Project report on

DESIGN OF A STEEL COMPOSITE BRIDGE

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to the

DEPARTMENT OF CIVIL ENGINEERING

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, H.P.

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CERTIFICATE

This is to certify that the work titled "DESIGN OF A STEEL COMPOSITE BRIDGE" submitted by Abhiral Gupta, Piyush Kumar Jain, Karan Singh Juneja, Rohit Panwar, Sakshi Singh and Sanchit Mittal for the partial fulfilment of the award of degree of Bachelor of Technology in Civil Engineering, Jaypee University of Information Technology, Waknaghat has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Supervisor

29/5/13

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CANDIDATES' DECLARATION

We hereby certify that the work which is being presented in this report, "DESIGN OF A STEEL COMPOSITE BRIDGE" in partial fulfilment of the requirement for the award of B.Tech degree, submitted in the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of our own work carried out from July 2012 to May 2013 under the guidance of Prof. Dr. Ashok Gupta, Head of Department. We have not submitted the matter embodied in the report for the award of any other degree.

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"Achievement is finding out what you would be doing, what you have to do. The higher the summit, higher will be the climb". It has been rightly said that we are built on the shoulders of others but the satisfaction that accompanies the successful completion of any task would be incomplete without the mention of the people who made it possible. We are very thankful to **Prof. Dr. Ashok Kumar Gupta** for his valuable suggestions and encouragement in working towards this project. We find it difficult to verbalize our deepest sense of indebtedness to our parents and friends for their boundless love and support, which has been a source of inspiration.

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ABSTRACT

These days the construction of bridges has rapidly increased. A bridge is mainly designed to span physical obstacles such as a body of water, a valley, or a road, for the purpose of providing passage over the obstacle. The main objective of this project was to document the design process for a steel composite bridge. We have designed a superstructure consisting of a concrete deck with reinforcement supported on steel girders. According to our design, the length of the bridge was kept as 54 m and was divided into three sections of equal length. The superstructure of the bridge is supported by abutments on both sides and has piers in the middle resting on pile foundations.

The project was mainly divided into two phases. The initial phase involved the analysis and design of a Steel Composite Bridge manually, whereas in the second phase the design was tested for safety using Staad Pro.

Manually, the composite bridge deck was designed with reinforced slab and a steel plate girder to cover a span of 18 m along with an abutment which supported both the terminals of the superstructure of the bridge and, at the same time laterally supported the embankment which serves as an approach to the bridge. Further, the project also encompassed the manual design process of the piers and pile foundations. In the software designing phase, using Staad Pro the superstructure of the bridge was designed along with the pier for supporting the superstructure.

CHAPTER 1: INTRODUCTION

1.1 GENERAL INTRODUCTION

A bridge is mainly designed to span physical obstacles such as a body of water, a valley, or a road, for the purpose of providing passage over the obstacle. There are many different designs that serve unique purposes and apply to different situations. Design of bridges vary depending on the function of the bridge, the nature of the terrain where the bridge is constructed, the material used to make it and the funds available to build it.

1.2 AIMS AND OBJECTIVES

The aim of this project is to successfully complete the design of steel composite bridge. The project is mainly divided into two parts, analysis and design of a Steel Composite Bridge manually and by using software.

Manually we designed composite bridge deck with reinforced slab and steel plate girder to cover a span of 18 m and, an abutment which supports one terminus of the superstructure of a bridge and, at the same time, laterally supports the embankment which serves as an approach to the bridge and pier and pile foundation.

In software designing using Staad Pro we designed the superstructure supported on pier.

Our objectives for this project are:-

- Design the cross-section of the deck slab, continuous over steel girders and cross section of the steel girders.
- Longitudinal elevation of the steel girders showing the details of shear connectors.
- Design the cross-section and elevation of the abutment.
- Design of cross section of pier
- Design of pile foundation

1.3 HISTORY

The first bridges were made by nature itself — as simple as a log fallen across a stream or stones in the river. The first bridges made by humans were probably spans of cut wooden logs or planks and eventually stones, using a simple support and crossbeam arrangement. Some early Americans used trees or bamboo poles to cross small caverns or wells to get from one place to another. A common form of lashing sticks, logs, and deciduous branches together involved the use of long reeds or other harvested fibers woven together to form a connective rope capable of binding and holding together the materials used in early bridges.

Dating to the Greek Bronze Age (13th century BC), it is one of the oldest arch bridges still in existence and use. Several intact arched stone bridges from the Hellenistic era can be found in the Peloponnese in southern Greece.

The greatest bridge builders of antiquity were the ancient Romans. The Romans built arch bridges and aqueducts that could stand in conditions that would damage or destroy earlier designs. Some stand today. An example is the Alcántara Bridge, built over the river Tagus, in Spain. The Romans also used cement, which reduced the variation of strength found in natural stone. One type of cement, called pozzolanas, consisted of water, lime, sand, and volcanic rock. Brick and mortar bridges were built after the Roman era, as the technology for cement was lost then later rediscovered.

Large Chinese bridges of wooden construction existed at the time of the Warring States, the oldest surviving stone bridge in China is the Zhaozhou Bridge, built from 595 to 605 AD during the Sui Dynasty. This bridge is also historically significant as it is the world's oldest open-spandrel stone segmental arch bridge. European segmental arch bridges date back to at least the Alconétar Bridge (approximately 2nd century AD), while the enormous Roman era Trajan's Bridge (105 AD) featured open-spandrel segmental arches in wooden construction.

Rope bridges, a simple type of suspension bridge, were used by the Inca civilization in the Andes Mountains of South America, just prior to European colonization in the 16th century.

During the 18th century there were many innovations in the design of timber bridges by Hans Ulrich, Johannes Grubenmann and others. The first book on bridge engineering was written by Hubert Gautir in 1716. A major breakthrough in bridge technology came with the erection of the Iron Bridge in Coalbrookdale, England in 1779. It used cast iron for the first time as arches to cross the river Severn.

With the Industrial Revolution in the 19th century, truss systems of wrought iron were developed for larger bridges, but iron did not have the tensile strength to support large loads. With the advent of steel, which has a high tensile strength, much larger bridges were built, many using the ideas of GustaveEiffe.

In 1927 welding pioneer Stefan Bryła designed the first welded road bridge in the world, which was later built across the river SłudwiaMaurzyce near Łowicz, Poland in 1929. In 1995, the American Welding Society presented the Historic Welded Structure Award for the bridge to Poland.

1.4 COMPONENTS OF A BRIDGE:

Pile — foundation

Abutment —supports at the beginning or end of the bridge integrated with the ground

Piers — intermediate support

Span—is the bridge between two supports

Girders—tall narrow beams

Support Structures— the part of the bridge that carries the load

1.5 TYPES OF BRIDGES

Function of a bridge is to carry a load across a distance. Due to gravity all loads have downward force (weight). Bridges can be classified:

On the basis of forces as follows:

1. Compression—Stone and concrete are strong in compression. A bridge that supports a weight in compression is an arch bridge.

2. Tension—wire rope and chains are strong in tension. Suspension and cable stayed bridges are tension bridges.

3. Tension compression (both)—A beam bends under the weight of load. When it bends the top half is compression and the bottom is tension. The taller the beam the stronger it is, but as a beam become taller n taller it becomes too heavy and costly. So we use Truss bridges.

Classification on the basis of material:

1.Timber and stone masonry bridges:-The Chinese were building stone arch bridges since 250 B.C. The Chao-Chow Bridge build around 600 A.D. is perhaps the most long lived vehicular bridge to day. This stone masonry bridge with a single span of 37.6 m and a central rise of 7.2 m with a road way of 9 m is situated about 350 km south of Bejing. The secret of its longevity is attributed to the accurately dressed and matching voussoirs without any mortar in the joints.

Notable examples of a series of stone masonry arched bridges across the river Siene in Paris are unique examples of human ingenuity. The proven durability of material and the long experience in intuitive proportioning made stone masonry arched bridges and the most popular form of construction in the early days of Railways until Iron and Steel bridges made their way in the 17^{th} century.

2.Iron and steel bridges:- The first iron bridge comprising of five semicircular arch ribs in iron joined together side by side to form a single arch span of 30 m was built at Coalbrookdale in 1779 over the Severen in England by Abraham Darby and John Wilkinson. Gradually wrought iron replaced cast iron in bridge construction during the period from 1840 to 1890. The development of steel by Bessemer in 1856 paved the way for extensive use of steel in road and railway bridges.

3.Reinforced concrete bridges:-The first reinforced concrete bridge was build by Adair in 1871 across the Waveney in England spanning 15 m. The adaptability of reinforced concrete in architectural form was demonstrated by Maillart in Switzerland in building arched bridges using reinforced concrete, utilizing the integrated structural action of thin arch slabs with monolithically cast stiffening beams.

The most common type is the slab deck used for short spans such as culverts for medium spans in the range of 10 to 20 m. Tee beam and slab deck is widely used. Bow String girder type bridges have been used for road bridges in the span range of 25 to 35 m. Continuous bridge decks with longitudinal girders of varying depth are found to be more economical in the span range of 20 to 40 m. Elegant arch bridges were built during the period from 1920 to 1950.

4.Prestressed Concrete Bridges: - A revolutionary and a path breaking achievement in materials technology was witnessed in 1928 designated as Prestressed Concrete which is ideally suited for construction of long span bridges. The big boom is prestressed concrete was witnessed after the Second World War.

Among the 500 and odd bridges built in post war Germany during 1949-53, seventy percent of them used prestressed concrete. At present the cantilever construction method is invariably used for long span prestressed concrete bridges mainly for quality control and rapidity of construction. During the last decade hundreds of fly overs built in the metropolitan cities of India have adopted the cantilever construction technique with minimal disruption of traffic.

5.Cable stayed bridges:-First modern cable stayed bridge is the Stromsund Bridge in Sweden around 1953. This innovation paved the way for the construction of number of famous Rhine family cable stayed bridges with spans up to and exceeding 300 m.

Cable stayed bridges are technically, economically, aesthetically and aerodynamically superior to the classical suspension bridges for spans in the range of 700 to 1500 m. The combination of cable stays with cellular box girder prestressed concrete decks have significantly extended the span range of highway bridges.

India's first cable stayed bridge is the Akkar Bridge in Sikkim completed in 1998 and extending over a length of 157 m with a single pylon of height 57.5 m.

Classification on the basis of structures:

1.Beam bridges: - Beam bridges are horizontal beams supported at each end by substructure units and can be either simply supported when the beams only connect across a single span, or continuous when the beams are connected across two or more spans. When there are multiple spans, the intermediate supports are known as piers. The earliest beam bridges were simple logs that sat across streams and similar simple structures. In modern times, beam bridges can range from small, wooden beams to large, steel boxes. The vertical force on the bridge becomes a shear and flexural load on the beam which is transferred down its length to the substructures on either side. They are typically made of steel, concrete or wood. Beam bridge spans rarely exceed 250 feet (76 m) long, as the flexural stresses increase proportional to the square of the length (and deflection increases proportional to the 4^{th} power of the length).

2.Truss bridges:-A truss bridge is a bridge whose load bearing superstructure is composed of a truss. This truss is a structure of connected elements forming triangular units. The connected elements (typically straight) may be stressed from tension, compression, or sometimes both in response to dynamic loads. Truss bridges are one of the oldest types of modern bridges. The basic types of truss bridges shown in this article have simple designs which could be easily analyzed by nineteenth and early twentieth century engineers. A truss bridge is economical to construct owing to its efficient use of materials.

3.Cantilever bridges: - Cantilever bridges are built using cantilevers—horizontal beams supported on only one end. Most cantilever bridges use a pair of continuous spans that extend from opposite sides of the supporting piers to meet at the centre of the obstacle the bridge crosses. Cantilever bridges are constructed using much the same materials & techniques as beam bridges. The difference comes in the action of the forces through the bridge. The largest cantilever bridge is the 549 m (1,801 ft) Quebec Bridge in Quebec, Canada.

4.Arch bridges: - Arch bridges have abutments at each end. The weight of the bridge is thrust into the abutments at either side. The earliest known arch bridges were built by the Greeks, and include the Arkadiko Bridge.

With the span of 220 m (720 ft), the Solkan Bridge over the Soča River at Solkan in Slovenia is the second largest stone bridge in the world and the longest railroad stone bridge. It was completed in 1905. Its arch, which was constructed from over 5,000 tonnes (4,900 long tons; 5,500 short tons) of stone blocks in just 18 days, is the second largest stone arch in the world, surpassed only by the Friedensbrucke (Syratalviadukt) in Plauen, and the largest railroad stone arch. The arch of the Friedensbrucke, which was built in the same year, has the span of 90 m (300 ft) and crosses the valley of the Syrabach River. The difference between the two is that the Solkan Bridge was built from stone blocks, whereas the Friedensbrucke was built from a mixture of crushed stone and cement mortar.

Dubai in the United Arab Emirates is currently building the Sheikh Rashid bin Saeed Crossing, which is scheduled for completion in 2012. When completed, it will be the largest arch bridge in the world, with a main span 667 metres (2,188 ft) long.

5.Tied arch bridges: - Tied arch bridges have an arch-shaped superstructure, but differ from conventional arch bridges. Instead of transferring the weight of the bridge and traffic loads into thrust forces into the abutments, the ends of the arches are restrained by tension in the bottom chord of the structure. They are also called bowstring arches.

6.Suspension bridges:- Suspension bridges are suspended from cables. The earliest suspension bridges were made of ropes or vines covered with pieces of bamboo. In modern bridges, the cables hang from towers that are attached to caissons or cofferdams. The caissons or cofferdams are implanted deep into the floor of a lake or river. The longest suspension bridge in the world is the 12,826 ft (3,909 m) Akashi Kaikyo Bridge in Japan. See simple suspension bridge, stressed ribbon bridge, bridge, suspended, and self-anchored suspension bridge.

7.Cable-stayed bridges: - Cable-stayed bridges, like suspension bridges, are held up by cables. However, in a cable-stayed bridge, less cable is required and the towers holding the cables are proportionately shorter. The first known cable-stayed bridge was designed in 1784 by C.T. Loescher. The longest cable-stayed bridge is the Sutong Bridge over the Yangtze River in China.

8.Fixed or movable bridges: - Most bridges are fixed bridges, meaning they have no moving parts and stay in one place until they fail or are demolished. Temporary bridges, such as Bailey bridges, are designed to be assembled, and taken apart, transported to a different site, and re-used. They are important in military engineering, and are also used to carry traffic while an old bridge is being rebuilt. Movable bridges are designed to move out of the way of boats or other kinds of traffic, which would otherwise be too tall to fit. These are generally electrically powered.

CHAPTER 2: DESIGN OF SUPERSTRUCTURE

2.1 AIM

To design a composite bridge deck with reinforced slab and steel plate girder to cover a span of 18 m.

2.2 DESIGN CONSIDERATIONS AND ASSUMPTIONS

Clear width of the road way: 7.5 m

Footpath: 1 m on either side

Spacing of main girder: 2 m

Materials: concrete M-20 grade and Fe-415 grade for steel, rolled steel sections with yield stress of 236 N/mm^2.

2.3 METHODOLOGY

To design the reinforced concrete deck slab and steel plate girders with shear connectors.

2.4 SCOPE

To be able to draw the following views to a suitable scale:

(a) The cross-section of the deck slab continuous over steel girders and cross-section of the steel girders.

(b) Longitudinal elevation of the steel girders showing the details of the shear connectors.

2.5 DESIGN

2.5.1 Cross-section the Deck

The cross-sectional details of the deck slab assumed are as shown in Fig. 1

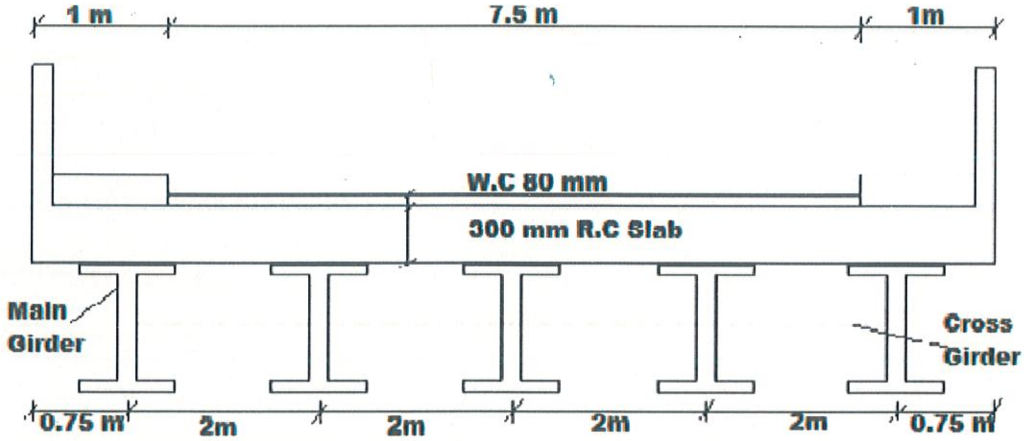


FIGURE 1 CROSS-SECTION OF DECK SLAB

2.5.2 Design of Deck Slab

Panel dimensions		=2 m × 4.5 m
Dead weight of slab	= (0.3×24.8)	=7.44 kN/m^2
Dead weight of W.C.	= (0.03×22)	=1.76 kN/m^2
Total load		= 9.2 kN/m^2

Live Load Bending Moment

Reference:- I.R.C. 6-2000 (Cl. 207.13)

Loading conditions = I.R.C. Class AA tracked vehicle referring to Fig. 2

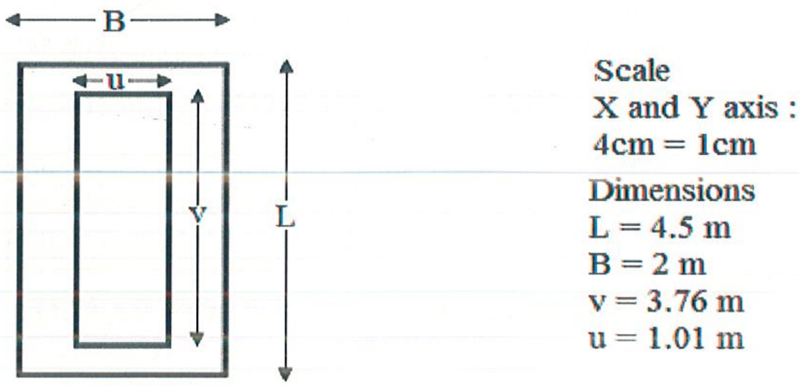


FIGURE 2 I.R.C. CLASS AA WHEEL LOAD

u	$=((0.85+2\times0.08)^2+0.3^2)^{1/2}$	= 1.10 m
v	$=((3.6+2\times0.08)^2+0.3^2)^{1/2}$	=3.85 m
(u/B)	= (1.1/2.0)	= 0.55
(v/L)	=(3.85/4.50)	= 0.85
K =(B/L)	=(2.0/4.5)	= 0.45

From Pigeaud's curves (Refer Fig. 3)

For K = 0.5 read out the values of

m_1=0.085

m_2= 0.017

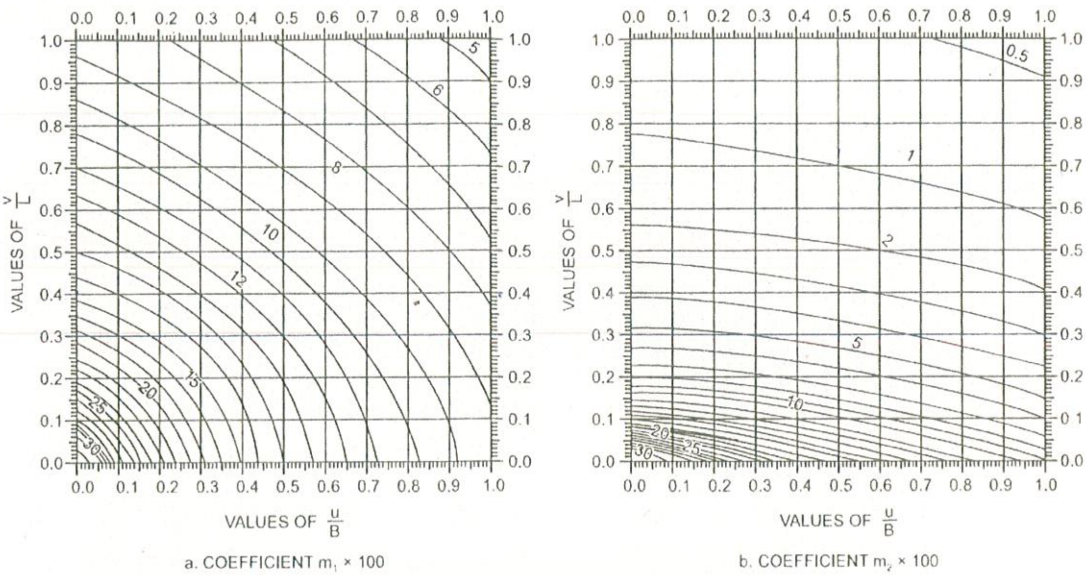


FIGURE 3 MOMENT COEFFICIENT M1& M2 FOR K = 0.5

Short span moment

M_B $=W(m_1+0.15\,m_2)$ $=350\,(0.085+0.15\times0.017)$ = 30.64 kN-m

B.M. including impact and continuity factor

Reference: I.R.C. 6:2000 (Cl 211.3) & D. Johnson Victor

M_B $=(1.25\times0.8\times30.64)$ =31kN-m

Long span moment

M_L $= W(m_2+0.015m_1)= 350(0.017+ 0.015 \times 0.085)$ $= 10.41$ kN-m

B.M. including impact and continuity factor

M_L $=(1.25 \times 0.8 \times 10.41)$ $= 10.41$kN-m

Dead Load Bending Moment

Dead load of deck slab $= 9.2$ kN/m^2

Total dead load of panel $= (9.2 \times 2 \times 4.5)$ $= 82.8$ kN

(u/B) =1

(v/l) =1

K $= (B/L)$ $= (2/4.5)$ $= 0.445$

(1/K) =2.25

Using Pigeaud's curves (Refer Fig. 4)

m_1 =0.047

m_2 =0.006

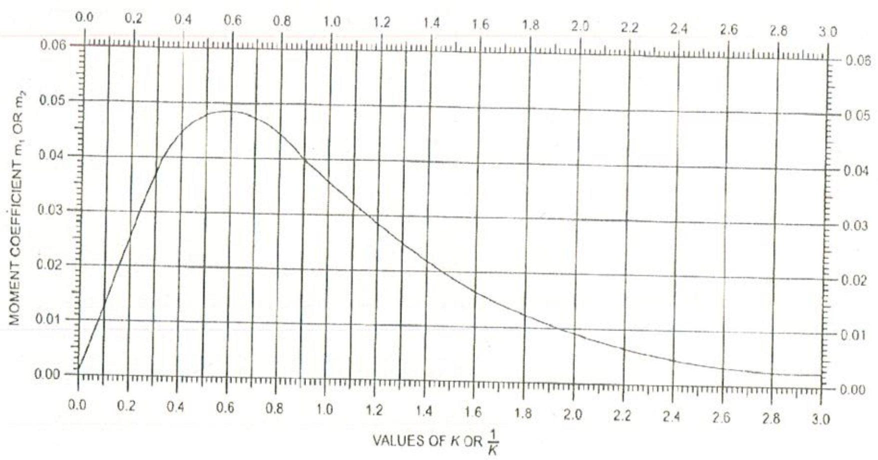


FIGURE 4 MOMENT COEFFICIENTS FOR SLABS COMPLETELY LOADED WITH UNIFORMLY DISTRIBUTED LOAD, COEFFICIENT IS M_1 FOR K & M_2 FOR 1/K

Short Span Moment

M_B	$= 82.8(0.047 + 0.15 \times 0.006)$	$= 3.96$ kN-m

Taking continuity into account

M_B	$= (0.8 \times 3.96)$	$= 3.17$ kN-m

Long Span Moment

M_L	$= 82.8\ (0.006 + 0.15 \times 0.047)$	$=1.08$ kN-m

Taking continuity into account

M_L	$=(0.8 \times 1.08)$	$= 0.86$ kN-m

The design bending moments

M_B	$= (30.64 + 3.17)$	$= 33.81$ kN-m
M_L	$= (10.41 + 0.86)$	$= 11.27$ kN-m

Design of Section

For M-25 grade concrete and Fe-415 grade for steel,

$Q = 1.184$,	$j = 0.894$,	σ_{st}	$= 200$ N/mm^2
$d = (M / Q\ .b)^{1/2}$	$= ((33.81 \text{ X } 10^6)/(1.185 \text{ x } 10^3\ ))^{1/2}$		$= 169.94$ mm
Overall depth			$=300$ mm
Effective depth			$= 260$ mm

Short Span

A_{st}	$=[(33.815 \times 10^6)/(200 \times 0.894 \times 260)]$	$= 727.45$ mm^2

Provide 10, 12mm ф bars at 140 mm centres.

Long Span

Effective depth for long span (using 10 mm ϕ bars) = (260-6-5) $\approx$250 mm

A_{st} =M/(σ_{st} $\times$ jd) =[(11.27 $\times$ 10^6)/(200 $\times$ 0.894$\times$249)] = 253.31 mm^2

Provide 5, 10 mm ϕ bars at 120 mm centres.

2.5.3 Design of Steel Plate Girder

Spacing of main girders		=2m
Spacing of cross girders		=4.5 m
Dead load on girder	= (9.2 × 2)	=18.4 kN/m
Self-weight of main girder	= (0.2L + 1)	= 4.6 kN/m
Total load (W)		= 23 kN/m
Self-weight of cross girders	=(2×1)	=2kN.

(assumed as 1 kN/m)

Dead Loads Moments

Refer to Fig 5.

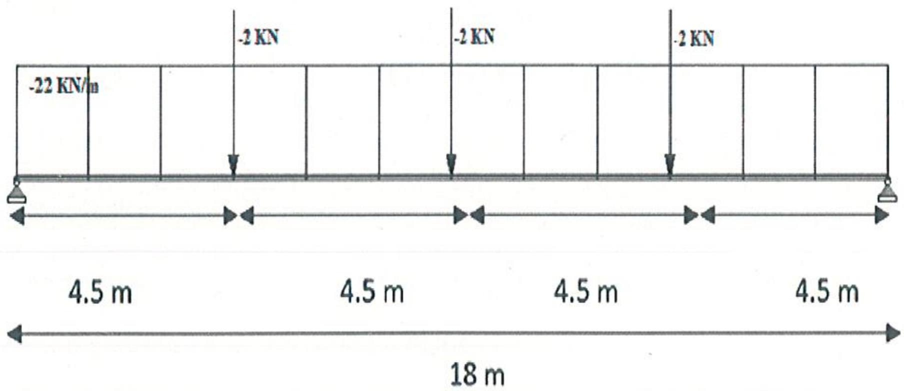


FIGURE 5 DEAD LOADS ON PLATE GIRDER

M_{max}= [(22 x 182)/8 + (2 x 18)/4 + (2 x 4.5)] = 950 kN.m

Live Load Moment

Referring to Fig. 6.

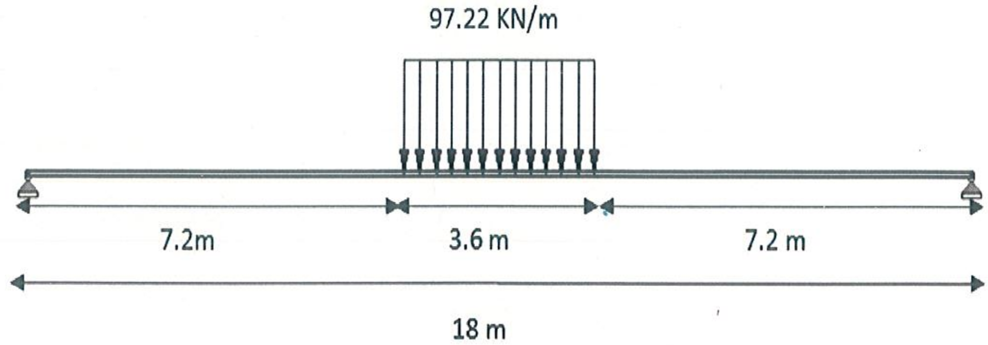


FIGURE 6 LIVE LOAD ON PLATE GIRDER.

M_{max}	$= [(350\times 9)/2 - (350 \times 0.9)12]$	= 1418 kN-m
Impact factor		= 10%
Live load B.M	$=(1418 \times 1.1)$	= 1560 kN-m
Dead load B.M		= 950 kN-m
Design B.M.	$=(950 + 1560)$	= 2510 kN-m

Shear Forces

Dead load shear	$= [(22 \times 18)/2 + 2 + 2/2]$	= 210 kN
Live load shear with impact factor	$= 1.1\ [(350 \times 16.2 \times 18]$	= 347 kN
Total design shear	$= (210 + 347)$	= 557 kN.

Proportioning of Trial Section of Web Plate

Approximate depth of girder

L/8 to L/10 span	$= 18/10$	= 1.8 m
Economical depth	$=5(M/\sigma_b)^{1/3}$	=1240 mm

Web depth based on shear considerations assuming 10 mm thick plate

d $= V/(\tau \times 10)$ $= [(557 \times 10^3)/(85 \times 10)]$ = 654.7 mm

Using web 1000 mm by 10 mm.

Flange Plates Approximate flange area required

$A_f = [(M/\sigma_b\, d) - (A_w/6)] = [(2510 \times 10^6)/165 \times 1000) - (10 \times 1000)/6]$ $= 13544.46\ mm^2$

Flange width B $= L/40$ to $L/45 = 450$ mm to 400 mm =500 mm

Thickness of plate $= (13544.46/500)$ $= 27.9$ mm

The section selected is shown in Fig. 7

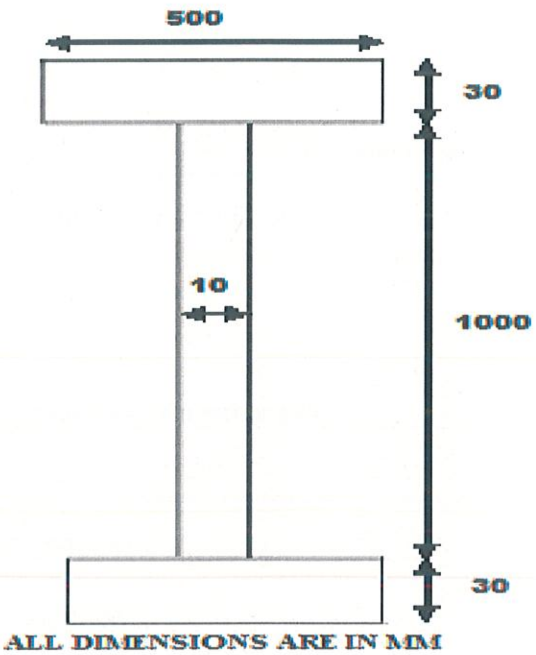


FIGURE 7 CROSS-SECTION OF PLATE GIRDER

Check for Maximum Stresses

$I = [(10 \times 1000^3)/12 + 2((500 \times 30^3/12) + 30 \times 500 \times 515^2)]$

$= 879 \times 10^7\ mm^4$

Bending tensile stress(σ_b)= $(My/I) = (2510 \times 10^6 \times 530)/(879 \times 10^7)$

$= 151.3\ N/mm^2$ $< 165\ /mm^2$

Average shear stress $= [(557 \times 10^3)/(1000 \times 10)] = 55.7\ N/mm^2$

Permissible average shear stress depends upon the ratio of $(d/t) = (1000/10) = 100$

Using stiffener spacing $c = 1000$ mm $= d$

Allowable average shear stress is $87\ N/mm^2$

Hence the average shear stress is within safe permissible limits.

Connection between Flange and Web

Maximum shear force at the junction of web and flange is given by

$\tau = (Vay/I))$ where

$V=557 \times 10^3 N$

$a = (500 \times 30) = 15{,}000\ mm^2$

$I= 879 \times 10^7\ mm^4$

$y=515\ mm$

$\tau= [(548 \times 103 \times 15 \times 103 \times 515)/(879 \times 10^7)] = 490\ N/mm^2$

Assuming continuous weld on either side, strength of weld of size ‘s’ is

$= (2 \times 0.7 \times s \times 102.5) \qquad = 145s$

$145\ s = 490, \qquad s = 3.37\ mm$

Use 5 mm fillet weld, continuous on either side,

Intermediate Stiffeners

Since $(d/t)= (1000/10)\ 100 > 85$

Vertical Stiffeners are required.

Spacing of Stiffeners = 0.33 d to 1.5 d

$= (0.33 \times 1000)$ to (1.5×1000)

= 333 mm to 1500 mm

Adopt 1000 mm spacing. Hence c = 1000 mm

The intermediate stiffeners are designed to have a minimum moment of inertia

$I = [(1.5\ d^3t^3)/c^2] = [(1.5 \times 1000^3 \times 10^3)/1000^2] = 15 \times 10^5\ mm^4$

Using 10 mm thick plate

Maximum width of plate not to exceed 12t

Reference - IS 800:1984 (Cl-6.7.4.4)

Use a plate 10 mm x 80 mm h = 80 mm

$I = [(10 \times 80^3)/3] = 17 \times 10^5\ mm^4 > 15 \times 10^5\ mm^4$

Connection of Vertical Stiffener to Web

Shear on welds connecting stiffener to web = $[(125 \times t^2)/h]$ kN/m

where , t = web thickness (mm)

h = outstand of stiffener (mm)

Shear on welds = $[(125 \times 10^2)/80]$ = 156.25 kN/m

= 156.25 N/mm

Size of welds (s) = $[156.25/(0.7 \times 102.5)]$ = 2.17 mm

Use 5 mm minimum size intermittent welds.

Effective length of weld 10 t = (10×10) = 100 mm

Use 100 mm long, 5 mm fillet welds alternately on either side.

End Bearing Stiffener

Maximum shear force = 557 kN

The end bearing stiffener is designed as a column (h/t) < 12

where, h = outstand

t = thickness

if, h = 180 mm

t = (180/12) = 15 mm

Use 180 mm by 15 mm size plate

Permissible bearing stress = σ_p = 189 N/mm^2 (IRC: 24 Cl-508.7.2)

Bearing area required = $[(557 \times 10^3)/189]$ = 2944.44mm^2

Two plates are used

Total area provided = $(2 \times 180 \times 15)$ = 5400 mm^2 > 2944.44 mm^2

The length of web plate which acts along with stiffener plates in bearing the reaction

= 20t = (20×10) = 200 mm

$I \quad = [(15 \times 370^3)/12 + (2 \times 200 \times 10^3)/12]$

$\quad = 6334 \times 10^4 \text{ mm}^4$

Area = A = $[(360 \times 15) + (400 \times 10)] = 9400 \text{ mm}^2$

$\lambda \quad$ = Slenderness Ratio = (L/r)

$r \quad = \sqrt{(I/A)} = \sqrt{(6334 \times 10^4)/9400} = 82$ mm

Effective Length of Stiffener = $(0.7 \times 1000) = 700$ mm

$\lambda \quad = L/r = (700/8^2) = 8.53$

From Table 8.3 (IRC: 24)

Permissible stress σ_{ac} in axial compression is obtained as 138 N/mm^2

Area required = $[(557 \times 10^3)/138] = 4036.23 \text{ mm}^2 < 9400 \text{ mm}^2$

Connection between Bearing Stiffener and Web

Length available for welding using alternate intermittent welds

	$= 2 \times (1000 - 40)$	= 1920 mm
Required strength of weld	$= [(557 \times 10^3)/1920]$	= 290.1 N/mm
Size of weld	$= [290.1/(0.7 \times 102.5)]$	= 4.04 mm

Use 5 mm fillet weld

Length of weld	$=(10 \times 10)$	= 100 mm

Use 100 mm long 5 mm welds alternately.

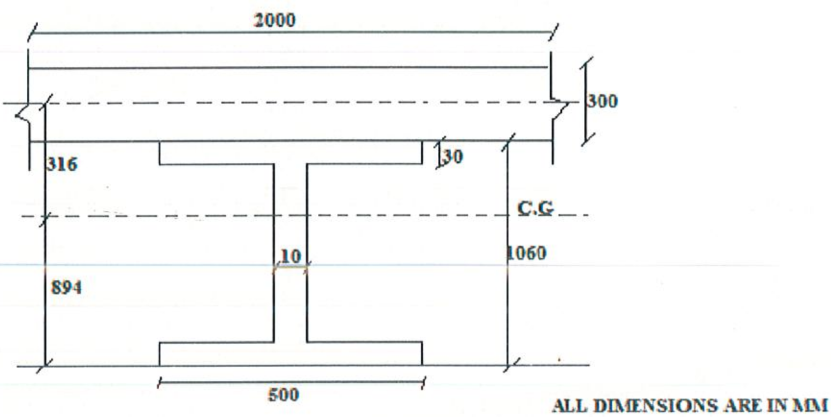


FIGURE 8 COMPOSITE SECTION (1)

Properties of the Composite Section

Referring to Fig. 8

$A_c = [(2000 \times 300)/13]$ $= 46154\ mm^2$

Modular ratio (m) $= 13$

The centroid of the composite section is determined by first moment of the area about the X-axis.

$Ay = [(46154 \times 1210) + (500\times 30 \times 1045) + (1000 \times 10 \times 530) + (500 \times 30 \times 15)]$

$= 77046340$

$A = [46154 + (500 \times 30) + (1000 \times 10) + (500 \times 30)]$ $= 86154\ mm^2$

$y = 894\ mm$

Concrete converting into steel

$m = Es/Ec = 13$

$Asc/13 = As$

$As = (2000\times 300)/13$ $= 46200\ mm^2$

$a = \sqrt{(46200)}$ $= 215\ mm$

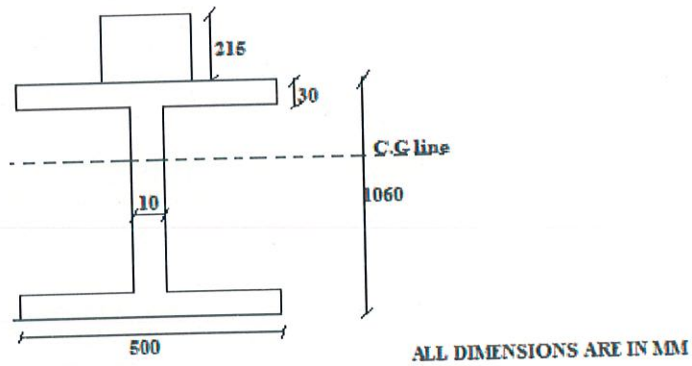


FIGURE 9 COMPOSITE SECTION (2)

Moment of inertia (I) = $1.772 \times 10^{10} mm^4$

Maximum shear force at junction of slab and girder is given by

τ = (Vay/I)

where V = 557 kN

a = 46154 mm^2

I = 1.772×10^{10} mm^4

τ = $[(557 \times 10^3 \times 46154 \times 316)1(1.772 \times 10^{10})]$ ≈ 458 N/mm

Total shear force at junction = (458×500) = 229000 N

Using 20 mm diameter mild steel studs

Capacity of one shear connector is given by

$Q = 196d^2 \times \sqrt{(f_{ck})}$

Where

$H = 5d = (5 \times 20) = 100$ mm

d= 20 mm

$f_{ck} = 25$ N/mm^2

$Q = 196 \times 20^2 \times \sqrt{25} = 392000$ N

Number of studs required in one row = (229000/392000) = 0.58 < 1

Provide a minimum of 2 mild steel studs in a row

Pitch of shear connectors = p = [(NQ)/(Fτ)]

Where

N = Number of shear connectors in a row

Q = Capacity of one shear connector

τ = Horizontal shear per unit length

F = Factor of safety = 2

$p = [(2 \times 392000)/(2 \times 458)]$ = 855.89 mm = 856 mm

Maximum permissible pitch is the least of

(i) three times the thickness of slab = (3×300) = 900 mm

(ii) 4 times the height of the stud = (4×100) = 400 mm

(ii) 600 mm

Hence adopt a pitch of 400 mm in the longitudinal direction.

The arrangement of shear connectors is shown in Fig.10.The cross section of the composite girder are shown in Fig.11

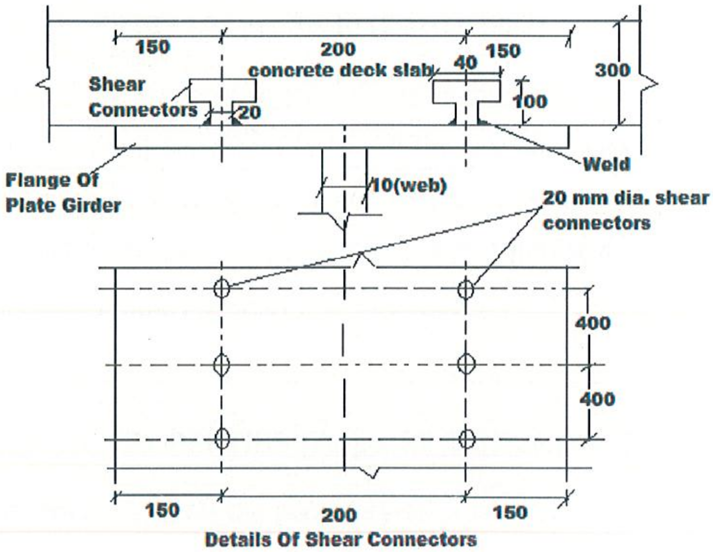


FIGURE 10

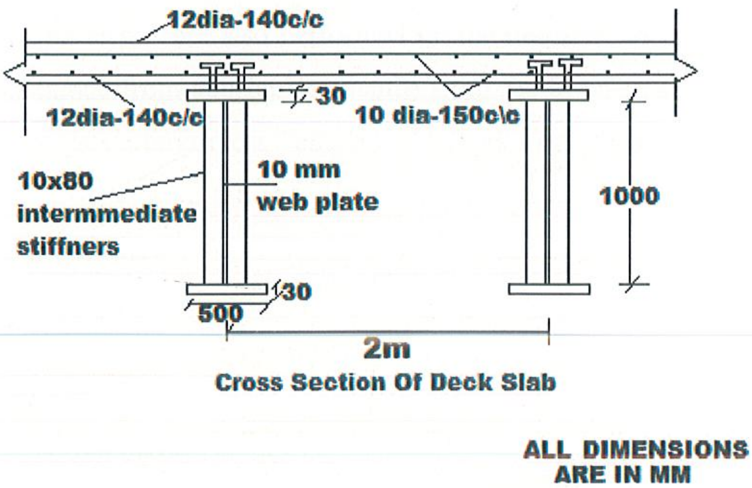


FIGURE 11

CHAPTER 3: SOIL PROPERTIES

The objective of the soil profiling is to obtain sequence & extent of the sub soil. To certain the characteristics of the sub soil to arrive at suitable parameters for the design of our structure.

3.1 INDEX PROPERTIES {As per SP 36 (Part-1)-1987}

The prerequisite of any soil investigation report is the classification of sub soil strata based upon laboratory classification tests. All the relevant classification tests on the samples obtained from the boreholes were carried out in the laboratory. The index properties obtained from such classification tests at different depths in the boreholes are reported in the borehole log chart and data sheets.

3.2 ENGINEERING PROPERTIES {As per SP 36 (Part-1)-1987}

It comprised of conducting all the necessary tests on the disturbed and undisturbed soil samples to evaluate different engineering properties of the sub soil. The unconfined compression strength tests were performed on undisturbed samples obtained from the cohesive strata so as to obtain undrained shear strength of the soil.

The consolidation tests were also performed on the undisturbed remoulded samples which were obtained from appropriate depths from the boreholes. Direct shear tests were performed on cohesionless samples obtained from different depths in each borehole so as to determine the shear strength characteristics of cohesionless strata met in various boreholes.

3.3 FAILURE MODES

A foundation can fail by two modes i.e.

1.Shear failure

2.Excessive settlement

Shear failure being catastrophic, an adequate factor of safety is applied to ultimate bearing capacity that can initiate this type of failure. BIS recommends a value of FOS = 2.5 to obtain the net safe bearing capacity 'q_{ns}' by using the physical characteristics of the foundation and relevant shear strength parameters of soil. Through settlement analysis a net loading intensity 'qn' is obtained by using the physical characteristics of the foundation and the relevant compressibility characteristics of the underlying soil. The value so obtained ensures that the foundation shall not settle more than that which is permissible as per BIS recommendations. The permissible settlement depends upon the type of superstructure and the nature of supporting strata.

The lesser of these computed values i.e. qns or qn is adopted as the allowable bearing capacity for proportioning the foundations of superstructures.

3.4 CORRECTION TO ACCOUNT FOR THE EFFECT OF OVERBURDEN PRESSURE

Where 'CN' is overburden pressure correction and is scaled off from the figure as provided in the code. 'N ' are the normalized blows.

3.5 SUBMERGENCE CORRECTION

Where 'Nc' is the final corrected value.

Nc= (15+Nn-15)/2 provided Nn is >15

Where ever both the overburden and submergence corrections are necessary, the overburden correction is applied first.

The overburden correction is meant for granular strata. Submergence correction IS applied only in case of sands and non-plastic silts.

3.6 BORE HOLE LOG CHART AND DATA SHEET

TABLE 1

Dr. GHUMAN AND GUPTA GEOTECH CONSULTANTS
(SOIL AND FOUNDATION ENGINEERS)

BOREHOLE NO: 1
PROJECT : CONSTRUCTION OF H.L. BRIDGE OVER MARKANDA RIVER AT JHANSA, HARYANA.

DIA OF CASING (mm): 100

BOREHOLE LOG CHART AND DATA SHEET

DEPTH FROM REF. (meter)	SYMBOLIC PRESENTATION	IS CLASSIFICATION	N - VALUES	N VALUES vs DEPTH PLOT	GRAIN SIZE ANALYSIS GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	WATER CONTENT (%)	BULK DENSITY (g/cc)	DRY DENSITY (g/cc)	VOID RATIO	UNDRAINED SHEAR STRENGTH C_u (kg/cm²)	C' (kg/cm²)	ø (degree)	COMPRESSION INDEX	REMARKS
			-	1.5m															
3.80		CL	-	3.0m															FIRM CLAY
			5	4.5m	-	9.0	78.4	12.6	28.4	20.2	17.0	2.0	1.71	0.58	0.40	-	-	0.13	
6.80		SM	6	6.0m															COMPACT SAND
8.40		M(NP)	17	7.5m	-	82.9	17.1	-	-	NP	12.6	1.86	1.65	0.62	-	0.0	32.0	-	COMPACT SILT
			18	9.0m	-	29.1	70.9	-	-	NP									
11.3		CL	19	10.5m	-	33.5	66.5	-	-	NP	12.1	1.85	1.65	0.62	-	0.0	32.0	-	CLAY
			14	12.0m	-	9.3	78.0	12.7	27.8	19.4	17.0	2.05	1.75	0.54	1.04	-	-	-	
			15	13.5m ▼SSWL=14.0M BELOW ESL															
			18	15.0m	-	13.1	71.7	15.2	30.4	21.0	19.2	2.12	1.78	0.52	1.38	-	-	0.11	VERY STIFF CLAY
17.4		CL	21	16.5m															
			29	18.0m	-	32.8	67.1	-	-	NP	22.5	2.05	1.67	0.60	-	0.0	32.3	-	
19.4		M(NP)	20	19.5m															COMPACT SILT
			22	21.0m	-	1.9	81.7	16.4	31.4	21.5	16.7	2.17	1.86	0.45	1.44	-	-	0.09	VERY STIFF CLAY
23.2		CL	24	22.5m															
25.0			23	24.0m	-	71.2	28.8	-	-	NP	22.5	2.05	1.67	0.60	-	0.0	32.4	-	IMPERVIOUS LAYER

gggc Dr. GHUMAN AND GUPTA GEOTECH CONSULTANTS
(SOIL AND FOUNDATION ENGINEERS)

BOREHOLE NO: 2
PROJECT: CONSTRUCTION OF H.L. BRIDGE OVER MARKANDA RIVER AT JHANSA, HARYANA.

BOREHOLE LOG CHART AND DATA SHEET

DEPTH FROM REF. (meter)	SYMBOLIC PRESENTATION	IS CLASSIFICATION	N - VALUES	N VALUES vs DEPTH PLOT	GRAIN SIZE ANALYSIS GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	WATER CONTENT (%)	BULK DENSITY (g/cc)	DRY DENSITY (g/cc)	VOID RATIO	UNDRAINED SHEAR STRENGTH C_u (kg/cm²)	C' (kg/cm²)	ø (degree)	COMPRESSION INDEX	REMARKS
			50(13)	1.5m	52.0	46.2	1.8	-	-	NP	14.1	2.0	1.75	0.52	-	0.0	40.7	-	
3.0		GP	13	3.0m	-	8.9	63.1	28.0	47.4	25.7	15.0	2.0	1.74	0.55	0.78	-	-	0.12	DENSE CLAY
4.20		CI																	STIFF CLAY
5.40		SP	26	4.5m	30.0	66.8	3.2	-	-	NP	8.3	1.93	1.78	0.50	-	0.0	33.6	-	COMPACT SAND
			39	6.0m															
8.20		GP	40	7.5m	75.0	23.2	1.8	-	-	NP	7.0	1.99	1.86	0.43	-	0.0	37.7	-	DENSE GRAVEL
10.0		GP	38	9.0m	65.0	33.6	1.4	-	-	NP	6.9	1.95	1.82	0.47	-	0.0	36.5	-	VERY STIFF CLAY
			18	10.5m	-	4.6	67.4	28.0	47.9	26.0	14.0	2.01	1.76	0.53	1.44	-	-	0.10	COMPACT SAND
13.2		CI	19	12.0m															
15.0		SM	20	13.5m	-	1.2	67.8	31.0	51.2	28.4	15.0	2.02	1.75	0.54	1.60	-	-	0.09	VERY STIFF CLAY
			28	15.0m															
18.0		CH	30	16.5m	-	72.4	27.6	-	-	NP	12.4	1.90	1.69	0.58	-	0.0	32.8	-	COMPACT SILT
			26	18.0m															
			26	19.5m		2.4	66.6	31.0	51.0	28.5	13.7	2.02	1.78	0.53	1.82	-	-	0.08	VERY STIFF CLAY
21.5		M(NP)	27	21.0m															
			40	22.5m															
25.0		CH	41	24.0m	-	20.7	79.3	-	-	NP	11.0	1.90	1.71	0.56	-	0.0	33.4	-	IMPERVIOUS LAYER

Note:-The value in Bracket is the penetration in cm in 50 Blows

Through field and laboratory investigations all the relevant parameters pertaining to various strata met in two boreholes were determined. Based upon these, borehole log charts and data sheets have been prepared.

TYPICAL DATA OF BOREHOLE -1

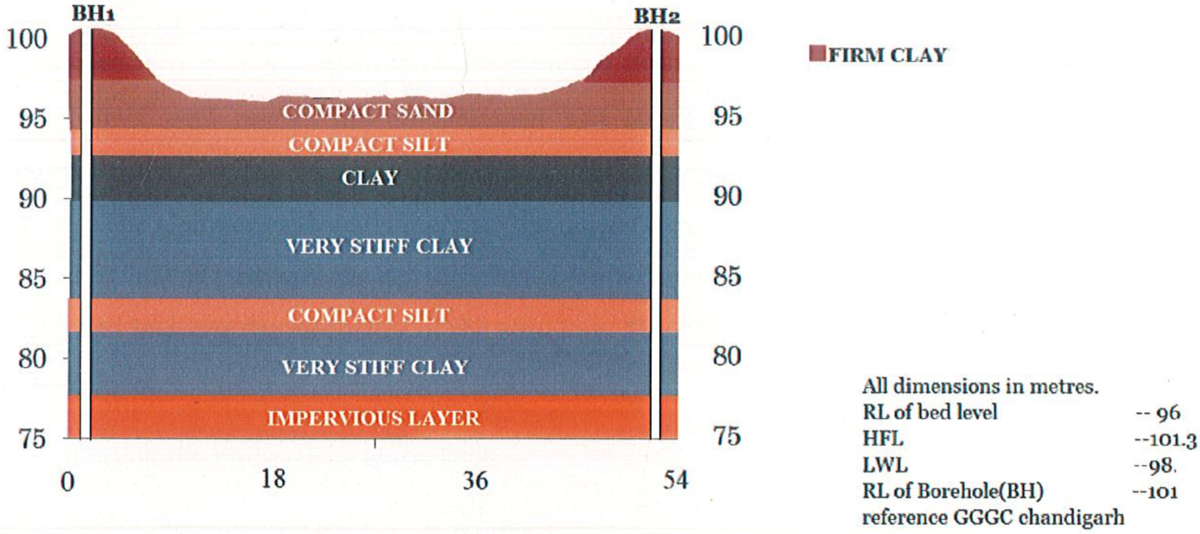


FIGURE 12

TABLE 2

3.7 OBSERVED AND CORRECTED SPT VALUES

Depth (m)	σ ó kg/cm²	C_N	N			N'			N''		
			BH-1	BH-1A	BH-2	BH-1	BH-1A	BH-2	BH-1	BH-1A	BH-2
7.5	1.35	0.90	18	18	17	16	16	15	16	16	15
9.0	1.62	0.84	19	19	18	16	16	15	16	16	15
10.5	1.89	0.78	-	-	19	-	-	15	-	-	15
15.0	2.70	0.67	-	22	-	-	15	-	-	15	-
16.5	2.85	0.64	27	27	-	17	17	18	16	16	16.5
18.0	3.00	0.63	29	-	29	18	17	18	16.5	16	16.5
22.5	3.45	0.61	-	28	-	-	17	-	-	16	-
24.0	3.60	0.59	29	31	33	17	18	19	16	16.5	17
30.0	4.20	0.53	-	32	-	-	17	-	-	16	-
33.0	4.50	0.51	-	33	-	-	17	-	-	16	-

CHAPTER 4: ABUTMENT

4.1 INTRODUCTION

Abutments are end supports of the superstructure, for example, deck units or girders. The ballast wall retains the embankment and supports the relieving slab. The abutment wing walls retain the embankment and provide anchorage for the bridge barrier. The abutment sidewalls increase the durability of the structure by separating the joints and the embankments, therefore keeping moisture away from the joints. They also improve the aesthetics of the structure. The basic function of an abutment is as follows:

- Supporting the bridge deck at the ends.
- Retaining the approach road embankment.
- Connecting the approach road to the bridge deck.
- Withstands any loads that are directly imposed on it.
- Provides vehicular and pedestrian access to the bridge.

The most common type of Abutment Structure is a Retaining Wall, although other types of Abutments are also possible and are used.A retaining wall is used to hold back an earth embankment or water and to maintain a sudden change in elevation.

In case of Retaining wall type Abutment bearing capacity and sliding resistance of the foundation materials and overturning stability must be checked.

4.2 SELECTION OF ABUTMENTS

The procedure of selecting the most appropriate type of abutments can be based on the following consideration:

- Construction and maintenance cost
- Cut or fill earthwork situation

- Traffic maintenance during construction
- Construction period
- Safety of construction workers
- Availability and cost of backfill material
- Superstructure depth
- Size of abutment
- Horizontal and vertical alignment changes Area of excavation
- Aesthetics and similarity to adjacent structures
- Previous experience with the type of abutment
- Ease of access for inspection and maintenance
- Anticipated life, loading condition, and acceptability of deformations

4.3 TYPES OF ABUTMENT

There are different types of abutments are:

1.Gravity Abutment:—A gravity abutment resists horizontal earth pressure from the rear, with its own weight, to be stable this leads to massive sized abutments. These abutments may be of mass concrete or stone masonry. A gravity abutment is composed of a back wall and splayed wing walls which rest on foundation.

2.U-abutment:—When the wing walls of gravity are placed at a right angle to the back wall the abutment is known as the U-abutment. The name U- abutment is due to the shape of the abutment in plan. The wing walls are typically cast monolithically with the abutment back wall and cantilevered both vertically and horizontally.

3.Stub abutment:— Stub abutments are relatively short abutments, which are placed on top of the embankment or slope. Sufficient rocky terrain must prevail at the site, so that the stub abutment can be supported on piles which extend through the embankment.

4.Counterfort abutment — A counterfort abutment is very much similar to a counterfort retaining wall. In counterfort abutments, a thin wall called counterfort connects the breast wall to the footing. These counterforts are spaced at regular intervals so that the breast wall is designed as a supported slab rather than as a cantilever. These are used when high abutments are required.

4.4 DESIGN REQUIREMENTS FOR ABUTMENTS

Height: - The height of abutment is kept equal to the piers.

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

2. Abutment width: - The top width should provide enough space for bridge bearings and bottom width is dimensioned as 0.4 to 0.5 times the height of the abutment.

3. Length of the abutment: - The length of the abutment must be at least equal to the width of the bridge.

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

4.5 FORCES ACTING ON ABUTMENTS

The various forces to be considered in the design of abutments are as follows:

1. Dead load due to superstructure.

2. Live load on the superstructure.

3. Self weight of the abutment.

4. Longitudinal forces due to tractive effort and braking.

5. Forces due to temperature variation.

6. Earth pressure due to the backfill.

4.6 FAILURE MODES FOR ABUTMENTS

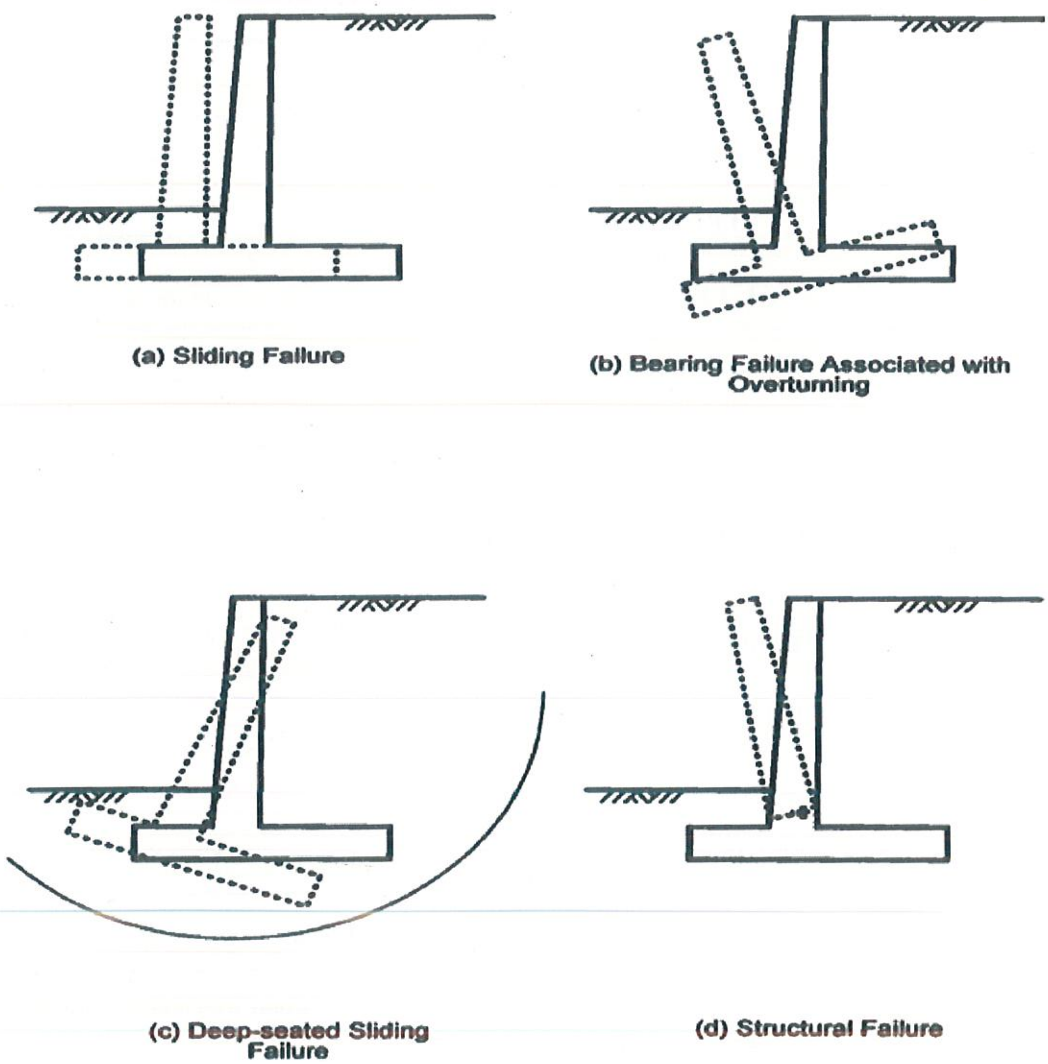
Abutments are subject to various types of failure, as illustrated in following figure. Failures can occur within soils or the structural members.

1.Sliding failure occurs when the lateral earth pressure exerted on the abutment exceeds the frictional sliding capacity of the foundation.

2.If the bearing pressure is larger than the capacity of the foundation soil or rock, bearing failure results.

3.Deep-seated sliding failure may develop in clayey soil.

4.Structural failure also should be checked.



Failure modes of abutments.

FIGURE 13

4.7 ABUTMENT DESIGN

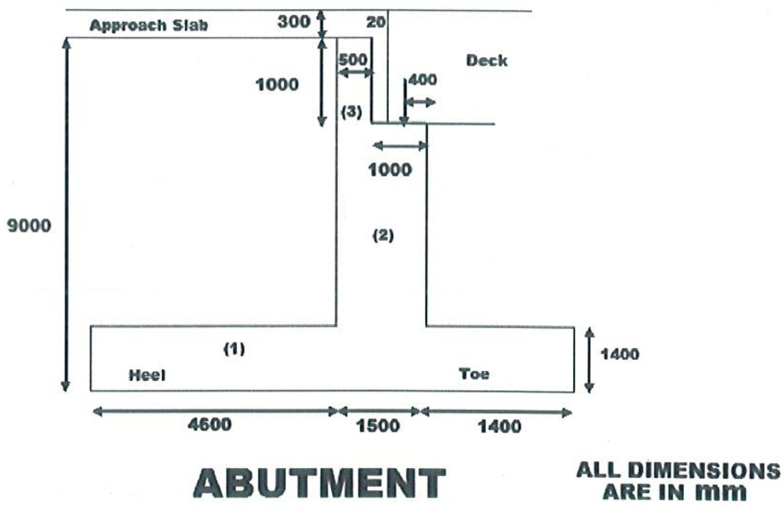


FIGURE 14

• Effective span	=16.8 m
• Overall length	=18 m
• Type of abutment	=R.C.C.
• Loading	=Class AA
• Backfill (angle of shearing resistance)	=32.2°
• Unit weight	=20.4 kN/m^3
• Z	=16.1°
• Dead load	=1050 kN
• Live load	=1735 kN
• Width	=9.5 m
• Neoprene bridge bearing pads	=320×400×65

• Embedded 5 plates of 3m thickness with 6m clearance.

• Bearing capacity of pads	=1N/mm^2

Self weight

- Part I

$7.5\times1.4\times24.8$ $=260.4$ kN/m

- Part II

$6.6\times1.5\times24.8$ $=245.52$ kN/m

- Part III

$0.5\times1\times24.8$ $=12.4$ kN/m

Longitudinal force (horizontal force)

- Force due to braking acts at 1.2m above road level ($0.2\times P_l=347$ kN)
- Force on one abutment wall $=347/2$

$=173.5$ kN/m

- Considering moderate +/- 17deg Celsius variation.
- Force due to temperature variation

❖ Strain due to temperature variation $=1.7\times11.9\times10^{-6}$

$=1.989\times10^{-4}$

❖ Strain due to shrinkage of concrete $=2\times10^{-4}$

•Total strain due to temp. & shrinkage=(1.989+2)×10-4 $=3.989\times10^{-4}$

•Horizontal deformation of deck due to temp. & shrinkage affecting one abutment

$=3.989\times18000/2$ $= 3.59$ mm

•Strain in bearing =Deformation/Elastomer thickness $= 3.59/65-(5\times3)$

$= 0.0718$

•Bearing capacity $= 1$ N/mm^2

•Horizontal force = (1.1×strain × area of plate in bearing × G × No. of bearing)/1000×width

$=(1.1\times0.0718\times1\times308\times700\times3)/(1000\times9.5)$ $= 5.377$ kN/m

•Vertical reaction due to braking $= 0.2P_L(1.2+1.3)/16.8\times9.5$ $= 5.43$ kN/m

Earth Pressure

•$P=0.5\ wh^2 \times ka$

•$ka=0.307$

•$Ø=32.2°$

•$\Theta = 90°$

•$Z = 16.1°$

•$\delta=0$ Inclination of earth fill surface

$P=0.307\times20.4\times9^2\times0.5=253.643KN/m$

•Height above the base of centre of pressure $=0.42\times 9$ $=3.78$ m

•Assuming Kp to be Neglected

Horizontal force due to live load surcharge

$= 1.2\times20.4\times0.307\times9$ $=67.638$ kN/m

Horizontal force due to approach slab

$= 0.3\times24.8\times0.307\times9$ $= 20.556$ kN/m

(Above two forces acting at 4.5m above base)

Vertical Load due to approach slab & Live Load surcharge

$=(1.2\times20.4+.3\times24.8)\times(4.6)$ $=146.832$ kN/m

Weight of earth on heel slab

$=20.4\times(9-1.4)\times4.6$ $=713.184$ kN/m

TABLE 3

Sno	Details	V kN	H kN	m Arm	M_V kN.m	M_H kN.m
1	D.L. from superstructure	1050	----	1.8	1890	----
2	Hor. force due to temp. & shrinkage	----	5.377	7.6	----	40.8652
3	Active earth pressure	----	253.64	3.78	----	958.7592
4	Horizontal force due to surcharge and approach slab	----	88.2	4.5	----	396.9
5	Vertical force due to surcharge and approach slab	146.832	----	5.2	763.5264	----
6	Selfweight part-1	260.4	----	3.75	976.5	----
7	Selfweight part-2	245.52	----	2.15	527.868	----
8	Selfweight part-3	12.4	----	2.65	32.86	----
9	Weight of earth on heel slab	713.184	----	5.2	3708.5568	----
10	**∑ Item 1 to 10(spanunloaded cond.)**	**2428.336**	**347.217**	—	**7899.3112**	**1396.5244**
11	L.L. from superstructure(class AA)	1735	----	1.8	3123	----
12	Vertical force due to breaking	5.4305	----	1.8	9.7749	----
13	Horizontal force due to breaking	----	173.5	8.00	----	1388
14	**∑ Item 11 to 14(span loaded cond.)**	**4168.766**	**520.717**	—	**11032.086**	**2784.5244**

CHECKS FOR ABUTMENTS

1) Check for stability

The forces and their position are shown in the figure-:

Two cases of Loading condition are examined :

a) Span Loaded Condition (Row 14)

Overturning moment about toe		2784.5244kN-m
Restoring moment about toe		11032.08619kN-m
Factor of safety		3.96192836 > 2 (safe)
Location Of Resultant from O		
Xo	$[(M_V-M_H)/V]$	1.978417762 m
e_{max}		1.25 m
e		1.1 < e_{max} safe

b) Span Unloaded Condition (Row 10)

Overturning moment about toe	1396.5244kN-m
Restoring moment about toe	7899.3112kN-m
Factor of safety	5.656407579 > 2 (safe)
Xo	2.677877691m
E	0.97 < e_{max} safe

2) Check for stresses at base

For span loaded condition

Total downward forces	4168.76655kN
Extreme stresses at base	1072.8289 kN/m^2 (max)
	68.48 kN/m^2 (min)

3) Check for sliding

Sliding Force	520.717 kN
Force resisting sliding	2501.25993kN
Factor of safety against sliding	4.803491974 > 1.5 (safe)

Hence the assumed section of the abutment is adequate

CHAPTER 5: DESIGN OF PIER

Highest flood level (H.F.L)	=5.3 m
Width of pier at H.F.L	= 2 m
Width of pier at the base of bed	=2 m
Height of pier	= 8 m
Width of bearing	=0.51 m
Length of pier	=10.5 m
Dead load from each span	= 210 KN
Reaction due to live load from each span	= 347 KN
Braking forces	=140 KN
Material of pier	= M25
Density of concrete	=24.8 KN/m^3

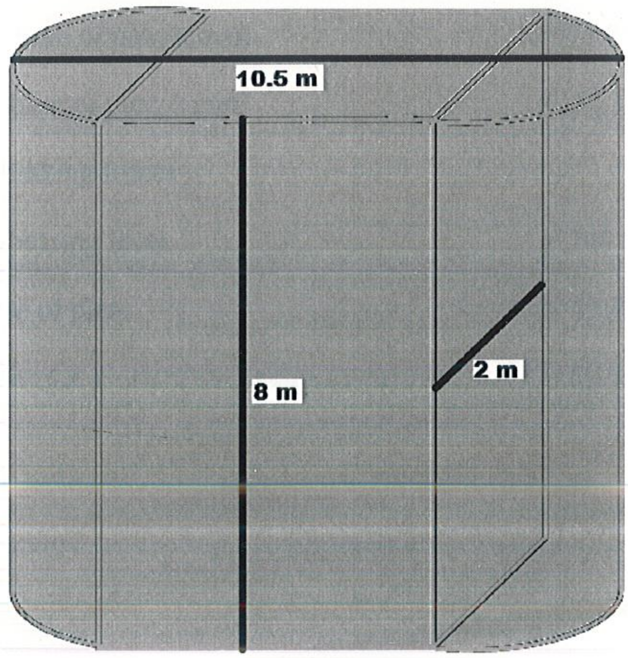


FIGURE 15

1.Stresses due to dead load and self weight of pier

Dead load, D.L	= 2100 KN
Self wt. of pier	= 8398.767 KN
Total direct load	= 10498.767 KN
Compressive stress at base of pier	= 499.9412857 KN/m^2

2. Effect of buoyancy

Width of pier at H.F.L	= 2 m
Submerged volume of pier	= 124.444 m^3
Reduction in wt. of pier due to buoyancy	= 1244.44 KN
Tensile stress at base due to buoyancy	= -59.25904762 KN/m^2

3.Stresses due to eccentricity of live load

Reaction due to live load, L.L	= 347 KN
Eccentricity	= 0.6 m
Moment about base, M	= 208.2 KN.m
Section Modulus, Z	= 21.33333333 m^3
Max. Stress at the base of pier, α max	= 75.8546131 KN/m^3
Min. Stress at the base of pier, α min	= 56.3358631 KN/m^2

4.Stresses due to braking forces

Breaking force at bearing level	= 140 KN
Moment about base of pier	= 1120 KN.m
Stresses at the base (±)	= 52.5 KN/m^2

Summary

Stresses	**Without Water**	**With Water**
Dead load and self wt	.=499.9412857	499.9412857 KN/m^2
Buoyancy	= 0	-59.25904762 KN/m^2
Eccentric live load	= 75.8546131	75.8546131 KN/m^2
Braking stress	= 52.5	52.5KN/m^2
Maximum stresses	= 628.2958988	569.0368512KN/m^2
Minimum stresses	= 523.2958988	464.0368512 KN/m^2

- The maximum permissible compressive stresses=2000KN/m^2 (M25 RCC)
- Hence stresses developed at the base of pier are within safe permissible limits.

Steel design:

Steel required for compression face

$P \times [\, e + D/2 - d' \,] = C_c + 0.87f_y \times A_{s1} - 0.87f_y \times A_{s2}$

$9255 \times 10^3 \times [600 + 1000 - 75] = 6673 \times 10^6 + 0.87 \times 415 \times 850 \times A_{st}$

$A_{st} = 24245.86\ mm^2$

Steel required for tension face

$P = (\, 0.36\ f_{ck} \times b \times d) + (0.87f_y \times A_{s1}) - (0.87f_y \times A_{s2})$

$9255 \times 10^3 = 0.36 \times 25 \times 10600 \times 2000 + 0.87 \times 415 \times 24245.86 - 0.87 \times 415 \times A_{s2}$

$A_{s2} = 220564.69\ mm^2$

$A_s = A_{s1} + A_{s2} = 244810.26\ mm^2$

$A_{smin} = 0.8\%$ of Area $= 169600\ mm^2$

$A_{s\ provided} = 245295.55\ mm^2 = 305$ no. of 32 mm ϕ bars.

So provide 32 mm bars a @ 45mm c/c

Shear Reinforcement (I.S. 456:200 Cl. 26.5.1.6)

Provide 12 mm 4 legged stirrup

$S_v = (0.87 \times f_y \times 4 \times \pi \times 12^2) / (\, 0.4 \times 10600 \times 4\,) = 38.522$ mm

So provide 4 legged 12 mm ϕ stirrups @ 35 mm c/c

CHAPTER 6: DESIGN OF PILE FOUNDATION

Total Load from Superstructure	= 10498.767 + 347	= 10845.767 kN
		= 10850 kN
Assuming 10 Piles		
Load on 1 Pile		= 1085 kN
Length of Pile		= 25 m
Length of pile above ground level		= 0.6 m
Total Length	= 25 + 0.6	= 25.6 m
L = 25.6 m	B = 0.8 m	
L/B	= 25.6/0.8	= 32
It will be designed as a long column since		32 > 12
Reduction coefficient	= 1.25 – (L/48B)	= 0.5833
Reference (I.R.C. 456:2000 Cl. B – 3.3)		
For M35 Grade concrete		
Permissible stress in concrete σ_{cc}		= 11.67 kN/mm^2
(I.R.C. 21 Table 10)		
Safe permissible stress in concrete	= 11.67 × 0.5833	= 6.807 kN/mm^2
Safe permissible stress in steel	= 200 × 0.5833	= 117 kN/mm^2
P	$= \sigma_{cc} A_{cc} + \sigma_{sc} A_{sc}$	
A_{sc}		= 9846.321 mm^2

Since 30 < L/B < 40

Longitudinal reinforcement

Should not be less than 1.5% of A_g

Reference (I.R.C. 78 – 2000 Cl. 709.48 (b))

1.5 % of A_g		$= 9600\ mm^2$
Provide 20 bars of 25 mm diameter.		
A_{st} provided		$= 9817.47\ mm^2$
A_{st} provided $> A_{st}$ min		
Clear Cover		= 75 mm
Lateral reinforcement		
Should not be less than 0.2 % A_g		
Using 8 mm ϕ as tie.		
Volume of tie	$= A_{sc} \times L$	$= 13000\ mm^3$
Volume of Pile per pitch	$= (800 \times 800) \times p$	
Pitch p		= 100 mm
Max permissible pitch	$= 0.5 \times 800$	= 400 mm
Distance of lateral reinforcement near pile head	$= 3 \times B$	= 2400 mm
Spiral reinforcement using 8 mm ϕ bars		
A_s		$= 50\ mm^2$
Volume of spiral per pitch		
Volume of helical reinforcement	$= \pi \times d \times A_s$	$= 92991\ mm^3$
Pitch p		= 35 mm
Lateral Reinforcement near end piles		0.4% of gross volume
Distance of lateral reinforcement near pile head	$= 3 \times 800$	= 2400 mm
Using 8 mm ϕ bars		
Volume of each tie	$= 5 \times (4800 - 8 \times 75)$	$= 13000 mm^3$
Volume per pitch length	$= 800 \times 800 \times p$	$= 13000\ mm^3$
Pitch p	= 101.56 mm	= 100 mm
Moment about Z-axis	$= (1.5W/2)-(0.55W/2)$	= 775 kNm
Effective depth required	$= \sqrt{(775 \times 10^6 / (0.874 \times 1750))}$	=720 mm

Clear cover = 55 mm

A_{st} per 1.5 m width $=(775 \times 10^6)/(230 \times 0.9 \times 720)$ = 5200mm^2

Spacing (using 25 mm ϕ bars) $= (1500 \times 491 / 5200)$ = 141.6 mm

Spacing = 120 mm

Adopt 25 mm ϕ bars at 120 mm c/c

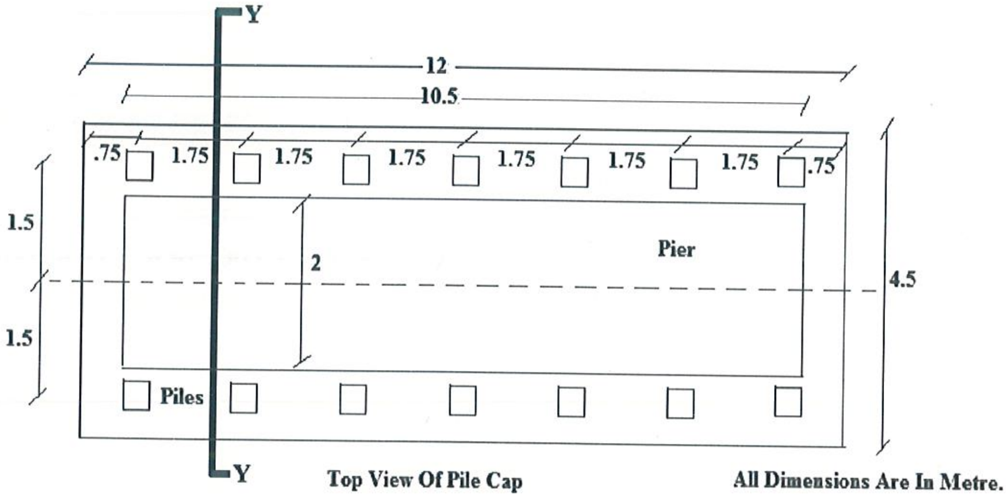


FIGURE 16

Moment about Z-axis $= (1.5W/2)-(0.55W/2)$ = 775 kNm

Effective depth required d $= \sqrt{(775 \times 10^6/ (0.874 \times 1750))}$ = 720 mm

Clear cover = 55 mm

A_{st} per 1.5 m width $=(775 \times 10^6)/(230 \times 0.9 \times 720)$ = 5200mm^2

Spacing (using 25 mm ϕ bars) $= (1500 \times 491 / 5200)$ = 141.6 mm

Spacing = 120 mm

Adopt 25 mm ϕ bars at 120 mm c/c

Distribution reinforcement $= 0.12$ of A_g = 1020 mm^2/m

Spacing $= (\pi \times 16^2 \times 1000/ 4 \times 1020)$ = 197.11 mm

Provide 16 mm ϕ bars at 190 mm c/c

Max Shear force		= 775 kN
Shear stress τ_v	= (V / bd)	= 0.62N/mm^2
From Table 19 I.S. 456:200		
τ_c		= 0.29 N/mm^2
Maximum shear that concrete can resist		= 3.7 N/mm^2
τ_{us}	$= \tau_v - \tau_c$	= 0.33 N/mm^2
V_{us}	$= 1750 \times 720 \times 0.33$	= 415.8 kN
Using 10 mm ϕ 8 legged stirrups		
S_v	$= (0.87 \times f_y \times A_s \times d)/V_{us}$	= 250.23 mm

Reference (I.S. 456:200 Cl.. 40.4(a))

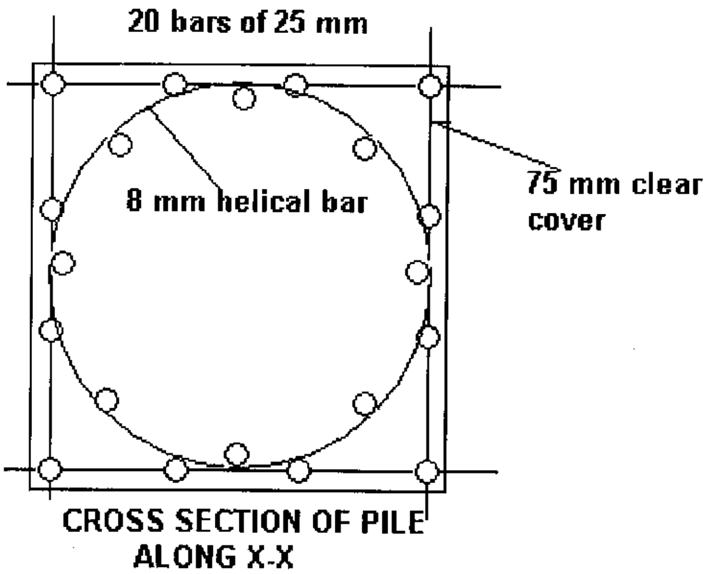


FIGURE 17

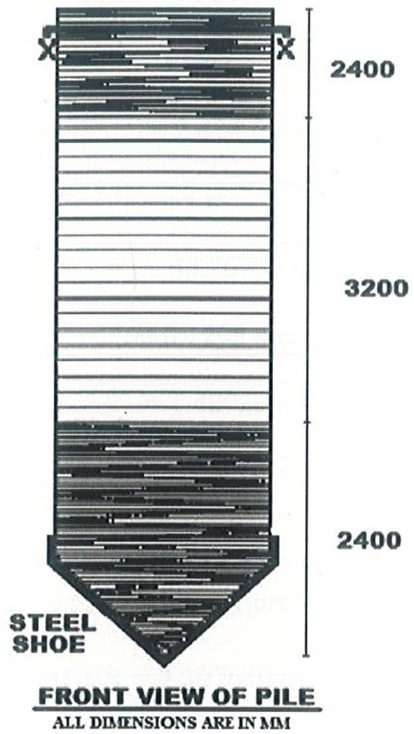


FIGURE: 18

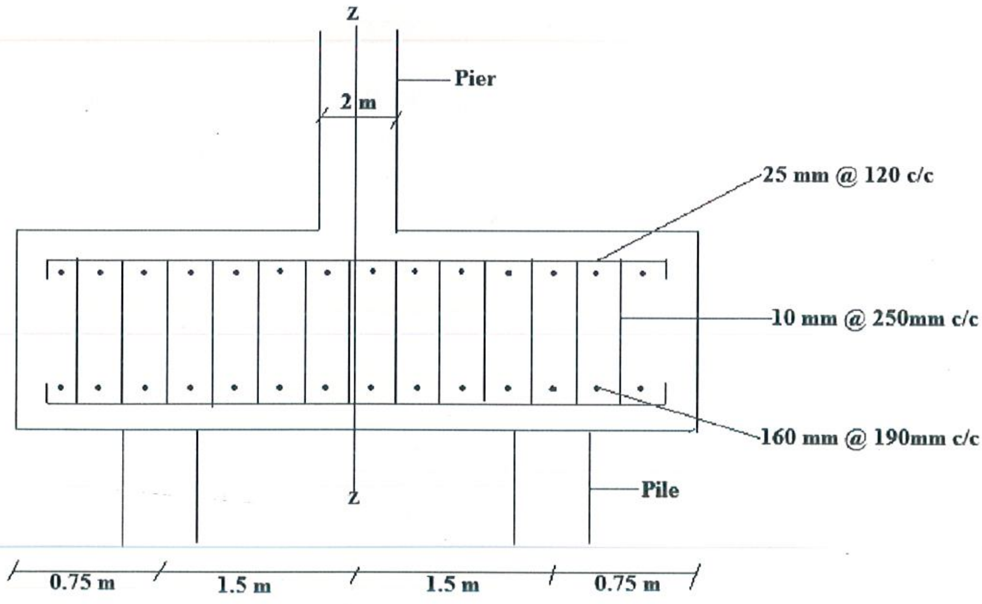


FIGURE 19

Maximum Scour Depth And Founding Level

	Value	Formula
Design discharge,D	= 2,426 m^2/s	
River width ,B	= 53 m	
Discharge intensity, q	= 45.76906 m^3/s per meter	q =D/B
Median size of sediment, d	= 0.1298 mm	
Flow velocity	= 1.489m/s	$v=(Qf^2/140)^{1/6}$
Highest flood level, H.F.L	= 5.3 m	
Lowest flood level, L.F. L	= 2 m	
LACEYS silt factor, f	= 0.634089	f=1.76 SQR d
Depth of flow (LACEYS),DLACEY	= 7.397649 m (below H.F.L)	$D=.473(Q/f)^{1/3}$
Maximum scour depth, Ds	= 14.7953 m (below H.F.L)	Ds=2DLACEY
Normal scour level	= 2.097649 m	
Deepest scour level	= 9.4953 m	

CHAPTER 7: SOFTWARE MODELING ON STAAD PRO

We are analyzing and modeling the superstructure and pier on the STAAD PRO.

To begin with STAAD.Pro V8i is the most popular structural engineering software product for 3D model generation, analysis and multi-material design. It has an intuitive, user-friendly GUI, visualization tools, powerful analysis and design facilities and seamless integration to several other modeling and design software products.

It is used For static or dynamic analysis of bridges, containment structures, embedded structures (tunnels and culverts), pipe racks, steel, concrete, aluminum or timber buildings, transmission towers, stadiums or any other simple or complex structure, STAAD-Pro has been the choice of design professionals around the world for their specific analysis needs. To see the new features in STAAD-Pro V8i, please go through the Release Report.

7.1 DESIGN CONSIDERATION

Span length	18m
Width of roadway	9.5m
Spacing of main girders	0 .75m on the corner and rest 2metres.
Surface deck thickness	0.3m
I section properties	
Width of flange	0.5m
Thickness of flange	.03m
Thickness of web	.01m
Depth of web	1m
Total depth of I-section	1.06m

7.2 DESIGN STEPS

- First we model and analyse the superstructure and then the Pier.
- After modeling both we merge them together and assess them both with two piers on each side and the bridge deck in the centre.
- Superstructure modeling:

- To begin with open STAAD pro create a new file.
- Specify the units.
- Define the nodes either by writing the coordinates or by using the snap node beam and grid method.
- Now join the nodes to form beams .
- On the left hand side go to support and create pinned support .
- Assign to the corner nodes on both the sides.
- Keeping the .75m segment overhanging.
- Now go to tools and on selecting create user table create an I SECTION with the user defined properties.
- Assigning it to all the beams.

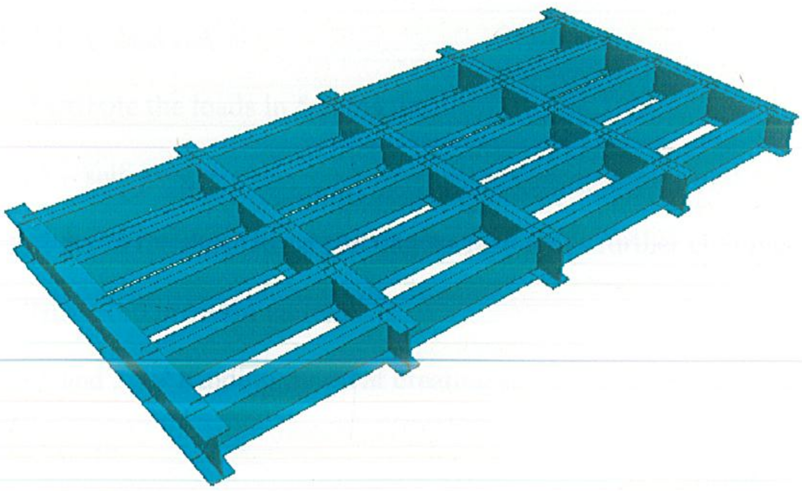


FIGURE 20

➢ Modeling of the upper deck

- Using the surface cursor select all the four edge nodes and create a surface over the beams.
- Thickness of the surface to fixed at 300mm.
- Material to be used is concrete.
- On assigning the thickness and the material we get the surface to be between the girder .i.e to rectify this we use the offset command to shift the beams down to a certain depth so that the deck lies above .
- Offset distance = depth of I/2 +deck/2
- Hence we give a offset of .68 in downward direction.

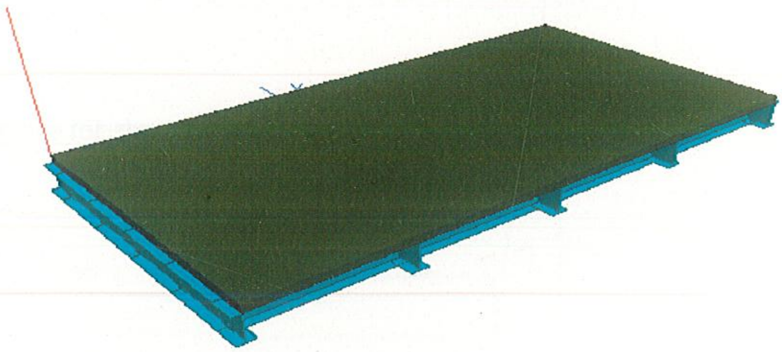


FIGURE 21

7.3 LOADINGS (class AA)

➢ We shall distribute the loads in 5 cases .i.e.:

➢ Load case 1 – self weight

➢ Load case 2 and 3 for shear force creating surface loads further creating a partially distributed pressure pattern

➢ Load case 4 and 5 for bending moment creating surface load in the same way but now in the centre.

➢ As the loading is for tracked vehicles we will distribute the load in such a way that there are two patches like the tracks of a tank.

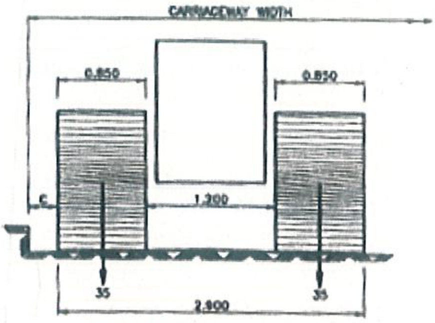
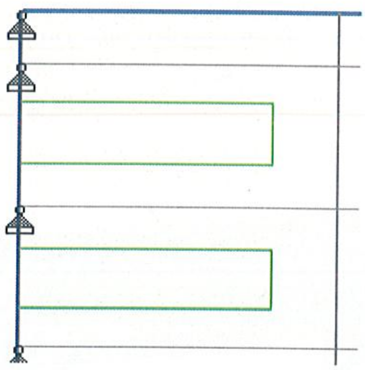


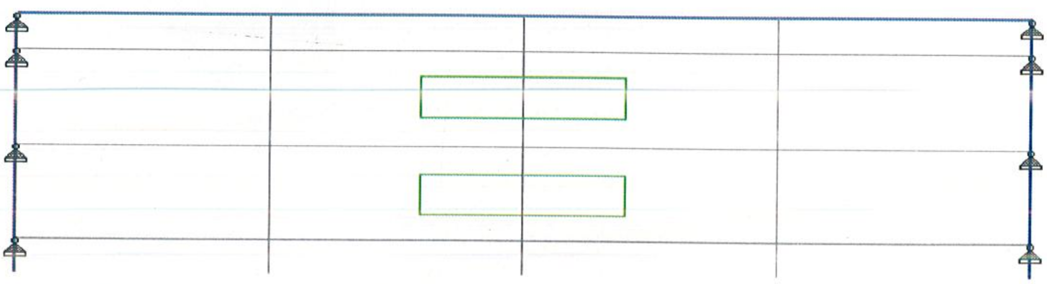
FIGURE 22

Further we will create two load combinations wherein we shall combine the self weight and shear force case in one combination and self weight and bending moment in the other.

➤ Load patches for shear force



➤ Load patches for bending moment



- Entering the steel design interface and check the structure in accordance with IS800.
- Using the take off optimize command we shall determine all the values of the steel sections.
- Performing analysis if there are no errors we will move on with the post processing.

7.4 PIER DESIGN:

- For the pier model we will select Run structure wizard
- Open solid model and create the solid model with height 8m and length and width 2m.
- Creating fixed support at the bottom nodes of the solid.
- Analysis the model and run it.

7.5 MERGING THE MODELS:

- Merging both the superstructure and the pier model together in such a way that it is bridge deck with piers on two sides.

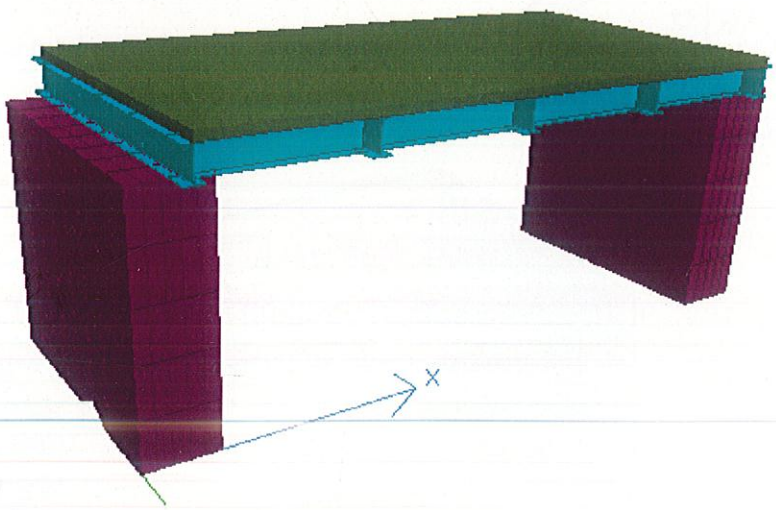
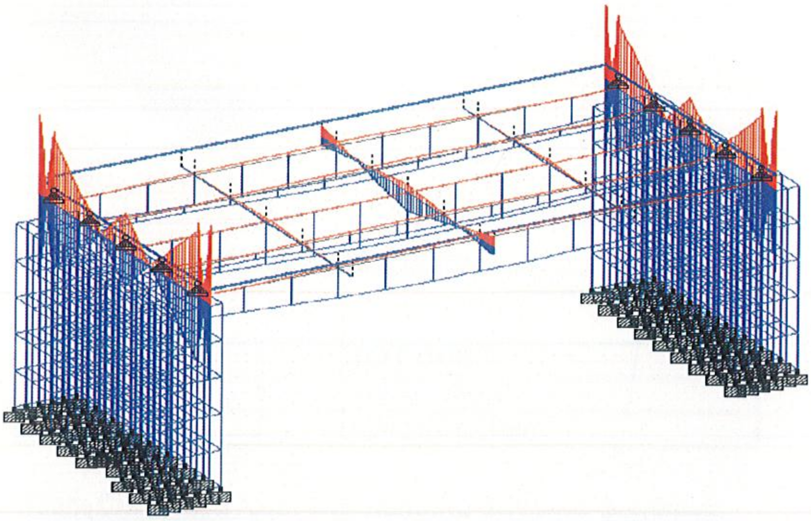


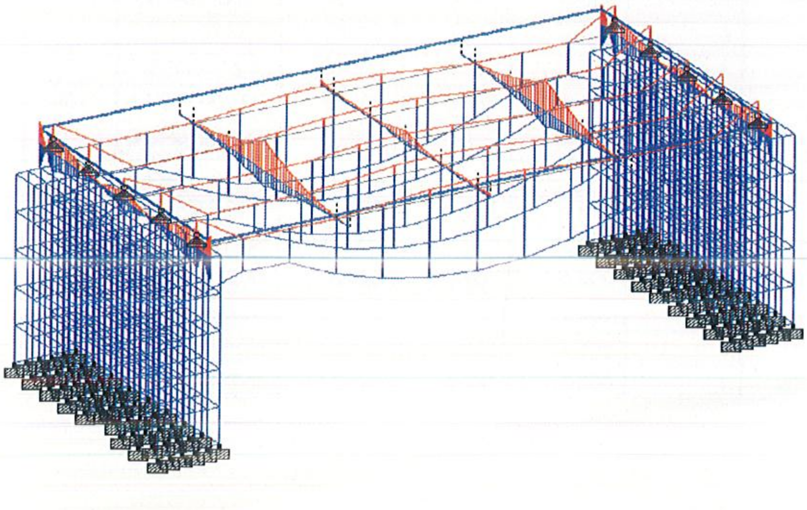
FIGURE 23

7.6 POST PROCESSING:

- On analyzing the whole structure we will enter the post processing mode.
- Here we will study the results of beam deflections, trusses , shear force and bending moment diagram.
- To begin with we study the stress diagram through the post processing panel of staad pro.
- Stress diagram for selfweight.



- Shear force load case .i.e. the corner load :



➢ Bending moment load case .i.e. the central load :

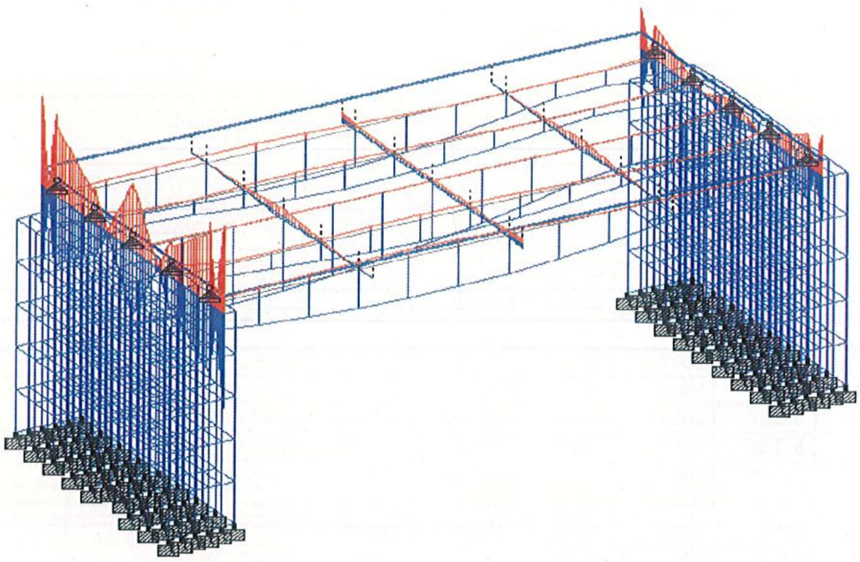


TABLE 4

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	LOAD CASE 1
Combination	3	SHEAR FORCE
Combination	7	BENDING MOMENT

Beam Combined Axial and Bending Stresses Summary

			Max Comp			Max Tens		
Beam	L/C	Length (m)	Stress (N/mm^2)	d (m)	Corner	Stress (N/mm^2)	d (m)	Corner
480	1:LOAD CASE	18.000	3.246	0.000	3	-9.249	9.000	4
	3:SHEAR FOR	18.000	3.170	4.500	2	-18.452	6.000	4
	7:BENDING M	18.000	4.207	9.000	2	-37.117	9.000	4
481	1:LOAD CASE	18.000	3.664	0.000	3	-8.528	9.000	3
	3:SHEAR FOR	18.000	4.918	18.000	3	-9.973	7.500	3
	7:BENDING M	18.000	5.987	18.000	4	-10.856	9.000	3
482	1:LOAD CASE	18.000	2.476	18.000	3	-9.590	9.000	3
	3:SHEAR FOR	18.000	3.447	18.000	3	-33.913	1.500	3
	7:BENDING M	18.000				-51.018	9.000	3
483	1:LOAD CASE	18.000	3.664	0.000	3	-8.528	9.000	3
	3:SHEAR FOR	18.000	4.918	18.000	4	-9.973	7.500	4
	7:BENDING M	18.000	5.987	0.000	3	-10.856	9.000	4
484	1:LOAD CASE	18.000	3.246	0.000	3	-9.249	9.000	3
	3:SHEAR FOR	18.000	3.170	4.500	1	-18.452	6.000	3
	7:BENDING M	18.000	4.207	9.000	1	-37.117	9.000	3

➢ Axial ,Shear force and Bending moment results

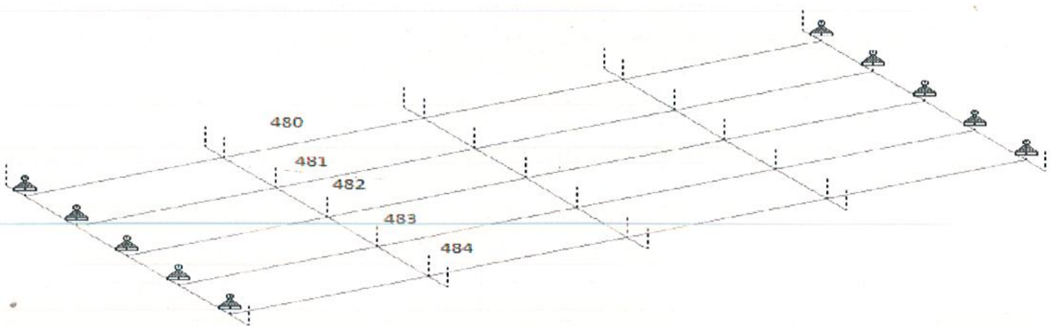
TABLE 5

Beam Force Detail

Sign convention as diagrams:- positive above line, negative below line except Fx where positive is compression. Distance d is given from beam end A.

			Axial	Shear		Torsion	Bending	
Beam	L/C	d (m)	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
480	1:LOAD CASE	0.000	-74.624	20.880	-0.000	0.000	0.342	42.958
		9.000	-74.624	-0.000	-0.000	0.000	0.342	-51.000
		18.000	-74.624	-20.880	-0.000	0.000	0.342	42.958
	3:SHEAR FOR	0.000	-159.260	17.286	-0.518	0.000	8.668	-11.701
		9.000	-159.260	-3.593	-0.518	0.000	4.002	-73.320
		18.000	-159.260	-24.473	-0.518	0.000	-0.664	52.976
	7:BENDING M	0.000	-368.708	20.880	-0.000	0.000	12.616	-34.599
		9.000	-368.708	-0.000	-0.000	0.000	12.616	-128.557
		18.000	-368.708	-20.880	-0.000	0.000	12.616	-34.599
481	1:LOAD CASE	0.000	-66.416	20.880	0.000	0.000	-0.105	45.123
		9.000	-66.416	-0.000	0.000	0.000	-0.105	-48.835
		18.000	-66.416	-20.880	0.000	0.000	-0.105	45.123
	3:SHEAR FOR	0.000	-73.442	19.892	0.299	0.000	-4.305	34.380
		9.000	-73.442	-0.988	0.299	0.000	-1.612	-50.688
		18.000	-73.442	-21.867	0.299	0.000	1.080	52.161
	7:BENDING M	0.000	-69.342	20.880	0.000	0.000	-3.942	44.351
		9.000	-69.342	-0.000	0.000	0.000	-3.942	-49.607
		18.000	-69.342	-20.880	0.000	0.000	-3.942	44.351
482	1:LOAD CASE	0.000	-83.264	20.880	0.000	0.000	-0.000	40.680
		9.000	-83.264	-0.000	0.000	0.000	-0.000	-53.278
		18.000	-83.264	-20.880	0.000	0.000	-0.000	40.680
	3:SHEAR FOR	0.000	-344.482	4.914	0.000	0.000	-0.000	-171.900
		9.000	-344.482	-15.965	0.000	0.000	-0.000	-122.168
		18.000	-344.482	-36.845	0.000	0.000	0.000	115.479
	7:BENDING M	0.000	-706.636	20.880	0.000	0.000	-0.000	-123.720
		9.000	-706.636	-0.000	0.000	0.000	-0.000	-217.678
		18.000	-706.636	-20.880	0.000	0.000	-0.000	-123.720

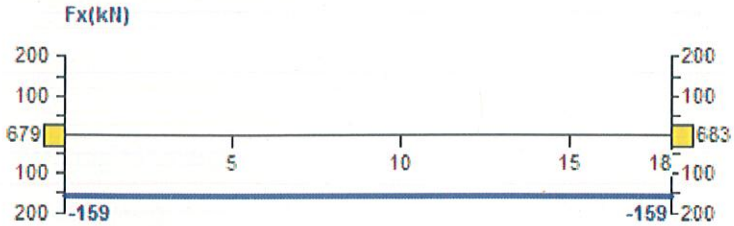
➢ Axial force, shear force and bending moment diagram for each load case and each main girder i.e. :



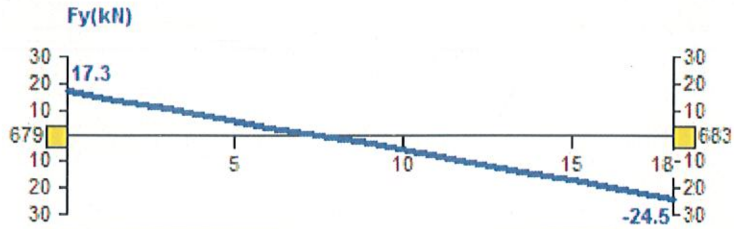
Shear Force Load case :

➢ For Beam 480:

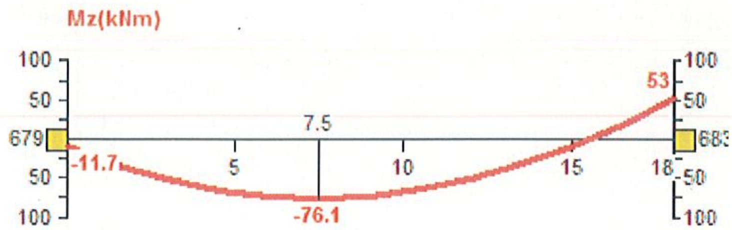
Axial force



Shear force

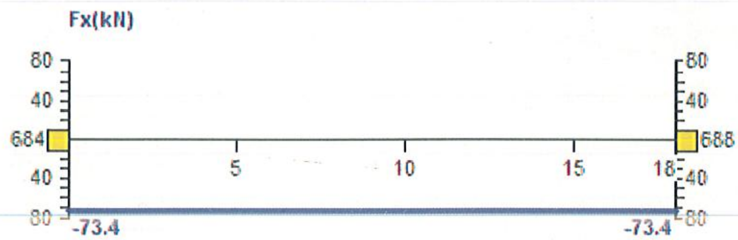


Bending moment

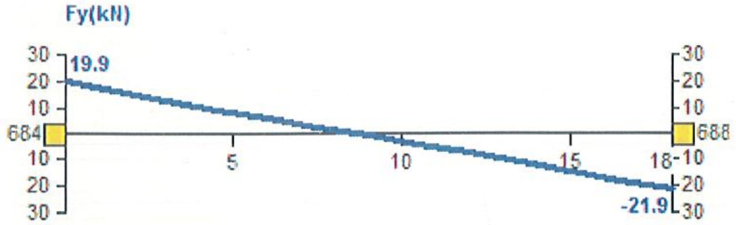


➢ For beam 481:

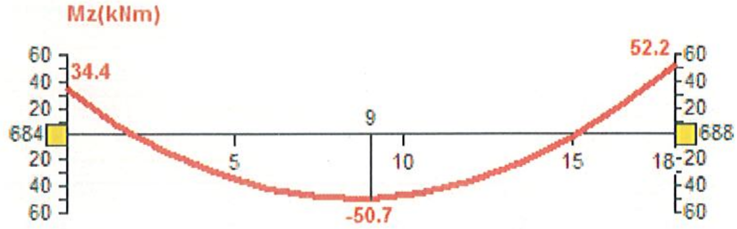
Axial force



Shear force

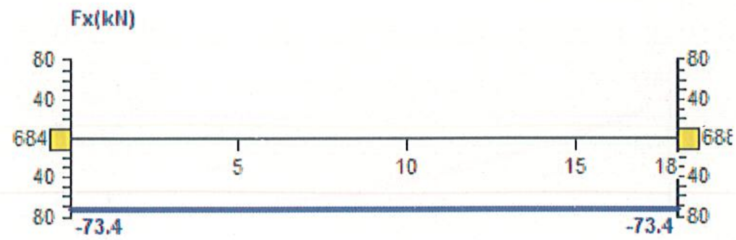


Bending moment

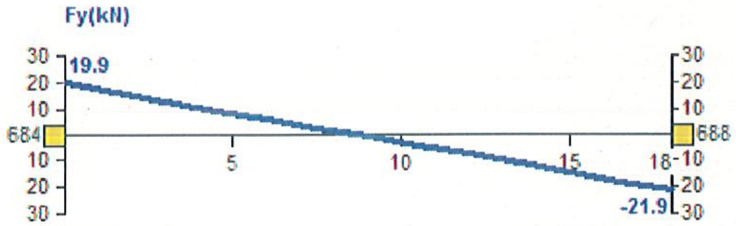


➢ For beam 482:

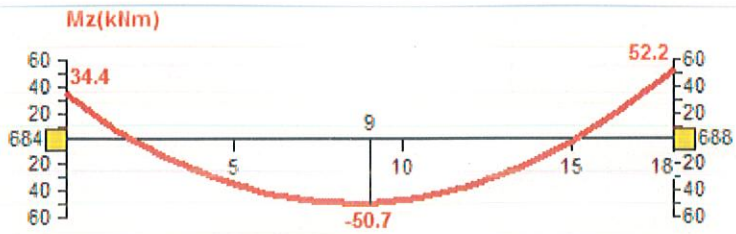
Axial force



Shear force

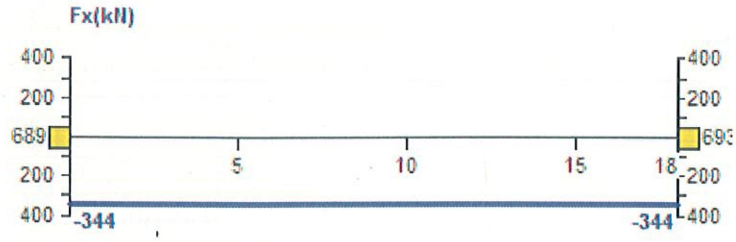


Bending moment

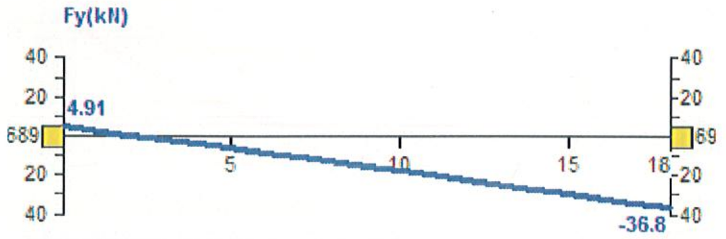


➢ For beam 483:

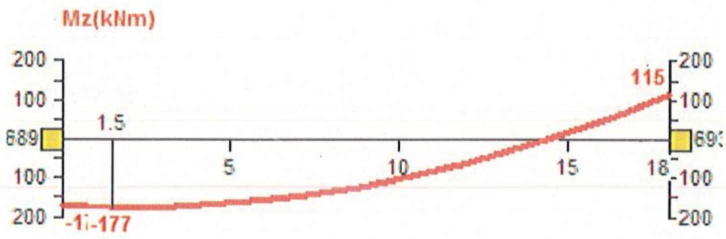
Axial force



Shear force

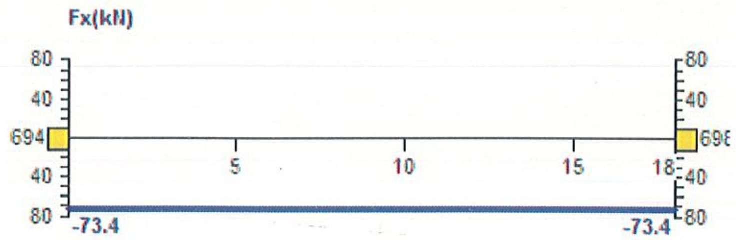


Bending moment

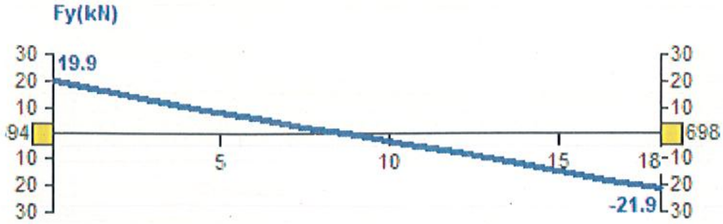


➢ For beam 484:

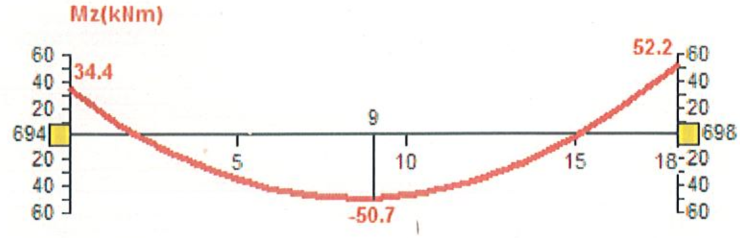
Axial force



Shear force



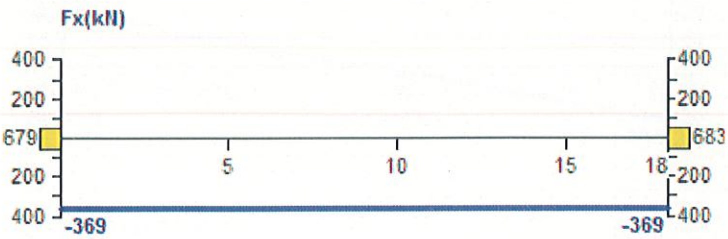
Bending moment



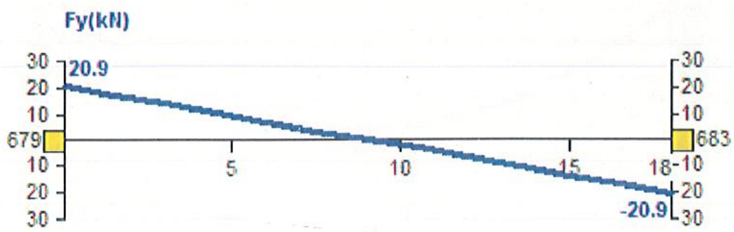
For bending moment load case :

➢ Beam 480

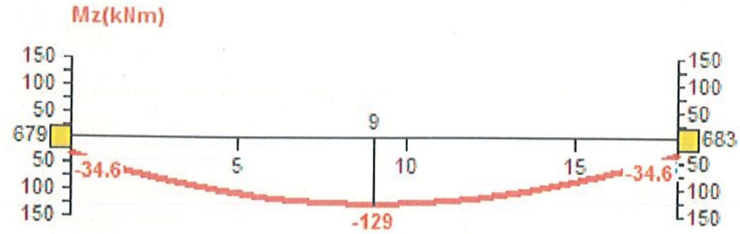
Axial force



Shear force

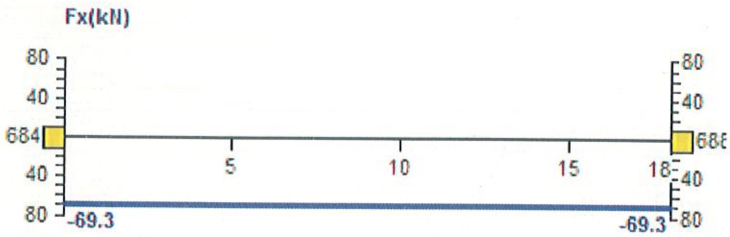


Bending moment

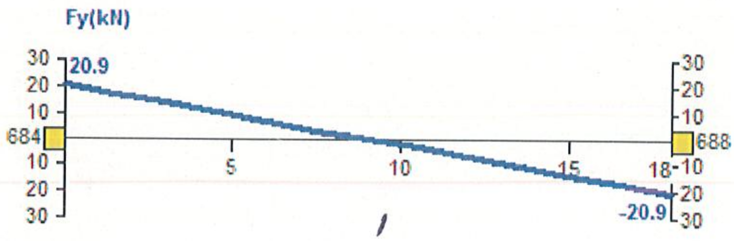


➢ Beam 481:

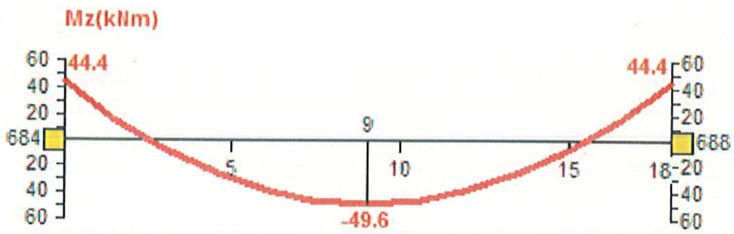
Axial force



Shear force

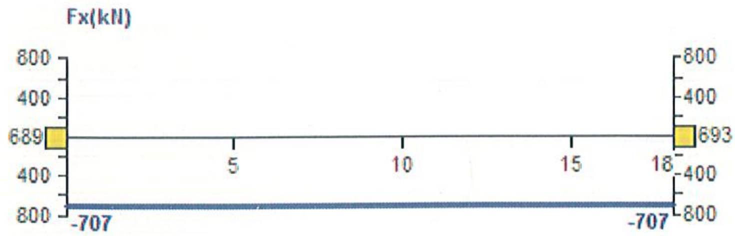


Bending moment

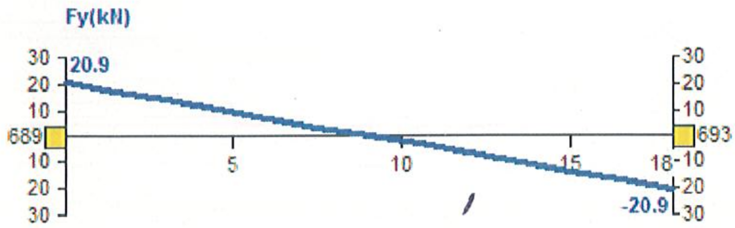


➢ Beam 482:

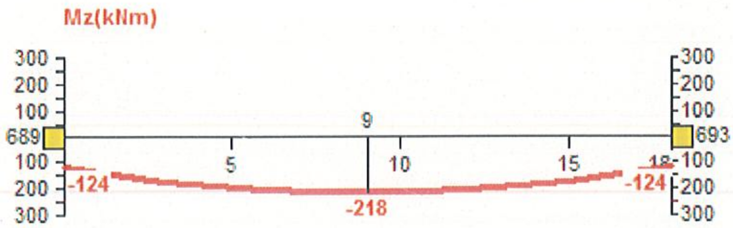
Axial force



Shear force

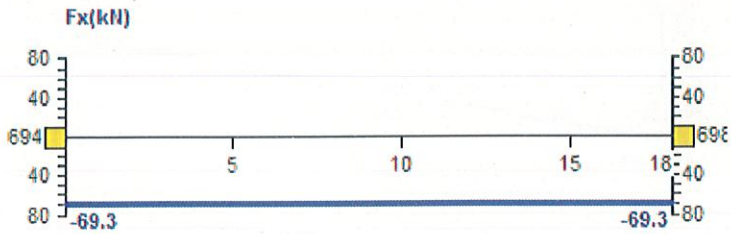


Bending force

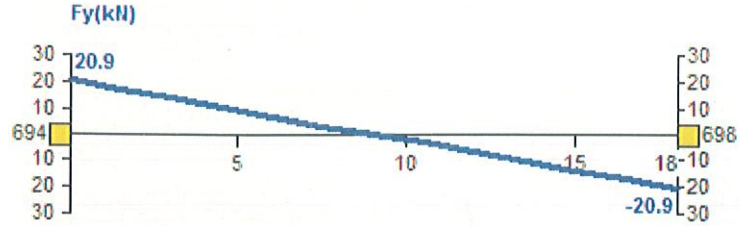


➢ For beam 483:

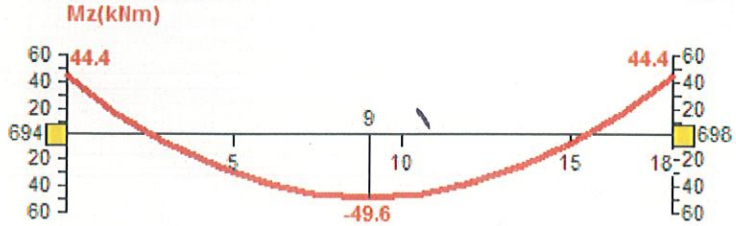
Axial force



Shear force

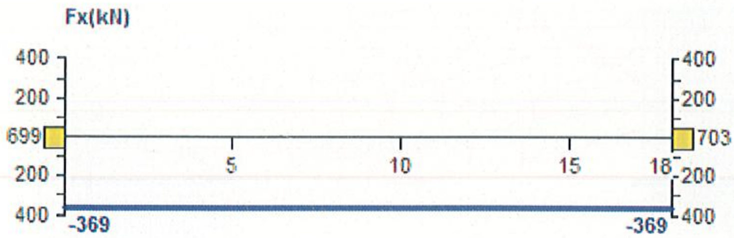


Bending moment

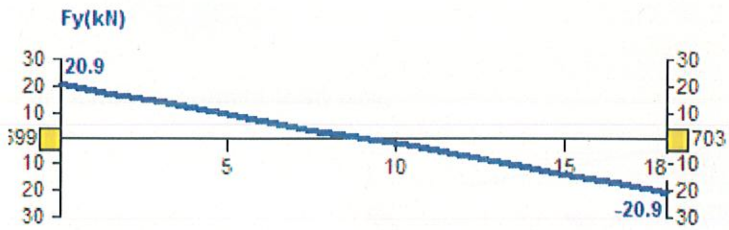


➢ For beam 484:

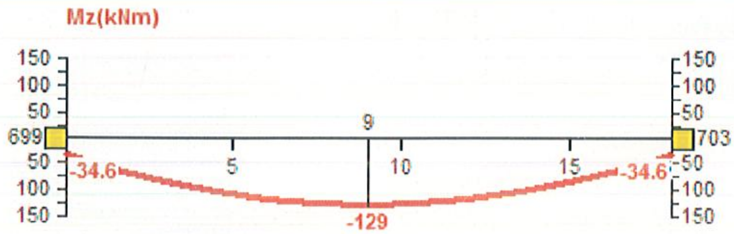
Axial force



Shear force

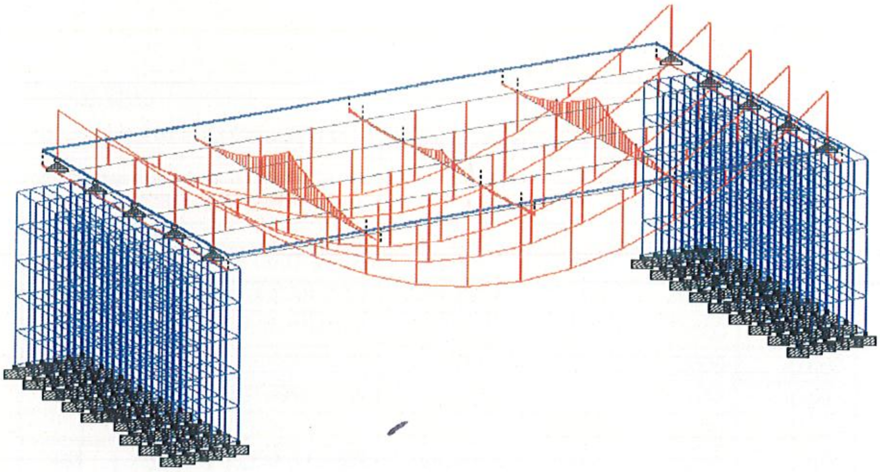


Bending moment

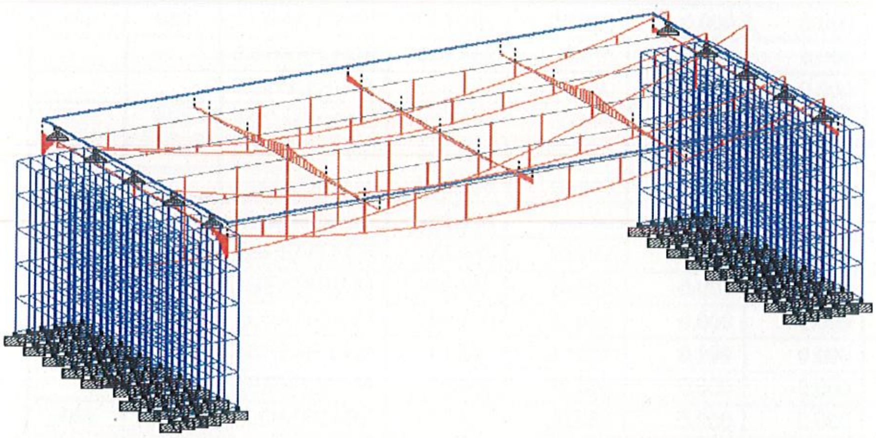


➢ Beam bending pattern in the Z direction :

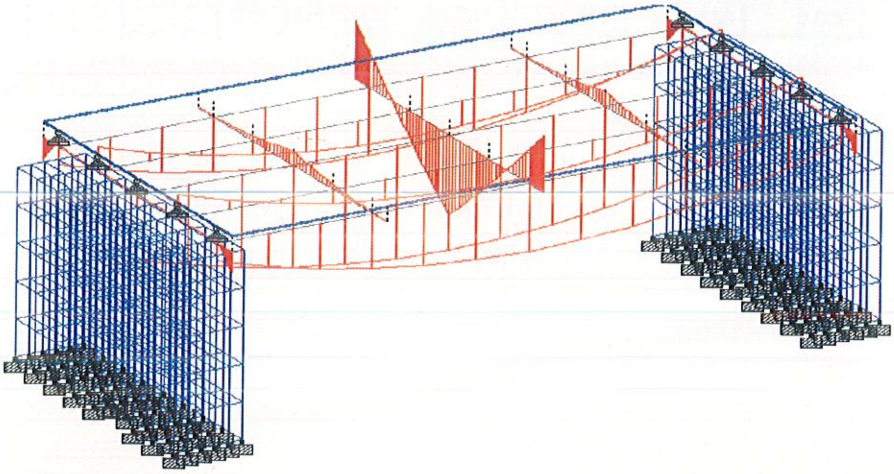
For self weight load case



For shear force load case



For bending moment load case



Beam end forces .i.e. forces at the corner nodes of each main girder:

TABLE 6

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
480	679	1:LOAD CASE	-74.624	20.880	-0.000	0.000	0.342	42.958
		3:SHEAR FOR	-159.260	17.286	-0.518	0.000	8.668	-11.701
		7:BENDING M	-368.708	20.880	-0.000	0.000	12.616	-34.599
	683	1:LOAD CASE	74.624	20.880	0.000	-0.000	-0.342	-42.958
		3:SHEAR FOR	159.260	24.473	0.518	-0.000	0.664	-52.976
		7:BENDING M	368.708	20.880	0.000	-0.000	-12.616	34.599
481	684	1:LOAD CASE	-66.416	20.880	0.000	0.000	-0.105	45.123
		3:SHEAR FOR	-73.442	19.892	0.299	0.000	-4.305	34.380
		7:BENDING M	-69.342	20.880	0.000	0.000	-3.942	44.351
	688	1:LOAD CASE	66.416	20.880	-0.000	-0.000	0.105	-45.123
		3:SHEAR FOR	73.442	21.867	-0.299	-0.000	-1.080	-52.161
		7:BENDING M	69.342	20.880	-0.000	-0.000	3.942	-44.351
482	689	1:LOAD CASE	-83.264	20.880	0.000	0.000	-0.000	40.680
		3:SHEAR FOR	-344.482	4.914	0.000	0.000	-0.000	-171.900
		7:BENDING M	-706.636	20.880	0.000	0.000	-0.000	-123.720
	693	1:LOAD CASE	83.264	20.880	-0.000	-0.000	0.000	-40.680
		3:SHEAR FOR	344.482	36.845	-0.000	-0.000	-0.000	-115.479
		7:BENDING M	706.636	20.880	-0.000	-0.000	0.000	123.720
483	694	1:LOAD CASE	-66.416	20.880	-0.000	0.000	0.105	45.123
		3:SHEAR FOR	-73.441	19.892	-0.299	-0.000	4.305	34.380
		7:BENDING M	-69.342	20.880	-0.000	0.000	3.942	44.351
	698	1:LOAD CASE	66.416	20.880	0.000	-0.000	-0.105	-45.123
		3:SHEAR FOR	73.441	21.867	0.299	0.000	1.080	-52.161
		7:BENDING M	69.342	20.880	0.000	-0.000	-3.942	-44.351
484	699	1:LOAD CASE	-74.624	20.880	0.000	0.000	-0.342	42.958
		3:SHEAR FOR	-159.261	17.286	0.518	-0.000	-8.668	-11.702
		7:BENDING M	-368.709	20.880	0.000	0.000	-12.616	-34.600
	703	1:LOAD CASE	74.624	20.880	-0.000	-0.000	0.342	-42.958
		3:SHEAR FOR	159.261	24.473	-0.518	0.000	-0.664	-52.977
		7:BENDING M	368.709	20.880	-0.000	-0.000	12.616	34.600

➢ Steel design details: the table shows the actual ratio , the allowable ratio and the normalized ratio of each main girder.

➢ The actual ratio is the ratio fixed during steel design according to the design parameters.

TABLE 7

Utilization Ratio

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (cm^2)	Iz (cm^4)	Iy (cm^4)	Ix (cm^4)
480	ISECTION	ISECTION	0.237	0.640	0.370	IS-7.1.1(A)	7	302.000	542E+3	41.7E+3	816.024
481	ISECTION	ISECTION	0.088	0.640	0.138	IS-7.1.1(A)	7	302.000	542E+3	41.7E+3	816.024
482	ISECTION	ISECTION	0.325	0.640	0.509	IS-7.1.2	7	302.000	542E+3	41.7E+3	816.024
483	ISECTION	ISECTION	0.088	0.640	0.138	IS-7.1.1(A)	7	302.000	542E+3	41.7E+3	816.024
484	ISECTION	ISECTION	0.237	0.640	0.370	IS-7.1.1(A)	7	302.000	542E+3	41.7E+3	816.024

CONCLUSION

In this project, a steel composite bridge has been designed. The two lane steel composite bridge is designed for I.R.C. class AA tracked vehicle loading. The bridge consists of three spans of 18 m and of width 9.5 m, which were manually analyzed and designed. The superstructure and pier was analyzed and designed in Staad Pro 2008. First, soil analyses were carried out followed by structural design. After analysis, the results were found to be within limits and reliable. The report involves different aspects which were taken into consideration at the time of designing the bridge as per Indian Road Congress and Indian Standard Codes.

The bridge designing was done utilizing MS-Excel spreadsheets. These spreadsheets cover all the design considerations of the bridge and also facilitate any future changes in the data without any programming. The advantage in using these sheets lies in the ease of use and the generation of reliable results. The design and analysis of superstructure and pier were confirmed using Staad pro and further all members were rendered safe in accordance with I.S. 800:2007.

All-in-all, the report provides a compilation involving almost every aspect related to steel composite bridge design and detailing.

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