - (M_L/Z_b)]

= [(10826.94*10^3)/(84.5*10^4) + {(10826.94*10^3*840)/(2.9*10^8)}-(2809.04*10^6)/(2.9*10^8)]

=16.11N/mm^2

At working load stage

σ_t = [(ɳP/A) – (ɳP_e/Z_b)+(M_D/Z_t)]

= [(0.85*10826.94*10^3)/(84.5*10^4) - {(0.85*10826.94*10^3*840)/(2.9*10^8)}+(8126*10^6)/(4.02*10^8)]=18.85N/mm^2

σb = [(ɳP/A) + (ɳPe/Zt)-(MD/Zb)] = [(0.85*`10826.94*10^3)/(84.5*10) - {(0.85*10826.94*10^3*840)/(4.02*10^8)}+(8126*10^6)/(2.9*10^8/)]= 0.129N/mm2

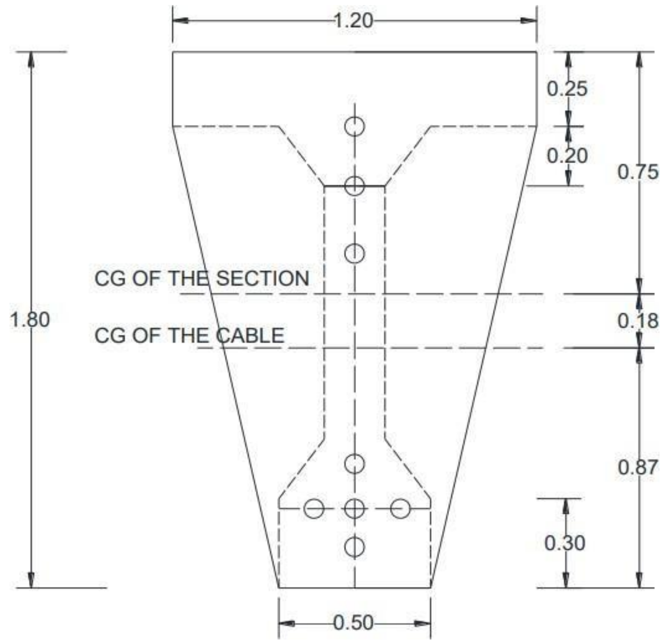


Fig.26 Arrangement of cables at support

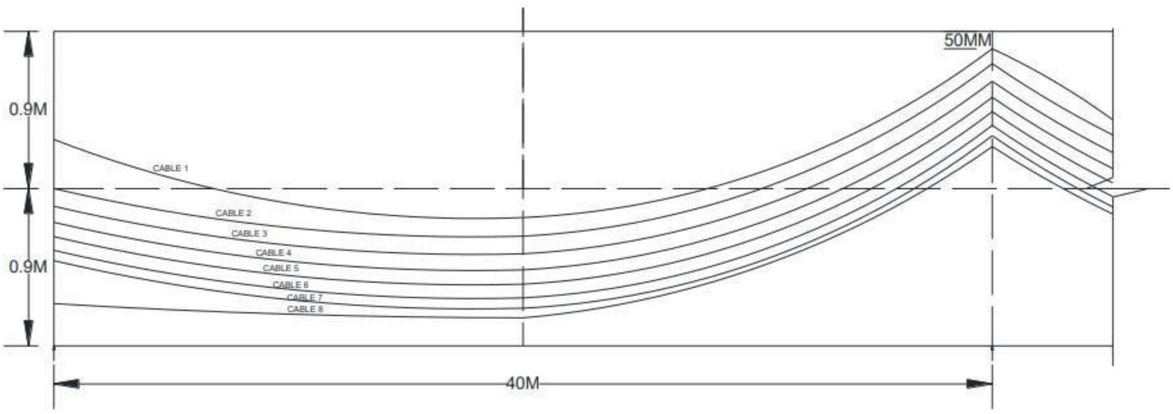


Fig.27

Check For Ultimate Flexural Strength:

For the centre of span section,

Ap = 8*7*140 = 7840mm^2

b = 1200mm; d= 1600mm

bw =300mm; fck= 50N/mm2

D_f = 250mm

According to IRC: 18-2000,

M_u =1.5M_G+2.5M_L = (1.5*8126*10^6) + (2.5*2809.04*10^6)= 19211.60*10^6

= 19211.60kN-m

The ultimate flexural strength is computed as,

iii) Failure by yielding of steel

M_u = 0.9*d*A_p*f_p= 0.9*1600*7840*1862 = 21021kN-m

iv) Failure by crushing of materials

Mu= 0.176$b_w d^2 f_{ck}$ + 0.667*0.8*(b-b_w)(d-D_f/2)D_f*f_{ck}== (0.176*300*1600^2*50) + 0.667*0.8*900*(1600-(250/2))*(250*250)=51008kN-m

Ultimate flexural strength at centre of span is calculated as

A_p = (A_{pw} + A_{pf})
=A_{pf} = 0.45fck(b-b_w) (D_f/f_p)= 0.45*50*(1200-300)*(250/1862)=2178.85mm^2

A_{pw} = 7840- 2718.85 = 5121.15mm^2
A_{pw}xf_p/b_wd.f_{ck}= 5121.85x1862/300x1600x50=0.4

Shear resistance required = 2292.525kN
Shear capacity of section=2102.3kn

Base shear = (2292.525 – 2102.3) = 190.2 kN

Using 10mm diameter 2 legged stirrups of Fe415 HYSD bars, the spacing Sv is obtained as,

Sv = (0.87*f_y*A_{sv}*d_t)/V= (0.87*415*2*79*1750)/(190.2*10^3)=435.2mm

Provide 10mm diameter stirrups at 150mm centre s near support and gradually

increased to 300mm towards the centre of span.

CHAPTER 6

SUBSRUCTURE DESIGN

Introduction

Substructure involves design of pier, abutment and its foundation

Design of pier

Pier design is based on IRC bridge design substructure specifications

Pier specifications:

Pier shape: Hammer head shape pier with circular ends of 4nos supporting 4 spans

Load classifications: Class AA tracked vehicle

6.2.1.

Step 1:Design criteria:

This involves defining material specifications, providing superstructure information and pier height

Material specifications : Concrete of m35 grade ,Steel of Fe415 steel

Density of m35 grade concrete:

28days characteristic compressive strength:35N/mm2

Superstructure data:

Girder type : I section girder

Girder spacing: 2m

No of girders: 4nos

Deck overhang: 1.5m

Span length: 29m

Parapet height=1m

Deck overhang thickness:250mm

Web depth:1.1m

Bottom flange thickness:300mm

Haunch thickness:450mm

Bearing height:0.4m

Superstructure depth: =Parapet wall height

$$+ \left(\frac{Deckoverhangthickness+haunchthickness+depthofweb+bottomflangethickness}{3.65m}\right)$$

$$=1m+ \left(\frac{0.25+0.45+1.1+0.30}{3.65}\right)$$

= 2m

Width of pier cap is decided based on superstructure data

6.2.2.

Step2: Selecting pier type:

Hammer head pier type is chosen based on site conditions

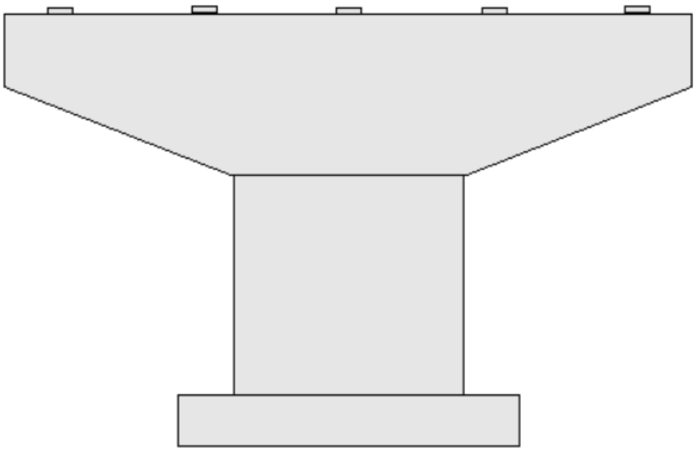


Fig.28 Typical cross section of hammerhead pier

6.2.3

Step3: Fixing preliminary pier dimensions:

Projection of cantilever slab on both sides of pier =lane width+ median width -1000=5000mm=5m

Web thickness= 4.6m

Total top width of slab projection on either side=14m

Bottom width of pier =7m

Thickness of bottom flange=1m

6.2.4

Step4:Calculation of dead load moments coming from structural elements above pier:

From superstructure:

Total load from external girder on to the pier cap : 7400KN

Total load from internal girder on to the pier cap : 6800KN

Pier cap dead load:

Sno	Dimensions of structure above	Load(KN)	Lever arm(m)	Moment(KN-m)
1	Deck slab(0.30x5x24)	36	5/2=2.5	90
2	Wearing coat=(0.080x2.75x24)	5.28	2.75	14.52
3	Kerb=(2.25x0.3x24)	16.2	3.87	62.6
4	Handrails	2	2.75x(2.25-0.080)=6	12
5	Traingular portion of hammer head(1.2x5x24)	144	1.7	245
6	Rectangular portion (1x5x24)	120	2.5	300
7	Rccposts(0.15x0.15x1x24)	0.54	5	2.7

Total dead load moment = 726.82KN

6.2.5

Step 5: Calculation of live load moments coming on to the pier cap:

Live load is taken as IRC class AA tracked vehicle load which is placed from 1300mm from the kerb and effective dispersion width of wheel load is given by

Be=1.2x+bw

X= distance of center to gravity of the concentrated load from the face of the cantilever support.

Be=1.2x0.1+Bw

Bw=0.85+2x0.080=1.01

Be=(1.2x0.1)+1.01=1.13

Live load per meter width including impact =(723×2)/1.13=1279KN

Design live load moment (Mq) = 1279×0.1 = 127.96KN-m.

6.2.6.

Step6: Calculation of design moment:

Design moment= Dead load moment+Live load moment=723Kn+127.96Kn=851KN-m

Factored moment=851x2.1=1702KN-m

6.2.7.

Step7: Design of reinforcement:

Effective depth required Qbd^2=max bm

$$d=\sqrt{\frac{maxbendingmoment}{Qxb}}=\sqrt{\frac{1702x10^6}{0.723x1000}}=1534.30$$

Effective depth req=2200-50=2150mm>1534.30mm

Therefore Depth adopted is enough

$$A_{st}=\frac{maximumbendingmoment}{\sigma_{st}xjxd}=\frac{1534.30x10^6}{200x0.91x2150}=3921.00$$

Use 25mm dia bars

$$a_{st}=\frac{\pi xd^2}{4}=\frac{\pi x25^2}{4}=490.87mm^2$$

No of bars=$\frac{3921}{491}$=8no's

6.2.8

Step8: Reinforcement details in hammer head

Top Reinforcement:

Provide 25no's of 25mm dia bars in 2layers

Side reinforcement:

Provide 9no's of 16mm dia bars of equal spacing

Inclined reinforcement:

Provide 9no's of 16mm dia bars of equal spacing

Shear reinforcement:

Provide reinforcement 10mm ø 4-legged stirrups @ 100 mm\cc.

6.2.9.

Step9: Design of circular pier:

Calculation of loads:

Weight of railing=(2x0.7)=1.4kn/m

of wearing coat = (0.080x7.5x22) = 13.2Kn.m

of deck slab=(0.300x12x24)=86.4KN-m

of Krebs = (2×0.3×2.25×1×24) = 32.4 KN-m

Pier cap is divided into 2 cantilevers and one rectangular section and hence weight is calculated

$=(2x\frac{1+2.2}{2}x5)x25=400$KN-m

Weight of rectangular portion = (2×2.2) × 25 = 110 KN-m

Therefore total weight of pier cap = 400+110 = 510 KN-m

Dead load of circular pier = $\frac{\pi x2^2}{4}x9x25= 706.5$ KN-m

Weight of IRC Class AA tracked vehicle is 700 KN

Total load= Dead load+live load=1450+706.5=2156.5KN

Load with impact=2156x2=4313kn

By considering dynamic effects such as wind load, longitudinal forces due to tractive effort of vehicles and longitudinal forces due to braking of vehicles a suitable factor of safety is made

Factor of safety = 2

Factored load = 4313×2 = 8626 KN

Factored load Pu=8626 KN

If vehicle is moving away from center line of bridge then eccentricity is taken as e=1.1m

Live load = 700×2 = 1400 KN

Maximum moment = 1400×1.1 = 1540 KN

Moment with impact = 700×1.1= 1400 KN

Factored moment = 1702×2.2 = 3404 KN-m

Therefore factored moment = Mu = 3404 KN-m

6.2.10

Step 10: Non dimensional parameters

$\frac{p_u}{f_{ck}d^2}=\frac{8626x10^3}{35x2000^2}$=61.61

$\frac{m_u}{f_{ck}d^3}=\frac{3404x10^3}{35x2000^3}$=12.15

Ratio = $\frac{D(Diameter of circular pier)}{d(clear cover)}=\frac{2000}{50}=40$

By referring chart number of 55 of SP 16

P=0.01*20= 0.2

Area of steel=$\frac{px\pi xd^2}{400}=\frac{0.2x3.14x2000^2}{400}$=6283.13mm^2

Use 25mm dia bars

$a_{st}=\frac{\pi x25^2}{4}$=490.87mm^2

No of bars=$\frac{6283}{490}$=13no's

Provide 30no's 0f 25mm dia bars around circumference of circular pier

Using 12mm dialateral ties

1. Least lateral dimension = 2000 mm

2. 16×25 = 400 mm

3. 300 mm
Hence provide 12mm dia bars @300mm Centre to Centre spacing.

Design of foundation of pier:

The foundation of pier is open foundation and further it is designed as pile foundation

Material properties:

Concrete grade (fck)= 35 N/mm²

Steel grade (fe) = 415 N/mm

Allowable stress of steel in tension and shear S_{st} = 240 N/mm²

Allowable stress of steel in direct compression Allowable stress of steel in direct compression S_{sc} = 205 N/mm²

Allowable compressive stress in concrete in flexure S_{cbc} = 10.0 N/mm²

Allowable comp. stress in concrete in direct compression S_{cc} = 7.5 N/mm²

Modular ratio (m) m = 10

Neutral axis factor k = 0.29

J= 0.90

The resisting moment coefficient R = 1.33

High Flood Level =200 m

Maximum Scour level for Pier = 201 m

Level of Deck Surface = 215.5 m

Thickness of Pier cap (overall Thickness) = 2 m

Top level of pier cap= 210m

Soil data:

Unit weight of backfill soil (γ) = 16 kN/m³

Unit weight of concrete Unit weight of concrete ωconc = 24 kN/m³

Angle between the wall and earth α = 0

Angle of internal friction of soil Ø= 32

Angle of friction between soil and wall δ = 16

Length of stem column (between the surfaces of the restrains) L = 10500 mm

Diameter of column D = 2600 mm

Effective length of column (IRC:21-2000, 306.2.1) L_e = 12600 mm

[effective length factor 1.2]

Forces at bottom of Footing

Dead Load

Dead Load From Superstructure = 8560 kN

Dead Load due to pier cap = 702.00kN

Dead Load of Pier Stem = 1083.10 kN

Dead load of footing = 3807.74 kN

Total Dead Load:

Moment Due to Breaking Force = 2324 kN-m

Effect of buoyancy [IRC:6-2000, 216.4 (a)]

Volume of submerged part of pier = 127.11 m^3

Net upward force due to buoyancy = -1271.12 kN

Live Load:

Live Load Excluding Impact = 2564.55 kN which will act at eccentricity = -0.001 m

Critical moment due to live load eccentricity = -1.379375 kN-m

Force due to water current

Exposed height to water current = 10.60 m

Perimeter Area exposed = 43.29 m

Maximum mean velocity m/sec = 2.2

Maximum velocity, Sqrt(2)*V, (IRC:6-2000,213.3), V = 3.11

Shape factor for square end (IRC:6-2000, 213.2), K = 0.66

Pressure intensity =$0.5KV^2$ (IRC:6-2000, 213.2) = 3.1944

Hence force due to water current = 92.19 kN

Moment due to water current = 977.24 kN-m

Length of Pile cap Along Brodge Axis = 7.40 m

Length of Pile Cap Across Bridge Axis = 13.40 m

Depth of Fixity from maximum Scour Level = 4.9 m

Diameter of Pile = 2 m

Depth of Pile = 20.00 m

No of Pile in one row = (Along Bridge Axis) 3

No of Row = 5

Total No of Pile (n) = 14

Embedded length of Pile = 14.80

Thickness of Pile Cap = 2.60 m

IRC 78:2000 Cl 709.5 OK

Factor of Safety FS = IRC 78:2000 Cl 709.3 2.5

offset of pile cap from the outer face of outermost pile = 0.10 m Ok

Center to center distance of the piles

Along Bridge Axis (Xi) = 3.00 m

Across Bridge Axis (Yi) = 3.00 m

Width of Pile Group (Outer Surface of The piles) along Axis (B) = 5.70 m

Width of Pile Group (Outer Surface of The piles) across Axis (L) = 5.70 m

Area Enclosed by pile Groups (Ag) = 45.60 m2

LOADS AND MOMENT CALCULATION:

Total Dead load= 1= 14152.84KN

Live load = 2564.55 KN

Moment along X-X=-1.38

Tractive or braking force=140kn and moment along Y-Y=2324KN

Total load=16857kn

Maximum load on individual piles

Maximum and Minimum Load is given by

$V_{max} = [V/n] + (M_{xx} * X_{max})/\sum X_i^2 + (M_{yy} * Y_{max})/\sum Y_i^2$

$V_{min} = [V/n] - (M_{xx} * X_{max})/\sum X_i^2 - (M_{yy} * Y_{max})/\sum Y_i^2$

Moment of Inertia of Piles

$\sum X_i^2 = 90.00\ m^2$

$\sum Y_i^2 = 270.00\ m^2$

Maximum Load will be on outermost pile

So, X max = 3.00

Y max = 6.00

Design of Pile:

Concrete grade (fck) =30 N/mm²

Steel grade (fe) = 500 N/mm²

Allowable stress of steel in tension and shearSst = 240 N/mm²

Allowable stress of steel in direct compression Ssc = 205 N/mm²

Allowable compressive stress in concrete in flexure Scbc = 10.00 N/mm²

Allowable comp. stress in concrete in direct compression Scc = 7.5 N/mm²

Modular ratio (m) m = 10

Neutral axis factor k= 0.29

j =0.90

The resisting moment coefficient R= 1.33

Cover= 80 mm

Horizontal Force Per Pile Non Seismic =19.5 kN

Seismic = 186.9 kN

Elasticity of Concrete=2.74E+04 MN/m2

Moment of Inertia= 4.91E-02 m4

Calculation for Cohesionless Soil Calculation for Cohesive Soil

Calculate Not Applicable

ηh = 5 MN/m3 k1 ((IS :2911/Part1/Sec2-2010/Table 4)) 20000 kN/m3

(Table 3 IS 2911) K or ηh 4

Stiffness Factor T = 3.06 m Relative Stiffness Factor, R= 4.3 m

Embedded length of Pile (Le)= 15.80 m Embedded length of pile, Le= 15.80 m

L1= 3.10 m L1 = 3.10 m

L1/T= 1.0 L1/R = 0.72

Lf/T 2 Lf/R 1.950

Lf = 6.12 Lf= 8.3 m

Summary of reinforcement of Pile Section

Provide 46 nos of 20 mm dia bars PLP1

Ast provided = 14452.00 mm2 ok

Lateral Ties

Minimum volume of lateral reinforcement per meter length of pile 0.3 %=2356194.49 mm3

Volume of tie of 10 mm tie = 207261.7 mm3

Number of Ties per meter of pile = 12 & Spacing = 84.00 mm c/c PLP2

Summary of reinforcement of Pile Section

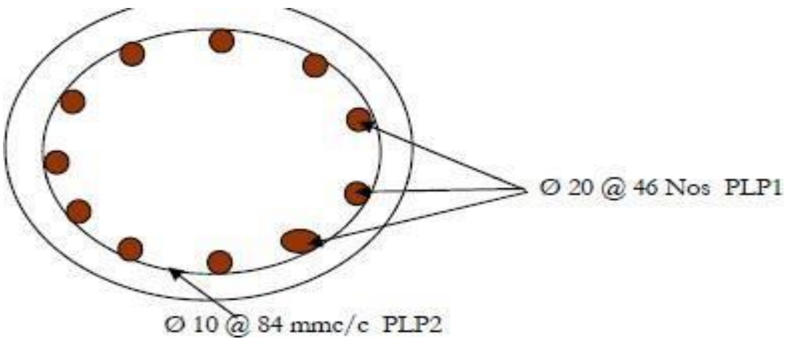


Fig.29 Pile Section

Design of abutment:

Material properties:

Grade of steel = Fe500=500N/mm^2

Grade of concrete=M35=35N/mm^2

Allowable stress of steel in tension and shear (S_{st})= 240N/mm^2

Allowable stress of steel in direct compression (S_{sc})= 205N/mm^2

Modular ratio m=10

Resisting moment R=1.33

Neutral axis factor k = 0.2 ;j= 0.90

High Flood Level = 10 m

Average Ground Level = 8m

Total depth of longitudinal Girder including Slab =2.00 m

Provided Clear free board = 1.5 m

Thickness of abutment cap= 0.9 m

Total height of abutment=11m

Length of abutment=11m

As per IRC : 6-2000, 217.1 for Equivalent live load Surcharge=1.2 m

Approach slab:

Thickness of approach slab=0.5m

Length of slab= 3.5m

Width of slab =9m

Dimensions of ballast wall:

Width of ballast wall: 0.5m

Length of ballast wall:10m

Thickness of wing wall=0.5m

Design of abutment stem:

Area and CG calculation with respect to stem of abutment

Symbol	Area (m)	CG – X	CG –Y	Weight (KN)
A1	1.76	0.20	5.45	464.64
A2	1.35	1.15	6.95	356.40
A3	9.750	1.13	3.25	2574.00
A4	0.98	2.00	2.17	257.40
A5	5.95	-1.17	8.77	57.12
A6	3.50	-1.75	10.40	33.60
A7	0.13	-0.13	10.35	33.00
TOTAL	23.41			3776.16

Forces on Abutment:

Total Dead Load from superstructur= 7000 KN Total

Critical Live load Excluding impact =1580.00 KN

Total Critical Live load including impact = 1680 KN

Earthpressure force:

Static earth pressure according to IRC 6:2000

Live load surcharge(Ý) =1.5

$0.5x\gamma xH_{eq}^2 xtan^2(45®-Ø/2)*L$

$=0.5x1.5x11^2 xtan^2(45®-35/2)*11$= 3400KN acting at distance of 0.42 from abutment base

=0.42x11=4.62m

Effect of buoyancy: [IRC:6-2000, 216.4 (a)]

Area of stem at top = 16.5 m^2

Depth of submerged part of abutment = 5.90 m

Area of stem at base = 19.8 m^2

Area of stem at HFL = 19.495385 m^2

Volume of submerged part of abutment = 115.92138 m^3

Taking 1/2 of the volume, Net upward force due to buyoncy = -579.6069 kN

Frictional force due to resistance of bearings:

For Pot Bearing:

Vertical dead Load=6800kn

Contact area of Pot Bearing (Assuming size 500mmX500mm)= 250000 mm^2

Contact Stress (sp)= 8.56 kN/mm2

Pot bearing constant (k)= 1.00

Maximum Friction Coefficient (µmax)= 0.065

Maximum Frictional Force = 138.36 kN

Total Lateral force due to frictional resistance of bearings=276.72 kN

Lateral force due to frictional resistance of bearings = 276.72 kN

Breaking Force:(As Per IRC:6-2000, 214.2)

Braking force = 20% of the weight of the design vehicle (Class AA)

And this force acts along the bridge at 1.2m above the road level 12.10 m from base

Total weight of the IRC Class AA vehicle = Total weight of the IRC Class AA vehicle = 700. kN

Therefore braking force length = 70kN

Load and moment calculation:

Loads	**Load coefficient as per irc**	**Vertical force (kN)**	**Horizontal Lever arm, (m)**	**Horizontal force (kN)**	**Vertical Lever arm, (m)**	**Moment (kN.m)**
Combination I	Dry case, Non-seismic Increment factor for allowable stresses					**1**
Superstructure dead load	1	6800kn	0.01			54.27
Live load	1	**1282.55**	**0.01**			**10.24**
Abutment	1	**3776.16**	**0.00**			**0.00**
Soil mass	1			**3491.46**	**5.08**	**17743.59**
Tractive/Braking force 1	1			**54.33**	**12.10**	**657.38**
Frictional force	1			**276.72**	**7.40**	**2047.76**
Total		**11858.71**		**3822.51**	**24.58**	**20493.14**

Design of abutment stem section:

Abutment Stem will be designed as compression member with uniaxial moment.

Overall Thickness of Stem at base D = 1800 mm

Length of the abutment L = 11000 mm

Gross cross sectional area of the stem Ag = 19800000 mm²

percentage of longitudinal tensile reinforcement (pst)= 0.25 %

the percentage of longitudinal compressive reinforcement (psc)= 0.13 %

Percentage of steel to be provided as per IRC:21 2000 306 2 2 Percentage of steel to be provided as per IRC:21-2000, 306.2.2 0 3. %

Total percentage of longitudinal reinforcement = 0.38 % OK

Then the initial total area of reinforcement Asc = 75240 mm²

Net area of concrete Ac = 19724760 mm²

Let the effective cover (referring to the CG of bars) cover (d')= 60 mm

Hence the effective depth deff = 1725 mm

Moment of inertia I = 4.788.E+12 mm^4

Section modulus Z = 5.519.E+09 mm³

Radius of gyration SQRT(I/Z*L) k = 501 mm

Height of the abutment (upto abutment cap) =6000mm

Effective length (height) factor (IRC:21-2000, 306.1.2, Table 13) = 1.75

Effective height of the abutment = 11375 mm

Ratio of Effective length : Radius of gyration = 22.71

Hence it is treated as a Short Column

Reinforcement Calculation:

Reinforcement	Area (mm^2)	Bar dia (mm)	Nos	Spacing (mm) c/c	Provided Nos
Tensile reinforcement (AS1+AS2)	49500	25	105	100 - AS1	110
Compressive Reinforcement (AS3+AS4)	25740	20	85	130 - AS2	85

Total area of tensile reinforcement Ast= 51542 mm²

Total area of compressive reinforcement Asc= 26704 mm²

Total provided area of longitudinal steel = 80700 mm²

Check For Shear:

Critical shear force at the base 3822510.67 N

Effective area of the section 19800000 mm²

Shear Stress 0.193 N/mm²

Permissible Shear Stress Permissible Shear Stress= 0 270 . N/mm² OK

[IRC:21-2000, Table 12B]

Distribution bar reinforcement calculation:

Let the percentage of distribution bars be 20 % of the total longitudinal reinforcement

Hence, area of distribution bars = 16139.932 mm²

Let's use bars of 16 mm Unit area = 201.06 mm²

Total number of distribution bars on each face of the stem = 41 nos Spacing @ 160 mm c/c

Provided spacing 160 mm and bar dia is 16 mm (AS3)

Distribution Bar calculation

Provided spacing 160 mm and bar dia is 16 mm (AS3)

No of Bar 56 on each face of stem

Development / Lap length to be provided where necessary = 1150 mm

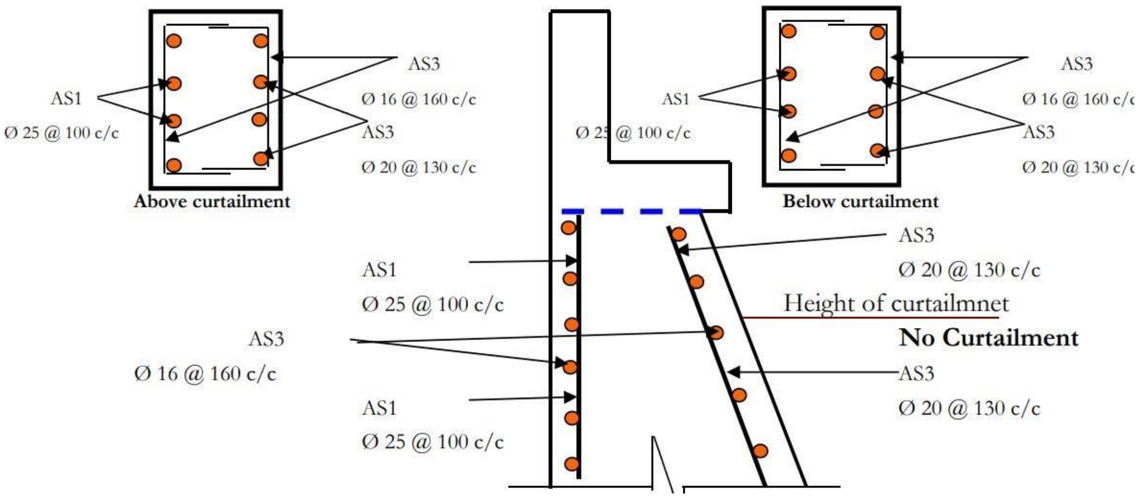


Fig.30 Reinforcement detailing

Design of abutment cap:

Calculation of Vertical Load

Superstructure Dead Load = 6800KN

Live Load Including Impact = 1282.6 KN

Total Load = 8282 KN

Total Load per Girder= 2070 KN

No of Longitidunal Girder=4

Depth of Abutment Cap D = 900 mm

Check For Punching Stress:

Bearing Size provided L= 500 mm

B= 500 mm

Allowable punching Stress = Allowable punching Stress = □au p = ks(0 16 au_p = ks(0.16 sqrt(fck))*sqrt(fck))

Where k s is minimum of 1 and 0.5 + bc and bc = B/L 1 So, ks = 1

Allowable punching Stress tau _p = 0.876 N/mm²

Total Punching Stress Developed Tau_ developed = V/Po*D

where P_o is perimeter of affected Area = 2 (2D+L+B)

P_o 5600 mm

So, Punching Stress Developed □au_ developed = 0.5518 N/mm²< 0.876 N/mm² Ok

As depth is safe for punching no additional reinforcement is required. Providing nominal reinforcement

Reinforcement Bar	**dia (mm)**	**Nos**	**Spacing (mm) c/c**	**provided Level**
Reinforcement along length of cap	12	28	175	AC1
Stirrups around the cap	10	62	175	AC2

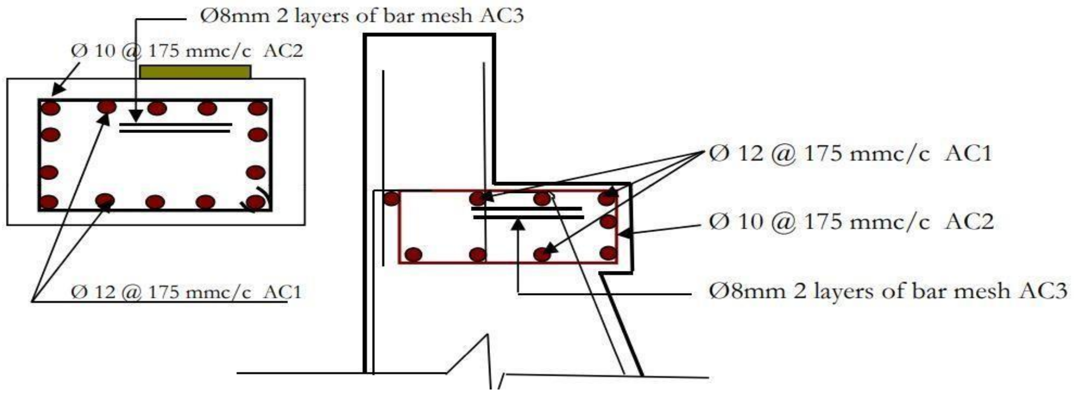


Fig.31 punching stress

Design of Back Wall/DirtWall:

Total Horizontal force due to earth pressure including live load surcharge is given by

0.5.ýs.(height of ballast wall+1.2(eq live load surcharge))2 .tan^2 (45°-Ø/2)*L= 420.16 KN which acts at a distance 0.42H from backwall base

Self weight of backwall = 316 8. kN these act at a distance from backwall toe of 0.2 m

Moment due to earth pressure about abutment base = 741.17 kN.m

Moment due to seismic forces = 132.35 kN.m

Moment due self weight= 63.36 kN.m

Total Moment about backwall toe = 936.88 kNm

Total Base Shear = 483.19kN

Providing 40 mm cover and total thickness of ballast wall is 400 mm &dia of main bar & Distribution bar are 25 mm & 12 mm respectively

So, available effective depth = 322.5 mm

Reinforcement	**Dia of Bar**	**Spacing (mm) c/c**	**provided Nos**	**Level**
Main Bar (Back Face)	25	210	53	AB1
Distribution Bar (Horizontal bar at each face)	12	300	11	AB3
Compression Bar (Front Face)	20	260	43	AB2

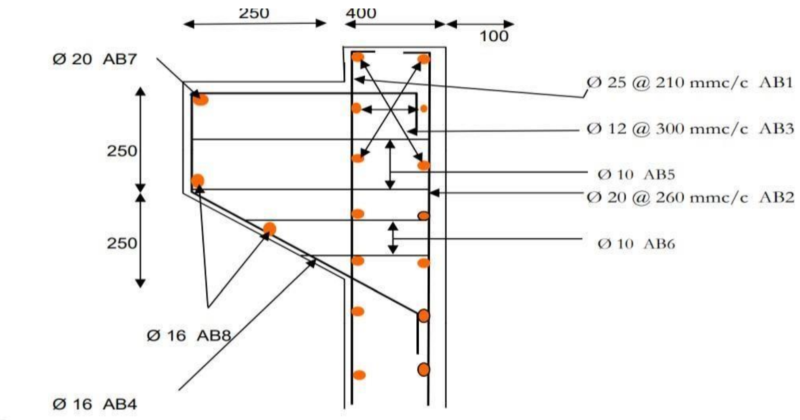


Fig.32 Reinforcement detailing for back wall

Design of abutment foundation:

Length of Pile cap Along Brodge Axis = 7.40 m

Length of Pile Cap Across Bridge Axis = 13.40 m

Depth of Fixity from maximum Scour Level = 6.12 m (IS 2911 part I section II, Appendix C, Adopting Max value)

Diameter of Pile = 1 m

Depth of Pile = 20.00 m

No of Pile in one row = (Along Bridge Axis) = 3

No of Row = 5

Total No of Pile (n) = 15

Embedded length of Pile = 12.86

Thickness of Pile Cap = 1.50 m

IRC 78:2000 Cl 709.5 OK

Factor of Saftey FS = IRC 78:2000 Cl 709.3 2.5

offset of pile cap from the outer face of outermost pile = 0.2 m Ok

Center to center distance of the piles

Along Bridge Axis (Xi) = 3.00 m

Across Bridge Axis (Yi) = 3.00 m

Width of Pile Group (Outer Surface of The piles) along Axis (B) = 6.70 m

Width of Pile Group (Outer Surface of The piles) across Axis (L) = 12.70 m

Area Enclosed by pile Groups (Ag) = 85.09 m^2

Area and C.G Calculation with respect to CG of Pile Cap:

Symbol	Area (m2)	CG-X	CG-Y	Weight (KN)
A1	**1.76**	**0.70**	**6.95**	**464.64**
A2	**1.35**	**-0.25**	**8.45**	**356.40**
A3	**9.75**	**-0.05**	**4.75**	**2574.00**
A4	**0.98**	**-0.70**	**3.67**	**257.40**
A5	**5.95**	**-1.63**	**10.27**	**57.12**
A6	**3.50**	**2.65**	**11.90**	**33.60**
A7	**0.13**	**1.03**	**11.85**	**33.00**
A8	**11.10**	**0.00**	**0.75**	**2930.40**
Total 34.51				**6706.56**

C.G from CL of cap -0.0064 3.427 m

Position of superstructure load point CG of pile cap= -0.01 m

Position of C.G From Superstructure Load Point= 0.12 m

Height of Abutment (H)= 10.90 m

Height of Abutment Including Cap (H')= 12.40 m

Length of Abutment (L)= 11.00 m

Over all Length of Cap (L')= 7.40 m

Maximum load on individual piles

Maximum and Minimum Load is given by

$V\ max = [V/n] + (Mxx*Xmax)/\sum Xi^2 + (Myy*Ymax)/\sum Yi^2$

$V\ min = [V/n] - (Mxx*Xmax)/\sum Xi^2 - (Myy*Ymax)/\sum Yi^2$

Moment of Inertia of Piles

$\sum X\ i^2 = 90.00\ m^2$

$\sum Y\ i^2 = 1134.00\ m^2$

Maximum Load will be on outermost pile

So, X max = 3.00

Y max = 6.00

Design of Pile

Concrete grade (fck) = 30 N/mm²

Steel grade (fe) = 500 N/mm²

Allowable stress of steel in tension and shear Sst = 240 N/mm²

Allowable stress of steel in direct compression Ssc = 205 N/mm²

Allowable compressive stress in concrete in flexure Scbc = 10.00 N/mm²

Allowable comp. stress in concrete in direct compression Scc = 7.5 N/mm²

Modular ratio (m) m = 10

Neutral axis factor k 0.29

J= 0.90

The resisting moment coefficient R= 1.33

Cover = 80 mm

Horizontal Force Per Pile Non Seismic= 254.8 kN

Seismic = 394.3 kN

Elasticity of Concrete, 2.74E+04 MN/m2

Moment of Inertia, 4.91E-02 m4

Soil Type Cohensionless Soil

Calculation for Cohesionless Soil	Calculation for Cohesive Soil
Calculate	Not Applicable

ηh = 5 MN/m3 k1 ((IS :2911/Part1/Sec2-2010/Table 4 20000 kN/m3

(Table 3 IS 2911) K or ηh =4000

Stiffness Factor T 3.06 m	Relative Stiffness Factor, R 0.8 m
Embedded length of Pile (Le) =12.86 m	Embedded length of pile, Le =12.86 m
L1 7.14 m	L1 7.14 m
L1/T 2.3	L1/R 8.88

Lf/T 2 Lf/R 1.950

Lf= 6.12 Lf = 1.6 m

Fixed End Moment,

Seismic case	Non seismic case
MF =1689.63	**2614.52 KNm**
Reduction Factor, m= 0.85 Fixed Head /Amend	**0.85 (IS :2911/Part1/Sec2-2010/Fig. 3,**
Actual maximum moment, M 1436.18	**2222.34 KNm**
Maximum Axial Force (kN) 1554.73	**1691.16**

Design For Non Seismic Case

Sectional area of pile = (Ag) 785398.2 mm²

Non Seismic Case

Let Provide main reinforcement 1.5 % of Sectional area

Total Area of reinforcement 11780.972 mm²

Let Provide 25 mm dia bars. Provided Number of Bar 24 (AP5)

Provide in one row

Spacing between the bars = 130 mm

Cover provided= 75 mm

Let provided diameter of transverse reinforcement = 10 mm

the diameter up to the line of reinforcement Dc = 850 mm

So Area of Steel Provided (As) = 11780.972 mm²

So Area of Concrete (Ac) = 785398.2 mm²

Check for Section capacity of Stem

Equivalent area of Section Ae = Ac+(1.5m-1)*As= 950331.8 mm²

Equivalent moment of inertia of section Ie = (PI*D^4/64) + (m-1)*As*Dc² / 8 =6.23E+10 mm4

Ze = 2*Ie/D = 124681958 mm3

Scc = P/Ae = 1.636 N/mm²

Scb = M/Ze = 11.519 N/mm²

(Scc/Sacc + Scb/Sacb) = 1.3700

Seismic Case: (Scc/Sacc + Scb/Sacb) = 1.346

Summary of reinforcement of Pile Section

Provide 24 nos of 25 mm dia bars APL1

Lateral Ties

Minimum volume of lateral reinforcement per meter length of pile 0.3 %= 2356194.49 mm3

Volume of tie of 10 mm tie 207261.7 mm3

Number of Ties per meter of pile = 12 & Spacing = 84.00 mm c/c APL2

Summary of reinforcement of pile section

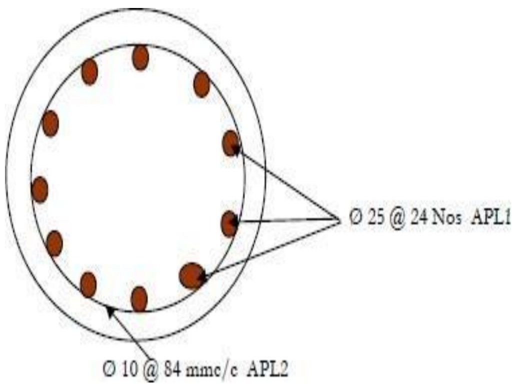


Fig.33 Pile section

CHAPTER 7

DESIGNING BRIDGE FOR DESIGN DISCHARGE, SCOURING AND SELECTION OF PIER SHAPE

Scouring and its phenomena:

Scouring is removal of stream bed in close proximity of structure constructed and cross a natural river and generally it takes place around bridge piers and abutments due to sudden change in flow pattern of water.Phenomenon of scour was studied briefly and effect of scour is known for the selection of appropriate pier shape which will experience least scour during the lifetime of the bridge.

Vortexes and formation around bridge pier

Depending upon the pier shape the flow pattern which consists of eddy currents can form.These vortexes causes local shear of bed and increase chance of scour and additionally scouring takes place when it is inclined to pier axis.

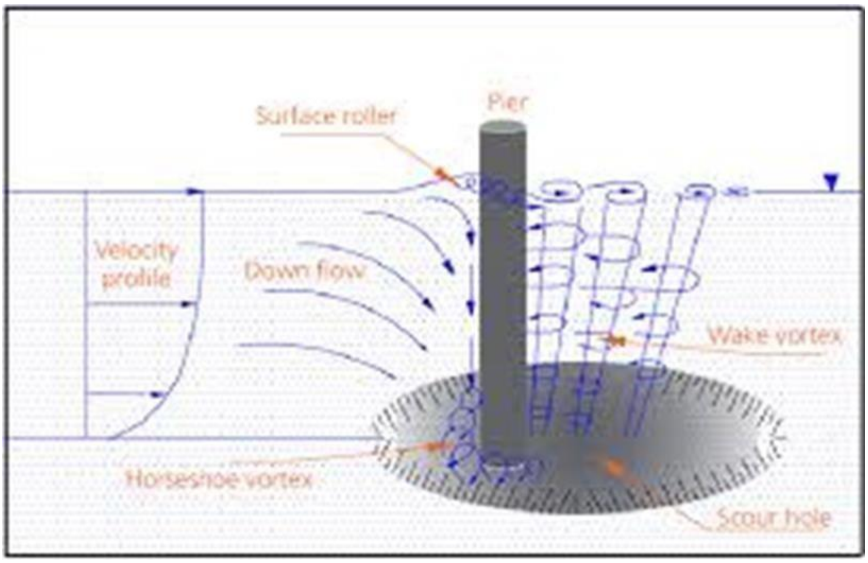


Fig.34 showing : Flow field around bridge pier

Conditions to be satisfied for minimum scour around pier:

Here, U = Sqrt .G.D.S

Uc = Shear velocity of bed material

D = depth of flow

S = river slope.

- *Based on theoretical analysis following points are derived which affect scour depth at bridge pier.*
- If down flow approaching pier is clear water occurs when U/U c <1.
- If down flow approaching pier carries sediment then U/Uc>1.

So in our case the scouring being **maximum** due to discharge exceeding design discharge the second case have been considered

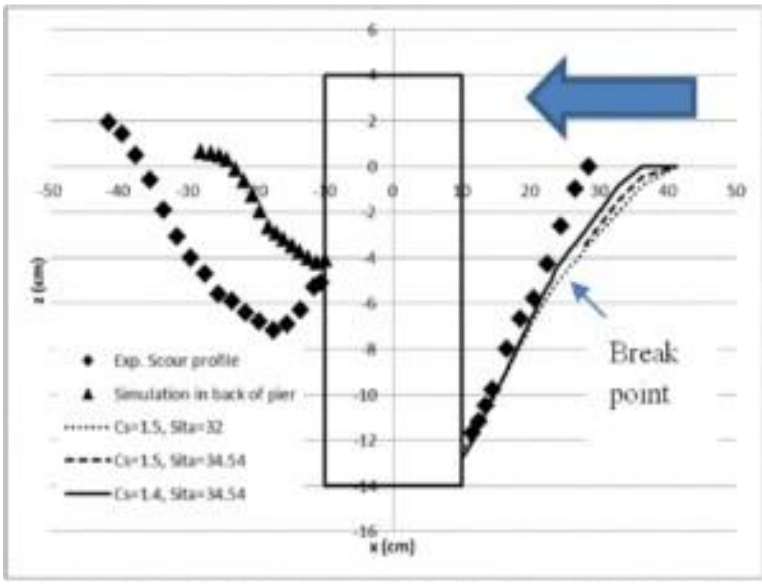


Fig.35showing downflow approaching pier

Design discharge of river and calculation of min scour depth

- Design discharge of Jhelum river = 25000m³/s
- Actual discharge during flood = 75000m³/s
- To ensure the safety of foundation of bridge the foundation must lie below normal scour depth and it can be calculated by following formula:

d= 0.473 (q/f)^1/3

d= 0.473 (25000/0.68)^1/3

Minimum scour depth, (d) = 15.62m

Therefore :

d =normal depth of scour

Q=design discharge

F= Lacey’s silt factor.

- Max. depth is calculated by taking 20% increase in design discharge = 1.20*15.62 = 18.75m

Selection of optimum pier shape:

- Since cross section of pier plays crucial role in reduction of stagnation of downflow around pier and there by reducing scouring

- Therefore there is a equation given by code (IRC – 6 -1966) to ensure minimum scouring pier by substituting the shape coefficients of different piers and there by deriving optimum shape of pier which is less prone to flow angle of downflow as this is main factor which Leads to scouring.

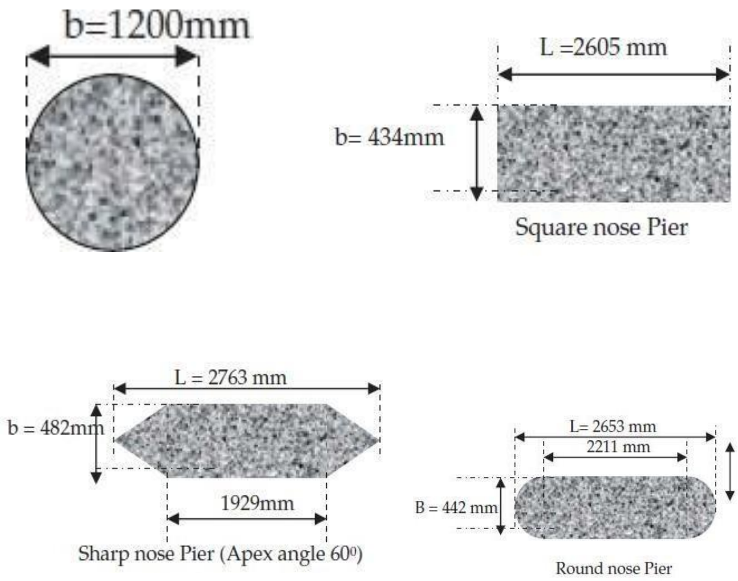


Fig.36 Following shapes were examined

Calculation of maximum scour for different pier types:

Maximum scour depths and discharge values for 2different conditions were assumed and mean velocity and flow depth values are obtained from previous records.

- Discharge : 75000 m3/s, Velocity v= 5m/s ,mean flow depth = 4.0m.

- Discharge: 25000m3/s, Velocity = 4m/s, mean flow depth = 4.0m.

Shape	Ks
Circular	1.0
Rectangular=2 to6	1.1
Square (2:1,3:1,4:1)	0.93, 0.79, 0.70
elliptical(2:1,3:1)	1.0, 0.86
Traingular	0.45

Effect of pier shape coefficients on maximum scour:

- *For calculating maximum scour depth above equation is used*

Ds/y = 2.0KS[b/y]^0.65*Fr1^0.43

Table showing: Average value of shape coefficients of pier shapes

Scour depth variation for flow condition 1&2:

Flow angle	Circular	Square nose	Round nose	Sharp nose
0 degrees	3.32	1.72	1.65	1.74
15 degrees	3.32	3.01	2.90	3.05
30 degrees	3.32	3.81	3.71	3.92
45 degrees	3.32	4.82	4.62	4.82

Flow angle	Circular	Square nose	Round nose	Sharp nose
0 degrees	3.02	1.56	1.50	1.58
15 degrees	3.02	2.73	2.63	2.77
30 degrees	3.02	3.51	3.38	3.56
45 degrees	3.02	4.37	4.2	4.42

These results are comparable because all pier cross sections are equal and it is evident that minimum scour depth occurs for circular pier and *maximum scour depth occurs for sharp nose pier* and scour depth doesn't change with scour angle for circular pier.But for other piers flow conditions are dependent on their cross sections and apex angle and scour depth will be larger as flow increases with flow angle.Therefore any bridge built across natural river experiences angular flow attack and it can be observed that flow conditions and sediment size does not have effect on circular pier and hence if pier shape is not a constraint we can opt for design of bridge with circular pier for safe and economic design across natural river crossings for the floods having maximum discharge.

CHAPTER 8

DESIGN OF BRIDGE IN CSI BRIDGE

8.1ABOUT CSI BRIDGE

Advanced modeling functions and complicated evaluation strategies account for dynamic effects, inelastic behavior, and geometric nonlinearity. Code-primarily based totally templates streamline the engineering method from version definition via evaluation, layout optimization, and the era of complete output reports. CSi Bridge is the most excellent software program for bridge engineering.

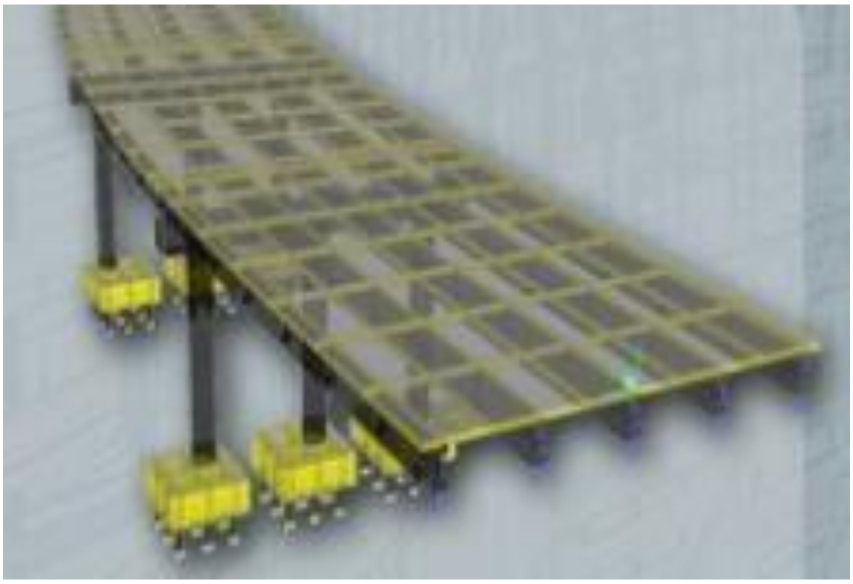


Fig.37 Bridge Model in CSI Bridge

8.2. Modeling of Bridge Systems

CSI Bridge - primarily based totally technique whilst growing analytical . permits meeting of objects (roadway superstructure, substructure, abutments, piers, basis system, etc.) routinely transfers the object-primarily based totally version right into a mathematical finite-detail version with the aid of using meshing the fabric area and assigning fabric properties. This object-orientated technique simplifies the want to immediately define

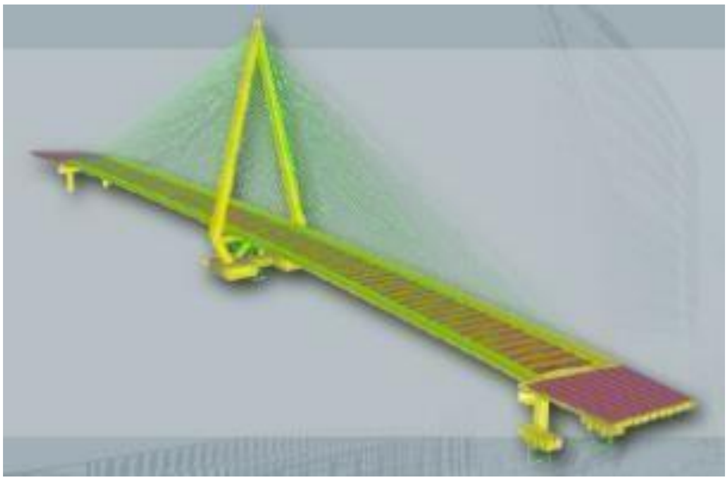


Fig.38 Modelling of Bridge

Loading and Analysis

After modeling, CSi Bridge offers alternatives challenge instances and consistent with constructing code (AASHTO LRFD, Canadian, etc.) and assigned consistent with version geometry.

A collection of templates make Csi Bridge object-primarily based totally version has been translated right into a finite-detail version and subjected to load instances and combinations, the evaluation technique follows directly. Analysis abilties cross properly past elastic overall performance into the and cloth nonlinearities offer perception into strength, ductility, and different overall performance measures vital to reaction beneath intense loading. Static-pushover and dynamic analyses (steady-state, reaction-spectrum, and time-history) offer similarly perception into earthquake resilience.

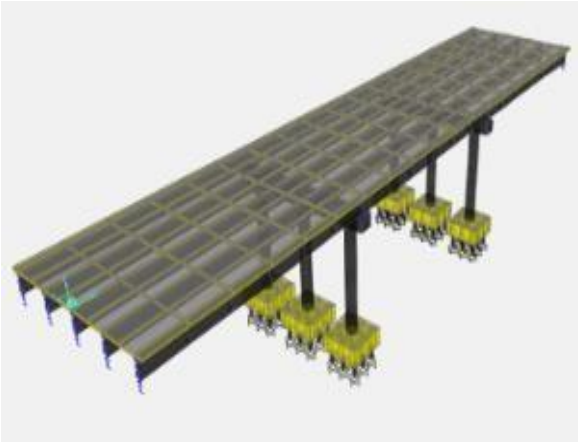


Fig.39 Loading and Analysis in CSI Bridge

design and output

computerized layout procedure evaluation processes to coordinate and optimize the resizing of bridge components.

reviews gift evaluation layout information in of formats.reaction record s and 3-d score in line with classification, influence-floor plots fordisplacement, reaction, and frame, shell, solid, or hyperlink reaction are all alternatives for output generation.

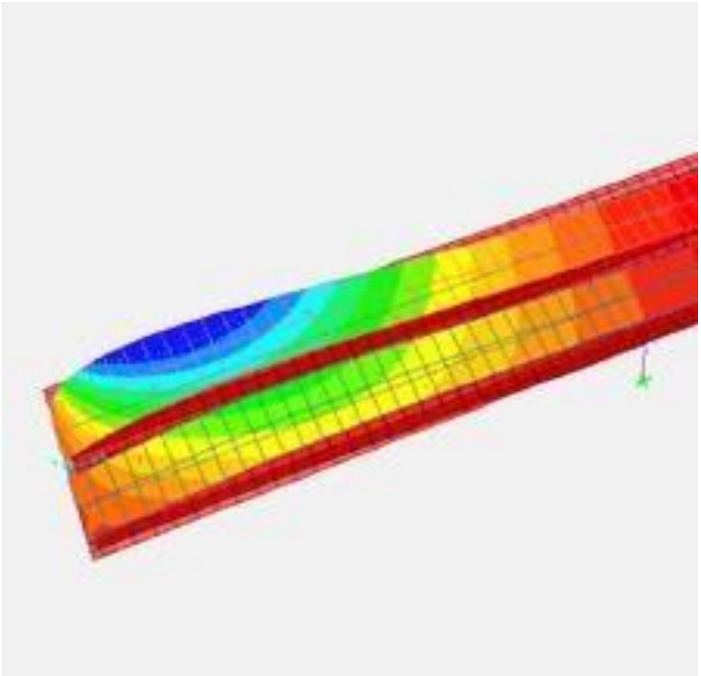


Fig.40. Design and Output

Analysis in Csi Bridge

I have designed bridge manually and compared it with csi bridge design and following outputs were obtained.

. Manual design calculations:

Bending moment	Dead load BM	Live load BM	Total BM	Units
Outer girder	3484.38	4246.2	7730.58	Kn-m
Inner girder	3484.38	3268.68	6753.06	Kn-m
Shear force	Dead load SF	Live load SF	Total SF	Units
Outer girder	790.02	928.46	1718.48	KN
Inner girder	790.02	645.12	1435.14	KN

Cross section of I girder:

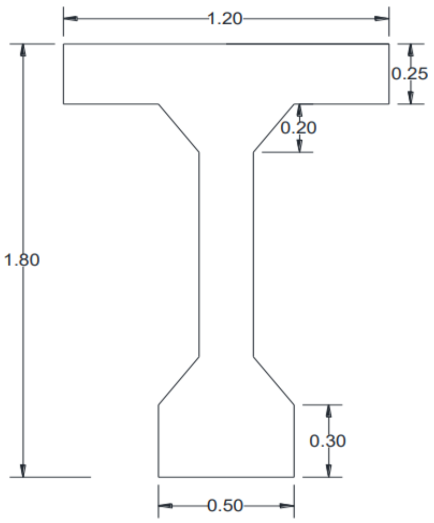


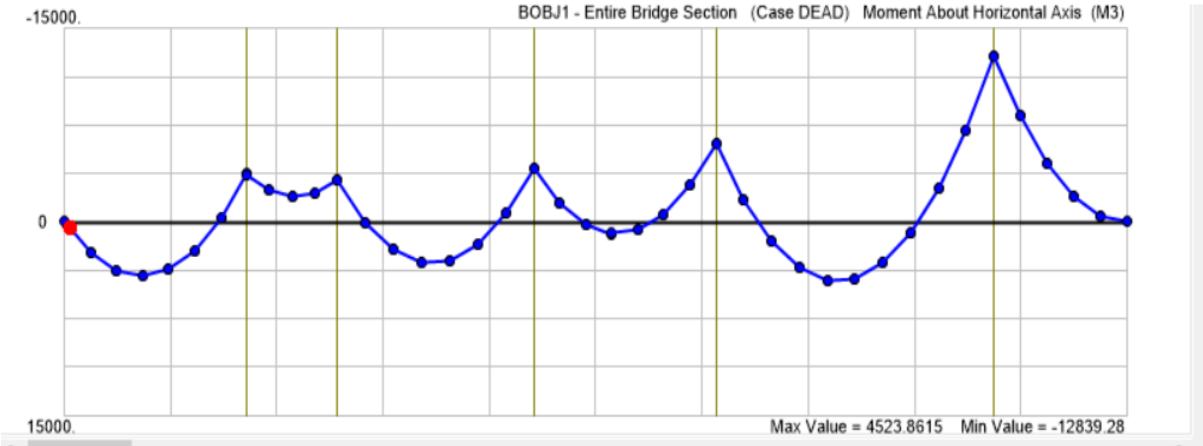
Fig.41 I Girder

Ultimate flexure and shear of I girder section:

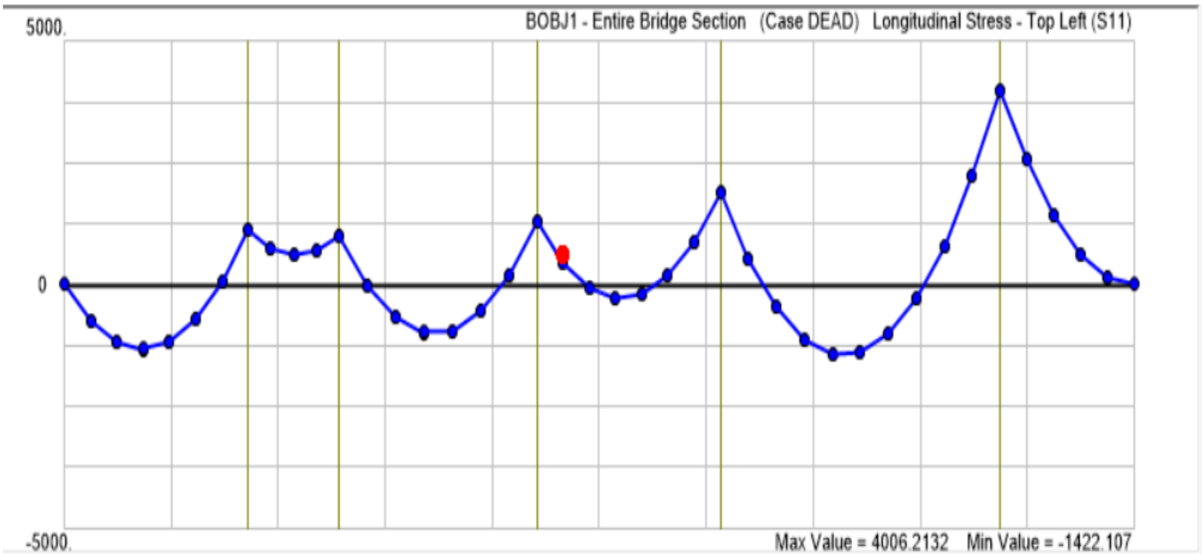
- Ultimate shear for I section including base shear = 4272.79 KN
- Ultimate flexure = 6380.95 kn-m

Csi bridge software result

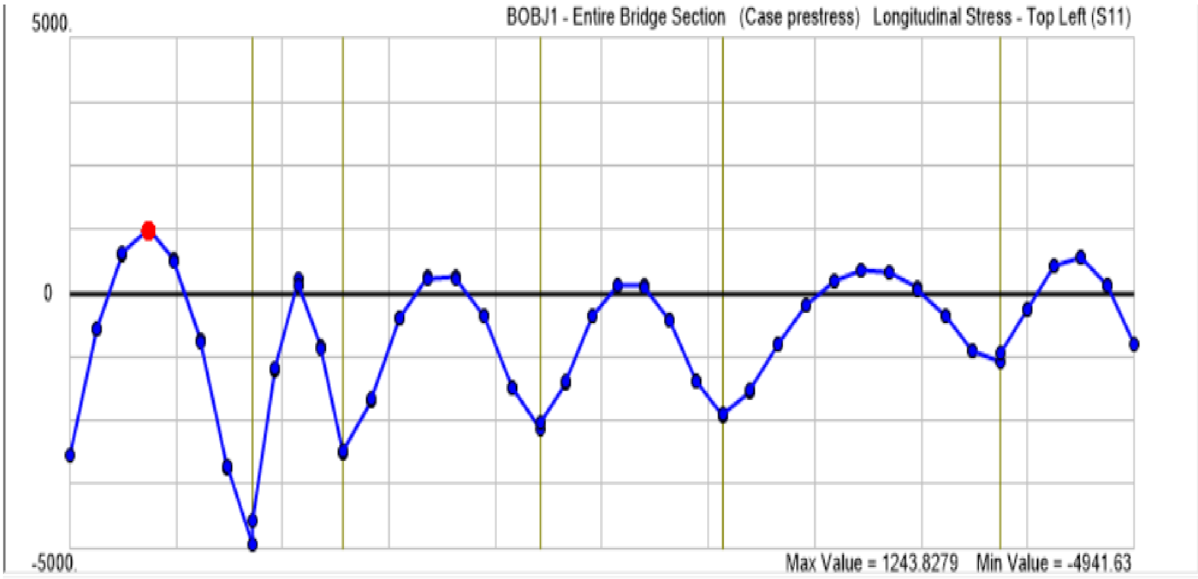
Dead load shear force as per design in software for entire superstructure length = 4523.86 KN (according to AASHTO)



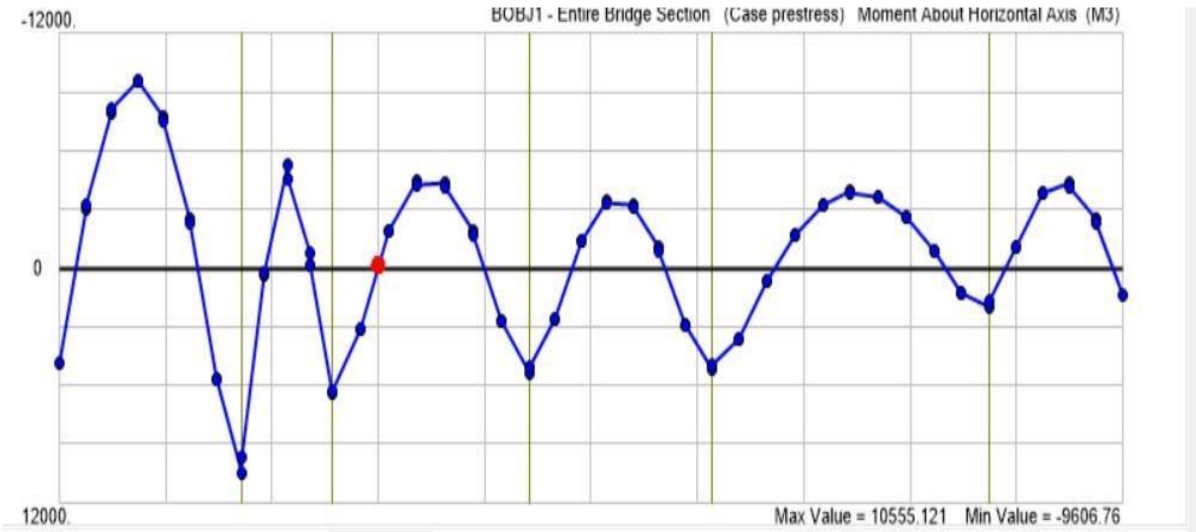
- Dead load bending moments as per design in software for entire superstructre length = 4006.2KN



Moment due to prestress in software = 1243.8 KN/m2



Shear force (including base shear) due to prestress = 10555.21kn-m



Superstructure Bridge Model:

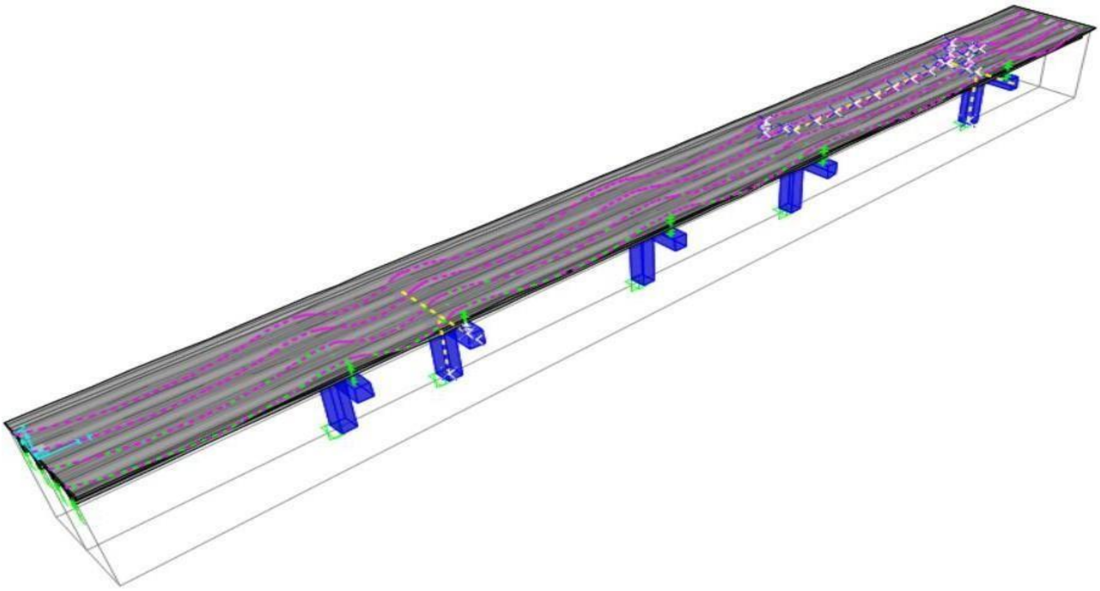


Fig.42. Showing final model of bridge in software

CHAPTER 9

CONCLUSIONS

- By designing the bridge and selecting optimum pier shape we have came to the conclusion that there is much need for inclusion of clauses that give deep elaboration of how to design bridge against flash floods. Since pier shape largely influences the flood effect on bridge precautions to be taken on proper selection of pier to minimize main factor called scouring .
- The overall maximum uplift force is found to be very large when the bridge substructure is totallyinundated.
- Upon many shapes that were examined the best shape that was found to be is circular shape which has less flow attack and inclination to its perimeter and plays a crucial role in reducing scour depth and reducing heavy downpour.
- To design bridge against flash floods we have selected a failure model of bridge which was failed due to flash floods which are result of high discharge flowing in jhelum river so we have took two discharge flow conditions and their velocities for purpose of design of bridge and
- After selecting flow conditions we have designed superstructure of bridge in csi bridge and compared it manually to obtain load coming fron superstructure After knowing the load which is coming from superstructure we have imposed that load on substructure for design purpose and also to decide the appropriate shape of pier which can resist flash floods For this purpose means for knowing which pier shape is best one we have took lacey equation which is given by irc:6 code for computation of scour depth
- In that equation we have substituted shape coefficients of different pier shapes in order to know the best pier shape
- Upon substituting the best shape obtained was circular shape because it has less angular attack of water flow to its surface and designed circular pier against uplift force resulting from flash floods

CHAPTER 10

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