IMPACT OF FLOOD GENERATED WAVE FORCES ON BRIDGE SUPERSTRUCTURE

A Project Report submitted in partial fulfillment of the requirement for the award of degree of

Master of Technology

In

Structural Engineering

under the Supervision of

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CERTIFICATE

This is to certify that the work which is being presented in the project title **“Impact of flood generated wave forces on bridge deck”** in partial fulfillment of the requirements for the award of the degree of Master of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by Ankush Thakur during a period from August 2014 to May 2015 under the supervision of **Mr. Chandra Pal Gautam** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

Date: - ………………………

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DECLARATION

I hereby declare that the research work presented in this Project entitled "*Impact of flood generated wave forces on bridge superstructure*" submitted for the award of the degree of Master of technology in the Department of Civil Engineering, Jaypee University of Information and Technology Wakhnaghat, is original and my own account of research. This research work is independent and its main content work has not previously been submitted for degree at any university in India or Abroad.

(Ankush Thakur)
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Last but not the least I would like to thank my parents, who taught me the value of hard work by their own example. I would like to share this bite of happiness with my mother and father. They rendered me enormous support during the whole tenure of my stay at JUIT, Wakhnaghat

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Date: **Ankush Thakur**

ABSTRACT

This study examines the effect of wave loading on a bridge model . The quantities of interest include the vertical uplift force generated due to the moments. Analysis is conducted for two stages with different height in each case: 1) intial impact 2.) Fully innudated Stage. The first stage starts from the time when the water free-surface elevation reaches the low chord of the bridge superstructure, the water free surface rises and reaches the top of the bridge barrier where it overtops the bridge and starts to flow on the bridge deck, and until the bridge is totally inundated . The second stage occurs when the bridge first becomes fully inundated, i.e., end of the first stage, and until the most critical events. The costing of the bridge superstructure is estimated. The Analysis is Done manually as well as on software (CSi BRIDGE).

For dead & live Load moments whereas the effect of wave is scene only in Software . The Bridge Design is done manually using Limit state Design (LSD) and checked using AASHTO LRFD using CSiBridge Software. Finally it is seen that whether the uplift force which Can be a cause of bridge failure exceeded the weight of bridge deck or not and results are Presented.

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Chapter 1 – SCOPE AND OBJECTIVE

Man has always pursued to cross the obstacles in his path by building bridges . His constant effort to innovate, invent and improvise has always created revolutionary materials and construction techniques to build bridges. One of the most influencing factors in any bridge design is its ability to resist lateral loads. It plays a very important role in choosing the best possible design as the amount of money that is required to build and rehabilitate a bridge is quite large as compared to other infrastructure projects. Hence, the need to find the cost-effective bridge arises.

Now-a-days, we see bridges that are constructed using either Pre-stressed Concrete or Steel. Pre- stressed Concrete is basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from external loads are counteracted to a desired degree. Pre-stressed concrete members are slender; therefore, weigh less than reinforced concrete members. They are durable as they don't have any cracks during service and are much cheaper than steel.

Steel is an alloy of iron and other elements that is mainly used for its strength weight ratio. Steel is strong both in resisting compression as well as tensio . Moreover, it weighs less than concrete thereby reducing the overall weight of the structure. It is ductile in nature so it is favored over concrete for earthquake resistance. However, it is costlier than concrete.

Both materials are very competitive, and it depends upon the purpose for which the bridge is being built. In this project I have made efforts to analysis & design the steel girder bridges with different depth of storm water . The site conditions, geometry and loading are kept constant for both cases so that I get a direct relationship between behavior of bridge during different stages. The design is based on Limit State Design (LSD) philosophy. I present the design using CSiBridge software that uses AASHTO Load Resistance Factor Design (LRFD). In the end I analyzed the effect of storms and flooded water on the Bridge superstructure and compare the moments and tried to give a valid design.

Chapter 2 – INTRODUCTION

2.1 Bridge-Definition

According to IRC: 5-1998, a bridge is defined as a structure having a total length of above 6 meters between the inner faces of the dirt walls for carrying traffic or other moving loads over a depression or an obstruction such as a channel, road or railway. These bridges are classified as given below

a. Minor Bridge: A minor bridge is a bridge having a total length of up to 60 m.
b. Major Bridge: A major bridge is a bridge having a total length above 60m.

2.2 Components of bridge

The main parts of a bridge structure are:

1. Decking, consisting of deck slab, girders, trusses etc.
2. Bearings for the decking.
3. Abutments and Piers.
4. Foundation for the abutments and piers.
5. River training works like revetment of slopes for embankment at abutments and aprons at river bed levels.
6. Approaches to the bridge to connect the bridge proper to the roads on either side.
7. Handrails & parapets

The figure on the next page shows the components of a bridge structure. The components above the level of bearings are grouped together as ***super-structure,*** while the parts below the level are classed as ***substructure***

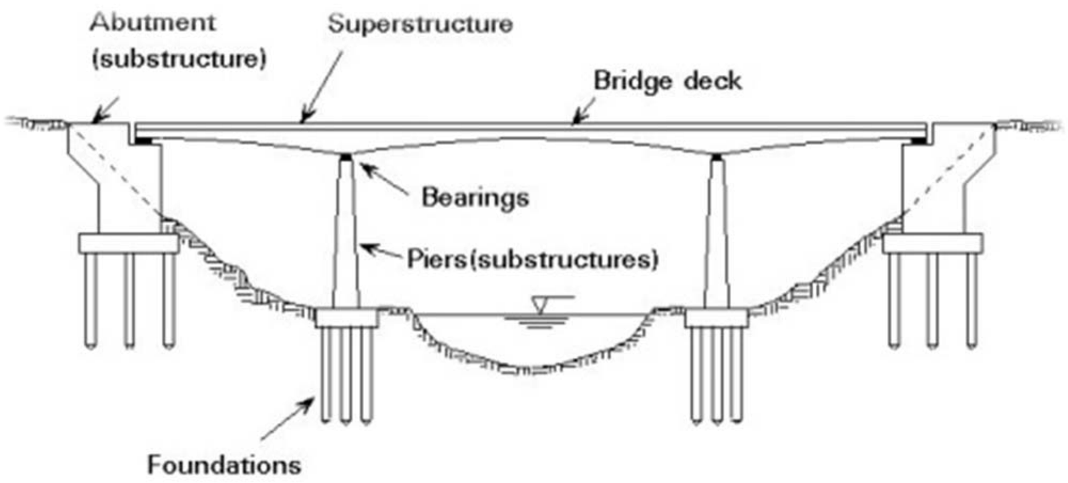

Figure 1: components of bridge

2.3 Flow of Forces

The loads that are acting on a bridge structure are first imposed on the deck slab. There are various types of decks:

1. Pre-stressed Concrete Bridge Deck
2. Reinforced Concrete Bridge Deck
3. Orthotropic Steel Bridge Deck

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.

Cables (as in cable stayed and suspension
bridge).

Arch Rib (as in arch bridge).

Box Girders.

Balanced Cantilevers.

The load is taken up by the above structural system and then transferred to the substructure in the form of shear forces and bending moments acting at the supports. The above system of force transfer decides the type of bridge. The loads are then transferred to the ground by piers/ bents and abutments.

2.4 Bridge Loading Standards

Highway bridge decks have to be designed to withstand the live loads specified by the Indian Road Congress (IRC). The standard IRC loads specified in IRC: 6-2000 are grouped under four categories as detailed below.

1. **IRC Class AA loading :** Two different types of vehicles are specified under this category grouped as tracked and wheeled vehicles. All the bridges located on National and State Highways have to be designed for this heavy loading. The IRC Class AA tracked vehicle (simulating an army tank) of 700 kN and a wheeled vehicle (heavy duty army truck) of 400 kN are shown:

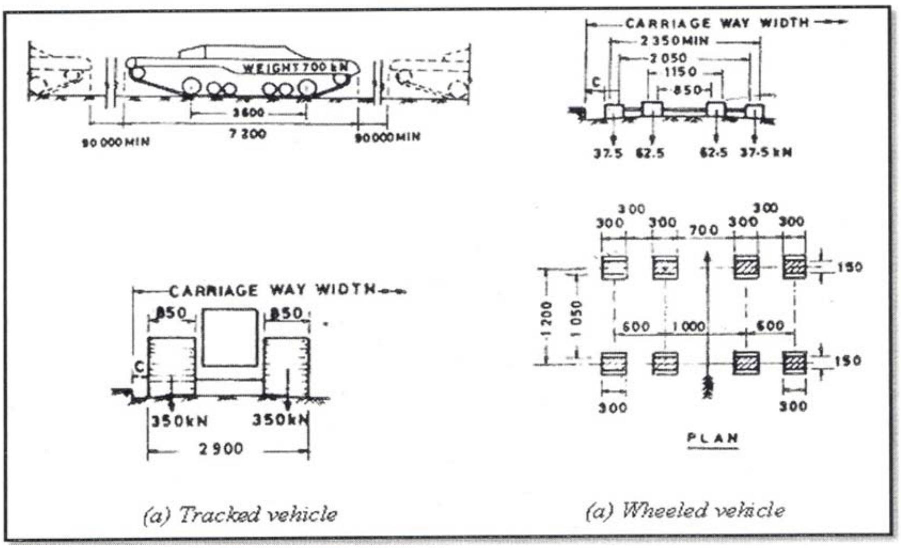

Figure 2 : IRC class AA & wheeled Vehicle

2. **IRC Class 70R Loading :** This loading consists of three types of vehicles.
   a) Tracked vehicle of total load 700 kN with two tracks each weighing 350 kN.
   b) Wheeled vehicle comprising 4 wheels, each with a load of 100 kN totaling 400 kN.
   c) Wheeled vehicle with a train of vehicles on seven axles with a total load of 1000 kN.

3. **IRC Class A Loading** : This type of loading consists of a wheel load train comprising a truck with a trailer of specified axle spacing and loads.
4. **IRC Class B Loading:** Class B loading is similar to Class A loading except that the axle loads are comparatively of lesser magnitude . This type of loading is

adopted for temporary structures and timber bridges

2.4.1 Impact Factor

Impact factors are generally applied to the moving wheel or distributed loads to enhance their magnitude to include their dynamic effects on the bridge deck. The impact factor is always inversely proportional to the length of the span and is different for reinforced concrete and steel bridges.

For IRC Class AA Tracked Loading, span of 9 m . or more, the impact factors for the following bridges is given below.

a) RC Bridges- 10% up to a span of 40 m.

b) Steel Bridges- 25% up to a span of 23 m.

2.5 Materials For Composite Steel Bridge

2.5.1 Grades of Concrete

According to IRC : 18-2000 , the minimum prescribed characteristic compressive strength of concrete should not be less than 35N/mm^2 • The code also stipulates that for Pre-stressed concrete construction , only “Design Mix Concrete” should be used. The concrete mix should be designed as per the Indian Standard Code IS: 10262- 1982 which sets out the guidelines for concrete mix design.

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 1-Properties & Basic Permissible Stress in concrete

2.5.2 Un tensioned Steel or Supplementary Reinforcement

Supplementary reinforcements are required in Pre-stressed concrete beams and slabs to safeguard against cracks and for resisting shear forces.

2.6 Composite Plate Girder Bridges

2.6.1 Introduction

A plate girder is simply a girder (beam) made up of steel plates which are connected by rivets or welds. Earlier, the elements of a plate girder used to be riveted together using high strength rivets, which in turn have given way to the welded plate girder. Plate girders can be used for simply supported spans from 20 m to 30 m and for continuous spans up to 250 m. The figure below illustrates some of the important components of a plate girder.

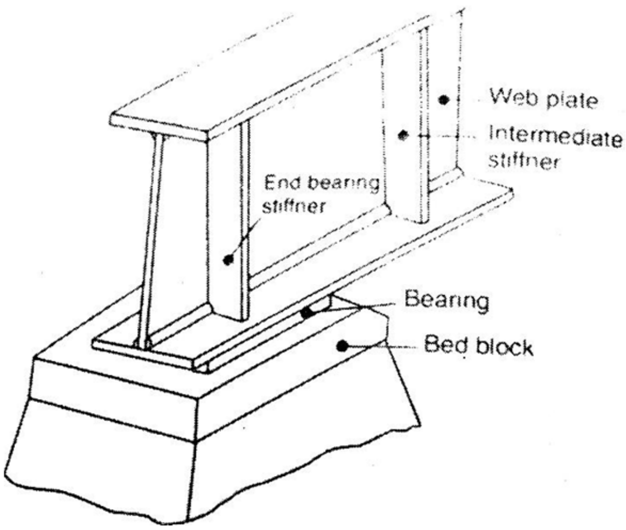

Figure 3: - Important Components of Plate Girder
(Source: Design of Bridge Structures by T.R. Jagadeesh & M.A. Jayaram)

2.7 Important elements of a Plate Girder

2.7.1 Web

The web of a girder can be of constant height or varying height. The girder with varying depth of web is called a *haunched girder*. In designing the web of a plate girder, we compute its depth (which is dependent on maximum bending moment) and its thickness (which is dependent on shear stress). The depth of the web can be decided based on 'Economic depth' of the plate girder. It is given by

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

M = design bending moment after incorporating impact effect as given by the impact factor

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 caused by corrosion. Inadequate dimensioning of a web may lead to web buckling.

2.7.2 Flanges

A flange should preferably be a single plate. The width of the plate depends on the span to width ratio which ranges from 40 to 45. The flanges should be connected to the web by welds to transmit the horizontal shear force combined with any vertical loads. The thickness may be calculated based on the approximate requirements of the flange area. The area of the 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 outstand of the flange should not be greater than 20 times the thickness of the plate . The section designed must be checked for critical stresses as stipulated by IS: 800-2007.

2.7.3 Intermediate Stiffeners

The web must be adequately supported laterally by stiffeners to avoid web failures (diagonal buckling). It also complied with lower web thickness. There are two general types of stiffeners -

- Vertical stiffeners located over the length of the span.
- Bending stiffeners located at the supports of the span.

2.7.4 Vertical Stiffeners

The vertical stiffeners are provided at spacing not greater than 1.5d and not less than *0.33d,* where d is the depth of the web . The web panel dimensions between two stiffeners should not be greater than 270 times the thickness of the web. The length of outstanding leg of the vertical stiffener may be taken as 12 times the thickness of the web. These vertical stiffeners should provide moment of inertia, which should not be less than

$$I = \frac{1.5d^3e^3}{C^2}$$

where

I= moment of inertia of the pair of stiffeners about the centre of the web or that of a single stiffener about the face of the web

t = thickness of the web

c = clear distance between vertical stiffeners

d = depth of the web

$$F=\frac{125}{h} \times t^2$$

Where

F= Shear Force in KN/m

i= Thickness of web in mm

h= outstand of stiffener in mm

2.7.5 End Bearing Stiffeners

End bearing stiffeners are provided at the points of supports. The end bearing stiffeners strengthen the web and transmit heavy reactive forces to the flanges of the plate girders. The end bearing stiffeners are designed as columns . The sectional area of an end bearing stiffener consists of the stiffener together with some length of the web (20 times the thickness of the web) on either side of the centre line of stiffeners. This area is used to determine the radius of gyration and to check the column stresses. Finally, the load bearing capacity of the stiffener as a column should be greater than the applied load or reaction.

2.7.6 Lateral Bracing

Lateral bracing is a system of cross frames located in the horizontal plane and installed for connecting flanges in order to resist lateral deformation. Lateral deformation is induced by wind loads, which act normal to the centre line of the web. Lateral bracing is required if the span exceeds 20m.

2.7.7 Shear Connectors

Shear connectors are provided to prevent the separation between the steel girder and the *in-situ* concrete slab by transferring the horizontal shear force along the contact surface without slip. The most common types of shear connectors used in composite bridge decks are –

a) The rigid connector comprising short length bars, stiffened angles, tees or channels welded on the flange of the steel girders. U-type hoops are welded to the shear connectors to provide a rigid connection as shown in fig 4

b) The flexible connectors consisting of studs or angles or channels or tees welded on the flange plates of prefabricated units.

c) Anchorage type shear connectors are provided for composite sections comprising of precast prestressed concrete girders and cast *in-situ* reinforced concrete slab.

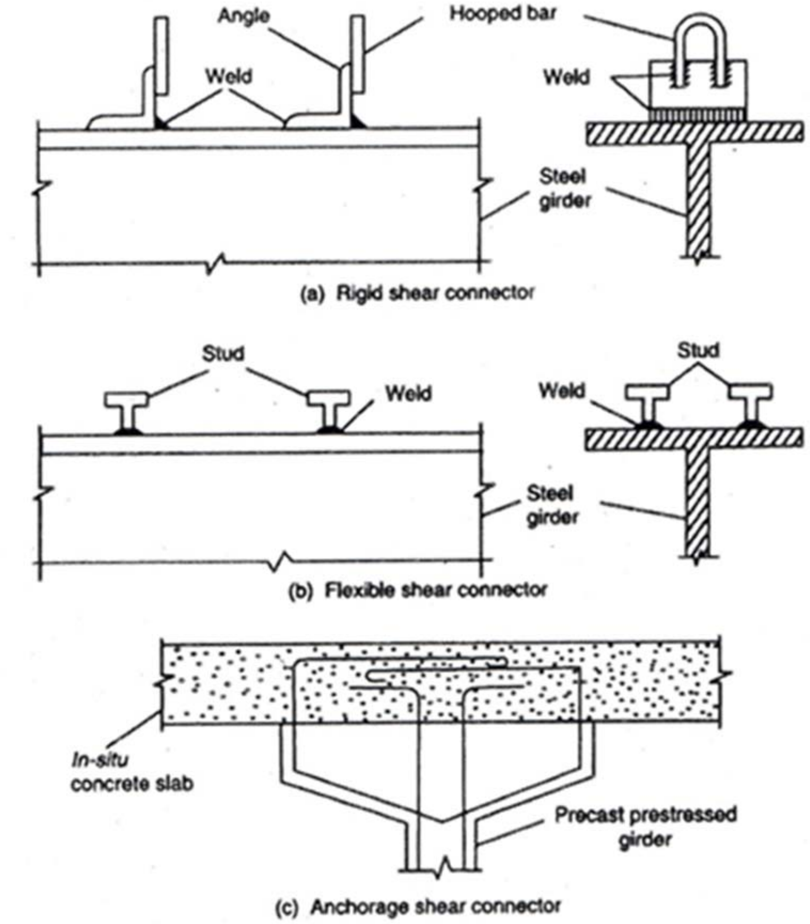

Fig 4 : Types Of Shear Connectors

Chapter 3 –LITERATURE REVIEW

3.1 General

In recent years, considerable attention has been paid to the wave impact in splash zones by many researchers and engineers all over the world. However, the wave impact phenomenon is fairly complex and involves strong nonlinearity of waves, traces of wave impact, fluid viscosity and turbulence. Hence, there is a continuous need for research in this area. On the other hand, wave forces acting on structures still remain an area of intensive research. Among these areas are loading on decks loaded with abrupt pressures and forces resulting from growing sea waves. The main reasons for these impact loads on deck slabs are local subsidence of the seabed during oil production that leads to lower deck clearance for bottom mounted structures and abnormal rise in sea level. Hence, an understanding on these extreme loads would lead to safe design of structures and increased reliability.

3.2 RESEARCH PAPERS

1. **CASE STUDY FOR TSUNAMI DESIGN OF COASTAL INFRASTRUCTURE SPENCER CREEK BRIDGE , OREGON**

   Solomon C. Yim, F; Yong Wei; Mohsen Azadbakht; Seshu Nimmala; and Tanarat Potisuk . In this paper it is Stated that most of bridges during past floods and earthquakes survived the Earthquake but destroyed by floods or tsunami's which indicates that seismic design criteria does not provide enough strength to resist loads acted on brides during flood and tsunamis loads. The paper presented the horizontal and vertical tsunami loads on the bridge deck. One of the main goals in the bridge design is to keep the superstructure on the supports; therefore, bridges can be used by traffic after natural hazard events.

2. **COLLAPSE OF STEEL BRIDGES**

   María Victoria Biezma and Frank Schanack Stated that Main causes of collapse of a steel bridge is

   - Force majeure :avalanche, flood, earthquake, terrorist attack etc.(65%)
   - Accidental overload and impact;(12%)
   - Structural and design deficiencies;(9%)
   - Scour;(9%)

- Construction and supervision mistakes; (3.5%)
- Lack of maintenance and inspection.(1.5%)

Paper mainly focuses on the failure of bridge due to design & structural deficiency . Examples are also illustrated which are chosen from a bibliographic research In the article explaining about how those bridges were collapsed and what was the reason.

3. Performance of Railway Bridges during the 2011 Tohoku Earthquake

Masato Abé and Makoto Shimamura talks about the performance of the railway bridges during tohoku earthquake (JAPAN) which are designed according to the upgraded seismic codes and planned seismic retrofits of . The codes are upgraded after the 1995 Kobe earthquake. The new bridges performed immensely well during earthquake and loss human losses and operation suspensions of the railway system were minimal. In this paper we also come to know that how Upgrading of seismic code requirements with planned (and executed) seismic retrofits contributed to reducing seismic dam-age incurred by bridges and civil structures; the fundamental strategy of the seismic retrofit (i.e., to increase the ductility of bridge columns) was shown to be effective. Typical damage patterns to bridge structures are illustrated in the figures cited.

4. Wave-Induced Pressures and Forces on Deck Slabs near the Free Surface.

K. Murali, V. Sundar , and Kannayya Setti Analyzed the hydrodynamic interaction of regular and random waves with a model offshore deck in a wave flume. The hydrodynamic pressures and forces on the model as well as its reflection and transmission characteristics are investigated in a dimensionless form. The experiments were carried out for three different clearances (e) between the still water level and the subface of the model. In order to investigate the effects of relative clearance of the deck (e/H in case of regular waves and e/Hs in case of random waves), different wave steepnesses (H/L in case of regular waves and Hs/Lp in case of random waves) were employed by varying wave height (H) and wavelength (L).

5. Simulation and Estimation of Tsunami Loads on Bridge Superstructures.

Mohsen Azadbakht and Solomon C. Yim, examines the estimated tsunami loads on five California coastal bridges. The quantities of interest include the horizontal and vertical forces and overturning moment. The simulations and analysis are conducted for two stages:

(1)initial impact and overtopping and (2) full inundation. The maximum uplift force during Stage 1 is found to occur when the tsunami water free-surface elevation reaches the top of the bridge barrier right before the water overtops the bridge and starts to flow onto the bridge deck. It is observed that the time interval representing the initial impact of the tsunami on the bridge superstructure leads to the maximum horizontal force, downward vertical force, and overturning moment. The overall maximum uplift force is found to be in tsunami scenarios in which the bridge superstructure is totally inundated, i.e., in Stage 2, if total inundation actually occurs. Analyzing a deck-girder bridge with a failed, i.e., removed, first seaward girder shows a 15% reduction in the maximum horizontal force. The uplift force is found to be approximately 25% larger for the bridge with a failed first girder. A design procedure is proposed to compute the maximum horizontal and vertical forces on bridge superstructures based on the simulation results. Good agreement between numerical predictions and formula estimations of the tsunami forces is observed. The proposed design procedure is intended to provide estimations of tsunami loads on bridge superstructures

6. **Analysis and Design of Arch-Type Pedestrian Bridge for Static and Dynamic Loads .**

James F. Welch, Mohammad A. Alhassan, ubna K. Amaireh presents the conceptual design of the pedestrian bridge considering four potential bridge concepts as well as the modeling, analysis and design details for the selected arch type pedestrian bridge. The selected concept for the pedestrian bridge was analyzed and designed using SAP2000 for static dead and live loads as well as for the wind loads according to AASHTO specifications and INDOT requirements.

Chapter 4 - PROJECT SPECIFICATION

4.1 Objective

To Determine the Effect Of wave on Composite Steel Girder Bridge And Determine The cost Effectiveness As well as checking the effect of submergence depth.

4.2 Specification

The Project Specification Are As Follows:

1. To Design A reinforced Concrete Bridge Deck With the following Specifications .
   - Clear Width Of roadway = 7.5 m
   - Span = 15 m
   - No. Of Traffic Lanes = 2 m
   - IRC Class AA Wheeled Live Load .
   - Wearing Coat thickness = 80 mm
   - M-35 Grade Of Concrete And Fe – 415 Grade Of Steel

2. To Design Composite Steel Plate Girders For Above Bridge Deck With the Following Specifications
   - Span = 15 m
   - Simply Supported
   - Rolled Steel Sections With Yield Stress Of 236 N/mm^2

3. To Check The Effect Of Submergence Depth on Maximum Wave Force .

4 To estimate cost of The following .
   - Steel Superstructure

4.3 Project Plan

The Project work was divided into two Semesters . In the first Semester I had completed the manual analysis And Designs Of the Bridges . In the Second Semester I had modeled Our Bridges in CSiBridge and Analyzed It And Checked Bridge for AASHTHO LRFD (Load Resistance Factor Design)

The project Was done in four Phases in Both Semesters . In the End results are Compiled At the end of this Report .

Chapter 5 :DESIGN APPROACH

5.1 Design Steps – Reinforced Concrete Slab

Load Calculation

- consider size of slab Panel . Assume a Depth
- calculate Dead Weight Of slab Panel + Dead Weight of Wearing Coat.
- Calculate Live Load. One wheel Is Placed at the centre of slab
- calculate dispersion of loive loads in slabs Spaning in two direction.

Live load analysis

- Using Pigeauds's curve to calculate live load moments in Short & long Span Direction including Impact Factor
- calculate Shear Force For Maximum Dispersion Of live Load.

Dead load analysis

- calculate deadLoad Moments and Shear Forces Using Pigeaud's Curves including Continuity Fcator
- Calculate Design Moments & Shear Forces(Dead+Live)

Design

- Calculate Reinforcement In Both Directions

Checks

- check the minimum depth of slab section for maximum moment using WSM
- check Shear Strength of Concrete Section

5.2 Limit State Design of Reinforced Concrete Bridge Deck Sections.

5.2.1 Design Philosophy

The Limit State Design is a method of designing structures based on statistical concept of safety and the associated statistical probability of failure. The limit state design philosophy recognizes the need to provide the structures which are serviceable at working loads and have the desired load factor against collapse.

5.2.2 Elastic Design Coefficients for RC sections

Based on the permissible stresses, the design constants are used for the computation of effective depth *'d'* of the structural element and the area of the steel *'Ast'* in the tension zone along with the neutral axis depth factor *'n'*, lever arm factor *'j'*, and the moment factor 'Q' expressed as a function of the permissible compressive stress 'σ_{cbc}' in concrete.

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

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

$$Q = 0.5\, \sigma_b\, n\, j$$

5.3 Analysis of Deck Slab

a) Dispersion of load along the span

The effective length of slab in the direction of the span is computed as the sum of the tires contact area over the wearing surface of the slab in the direction of the span and twice the overall depth of the slab inclusive of the thickness of the wearing surface.

If D =depth of the wearing coat

H = depth of the slab

x = wheel load contact area along the span

v = effective length of dispersion along the span We have the relation,

$$v = x + 2(D+H)$$

b.) Dispersion of slabs Spanning In Two Direction

In bridge decks comprising slab integrally cast with longitudinal and cross girders as in the case of Tee Beams and Slab Decks, the moments develop due to wheel loads on the slab both in the longitudinal and transverse directions. These moments are computed by using the design curves developed by M. Pigeaud. Pigeaud's method is applicable to rectangular slabs supported freely on all the four sides and the slab should be symmetrically loaded. The following notations are used in calculating the dispersion width and moments due to concentrated wheel loads on slabs. L = Long span length

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

$M2$ = Moment in the long span direction

$m1$ & $m2$ = 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

The dispersion of the wheel or track load may be assumed to be at 45 degrees through the wearing coat and structural slab according to Clause 305.16.3 of IRC: 21-2000 specifications.

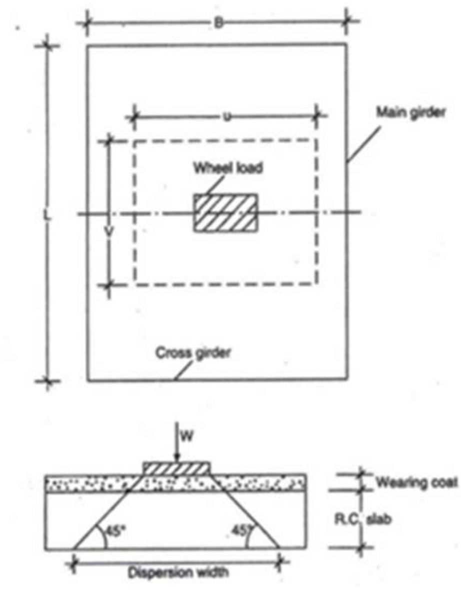

Figure 5:Disperesion Of wheel load Through Wearing Coat & Deck at 45 degree

The bending moments in the short and long span directions are expressed as

$$M1 = (mi + pm2)$$
$$M2 = (m2 + p\ m1)$$

The values of the moment coefficients m1 and m2 depend upon the parameters *(u/B)* and *(v/L)* and the value of *K = (BIL)*. Pigeaud's curves are used for the estimation of moment coefficients *m1* and *m2* for various values of *K*. 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.

5.4 Design Steps – Simply Supported Steel Girder Bridges

Dead Load Analysis

- calculate Dead Load (Slab)+Dead Load(cross Girders)+Self Weight
- Calculate Dead Load Bending Moment & Shear Force

live load Anlaysis

- Pllace Live Load On the centre of span to create Maxm Bending Moment
- calculate max. Shear force by Placing Load Near on of the Supports

Proportioning Trial Section

- Calculate Web Depth Based On shear considerations
- Proportioning Trial section of flange by calculating approximate flange area required.
- connection between flange and web by calculating maximum shear force at junction

Providing Stiffners

- checking if vertical stiffners are required
- calculating shear on welds connecting stiffner to web
- designing end bearing stiffners as columns
- calculating length of weld required for connectiing end bearing stiffners to web

Checks

- check for maximum Bending Tensile Stress

5.4.1 Design Principles for Composite Steel Plate Girder Bridge

The design of a plate girder involves the section of the cross-section and design of connection between flanges and web, together with the design of intermediate and bearing stiffeners and their connections to the web of the plate girder. The various design steps involved are as follows –

1. Computing the live load and dead load moments and shear forces. The self-weight of the girder may be assumed as *(0.2L* + I) kN/m, where *L* = span of the girder.
2. The design moments and shear forces are computed by applying impact factors to the live load moments and shears. The impact factor for steel bridges prescribed in IRC 24-2001 and IRS bridge rules are outlined in above sections.
3. Approximate Depth Of Girder=1/8 to 1/10 of span

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

Where M = design bending moment

σ_b= Permissible bending stress in steel as specified in IRC : 24-2001 . For plate girders, the permissible stress is 141 N/mm^2 and 150 N/mm^2 for clear depth to thickness ratio of web is greater or less than 35 respectively.

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 with an yield stress of 236 N/mm^2 as per IRC: 24-2001.

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

4 Approximate flange area required is

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

Flange width = *L/40* to *L/60*, outstand of flange beyond the flange angle should not exceed 16t for mild steel and 14t for high tensile steel, where *'t'* is the thickness of the thinnest flange plate in the case of riveted connections . For welded connections the flange plate should not project beyond the line of connections to the web by more than 12t.

5 The proportioned section is checked for permissible stresses as per the specifications of IRC 24-2001

6 The connections between flange and web is designed to resist a maximum horizontal shear force given by,

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

Where *V* = Shear force at the section

a = area of the flange

y = distance of the centroid of area *V* from neutral axis

I = second moment of inertia

The size of weld is designed to resist this horizontal force.

7 Spacing of intermediate stiffeners is given by *'c'* computed as not greater than *l.5d* and not less than *0.4d, where 'd '* is the unsupported depth of the web.

The unsupported Stiffeners are Designed to have a minimum moment of inertia given by

$$I = \frac{1.5d^3t^3}{C^3}$$

Where *d* = depth of web

t = thickness of web

c = spacing of stiffeners

The outstand of stiffener shall not be more than 16d for rolled sections and 12t for flats where *'t'* is the thickness of the section or flat. The connection of intermediate stiffener to web is designed to resist a horizontal shear force of not less than ($125t^2/h$) kN/m,

Where t =thickness of web (mm)

h = outstand of stiffener (mm)

8 The end load bearing stiffener is designed as a column assuming the section to consist of the pair of stiffener together with a length of web on each side of the centre line of the stiffener and equal to 20 times the web thickness. The permissible stresses are checked as that for compression member assuming an effective length equal to 0.7 times the length of the stiffener.

9 According to IRC 22-1986, the safe shear resistance of high tensile steel connectors iscomputed by empirical relations specified in the code depending upon the type of connectors.

a) For mild steel shear connectors, the safe shear resisted by each connector is computed by the following empirical relation-

For channel, Tee or angle shear connectors of mild steel:

(*fst* = 420 to 500 N/mm^2 and t = 230 N/mm^2),

$$Q = 107(h_f + 0.5t)L\sqrt{f_{ck}}$$

b) For welded stud connectors of mild steel with};, = 420 N/mm^2 and_t;, = 350 N/mm^2 and having a ratio of *(hid)* less than 4.2,

$$Q=48hd\sqrt{fck}$$

c) For welded stud connectors of mild steel having a ratio of *(hid)* equal to or greater than 4.2,

$$Q=196d^2\sqrt{fck}$$

Where Q= Safe shear resistance of one shear connector (N)

fck = Characteristic compressive cube strength of concrete (N/mm^2)

$h1$= Maximum thickness of flange measured at the faces of the web

(mm) L = Length of shear connector (mm)

h= Height of stud connector (mm)

d=Diameter of stud (mm)

t = Thickness of web of shear connector (mm)

c) When anchorage type shear connectors are used to connect the concrete slab deck with precast prestressed concrete girders, the ultimate shear resistance of one connector is given by the empirical relation,

$$Qu = As\sigma_u 10^{-3}$$

Where Q_u= ultimate Shear Resistance Of Each Connector (KN)

As = Cross sectional area of each connector (mm^2)

σ_u= Ultimate tensile strength of steel of the anchorage connector (N/mm^2)

The ultimate bond stress at the interface should not exceed 2.1 N/mm^2 and the interface should be roughened for effective bonding.

The spacing of the shear connector is computed by the relation,

$$p = \frac{\Sigma Q_u}{V_{L}}$$

Where p =Spacing of the shear connector (mm)

Q= Safe shear resistance of one connector (kN)

Qu = Ultimate shear resistance of once connector (kN)

V_L = Longitudinal working shear per unit length

V_{lu} =Ultimate longitudinal shear per unit length.

The longitudinal shear (working or ultimate) is calculated using the equations,

$$V_{Lu} = \left(\frac{V_u A_c \bar{Y}}{I}\right)$$

$$V_L = \left(\frac{V A_C \bar{Y}}{I}\right)$$

Where V = Vertical shear due to dead load placed after composite section is effective and working live load with impact

Vu = Vertical shear due to ultimate loads computed with load factors of 1.5 for dead load and 2.5 for live load.

Ac = Transformed compressive area of concrete above the neutral axis of the composite section

Y = Distance from neutral axis to the centroid of the area Ac

I = Second moment of area of the whole transformed composite section

Chapter 6 -Composite Plate Girder Bridge

6.1 Analysis And Design Of Slab – Manual (Microsoft Excel)

The Analysis and Design Of 15 meter Bridge was done by hand for live load and dead load Shear forces and bending moments Presented On the Next Page.

COMPOSITE BRIDGE DECK			
General Details		**REMARKS**	**FORMULAE**
Bridge Span	15	in meters	
Clear width of roadway	7.5	in meters	
foothpath/kerb	0.6	in meters	
no. of steel girders	4	in meters	
cross girder Spacing	3	in meters	
Design of deck Slab			
Panel Width , B	2.5	in meters	
panel Length , L	3	in meters	
Depth of Slab , t	0.2	in meters	
Thickness Of Wearing Coat , h	0.08	in meters	
Density of concrete ,D	24	KN/M^3	
Density of Wearing Coat ,D_W	22	KN/M^3	
Stress Of Steel ,	250	N/mm^2	
Dead Weight of Slab, DL_1	4.8	KN/M^2	$DL_1 = t \times D$
Dead Weight of Wearing Coat , DL_2	1.76	KN/M^2	$DL_2 = h \times D_w$
Total Load , DL	6.56	KN/M^2	$DL = DL_1 + DL_2$

COMPOSITE BRIDGE DECK			
LIVE LOAD B.M		**REMARKS**	**FORMULAE**
		LOAD IRC CLASS AA-T FOR SEVERE RESULTS	
Wheel Load , W_L	350	in Kn	
Wheel Contact along B , x	0.85	in meters	
Wheel Contact along L , y	3.6	in meters	
contact areaAvailable on Slab	3	in meters	
U	1.01	in meters	u=(x + 2 x h)
V	3.76	in meters	v=(y + 2 x h)
u/B	0.404		
v/L	1		
K	0.833333333		K=B/L
Coefficients for moments Along	for given valvue of K , refer to pigeaud's curves		
Short Span , m_1	0.08		
Long Span , m_2	0.0425		
Poisson Ratio For Concrete , μ	0.15		
continuity Factor , *f*	0.8		
Impact factor , I_f	1.25		
calculated Value	279.2553191	KN	
Actual Load To Be Considered	279.2553191	KN	Minimum of Calculated & Default
Short Span Moment , M_B	24.12067819	kN-m	M_B=W_L (m_1 + μ$m_{2)}$
B.M. Including Impact & continuity Factor	**24.12067819**	kN-m	f x I_f x M_B
Long Span Moment , M_L	15.21941489	kN-m	M_B=W_L (m_2 + μm_1)
B.M. Including Impact & continuity Factor	**15.21941489**	kN-m	f x I_f x M_L

COMPOSITE BRIDGE DECK			
DEAD LOAD B.M		**REMARKS**	**FORMULAE**
Dead Load of Deck Slab	6.56	in kN/m^2	
Total Dead Load/Panel	49.2	kN	
u/B	1		
v/L	1		
B/L	0.833333333		
Coefficients for moments along	using Pigeaud curve for k=0.8 & 1/K=1.25		
Short Span , m_1	0.045		
Long Span , m_2	0.028		
Poisson Ratio For Concrete , µ	0.15		
continuity Factor , *f*	0.8		
Short Span Moment , M_B	2.42064	kN-m	$M_B=W_L\ (m_1 + \mu m_{2)}$
B.M. Including continuity Factor	**1.936512**	kN-m	$f \times M_B$
Long Span Moment	1.7097	kN-m	$M_B=W_L\ (m_2 + \mu m_1)$
B.M. Including continuity Factor	**1.36776**	kN-m	$f \times M_L$
Design Bending Moments			
Short Span Moment , M_B	26.54131819	kN-m	
Long Span Moment , M_L	16.58717489	kN-m	

DESIGN OF SLAB					
Design Coefficient for Flexural Members				**Remarks**	**Formulae**
Moment Factor , Q			2.31		Coefficient values For M40 Grade Concrete and Fe415 grade tor steel
Lever Arm Factor , *j*			0.9		
Permissible Stress In steel , σ_{st}			200	in N/mm^2	
Effective Depth Required , d_{min}			107.1902427	in mm01	$d=\sqrt{(M/Q . b)}$
Providing Overall Depth			200	in mm	
clear cover			40	in mm	
Effective Depth , d_{prov}			154	in mm	
Area Of Steel Along Short Span , A_{it}			957.4790112	mm^2	$A_{it} =\sqrt{M_B} /(\sigma_{st} .j.d)$
Provide Bars of Dia , ϕ			12	in mm	
at centre-to-centre spacing of			100	in mm	
Effective Depth Along Long Span using 10mm ϕ bars , d_o			143	in mm	$d_o=d_{prov}$ -6 -5
Area Of Distribution Steel , A_{st}			644.412389	in mm^2	

DESIGN OF STEEL GIRDER			
GENERAL DETAILS		**Remarks**	**Formulae**
Bridge Span	15	in m	
Wheel Load , W_L	350	kN	
Wheel Contact Along B , x	0.85	in m	
Wheel Contact along L , y	3	in m	
No. Of Main Steel Girders , n_m	4		
No. Of Cross Steel Girders , n_c	4		
Spacing Of main Girders , S_m	2.5	in m	
Spacing Of Cross Girders , S_c	3	in m	
Dead Load On Girder , DL_g	16.4	in kN/m	$DL_g = DL \times S_m$
Self Weight of main girder , SW_{mg}	4	in Kn/m	$SW_{mg} = (0.2L+1)$
Total load , W	20.4	in Kn/m	$w = DL_g + SW_{mg}$
Self Weight Of cross Girders (Assumed as 1 Kn/m) , SW_{cg}	2.5	in Kn/m	
a.) Dead Load Moments			
Max. Dead Load Moment , M_{max}	590.625	in kN-m	M = $(WL^2/8)$ + (SW L/4) + (SW x S_{max}
b.) Live Load Moments			
Max. Live Load Moment	1518.489		$M_{max} = (W_L L/4) + ((W_L/x) + (x/4))/2$
Impact Factor I_f	10	Percent	
live Load B.M	1670.337	kN-m	$M_{max}(live) \times I_f$
Dead Load B.m	590.625	kN-m	
Design Bending Moment	**2260.962**	kN-m	$B.M_{live} + B.M_{dead}$

C.) Shear Forces		Remarks	Formulae
Dead Load Shear Force , SF_{dead}	158	In kN	$SF_{dead} = (WL/2)+(N_C S_{WG}/2)$
Live Load Shear Force , SF_{live}	175	In kN	$SF_{LIVE} = W_L/2$
Live Load Shear with Impact	192.5	In kN	SF_{LIVE} X I_f
Design Shear Force , V	350.5	In kN	$V = SF_{LIVE} + SF_{DEAD}$
d.) Proportioning Of Trial Section Web ,			
Permissible bending Stress in Steel , σ_b	165	in N/mm^2 ; from table 8.2 in IRC:24 - 2002	
Aprroximate Depth Of Girder	1.5	in meteres ; 1/8 to 1/10 Span	
Economical Depth , d_{eco}	1132.706	in mm	$depth_{eco} = 5\sqrt[3]{M/\sigma_b}$
assuming Thickness Of web , t	10	in mm ; not less than 10mm	
Average Shear Stress , τ_v	85	in N/mm^2 ; As per IRC:25 for Mild Steel	
Depth Of Web , d	412.3529	in mm	$d = V / \tau_V .t$
Adopting Web With Depth , d_W	1000	in mm	
& thickness , t_W	10	in mm	
Area Of Web , A_W	10000	in mm^2	

e.) FLANGE PLATES		**Remarks**	**Formulae**
Approximate Area Required , A_f	12036.13614	in mm^2	$A_f = [M/\sigma_b .d) - (A_w/6)]$
Range Of flange Width , B			
L/40	375	In mm	
TO L/45	333.3333333	In mm	
Adopt Flange Width As	450	In mm	
Thickness of Plates	30	Mm	
f.) Check For maximum Stresses			
Distance Of Neutral Axis to the centre of A_f , $\bar{y}$	515	in mm	
Moment Of Inertia Of Section About N.A , I	7996433333	mm^4	$I_{xx} = [(t .d^3/12) + 2((b((h-h)/2)^3/12)+Ay^2)]$
Distance from neutral axis to the top , y	530	Mm	
Bending tensile Stress , σ_b	149.8555738	N/mm^2	$\sigma_b = (My/I)$
CHECK For Bending	**Section Is O.K**		
Average Shear Stress	35.05	N/mm^2	$V/d_w.t_w$
permissible average Shear Stress depends upon d/t ratio	100		
Using Stiffner Spacing , c	1000	Mm	c = 1 d_w ; using table 6.6 of IS800
allowable Average Shear Stress	87	N/mm^2	using table 8.6 in IRC:24 - 2001
Check For Shear	**Section Is O.K**		

connection Between Flange & Web		Remarks	Formulae
Total Design Shear, V	350.5	in kN	
Area Of Flange Plate, A	13500	in mm^2	
Moment Of Intertia , I	7996433333	in mm^4	
Distance from Neutral Axis to the centre of A_f , ȳ	515	in mm	
Adopted Web With Depth , d_w	1000	in mm	
and Thickness t_w	10	in mm	
Permissible Shear Stress In Weld , τ_p	102.5	in N/mm^2	
Max. Shear Force at The Junction of Web & flange ,τ	304.7422705	N/mm	τ= V a ȳ /I
Constant K for Weld Strength	0.7	Table 22 of IS:800-2007 ; Angle Between Faces 60-90^o	
Size Of Weld , S(calculated)	2.442508711	in mm	S= V/ 2 x K x τ_p
Size Of Weld Provided , S	5	in mm	
h.) Intermediate Stiffner			
Ratio d/t	100	> 85 (Refer Clause 508.2.1 (a) Pg.30 of IRC:24-2001	
Hence , Vertical Stiffners Are Required			
Spacing of Stiffners Can Be From			
$0.33d_w$	330	in mm	$0.33*d_w$
$1.5d_w$	1500	in mm	$1.5*d_w$
Adopting Spacing , c	1000	in mm	

As per IS:800, Intermediate Stiffner have to povide Moment Of Inertia ,I		Remarks	Formulae
moment of Inertia , I	1500000	in mm^4	
Assumed Thickness of Plate , t	10	Mm	
maximum width not to exceed 16t	160	in mm ; as per IRC:24-2001	16 x t
adopting Plate Outstand	100	in mm	
Moment Of Inertia , I	3333333	in mm^4	$I= t \times h^3/3$
Check	**O.K**		
i.) Connection Of Vertical Stiffner To Web			
Web Thickness , t_w	10	in mm	
Outstand Of Stiffner , h	100	in mm	
Shear On Welds Connecting Stiffner To Web	125	N/mm	Shear= $125t^2/h$
size of welds , s(calculated)	1.74216	in mm	s=Shear/(k.τ_p)
size of welds , s(adopted)	5	in mm	
effective Length Of Weld , 10t	100	in mm	10 x t_w
Spacing Provide	100	in mm	
using 100 mm long , 5 mm fillet Welds Alternately on Either Side			

j.) End Bearing Stiffner			Remarks	Formulae
	350.5	In kN		
Max. Shear Force V				
Assuming Outstand Length , h	200	mm	outstand Length /Thickness Shouldn't Increase by 12	
Thickness , t(calculated)	16.66666667	mm		t=h/12
Thickness , t(adopted)	20	mm		
Using 200mm x 20 mm Size Plate				
Permissible Bearing Stress , σ_p	187.5	in		$0.75f_y$
Bearing Area To Be Provided , A_{reqd}	1869.333333	mm^2		
Bearing Area Provided , A_{prov}	8000	mm^2	**OK**	2 x h x t
length of Web which Take Reaction , L_w	200	in mm	Assumed That Portion Which Take Reaction Is $20t_w$	$20 \times t_w$
Moment Of inertia of Bearing Area w.r.t	114901666.7	mm^4		$I=(t.(2h+t_w)^3/12)+(2xL_w xt_w^3/12)$
Area , A	12000	mm^2		$A= (2xhxt)+(2xL_w xt_w)$
Radius Of Gyration	97.85263864	in mm		$r = \sqrt{I/A}$
Effective Length Of Stiffener	700	in mm	Held in Position But Restrained Against Rotation	$0.7*D_w$
Slemderness Ratio , λ	7.153613942			
Permissible Stress ,σ_{ac} (Mpa) in Axial	150	N/mm^2	Refer to table 5.1 of IS:800 For F_y=250 MPa	
Area Actually Needed	2336.666667	mm^2		
CHECK		**O.K**		

K.) Connection Between Bearing Stiffner And WEB		Remarks	Formulae
Length Available for welding using alternate intermittent Weld	1920	mm	
Required Strength Of Weld	182.5521	N/mm	v/length
Size Of Weld , S	2.54428		S=Strength/K x τ_p
use Fillet Weld Of Size	5	mm	
Length Of Weld , 10t	100	mm	10 x t
use 100 mm long 5 mm welds intermittently			

6.2 Analysis – Software (CsiBridge)

Analysis of composite Bridge was Performed in csiBridge software .Models was Shown Below and The dead , Load Live Load And wave Load Is simulated .

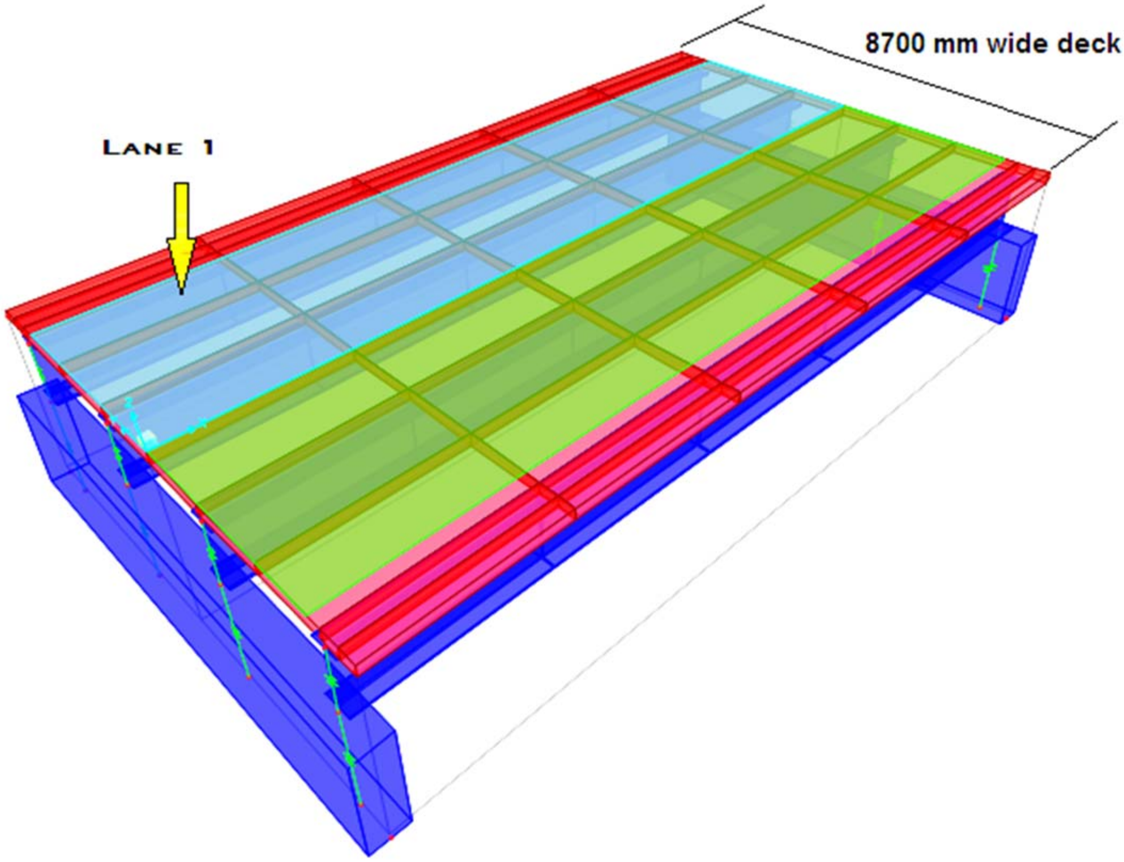

Figure 6 -3d rendered view of composite steel girder Bridge

6.3 Loadings

Various Loads are applied on this Bridge

a.) Dead Load

b.) Live Load

c.) Parapet Loads

d.) Wearing Surface

e.) Wave Load (at different Heights)

6.4 Deflection due to various Loads

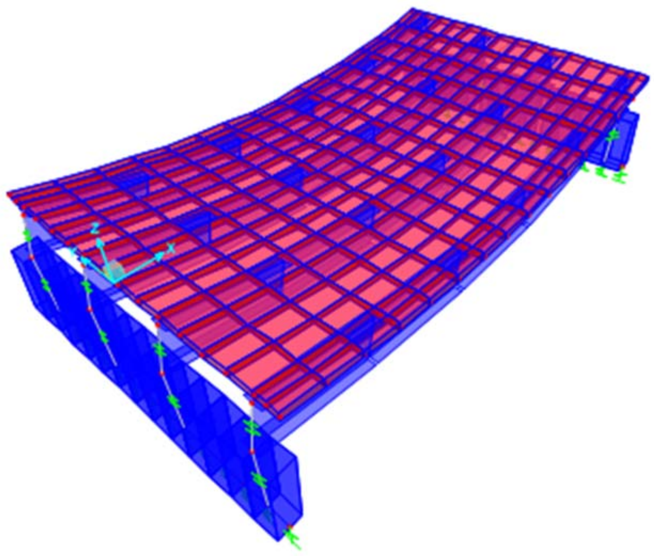

Figure7 - Deformed Shape Due to Dead Load +SIDL

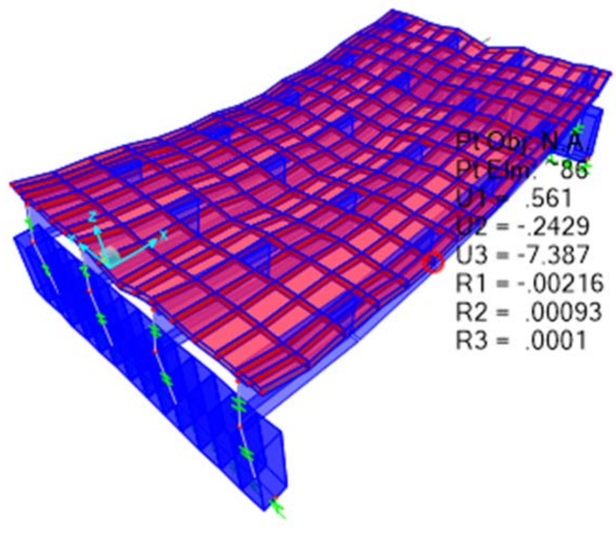

Figure 8– Deformed Shape Due to Live Load

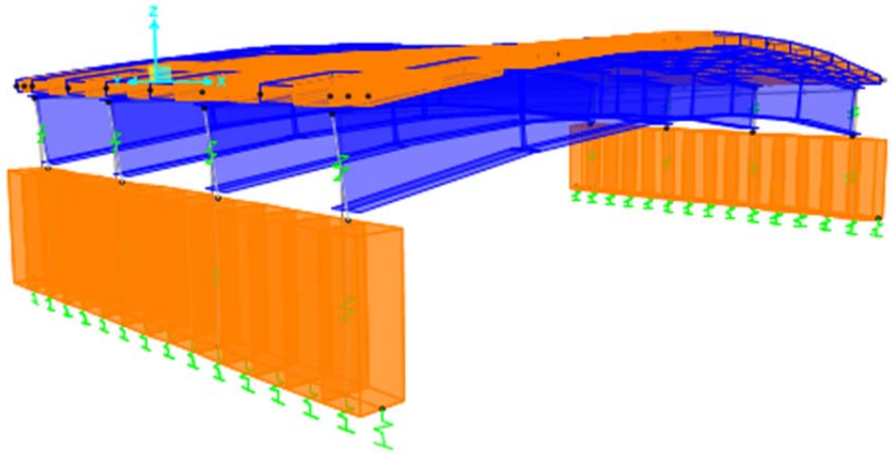

Figure 9 - Deformed Shape Due to Wave

TABLE 2

DEFLECTION			
S.NO	**DEAD LOAD**	**LIVE LOAD**	**WAVE**
1.	*2.93mm*	*3.7mm*	*-5.96mm*

6.5 Moments Diagrams

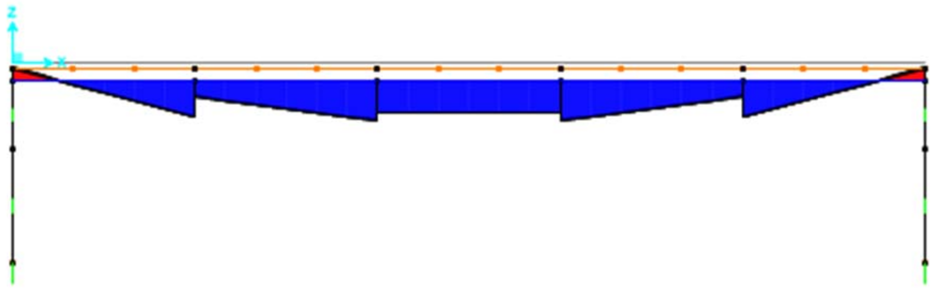

Figure 10 - Moment Diagram Of Dead Load

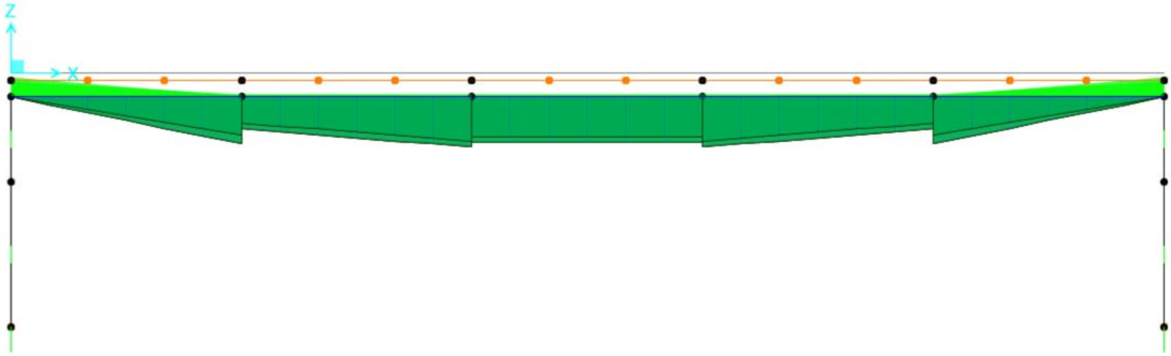

Figure 11 - Moment Diagram Of Live Load

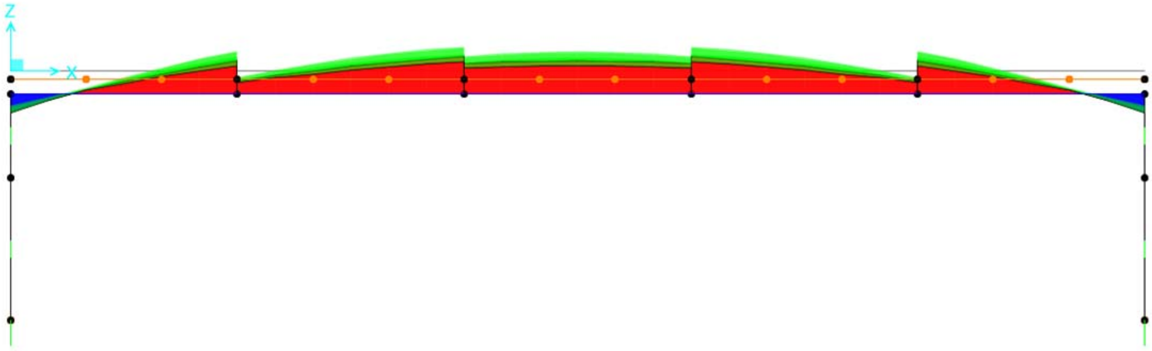

Figure 12 - Moment Diagram of Wave Load

Table 3

S.no	Bending Moments					
	Dead+SIDL		Live Load		Wave	
	Max	Min	Max	Min	Max	Min
1.	442.62Kn-m	0.015kn-m	1326.89kN-m	-4.95kN-m	0.0321Kn-m	-308.05Kn-m

6.6 Comparison –Manual & Software

A comparison of important result of analysis is obtained using manual analysis and Software analysis

Table 4

	Microsoft excel	CsiBridge
Shear Force		
a.) Dead load	158 kN	124 kN
b.) Live Load	175 kN	347 kN
Bending Moment		
a.) Dead Load	590.625 Kn-m	445.8 kN-m
b.) Live Load	1518.48 kN-m	1344 kN-m

As We can See the difference between the results calculated both in software as well as manual its just because few things like wearing surface , forms , manufacturing dead load was not modelled in csi bridge .

6.7 Modelling Wave Forces On Bridge Decks

Case I: storm-surge elevation at the bottom of the bridge deck

In this section, the selected bridge deck bottom is at the water surface and the interaction between wave and bridge is simulated by the wave-loading model. Figure 13 shows the wave plot of the bridge deck within a wave period (3 s) during the wave–structure interaction.

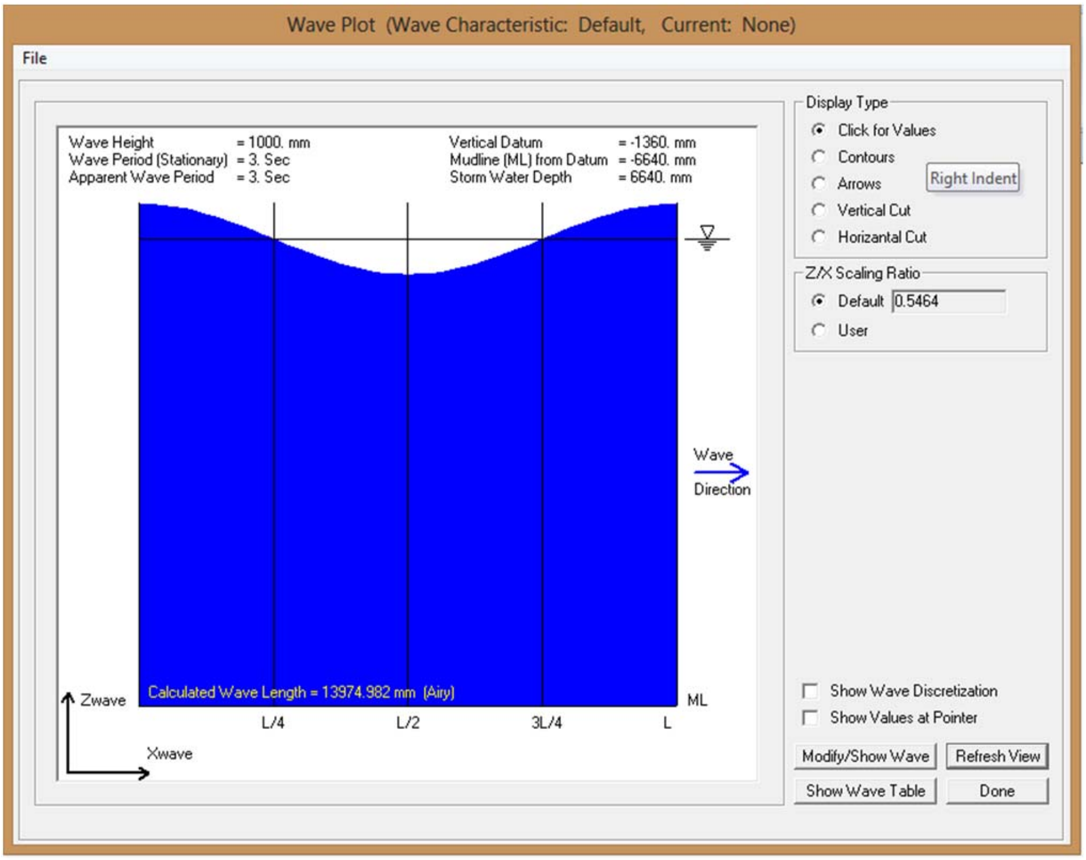

Figure 13 - Wave Plot(wave level Below bridge deck)

The phenomenon of negative pressure can be explained as follows. As the trough advances towards the deck, the water falls below the equilibrium position. This drop of water surface under the deck bottom creates a vacuum in the space between the deck and the new water surface, and an equal volume of air fills this space. This process creates a suction force as a negative uplift force. The modelling result shows the maximum uplift force for the emerged case (where the deck bottom is located at surge water surface) is too low that it does not exceed the bridge span weight of 71.1 metric tons.

Case II: storm-surge elevation at the middle height of the bridge deck

In this case, the selected bridge deck elevation is reduced by ½ of the deck height (Hb = 1.26 m) compared with Case I, and the surge water elevation is at the middle height of the bridge deck The process of the interaction between wave and bridge deck at different phases within a wave period (3 s) is simulated.

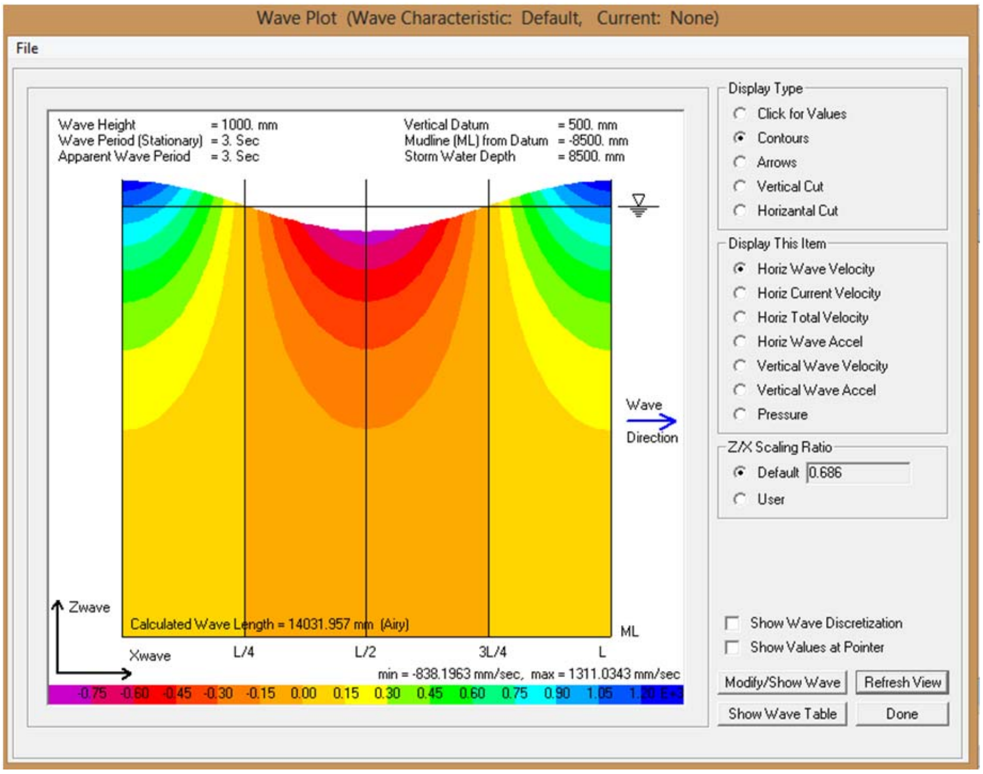

Figure 14- contours Showing Horz. Wave Velocity

Program calculated Velocity & Pressure Distribution Is Applied On Bridge Superstructure and Results are calculated. It is noted that The Vertical Uplift Force Still not Exceede the Weight of the Superstructure and Hence It Is safe..

Case III: storm-surge elevation at the top of the bridge deck

In this case, the top of the selected deck is now in contact with the surge water surface. Bridge deck is completely submerged under the water surface. As the bridge deck is fully submerged, the effect of wave–bridge interaction is more evident than that in previous case. The weight of the bridge span is 71 metric tons and the estimated maximum uplift force exerted on the bridge is Still less than the weight of the bridge span. Damage To the Bridge Deck is mainly caused by the uplift force which in this case still at lower level hence it's safe enough to withstand such forces . Design check was also performed to ensure the safety of bridge in dead and live loads.

6.8 Effect of submersion depth on the maximum wave force

The maximum wave force exerted on bridge decks is dependent on the elevation of the bridge deck related to surge water elevation. In this section, the effects of submersion depth on the maximum wave force are examined by changing the deck elevation submerged under water and fixing all other variables (water depth, wave period, etc.).

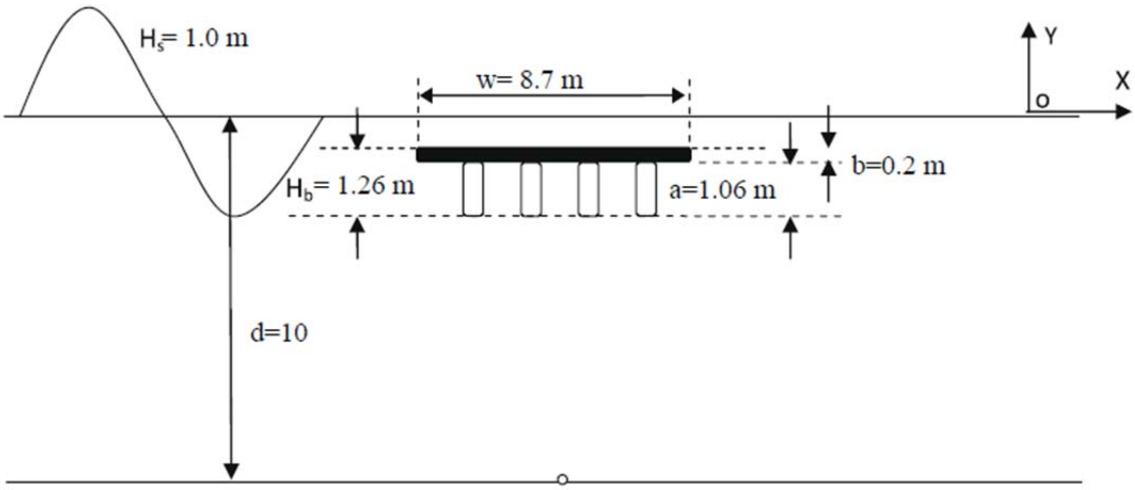

Figure 15 - Sketch of the model setup for simulating wave and bridge interaction

As shown in , the submersion depth (y) is defined as the distance from the surge water surface to the bottom of girders supporting the bridge deck. The submersion coefficient (Cs) is defined as Cs = y/H_b, where H_b is the height of the bridge deck . In order to investigate the effect of submersion depth on the maximum uplift force, five different submersion depths in reference to bridge height (Hb = 1.26 m) are considered, which are Cs = y/Hb = 0.0, 0.5, 1.0, 1.5, and 2.0, respectively, as shown .The water depth (d) and significant height (Hs) remain as, d = 7.22 m and H_s = 1.0 m. Other parameters, such as dimensions of the computational domain, grid sizes, and time steps remain unchanged. For each of the submersion depths, the process of interaction between the wave and bridge deck is simulated.

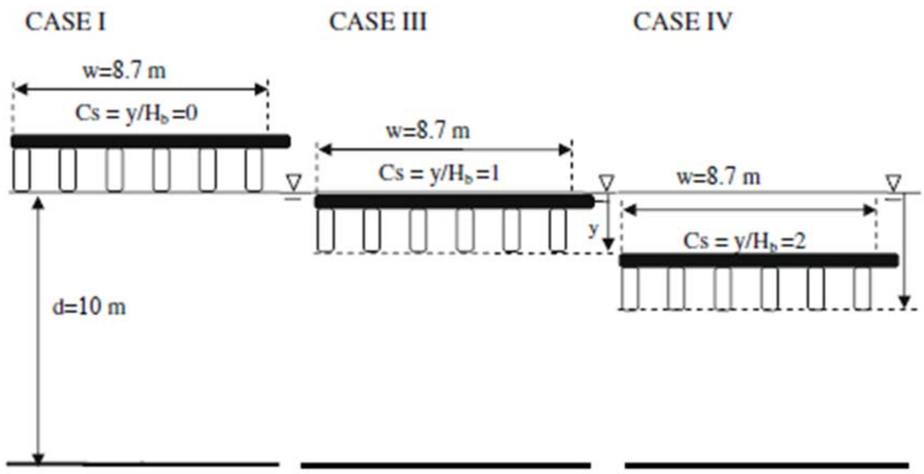

Figure 16 Model setups to investigate the effect of submersion depth. (Hb = 1.26 m is the height of the bridge span).

To better understand the effect of submersion depth, the maximum negative uplift force for each of the case is obtained . By comparing one can see the effect of submersion depth on the maximum uplift force. The results indicates the maximum positive normalized total uplift forces per span width are strongly dependent on the submersion depth. As the submersion depth increases, the maximum normalized negative force increases. The maximum uplifting wave force, when the bridge deck or girder bottom is at surge water elevation (Cs = 0.0), is 0.01 of the bridge span weight. From Cs = 0.5, the maximum uplift force starts to increase. When the submersion coefficient increases from Cs = 0.5 to 1.0, the normalized maximum uplift force increases from 0.4 to 0.89.

This phenomenon can be explained as follows. When the bridge deck bottom is level with the surge water surface, i.e. Case I with Cs = 0.0, only the wave crest has direct contact with the bridge deck, and a large portion of the wave energy cannot be conveyed to the deck. Therefore, the maximum uplift force is comparatively small at Cs = 0.0. When the submersion coefficient Cs increases to around 1.0, a majority of the wave force is concentrated in the vicinity of the water surface, and the maximum uplift force occurs at this submersion depth.

CHAPTER 7- COSTING OF COMPOSITE BRIDGE GIRDER

7.1 Cost of 1 m^3 of cement concrete (M60,M35)

Table 5

Details of 1m^3 of cement concrete (M60) ; 1:3.5:2.19:0.29:0.8$^{\theta}$						
Material	**Quantity**[#]	**Unit**[*]	**Rate/unit**		**cost**	
Stone Aggregate	782.964	Kg	0.75	Rs	587.223	Rs
Sand	350.378	Kg	0.32	Rs	112.121	Rs
Cement	570	Kg	7.5	Rs	4275	Rs
Water	131.1	Kg	0.1	Rs	13.11	Rs
Plasticizer	6.024	Kg	50	Rs	301.2	Rs
Silica Fumes	30.025	Kg	30	Rs	900.75	Rs
			Total		**6189.404**	**Rs/m^3**

Details of cost of 1m of cement concrete (M35)						
Material	**Quantity**	**Unit**	**rate/unit**		**cost**	
Stone Aggregate	619	Kg	0.75	Rs	464.25	Rs
Sand	600	Kg	0.32	Rs	192	Rs
Cement	400	Kg	7.5	Rs	3000	Rs
Water	172	Kg	0.1	Rs	17.2	Rs
			Total		**3673.45**	**Rs/m^3**

Notes:

**Rates have been taken from the article on Effect of fineness of sand on the cost and Properties of concrete by Prashant Agarwal , QC Manager , HCC Ltd.*

#Quantity have been Picked from B.tech Project Thesis : Study Of Cost Effectiveness in Design of Superstructures With HPC by sunil kumar sahoo Deptt. Of cvil engineering NIT Rourkela .

Θ cement : fine aggregate : coarse aggregate : water :super plasticizer

7.2 Cost of overall Composite Steel Girder Bridge

Cost Of composite Steel Bridge									
				a.) Cost Of Slab					
Section	**Depth(m)**	**width(m)**	**Length(m)**	**Quantity**	**unit**	**Rate/unit**	**cost**		
Slab	0.2	8.7	15	26.1	m^3	3673	**95866**	rs	
				b.) cost of steel					
section	**Quantity**	**unit**	**length(m)**	**steel (MT)**	**rate/MT***	**cost**			
Slab Transverse	957.47	mm^2	9.5	0.71	52000	3711	rs		
Slab Longitudnal	143	mm2	15	0.017	52000	874	rs		
				total		**4585**	**rs**		
				c.) cost of Girders					
Girders	**grade**	**width/depth (in m)**	**thickness in (m)**	**length(in m)**	**Kg/m3**	**quantity**	**rate/MT**	**Total**	
Top Flange	Fe250	0.45	0.03	15	7850	1590	45000	71534	rs
bottom Flange	Fe251	0.45	0.03	15	7850	1590	45000	71534	rs
Web	Fe252	1	0.01	15	7850	1176	45000	52920	rs
wastage(10%)						430		19602	rs
total cost								215590	rs
Total Cost Of 5 girders								1077947	rs
Total Cost Of Composite Steel Bridge								**1178397**	rs

Table 6

Chapter 8. Results

In the end we get the following results:

Description	Steel Plate Girder Bridge
Self weight of cross girder	2kN
Self weight of main girder	4 kN/m
Design Dead load	26.54kN/m
Max. Dead Load Moment	442.6 kN-m
Max. dead Load Shear	116.5kN
Max LL moment	1326 kN-m
Max LL Shear	434.71 kN
Cross Sectional Area of Section	$0.037m^2$
Depth of Section	1.06m
Material Cost Of Superstructure	Rs. 1178397

Table 7: Results(Dead Load , Shear Force And Costing)

Moment Due to different Height Of Wave Is Tabulated Here

Height Of Wave	Max.(+ve)	Max(-ve)
$y/h_b = 0$	$3.36e^{-14}$ kN-m	$-3.31e^{-14}$kN-m
$y/h_b = 0.5$	18.97 kN-m	$-3.28e^{-10}$kN-m
$y/h_b = 1$	6.863 kN-m	-308.80kN-m
$y/h_b = 1.39$	$1.221e^{-05}$ kN-m	-554.48kN-m
$y/h_b = 2$	$6.08e^{-08}$kN-m	-545kN-m

Table 8 : Results Of Different cases of Wave

Chapter 9. Conclusion

In order to analyze the effect of submersion depth on the uplift force, the interactions between a linear wave train and the bridge deck with five different elevations (Cs = y/Hb = 0.0, 0.5, 1.0, 1.39, and 2.0) have been studied using the present wave-loading model..

The results show that the

- The initial stage of Flood generated loading on a bridge superstructure leads to a combination of lateral (horizontal) and uplift (upward vertical) forces. These forces gradually increase as the water free-surface elevation rises.
- Maximum uplift force is the largest when the bridge deck is induntated (Cs = 1.39), or the surge water elevation is at the deck top. The maximum uplift force reached 77% of the bridge span weight (71.1 metric tons). From this bridge elevation Cs = 1.0, it improves our understanding of wave forces at varying bridge elevations and wave–bridge interaction, which is very useful for the assessment of the potential risk of coastal bridges exposed to storm surges and extreme wave conditions
- The model present in this study can also provide a quicker assessment of wave forces before conducting further more expensive 3D numerical modeling or large-scale physical experimental studies.
- The overall maximum uplift force is found to be in the full inundation stage when the bridge superstructure is totally inundated.

RESEARCH SCOPE

In this project I have determined the effect of wave loading on a bridge model & The costing of the bridge superstructure is estimated but still there is a wide scope for research and to learn more about Bridges. Following are the few other options on which work can be barried out.

[1] Effect Of Submersion Depth On Steel Arch Bridge

[2] Cost Variation of Steel & PSC girder bridge.

[3] Experimental Test Can be Conducted in Labs.

[4] Economizing the Section With Number of girders and Depth.

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