ANALYSIS AND DESIGN OF BRIDGE SUBSTRUCTURE AND FOUNDATION

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Joshi Bhagirath G.
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Guide
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Ahmedabad 382481
May 2006

CERTIFICATE

This is to certify that the Major Project entitled "**Analysis and Design of Bridge Substructure and Foundation**" submitted by **Mr. Bhagirath G. Joshi (04MCL004)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering** (**Computer Aided Structural Analysis and Design**) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Date of Examination

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ABSTRACT

Bridge is consisting of mainly two components like superstructure and substructure. Till nowadays lot many investigations and research work is done on Bridge superstructure's type and shape. But the substructure topic is not being in light till now. Economy of bridge depends on the cost of substructure and superstructure and therefore the bridge should be designed in such a manner that the cost of substructure and superstructure should be almost equal.

Substructure and Foundation design consist of:

- Pier cap design
- Pier design
- Pile cap design
- Pile foundation design for central span
- Well foundation design for end span

Pier cap is designed and reinforced to take care of loads of superstructure dispersing in pier. According to positions of bearings and top section of pier, when bearings are placed centrally over the pier, the load from bearings is directly transferred to pier and the pier cap need not to be designed for flexure but when bearings are not placed centrally over the pier, the part of pier cap have to be designed as cantilever portion.

The forces considered for pier design are Dead load, Buoyancy Force Live load (70 R Wheeled Vehicle), Impact Load, Braking force, Seismic Force, Wind Force, Water Current Force etc. The various load combinations are done for construction condition as well as for service condition. The pier behaves as an ordinary R.C.C. column and is designed as a R.C.C. column subjected to biaxial bending.

A rigid pile cap in reinforced concrete should be provided to transfer the load from the pier to the piles as uniformly as possible under normal vertical loads. Pile cap can be designed either by Truss analogy method or by bending theory. Here the pile cap is designed by Truss analogy method. In truss analogy method

pile cap area is divided into various strips in both the directions considering number of pile and pile diameter. The truss is in triangular form with a node at the centre of loaded area. The lower node of the truss lies at the intersection of the centre line of the piles. Strips in both directions are designed as beam elements while remaining portion are designed as slab element.

For the analysis of pile, the forces considered are Dead load, Buoyancy Force Live load (70 R Wheeled Vehicle), Impact Load, Braking force, Seismic Force, Wind Force, Water Current Force. The various load combinations are done for construction condition as well as for service condition. Load distributed on individual pile is compared with load carrying capacity of individual pile which should be higher than the load on individual pile. The pile behaves as an ordinary column and is designed as a column subjected to biaxial bending.

The forces considered for analysis of Abutment wall are Dead Load, Live Load, Braking Force, Seismic force and Earth Pressure. For Earth pressure any rational theory shall be accepted subjected to modification that the centre of pressure exerted by the backfill is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of that height (As per IRC Provisions). The various load combinations are done for construction condition as well as for service condition. Abutment wall is also considered as column, so it is designed as a column with biaxial bending.

Abutment cap should be suitably designed and reinforced to take care of concentrated point loads dispersing in abutment. Some part of the Abutment cap is designed as corbel portion. A dirt wall is provided to prevent the earth from approaches spilling on the bearings. Dirt wall is designed for self weight, Earth Pressure, Live load acting directly on dirt wall and braking force due to Live Load.

The main components of well foundation are Well Cap, Steining, Well Curb, and Bottom Plug. The forces considered for analysis of Well foundation are Dead Load, Live Load, Impact Force, Seismic Force and Earth pressure. A well cap is needed to transfer the loads and moments from the pier or abutment to the well or wells below. Well cap is designed by bending theory. Lateral stability of well is most important check for well foundations. Lateral Stability of well is checked by Elastic approach and Ultimate resistance approach.

The parametric study for “Loads and moments on Pier and Pile” is carried out for span of superstructure ranging between 15m to 40m is carried out. For parametric study the program is prepared in Visual Basic which gives design loads and moments for design of pier and pile.

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NOTATIONS

V	=	Maximum mean Velocity of Water Current (m/sec)
Q_a	=	Capacity of Individual Pile
P_{max}	=	Calculated maximum Load on Individual Pile
A_g	=	Gross Area of Section
f	=	The minimum clearance between outer edge of the wheel and the roadway face of the kerb
g	=	The minimum clearance between the outer edges of passing or crossing vehicles
Φ	=	Angle of internal friction of soil
δ	=	Angle of Wall friction
k_a	=	Co-efficient of Active Earth Pressure
k_p	=	Co-efficient of Passive Earth Pressure
P	=	Vertical Load (kN)
HL	=	Horizontal Load in Longitudinal direction (kN)
HT	=	Horizontal Load in transverse direction (kN)
ML	=	Longitudinal Moment (kN-m)
MT	=	Transverse Moment (kN-m)
F_{eq}	=	Horizontal Seismic Force (kN)
Z	=	Zone factor
I	=	Importance Factor
R	=	Response Reduction Factor
Sa/g	=	Average response acceleration coefficient
μ	=	Coefficient of friction between soil and concrete of curb
N	=	weight of steining in kN per m run
H	=	Hoop Tension
C	=	Hoop Compression

1. INTRODUCTION

1.1 General

The civil engineer makes a vital contribution to the development of the modern world. Society relies heavily on the abilities of civil engineers to create a sustainable environment which is both functional and pleasing. The need of bridge is felt by people and it is communicated to Government through public representatives. Bridge structure is designed to provide continuous passage over an obstacle. Bridges commonly carry highways, railroad lines, and pathways over obstacles such as waterways, deep valleys, and other transportation routes. Bridges may also carry water, support power cables etc

A good bridge is one that is simple in order, has functional performance, graceful in view, balanced in distribution of mass, harmonious in proportion, orderly in lines, integral with the environment and serene in character. The subject bridge engineering is being given greater emphasis in the new curricula for engineering studies. A successful bridge engineer has to have an appreciation of aesthetics and economics, besides ability in analysis and dexterity in design.

1.2 Need of Study

Economy of bridge depends on the cost of substructure and superstructure. A bridge should be designed in such a manner that the cost of substructure and superstructure should be almost equal. Till today many investigation and research work is done on Bridge superstructure's type and shape. But the substructure topic is not been explored till now and therefore aim of study is to prepare Excel spreadsheets for analysis and design of bridge substructure and foundation components like pier cap, pier, pile cap, pile, abutment cap, abutment wall, dirt wall, well cap and well foundation. The parametric study for "Loads and moments on Pier and Pile" is carried out for span of superstructure ranging between 15m to 40m is carried out. For parametric study the program is prepared in Visual Basic which gives design loads and moments for design of pier and pile.

1.3 General Design consideration

A good design for a bridge structure should satisfy the two main requirements first is economy and second is aesthetics. The function of a bridge is to provide a passage to the intended traffic over the particular obstruction with almost safety and convenience. The load carrying capacity of the structure should be adequately above the normal service loads so that the probability of failure during the specified lifetime of the bridge is below the limit specified. The safety and serviceability of the bridge are ensured by limiting the deflections and vibrations during its service life.

The aesthetic requirements of a bridge structure, through very important, are difficult to codify. The designer should attempt integration of the structure and the visual form rather than treating them as separate parts.

The need for the consideration of economy in bridge structures cannot be overemphasized. The criteria for economy vary from place to place and no rigid rules can be specified. The bridge engineer should consider several preliminary designs using different span arrangements, structural materials and construction techniques. The designer should examine the features of the above designs for function, aesthetics and economy and narrow down the choice to two or three alternatives.

Steps for design of bridge:

The below mentioned steps should be followed while designing a bridge

1. Survey and selection of bridge site
2. Collection of design data for bridge projects.
3. Select type of bridge as per requirement.
4. Choice of span for bridge.
5. Select suitable section of superstructure.
6. Select suitable type and shape of substructure as per geological condition.
7. Select suitable type and size-shape of foundation as per soil condition.

1.4 Organization of Report

The dissertation work is organized in the following manner.

Chapter 1 covers the introduction part, Need of study of the topic “bridge substructure and foundations” and general design considerations

Chapter 2 deals with Literature review. It includes the details of various literatures covered in journals, papers and books regarding bridge substructure and foundation.

Chapter 3 covers introductory part of superstructure, details of various components of superstructure and Loads acting on bridges.

Chapter 4 covers forces acting on pier cap and pier as per IRC-recommendation, IRC-code provision for design of pier and pier cap, Analysis and Design of pier and pier cap and stability check for pier.

Chapter 5 deals with forces acting on pile cap and pile as per IRC-recommendation, IRC-code provision for design of Pile foundation and pile cap, Analysis and Design of Pile foundation and pile cap.

Chapter 6 covers forces acting on abutment as per IRC-recommendation, IRC-code provision for design of Abutment and Abutment cap and Analysis and Design of Abutment and Abutment cap.

Chapter 7 deals with forces acting on well foundation as per IRC-recommendation, IRC-code provision for design of Well foundation and Well cap, Analysis and Design of Well foundation and Well cap and stability check for well foundation.

Chapter 8 covers parametric study of pier and pile to calculate design moments and loads with varying span of superstructure from 15m to 40m.

Chapter 9 covers conclusion and future work of study

Appendix 1 covers with Visual basic program's form and source file for pier

Appendix 2 covers with Visual basic program's form and source file for pile

2. LITERATURE SURVEY

2.1 General

Literature survey is carried out to review various applications of analysis and design of substructure and foundation of bridge. In literature survey, main emphasis is given on various books, IRC codes, published papers that describe the implementation of bridge substructure and foundation design.

2.2 Literature Survey

Rakshit K. S., “Design and construction of highway bridges”, New central book Agency, Calcutta, India.

The book covers various types of piers and abutments, design considerations for pier and abutment. It also covers shallow and deep foundations for bridge and their design. In particular deep foundations like pile foundation and well foundation design are covered. It explains load transfer mechanism of pile and its load carrying capacity.

Swami Saran, “Analysis and design of substructures- Limit state design”, Oxford & IBH Publishing Co. Pvt. Ltd., New Delhi, India.

It coves analysis and design of substructure and foundation of bridge. Pier design problem and well foundation design problem is also covered in this book. It also covers lateral stability of well foundation by elastic analysis and ultimate resistance approach and also stresses on well steining and well curb.

Victor D.J., “ Essentials of bridge Engineering”, Oxford & IBH Publishing Co. Pvt. Ltd., New Delhi, India.

This book deals with various types of pier and abutment and their design. It covers forces due to water current and collision on pier and abutment. Examples of design of pier and abutment are discussed in this book. It also covers various types of foundation- pile foundation, well foundation and shallow foundation and their design.

Raina V. K., "Concrete bridge handbook", Galgotia Publication Pvt. Ltd.
This book covers the basic principle of bridge structural analysis and design, forces to be considered in the analysis for the design of a bridge substructure and foundation. It covers analysis and design of slender exposed piles in a group. It also deals with evaluation of base pressures and contact area under foundations subjected to direct load and any axis bending. It also covers design procedure for short-cantilever portion of pier cap and pile cap.

IRC-6-2000, "Load and Stresses", Standard specifications and code of practice for road bridges, The Indian Road Congress.
This code deals with different Loads, stresses and forces acting on bridge substructure and foundation. It explains how to calculate various forces acting on bridge substructures. All substructures unit and foundation are analyzed and designed using this codal provisions.

IRC-78-2000, "Foundation and substructure", Standard specifications and code of practice for road bridges, The Indian Road Congress.
This code deals with the design foundations and substructures of road bridges. It covers design consideration for different types of piers, abutments, dirt wall, pier and abutment caps, pile foundation, well foundation etc. It also covers reinforcement requirements of components of substructure and foundation like pier, pier cap, pile and pile cap. It includes load carrying capacity determinations of individual pile in soil and rock.

Ministry Of Surface Transport (MOST) Classification

Ministry of surface transport developed the classification for the bridge superstructure. In this classification the standard sections of bridge superstructure's components like parapet, kerb, footpath, central verge, deck slab, longitudinal and transverse girders, diapharms are covered. The detailed standard sections of bridge superstructure for span ranging from 15m to 40 are covered in MOST classification.

Mark E. Williams, Marc I. Hoit, "Bridge pier live load analysis using neural networks", Science-direct Journal, Advance in Engineering Software, Vol. 35-2004

The pier structure is modeled using either linear elastic beam elements or non-linear discrete elements. The positioning of the vehicular live loads on the bridge follows the guidelines of the AASHTO-LRFD bridge design code. For highway bridges, the worst load positioning for the superstructure design usually does not produce the worst force effects for the pier design. This paper investigates to predict the worst load positioning for the bridge pier. Four unique force effects pier design that must be determined namely maximum force combination in a pile, maximum force combination in the pier column, maximum shear force in pier cap, and maximum bending moment in pier cap.

Kyle M. Rollins, Robert J. Clayton, Rodney C. Mikesell, and Bradford C. Blaise, "Drilled Shaft Side Friction in Gravelly Soils" Journal of Structural Engineering, Vol. 131, No. 8, August 1, 2005

To evaluate side friction, 28 axial tension uplift load tests were performed on drilled shafts in soil profiles ranging from uniform medium sand through well-graded sandy gravel. Typical load–displacement curves for skin friction in gravelly soils were developed. Measured load capacities were compared with capacities from rational theory equations. Reasonable agreement between measured and computed capacities was generally found for sandy profiles. However, measured capacities were typically two to four times higher than predicted at sites where the gravel fraction was over 50%. Additional load test data for gravelly soils were collected and combined with the data from Utah load tests. Based on this data set, modifications to the design equations were then developed to predict ultimate side friction capacity better while still maintaining a margin of safety.

Al-Homoud A.S.,Whitman R. V.," Seismic analysis and design of rigid bridge abutments considering rotation and sliding incorporating non-linear soil behavior", Science-direct Journal, Computers and Structures, Vol. 18-1999.

This paper is useful for abutment design. A two-dimensional (2D) finite element analytical model is developed to analyze the seismic response of rigid highway bridge abutments and founded on dry sand. The soil is modeled by a 2D finite element grid and the bridge abutment is modeled as a rigid substructure. The result from paper shows that the maximum dynamic earth force is acting higher than 0.6H above the base.

3. SUPERSTRUCTURE

3.1 Introduction:

The main forms of bridge superstructure are Arches, Slabs, Girder in form of beams, Trusses, Suspension bridges and the latest being cable-stayed bridges. Arches can be of masonry, cast iron or concrete. Slabs can be of stone or made up of reinforced concrete. The girder/beam as well as the truss can be made up of either timber, steel or concrete, or can be made up of combination of steel and concrete. Suspension bridges are made up of high tensile steel cables strung in form of catenary to which the deck is attached by steel suspenders, which are mainly made up of steel rods/members/cables. The decking can be of timber, concrete or steel spanning across the stiffening girders transmitting load to suspenders. Cable-stayed bridges are similar to the suspension bridges excepting that there will be no suspenders in the cable-stayed bridges and a number of cables stretched from support towers directly connect the decking.

3.2 Components of superstructure:

The Figure No. 3.1 shows different components of superstructure.

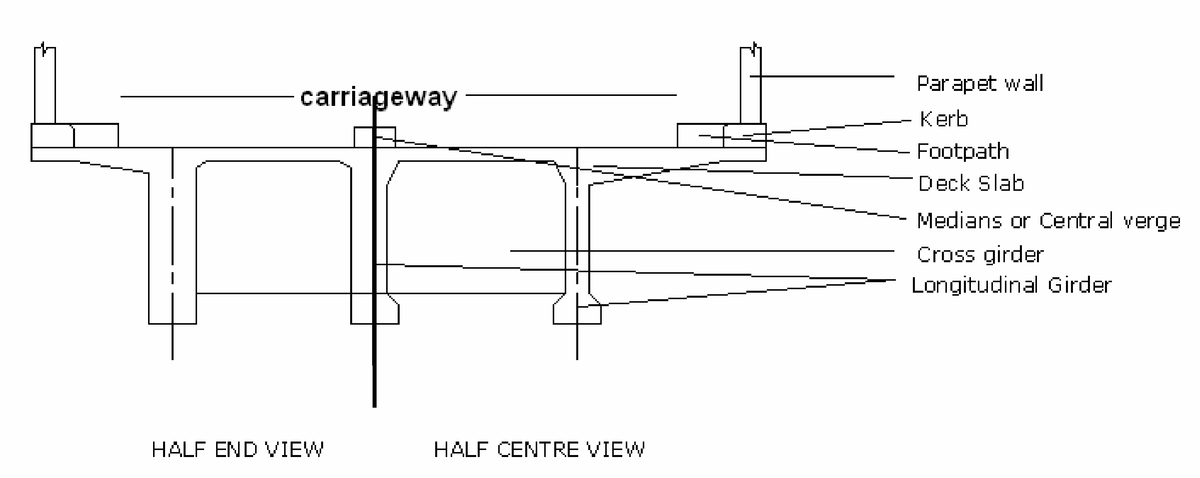


FIGURE 3.1 COMPONENTS OF SUPERSTRUCTURE

Carriageway:

The minimum clear widths of carriageway to be adopted for various types of traffic are as below:

- Single Lane Bridge – 4.25m
- Two lane Bridge – 7.5m
- Multilane Bridge – 7.5m plus 3.5m for every additional lane over two lanes.

Medians OR Central Verge:

In case of bridges with four or more number of lanes, it would be desirable to have a raised median along the centre line of bridge. The median serves to physically separate the traffic from opposing directions and improve road safety. For economic reasons, the width of the median may be kept low, but should not be less than 1.2m

Kerbs:

It is desirable to provide a kerb of 600 x 225mm on either side of the roadway on the bridge. The roadside edge of the kerb will have a slope of 1 in 8 for 200mm height and a curved edge with a radius of 25mm at top.

Footpath:

The width of footpath is minimum of 1.5m on bridges in rural areas and it is increased suitably in urban areas. The capacity of the side walk is taken as 108 persons per minute. The width should be increased in steps of 0.6m for every additional capacity of 54 persons per minute.

Parapets or Railings:

Railing should be for a height of 1.1m less one half of the horizontal width of the top rail. The space between the bottom rail and the top rail should be filled by means of closely spaced horizontal or inclined members.

Girders and Slabs:

Concrete bridge decks which are transversely connected only by situ slabs are usually referred to as Girder and Deck bridge. These bridges are by far most commonly adopted type in the span range of 10 to 50m. The majority of girders and slab decks have number of girders spanning longitudinally between pier to pier or pier to abutment with a thin slab spanning transversely across the top. The longitudinally spanning girder is known as longitudinal girder and transversely spanning girder is known as transverse girder or cross girder. Usually I-section or T-section is used for the girder. T-section girders are one of the most common example under this category and very popular because of their simple geometry, low fabrication cost, easy erection or casting and smaller dead loads. T-section girder bridges with cross beams extending into and cast monolithically with deck slab is found to be more efficient and is recommended for adoption. Simply supported R.C. T-section girder is normally adopted for spans up to 25m. Span-depth ratio is generally kept as 10 for simple spans and 12 to 15 for continuous span. In some cases, single cell or multicell box girder bridges are also preferred. For more than 25m span I-section girder is generally provided. One typical section and plan of a 40m span bridge superstructure as given by MOST (Ministry Of Surface Transport) is shown in Figure 3.2

3.3 Superstructure Loads

3.3.1 Dead Load:

The dead load of superstructure consists of weight of rail, footpath, kerb, deck slab, girder etc. These dead loads of superstructure are then transfered on pier and abutment. The MOST (Ministry Of Surface Transport) gives these superstructures dead load for span ranging from 16m to 40m as given in table 3.1. These superstructure dead loads are for various standard sections of girder, deck slab, rail and kerb depending on span and type of bridge.

TABLE 3.1 DEAD LOAD ON PIER AND ABUTMENT (AS PER MOST)

SPAN (m)	Dead load of superstructure on pier (kN)	Dead load of superstructure on Abutment (kN)
16 – R.C.C.	2849	1425
18– R.C.C.	3324	1662
21– R.C.C.	4274	2137
24– R.C.C.	5108	2554
30 - PRESTRESS	6040	3020
35- PRESTRESS	7460	3730
40- PRESTRESS	8960	4480

3.3.2 Live Load

As per IRC-6, Road bridges are designed for following live loads:

- **Class A Loading:**

  This loading is to be normally adopted on all roads on which permanent bridges are constructed.

- **Class B Loading:**

  This loading is to be normally adopted for temporary structures and for bridges in specified areas.

- **Class AA Loading**:

  This loading is to be adopted within certain municipal limits, in certain existing or contemplated industrial areas, in other specified areas, and along certain specified highways.

- **Class 70 R Loading:**

  This loading is to be adopted within certain municipal limits, in certain existing or contemplated industrial areas, in other specified areas, and along certain specified highways.

For each standard vehicle or train, all the axles of a unit of vehicles shall be considered as acting simultaneously in a position causing maximum stress.

IRC Class A & B Loading

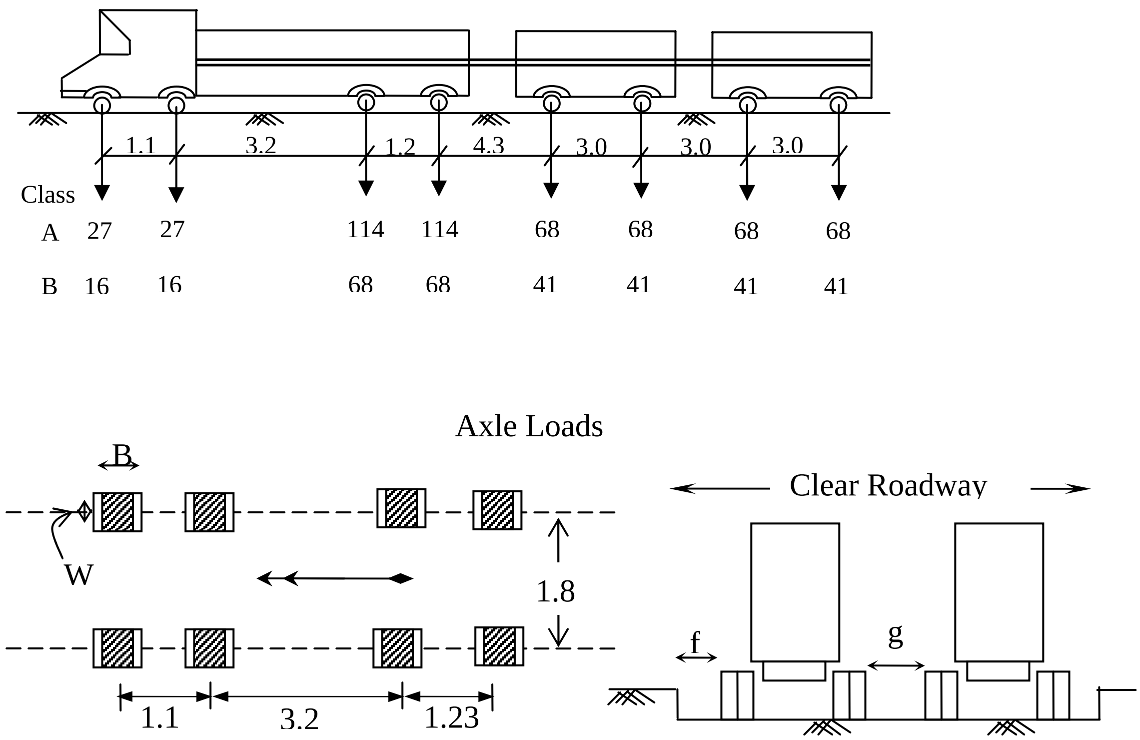


FIGURE 3.2 IRC CLASS A & B LOADING

- All values are in m and kN unless otherwise stated
- The nose to tail distance between successive trains shall not be less than 18.4m
- No other live load shall cover any part of the carriage way when a train of vehicles is crossing the bridge.
- The ground contact are of the wheels shall be as per table 3.2:

TABLE 3.2 GROUND CONTACT AREAS FOR CLASS A & B VEHICLE

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

- The minimum clearance, f ,between outer edge of the wheel and the roadway face of the kerb, and the minimum clearance , g , between the outer edges of passing or crossing vehicles on multi-lane bridges shall be as per table 3.3:

TABLE 3.3 CLEARANCES FOR CLASS A & B VEHICLE

Class A vehicle			**Class B vehicle**		
Carriageway width	g	f	Carriageway width	g	f
5.5m to 7.5m	0.4-1.2m	150mm	5.5m to 7.5m	0.4-1.2m	150mm
Above 7.5m	1.2m	150mm	Above 7.5m	1.2m	150mm

IRC Class AA Loading:

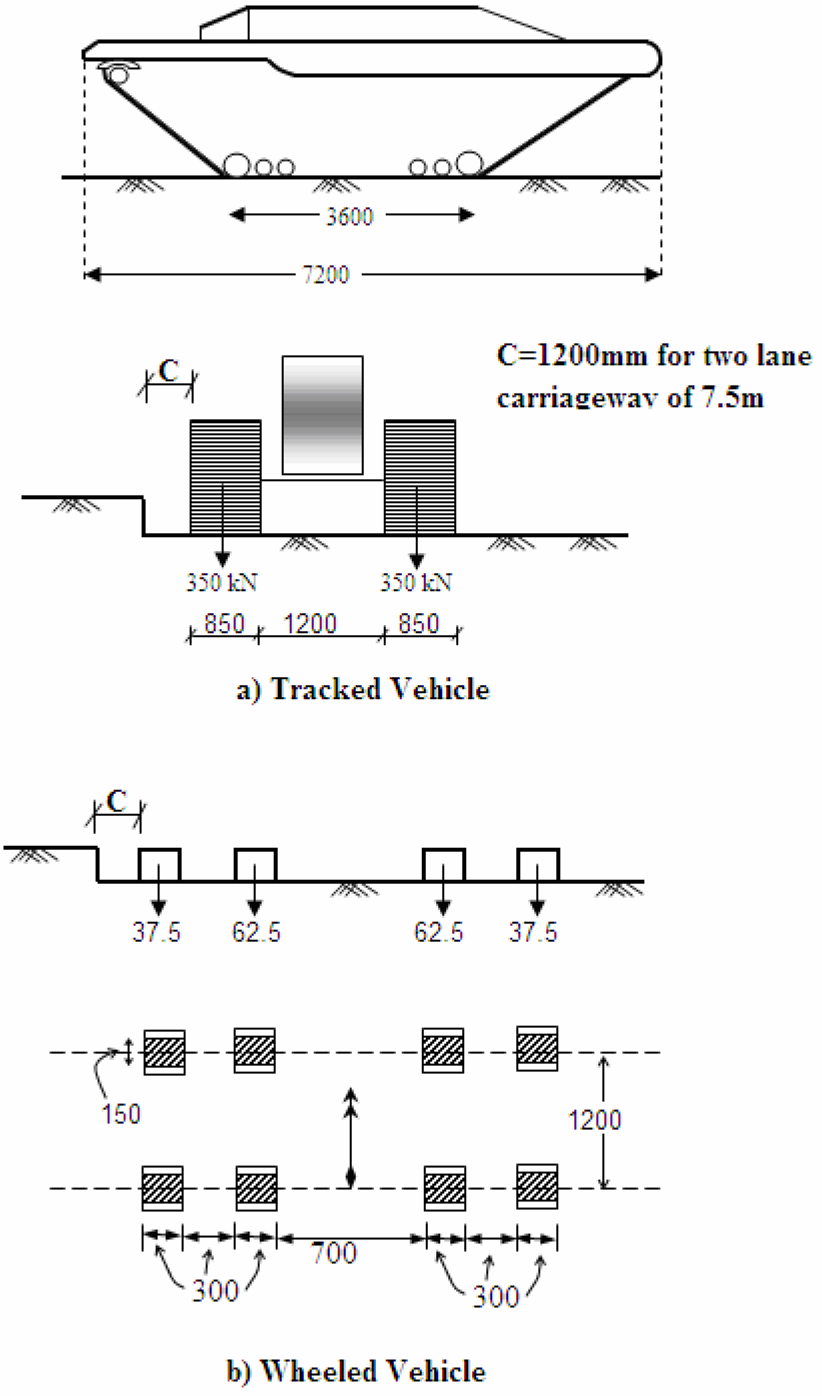


FIGURE 3.3 IRC CLASS AA LOADING

- All dimensions are in mm unless otherwise stated
- The nose to tail spacing between two successive vehicles shall not be less than 90m
- For multi-lane bridges, one train of class AA tracked or wheeled vehicles which ever creates severe conditions shall be considered for every two traffic

lane width. No other live loads shall be considered on any part of the two lanes wide carriage way of the bridge where above mentioned train of vehicle is crossing the bridge.

- The minimum clearance between the road face of the kerb and the outer edge of the wheel or track , C, shall be as under:

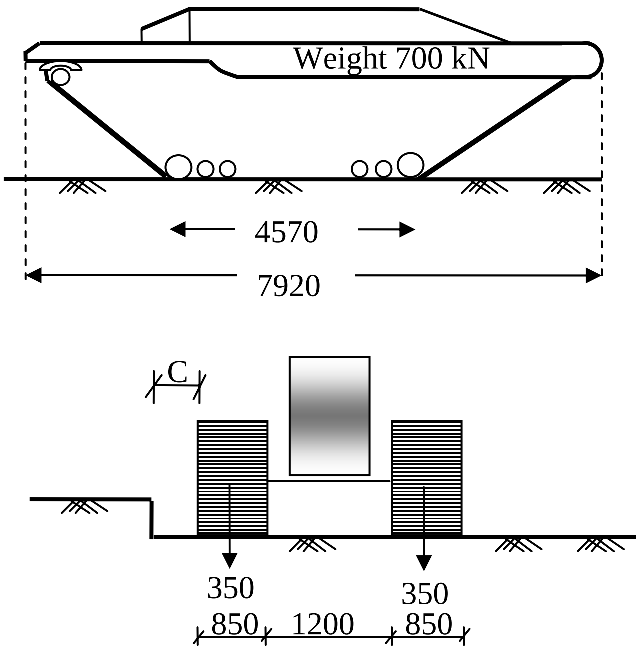
  Single lane bridge 0.3 m

  Multi Lane Bridge

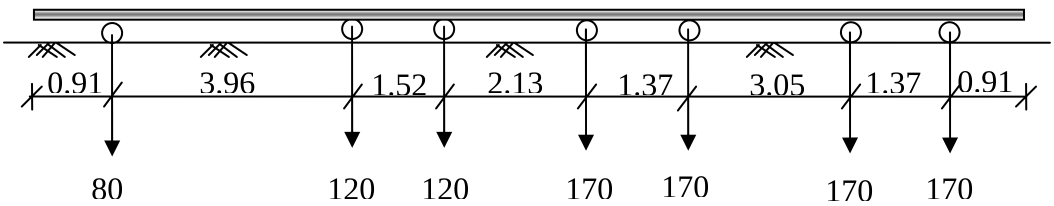
  Carriageway less than 5.5 m 0.6 m

  Carriageway 5.5m or above 1.2 m

IRC Class 70 R Loading



a) Tracked Vehicle



b) Wheel Vehicle

FIGURE 3.4 IRC CLASS 70-R LOADING

- All values are in m and kN unless otherwise stated
- The nose to tail spacing between two successive vehicles shall not be less than 90m
- For multi-lane bridges, one train of class AA tracked or wheeled vehicles which ever creates severe conditions shall be considered for every two traffic lane width. No other live loads shall be considered on any part of the two lanes wide carriage way of the bridge where above mentioned train of vehicle is crossing the bridge.
- The minimum clearance between the road face of the kerb and the outer edge of the wheel or track , C, shall be as under:

  Single lane bridge 0.3 m

  Multi Lane Bridge

  Carriageway less than 5.5 m 0.6 m

  Carriageway 5.5m or above 1.2 m

3.4 Combinations of Live Load as per IRC:

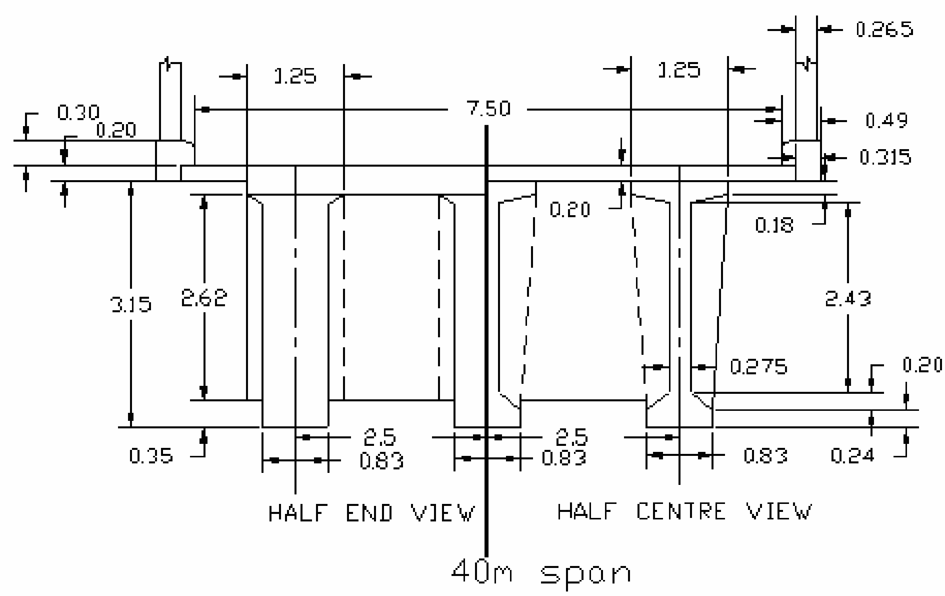
The carriageway live load combination shall be considered as per Table 3.4

TABLE 3.4 COMBINATION OF LIVE LOAD

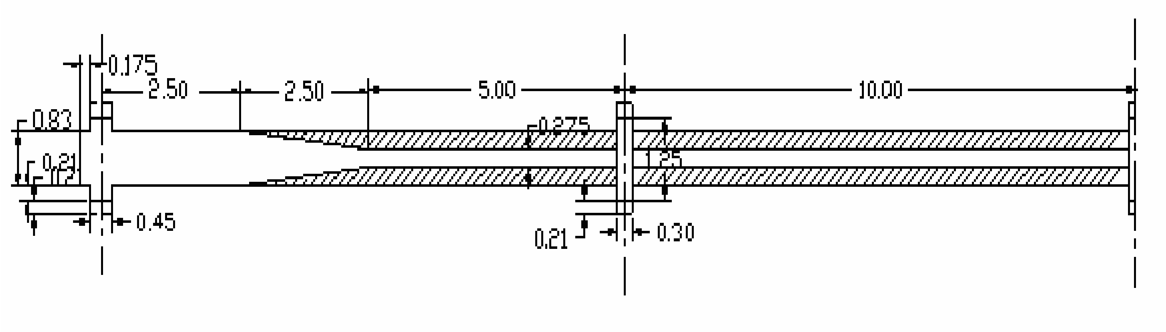
<table>
<tr><th>Sr. No</th><th>Carriageway width</th><th>Number of Lanes For design purpose</th><th>Load combination</th></tr>
<tr><td>1.</td><td>Less than 5.3m</td><td>1</td><td>One lane of Class A.</td></tr>
<tr><td>2.</td><td>5.3m to 9.6m</td><td>2</td><td>One lane of Class 70 R or Two lanes of Class A.</td></tr>
<tr><td>3.</td><td>9.6m to 13.1m</td><td>3</td><td>One lane of Class 70 R with One lane of Class A or Three lanes of Class A.</td></tr>
<tr><td>4.</td><td>13.1m to 16.6m</td><td>4</td><td rowspan="3">One lane of Class 70 R for every two lanes with one lane of Class A for the remaining lanes, or One lane of Class A for each lane.</td></tr>
<tr><td>5.</td><td>16.6m to 20.1m</td><td>5</td></tr>
<tr><td>6.</td><td>20.1m to 23.6m</td><td>6</td></tr>
</table>

3.5 Superstructure details for further substructure analysis and design

For the analysis and design of bridge substructure components and for foundation design the span of superstructure considered is 40m. For 40m span of superstructure dead load of superstructure is 8960 kN including all the components of superstructure like rail, kerb, deck slab and girders. Area of superstructure seen in elevation along flow direction due to deck , handrails and girder is 146 m^2. One typical section and plan showing superstructure details for a span of 40m (As per MOST) is given in fig 3.5. For the case of live load 70-R Wheeled vehicle is considered for further analysis and design of bridge substructure and foundation.



Typical sectional view



Sectional Plan

All dimensions are in meter

FIGURE 3.5 TYPICAL SECTIONS AND PLAN OF SUPERSTRUCTURE OF SPAN 40m (AS PER MOST)

4. PIER AND PIER CAP

4.1 Pier

4.1.1 Introduction

The main function of the pier is to transfer the vertical loads to the foundation and to resist all horizontal forces and transverse forces acting on the bridge. Pier bodies can be constructed in plain concrete, reinforced concrete or in masonry (brick, stone or block), and in some special cases in timber, steel or prestressed concrete.

In general, the shapes of pier should be such as to cause minimum obstruction to the flow of water. Piers shall be designed to be safe under the worst combination of loads and forces during construction and service conditions.

4.1.2 Type, Shape and Size of pier

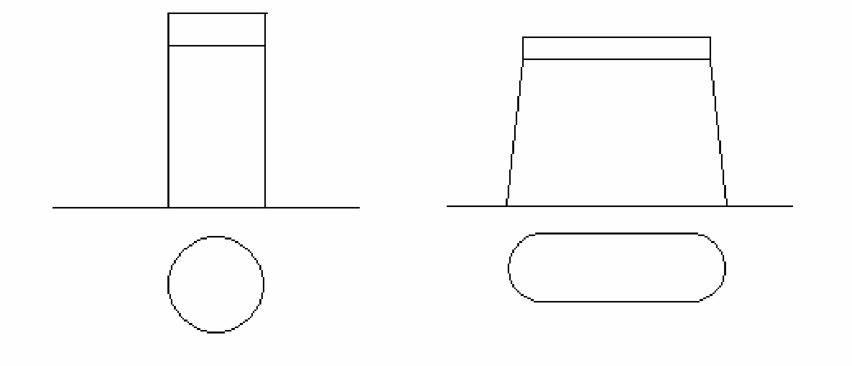
Selection of type of piers depends upon the site conditions, soil conditions and hydraulic data. Type of pier is also affected by type of superstructure. While constructing road over bridges or flyovers especially in the city areas and shopping areas, the aim should be to avoid too many pillars in the middle so that the driver can have a clear unobstructed view. In a similar manner, when providing a bridge on river, it will be advisable to have slender and tall piers so that the view of the scenery is not obstructed.

Size of pier depends on total width of superstructure and High flood level of river. Width of superstructure gives dimensions in flow direction, while HFL gives height of pier. Shape of pier also depends upon flow velocity.

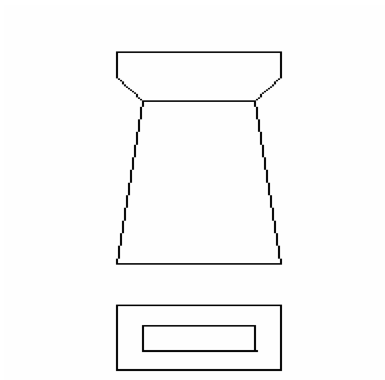
The top width of the pier depends on the size of the bearing plates on which the superstructure rests. The length of the pier at the top should not be less than 1.2m in excess of the out to out dimension of the bearing plates measured perpendicular to the axis of the superstructure.

Different shapes of piers are shown in figure

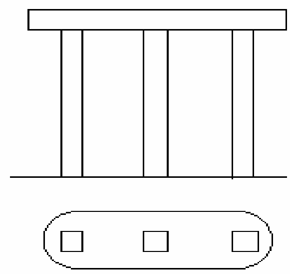
1. Solid masonry -concrete pier shaft and Solid circular pier



2. Hammerhead type pier



3. Trestle type pier



4 Framed Piers

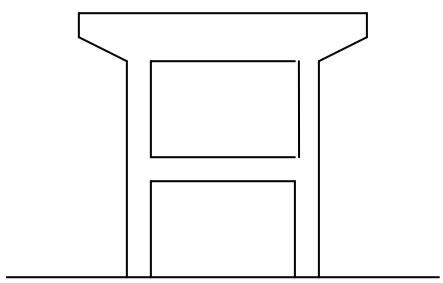


FIGURE 4.1 DIFFERENT TYPES OF PIER

4.2 Forces on Pier

The forces acting on a pier are as follows:

- Dead load of superstructure and pier itself.
- Live load
- Impact effect
- Buoyancy force
- Effect of wind on moving load and on the superstructure
- Forces due to water current
- Longitudinal forces due to braking effect of vehicles
- Seismic force.
- Centrifugal force
- Temperature force
- Earth pressure

➢ **Dead Load :**

Dead load on pier depends upon the weight of superstructure and bearings. Weight of superstructure can be found out by multiplying volume of its components with its unit weight. Self weight of pier is also included in dead load.

➢ **Live load:**

Live load can be calculated as explained in IRC-6 Cl. no.207. For worst condition, for two lanes, one lane of Class 70R or two lanes of Class A, and for four lanes, one lane of Class 70R for every two lanes with one lane of Class A for the remaining lanes should be considered.

➢ **Impact effect:**

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

- For Class A or Class B loading

Impact factor fraction for R.C.C. bridge = {4.5/ (6+L)}

Where L is length of span in meters.

- For Class AA loading and Class 70R loading
    a. For span less than 9m:
        i. For tracked vehicles: 25 percent for spans up to 5m linearly reducing to 10 percent for span 9m.
        ii. For wheeled vehicles: 25 percent.
    b. For span of 9m or more:
        i. For tracked vehicles: 10 percent up to a span of 40m and in accordance with the curve in Fig. 5 of IRC-6 for spans in excess of 40m.
        ii. For wheeled vehicles: 25 percent for spans up to 12m and in accordance with the curve in Fig. 5 of IRC -6 for spans in excess of 12m.

➢ **Buoyancy force:**

In the design of submerged masonry or concrete structures the buoyancy effect may be limited to 15 percent of fully buoyancy.

➢ **Wind force:**

Wind force shall be considered to act horizontally and in such a direction that the resultant stresses in the member under consideration are the maximum. The intensity of the wind force shall be based on wind pressures and wind velocities shown in table 4.1 and shall be allowed for design. The pressure given therein shall however, be doubled for bridges situated in areas such as the kathiawar and Bengal and orissa coasts as per Fig. 6 of IRC-6.

TABLE 4.1 WIND PRESSURES AND WIND VELOCITIES

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

Where H= the average height in meter of the exposed surface
V= horizontal velocity of wind in kilometer per hour at height H
P= horizontal wind pressure in kg/m^2 at height H.

- **Water current force:**

Any part of a road bridge which is submerged in running water shall be designed to sustain safety the horizontal pressure due to force of the water current. On piers parallel to the direction of the water current, the intensity of pressure shall be calculated from the following equation

$$P = 52\ K\ V^2$$

Where P = Intensity of pressure due to water current in kg/m^2
V = The velocity of the current at the point where the pressure intensity is being calculated in meter per second.
K = A constant having the different values for different shapes of piers mentioned in IRC-6 cl. No. 213.Which is given below

(A)	Square Ended Piers	1.5
(B)	Circular piers of piers with semi-circular ends	0.66
(C)	Piers with triangular cut and ease waters, the angle included between the faces being 30° or less	0.5
(D)	Piers with triangular cut and ease waters, the angle included between the faces being more than 30° but less than 60°	0.5-0.7
(E)	Piers with triangular cut and ease waters, the angle included between the faces being more than 60° but less than 90°	0.7-0.9

- **Longitudinal force due to braking effect:**

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

(a) Single lane or two lane bridge:

20 percent of the first train load plus 10 percent of the load of the succeeding trains or part thereof, the train loads in one lane only being considered for the purposes. Where the entire first train is not on the full span, the braking force shall be taken as equal to 20 percent of the loads actually on the span

(b) More than two lane:

As in (a) above for the first two lanes plus 5 percent of the loads on the lanes in excess of two.

- **Seismic force:**

Bridges in seismic zones II and III need not be designed for seismic force provided both conditions are met, first one is span is less than 15m and second is total bridge length is less than 60m. All ether bridge shall be designed for seismic force. For the purpose of determining the seismic force, the country is classified into four zones. The horizontal seismic forces to be resisted shall be computed as follows.

$$F_{eq} = A_h * (\text{Dead load}).$$

Where F_{eq} = Seismic force to be resisted.

A_h = horizontal seismic coefficient

$$= \left(\frac{Z * I * Sa}{2 * R * g} \right)$$

Z = zone factor as given in table 5 of IRC-6 -2000.

I = Importance factor, for important bridges…..1.5

For other bridges ………1.0

T = Fundamental period of the bridge member for Horizontal vibration

R = response reduction factor

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

- **Centrifugal force:**

Where a road bridges is situated on a curve, all portions of the structure affected by centrifugal action of moving vehicles are to be proportioned to carry safely the stress induced by this action in addition to all other stress to which they may subjected. The centrifugal force shall be determined from the following equation

$$C = WV^2 / 127 R$$

Where C= centrifugal force

W= live load

V= the design speed of the vehicles using the bridge in km/Hour

R= the radius of curvature.

- **Earth pressure:**

  Structures designed to retain earth fills shall be proportioned to withstand pressure calculated in accordance with any rational theory. Coulomb's theory shall be accepted subjected to modification that the centre of pressure exerted by the backfill, when considered dry, is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of that height. All abutments and return wall and sometimes piers shall be designed for a live load surcharge equivalent to 1.2m earth fill.

4.3 Various Load Combinations

The various load combinations for analysis and design of pier as per IRC-6 are given in following table no. 4.2.

TABLE 4.2 VARIOUS LOAD COMBINATIONS FOR PIER DESIGN

No.	Dead Load	Live load	Impact force	Impact Floating bodies	Wind force	Water current force	Braking force	Bearing Rigidity force	Centrifugal Force	Buoyancy force	Seismic force	Due to Temperature	Secondary Effects	Remarks
1	1	1	1	-	-	1	1	1	1	1	-	-	-	Service Conditions
2	1	1	1	-	-	1	0.5	0.5	0.5	1	-	1	1	
3	1	0.5	0.5	-	-	1	0.5	0.5	0.5	1	-	1	1	
4	1	1	1	-	1	1	1	1	1	1	-	1	1	
5	1	1	1	1	1	1	1	1	1	1	-	1	1	
6	1	0.5	0.5	-	-	1	0.5	0.5	0.5	1	1	1	1	
7	1	-	-	-	1	1	1	-	-	1	-	-	-	Construction Condition
8	1	-	-	-	-	1	1	-	-	1	0.5	-	-	

4.4 Steps for Pier design

To design a pier the following steps should be followed:

Select type of pier 1

↓

Select shape and size of pier 2

↓

Select dimension of pier cap 3

↓

Calculate loads and moments on pier 4

↓

Consider Various Loads and moments combinations under service and construction condition and select combination that govern the design 5

↓

Reinforcement Design using governing moment values 6

↓

Check for Stresses at the base of pier 7

4.5 Check for Stresses at the base of Pier

For the design purpose, the section passing through the base of the pier is most critical. So the check at the base should be carried out. For that determine area of cross-section of the pier base (A), its moment of inertias about x-x axis and y-y axis (I_{xx} and I_{yy}).

Then Calculate

$$f_{1,2} = \frac{V}{A} \pm \frac{M_{xx}}{I_{xx}} * y \pm \frac{M_{yy}}{I_{yy}} * x$$

Where $f_{1,2}$ = Stresses in tension or in compression depending upon sign conventions

V = Total Vertical Load at the base of pier

M_{xx} = Moment in x-x direction at the base of pier

M_{yy} = Moment in y-y direction at the base of pier

x= Eccentricity in x-direction

y= Eccentricity in y-direction

For safety, there should be no tension stresses at the base of a pier.

4.6 IRC Code Provisions for pier

The important IRC codes provisions for design of pier are as follows:

- IRC-78-2000, "Foundation and substructure", Standard specifications and code of practice for road bridges, The Indian Road Congress.
- IRC-6-2000, "Load and Stresses", Standard specifications and code of practice for road bridges, The Indian Road Congress.

➢ IRC-78-2000, "Foundation and substructure" includes general features and design consideration of foundation and substructure. The main important clause of IRC-78 is described below:

- In case of hollow concrete piers the thickness of the wall shall not be less than 300mm.
- In case of bridge having multi column piers across rivers carrying floating trees or timber it will be necessary to brace such structure by means of diaphragm walls which shall not be less than 150mm.
- Cut and case waters where provided shall extend up to affluxed HFL or higher, if necessary, from consideration of local conditions like waves, etc.
- When supports are made with two or more columns spaced closer than three times the width of columns or 2 meters whichever is less, across the direction of flow, the group shall be treated as a solid pier of the same overall width

and value of K taken as 1.25 for working out the intensity of pressure according to relevant clause of IRC-6.

- In the design of submerged masonry or concrete structure buoyancy effect through pore pressure shall be limited to 15 per cent of full buoyancy.
- The lateral reinforcement of the walls of hollow RCC pier shall not be less than 0.3 per cent of the sectional area of the wall of the pier.

IRC-6-2000, "Load and Stresses" includes Loads and Stresses on bridge. It includes various loads which can be applied on pier which are described earlier in chapter 4, section 4.2.

4.7 Pier cap

4.7.1 General

Pier cap should be suitably designed and reinforced to take care of concentrated point loads dispersing on pier. Caps cantilevering out from the supports or resting on two or more columns shall be designed to cater for lifting of superstructure on jacks for repair/replacement of bearings. The locations of jacks shall be predetermined and permanently marked on the caps.

4.7.2 Analysis and Design of pier cap

In case when bearings are placed centrally over the columns, the load from bearings will be considered to have been directly transferred to columns and the cap beam need not be designed for flexure, only minimum reinforcement for slab should provide in both ways as well as in both layer top and bottom. When the distance between the load/center lines of bearing from the face of the support is equal to or less than the depth of the cap (measured at the support) the cap shall be designed as corbel. Where a part of the bearing lies directly over the pier, calculation of such reinforcement should be restricted only for the portion which is outside the face of the pier. Moreover, in such cases the area of closed horizontal stirrups may be limited to 25 percent of the area of primary reinforcement.

4.8 Analysis and design of Pier Cap

Pier cap should be suitably designed and reinforced to take care of load coming from superstructure and dispersing on pier. The size of pier cap depends on bearing positions and top section area of pier. Along flow direction numbers of bearings provided are three, which are placed 2.5m c/c and the width of pier along flow direction is 9.3m. The projections 1.1m are given in both sides of bearings.

Width of pier cap along flow direction = width of pier + Projection

= 9.3 + 2 (1.1) = 11.5 m

Along traffic direction numbers of bearings are two, which are placed 1.6m c/c and the width of pier along traffic direction is 1.8 m. The projections 0.6 m are given in both sides of bearings.

Width of pier cap along traffic direction = width of pier + Projection

= 1.8 + 2 (0.6) = 3.0 m

The depth of pier cap is assumed as 1.2 m which is checked out as per IS 456-2000 and it is satisfying the depth requirements

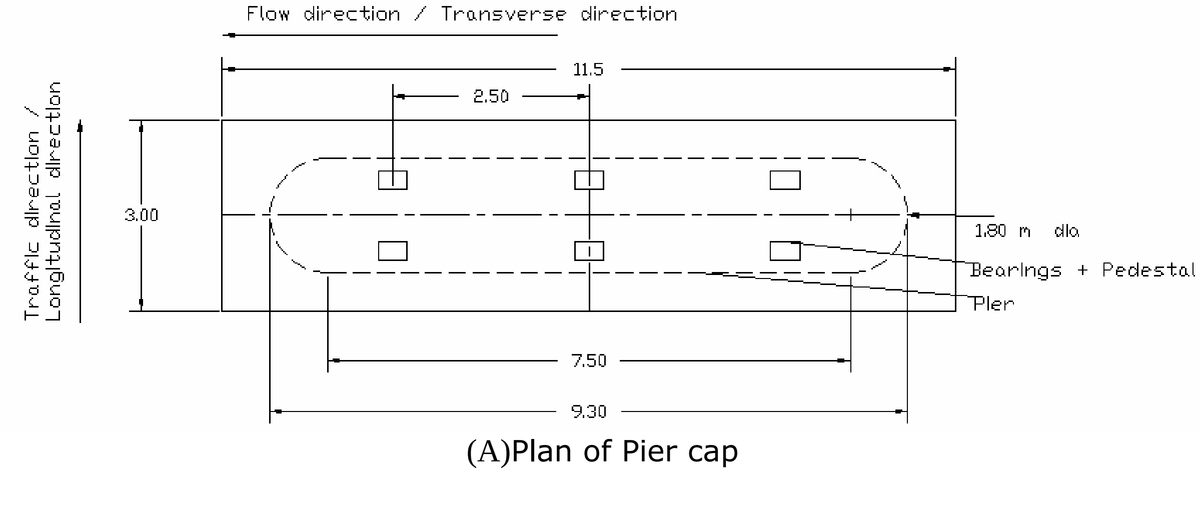
Data for pier cap analysis and design

The plan and section of pier cap is shown in fig no. 4.2

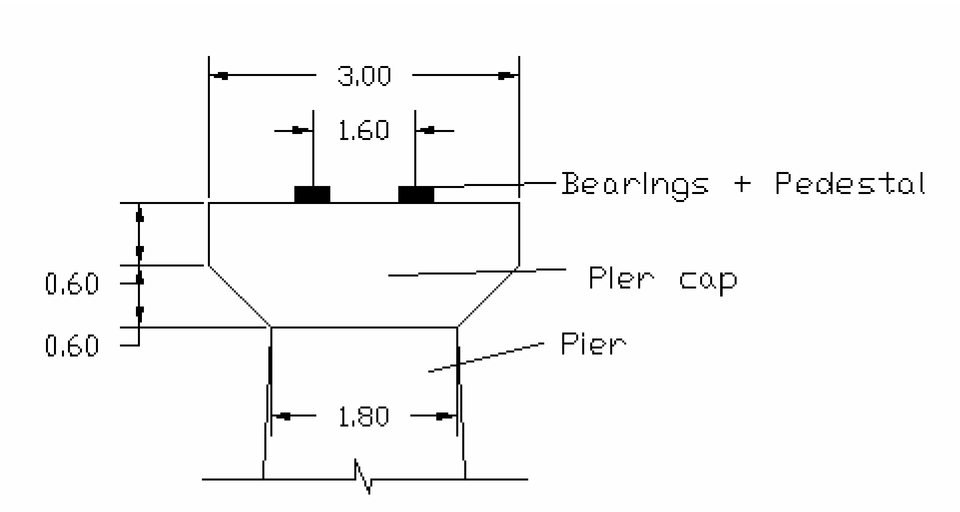
Pier cap Size = 11.5 x 3 x 1.2 m

Grade of concrete = M 25

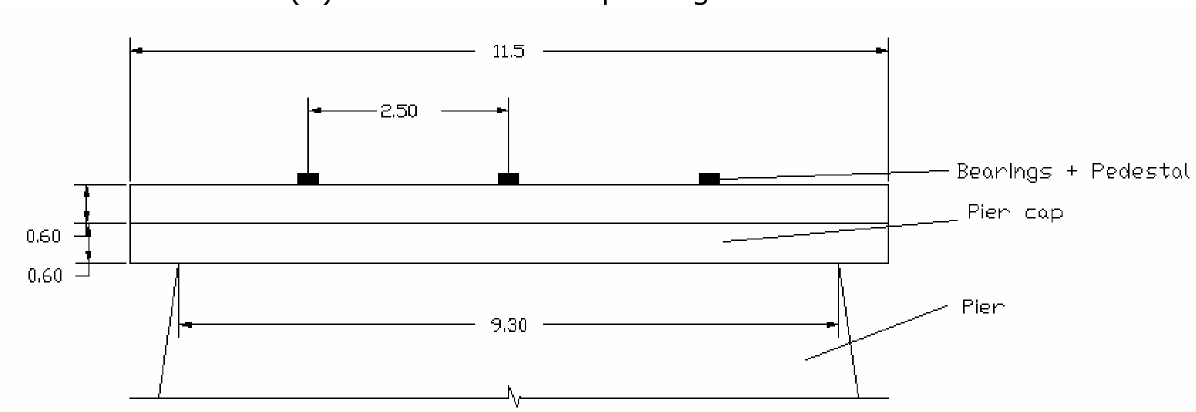
Grade of steel = Fe 415



(A)Plan of Pier cap



(B) Section of Pier cap along flow direction



(C) Section of Pier cap along traffic direction

All dimensions are in meter

FIGURE 4.2 PLAN AND SECTIONS OF PIER CAP

As per IRC-78, 2000 Cl. No. 710.8.4,

"In case bearings are placed centrally over the pier, the load from bearings will be considered to have been directly transferred to piers and the cap beam need not be designed for flexure and minimum steel area as per IRC-21,2000 Cl. No. 305.19 (0.12% should provided in both layer as well as in both ways)"

Adopting 0.12 % steel area (As per IRC-21, 2000, Cl.No. 305.19) for the pier cap

Depth required

Modification factor = 2.0 (As per IS-456, 2000, fig -4)

Span to effective depth ratio = 20 (As per IS-456, 2000, Cl. No. 23.2.1)

Span to effective depth ratio permissible = 2.0x 20 = 40

Depth required (d_{req}) = Width of pier cap along flow direction / Span to effective depth ratio permissible

= 11.5 * 1000 / 40

= 287.5 mm < 1200 mm (Depth Assumed) …..O.K

Provide 0.12 % steel along flow direction

Steel Area required A_{st} = 0.12 x 1200 x 1000 / 100

= 1440 mm^2

Provide 16mm dia bars @ 130mm c/c

Ast provided = 1547mm^2

Provide 0.12% steel along traffic direction

Steel Area required A_{st} = 0.12 x 1200 x 1000 / 100

= 1440 mm^2

Provide 16mm dia bars @ 130mm c/c

Ast provided = 1547 mm^2

The reinforcement details of pier cap are shown in fig.4.3

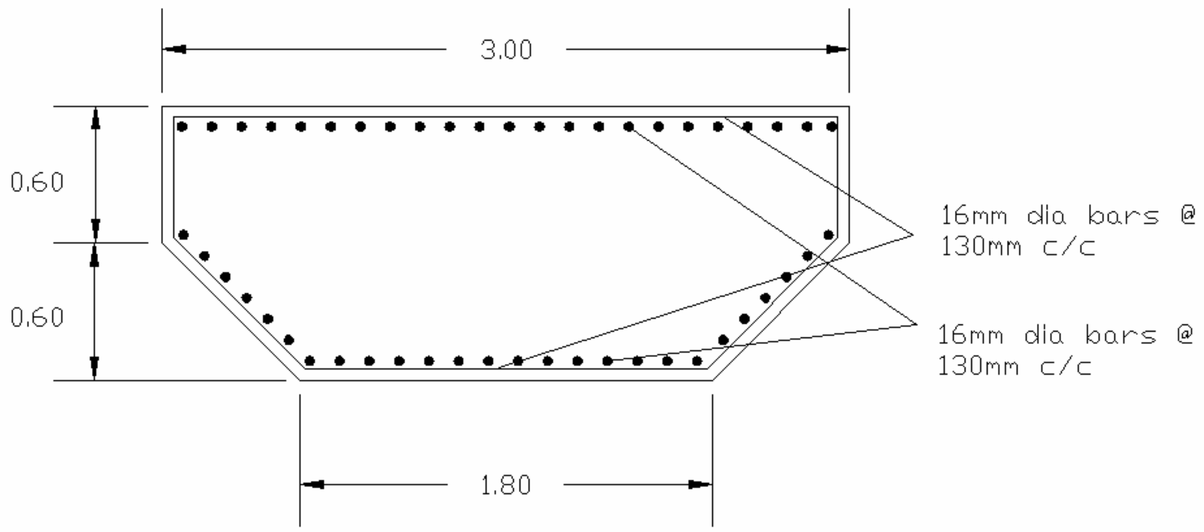


FIGURE 4.3 REINFORCEMENT DETAILS OF PIER CAP

4.9 Analysis and design of pier

For analysis and design, wall type pier section is selected. The top width of pier is 9.3 m with two semicircular ends of 1.8m diameter. The bottom width of pier is 10.2 m with two semicircular ends of 2.7m diameter. The height of pier is 9m and high flood level is at 8.1m above the base of pier.

Data

Superstructure = simply supported Prestressed I girder
Span = 40 m
Pier = Wall type pier
Foundation = Pile Foundation
Dimensions of pier are as shown in figure 4.4

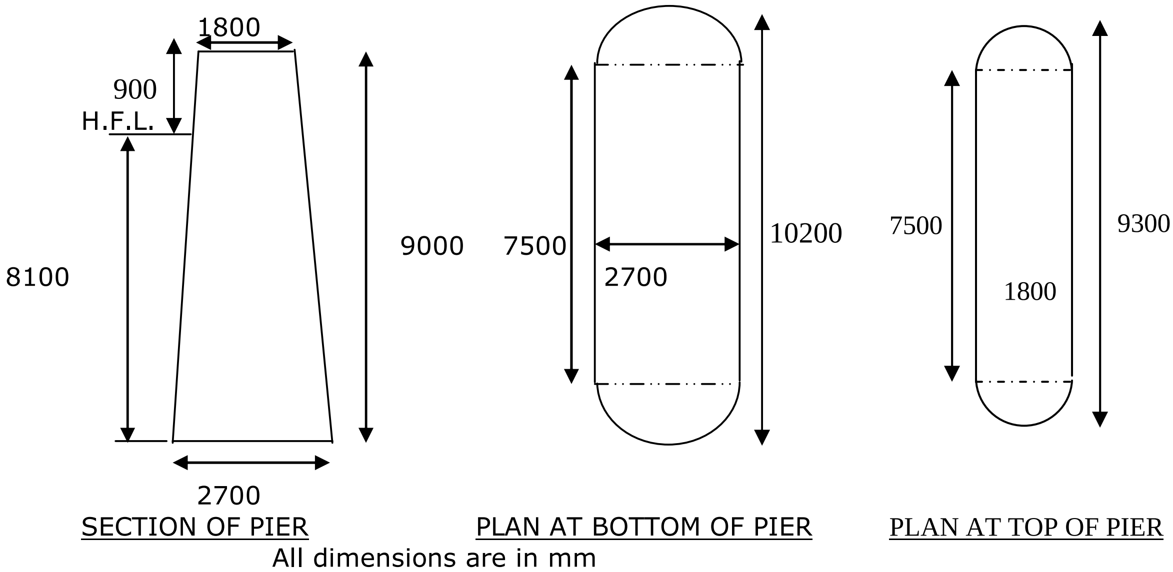


FIGURE 4.4 SECTION, TOP AND BOTTOM PLAN OF PIER

Dead load from each span = 4480 kN (As per MOST (40m span))
Maximum mean velocity of water = 3.6 m/sec
Material for pier = Cement concrete M 20 grade
Live load: IRC Class 70 R Wheeled vehicle
The size of pier cap = 11.5 x 3 x 1.2 m

4.9.1 Calculation of forces on pier

Dead Load

Dead Load from Superstructure = 2 x 4480

= 8960.00 kN

Volume of pier cap = 11.5 x 3 x 1.2

= 41.4 m^3

Dead Load from pier cap = Volume of pier cap * Density

= 41.4 x 25

= 1035.00 kN

Self weight of pier

Area at base = 7.5 x 2.7 + 3.14 x 2.7 x 2.7 / 4

= 25.97 m^2

Area at top = 7.5 x 1.8 + 3.14 x 1.8 x 1.8 / 4

= 16.04 m^2

Average Area = 21.01 m^2

Total Volume = Average Area * Height

= 21.01 x 9

= 189.07 m^3

Self weight = Volume of pier * Density

= 189.07 x 25

= 4726.81 kN

Total dead load = Load of superstructure +Load of Pier cap +Self weight of pier

= 8960 + 1035 + 4726.81

= 14721.81 kN

Dead Load for one span disloged condition

= One side span load of superstructure + Load of pier cap + Self weight of pier

= 4480 + 1035 + 4726.81

= 10241.81 kN

Longitudinal moment for one span disloged condition

= Dead Load of superstructure of one span * eccentricity between bearing and center line of pier (fig 4.5)

= 4480.00 x 0.8

= 3584.00 kN-m

Buoyancy Force

Height of H.F.L. from top of pier = 0.9 m

Width of pier at H.F.L. = 1.89 m

Submerged volume of pier (Height is from base to H.F.L)

$= [7.5\text{x} \{(1.89 + 2.7) / 2\} \text{ x } 8.1] + [3.14 \text{ x } \{2.7 + 1.89\}^2 \text{ x } 8.1/ 16]$

$= 172.91 \text{ m}^3$

Total submerged weight = Volume * Density

=172.91 x 25

=4149.88 kN

Buoyancy force is taken as 15% of submerged weight

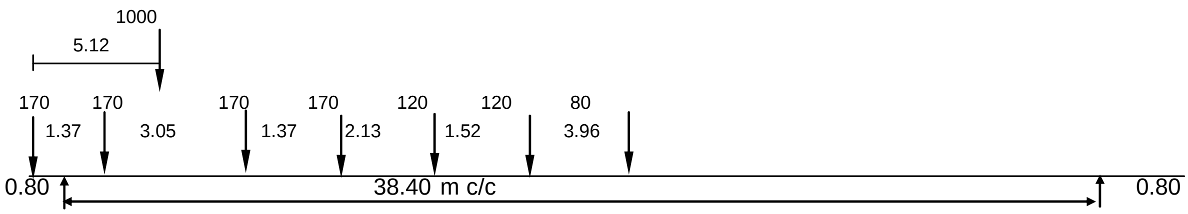
(As per IRC-6, 2000 Cl. No. 213.5)

Buoyancy Force = 0.15 x 4149.88

= 622.48 kN

Live Load

A) 70 R (W) (a) (One Span disloged condition)



All loads are in kN and all distances are in meter

$$\text{Reaction @ left} = \frac{1000 \text{ x } (38.40 + 0.8 - 5.12)}{38.40}$$

$$= \frac{1000 \text{ x } 33.88}{38.40} = 891.58 \text{ kN}$$

Total live load = 891.58 kN

Longitudinal moment=Live Load * eccentricity between bearing and center line of pier (Longitudinal eccentricity) (Refer fig. 4.5)

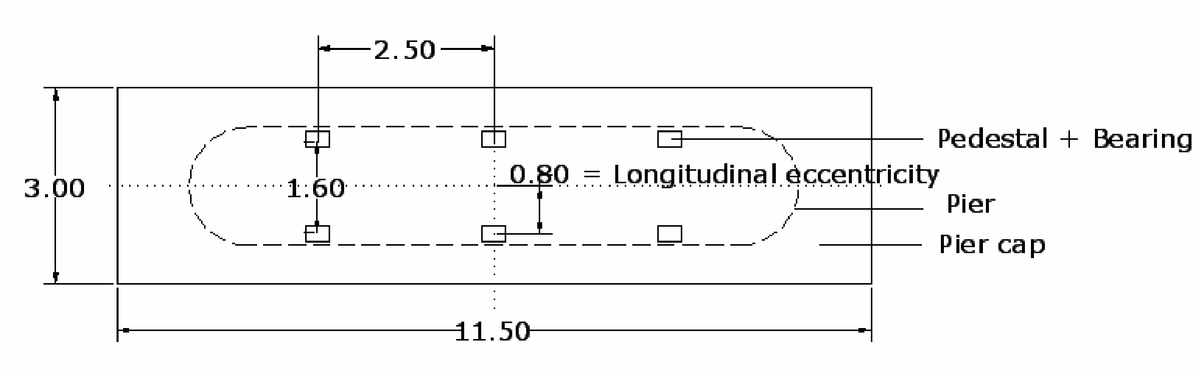


FIGURE 4.5 ECCENTRICCITY FOR LONGITUDINAL MOMENT

Longitudinal moment = 891.58 x 0.8

= 710.00 kNm

Maximum Transverse eccentricity (fig 4.6) = 8.50/2 - 0.50 - 1.20 - 2.79 / 2

= 1.155m

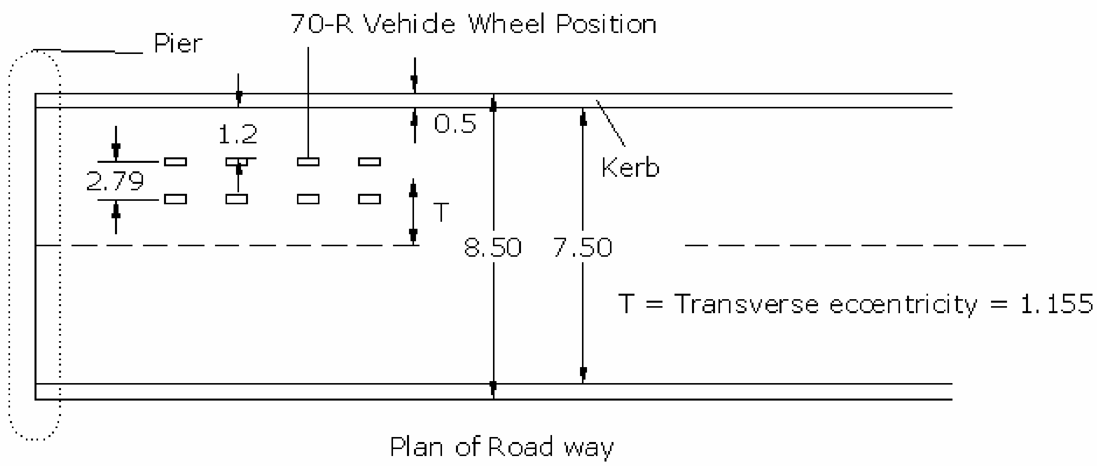


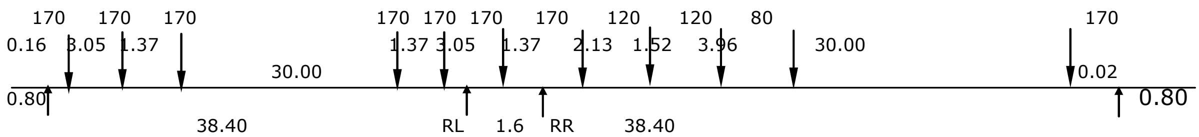
FIGURE 4.6 PLAN OF ROAD WAY FOR PIER

Transverse moment = Live Load * Transverse eccentricity (Refer fig. 4.6

= 891.58 x1.155

= 1029.77 kN-m

70 R (W) (b) (Both span loaded condition)



All Loads are in kN and distances are in m

RR = 536.04 kN

RL = 436.09 kN

Total reaction = RL + RR = 536.04 + 436.09 = 972.13 kN

Longitudinal moment = 536.04 - 436.09 + 170 x 0.8

= 270.00 kN-m

Maximum Transverse eccentricity (fig 4.6) = 8.50 / 2 - 0.50 - 1.20 - 2.79 / 2

= 1.155 m

Transverse Moment = Total reaction * transverse eccentricity

= 972.13 x 1.155

= 1122.81 kN-m

Impact Force

Impact Factor = 0.08 (As Per IRC - 6, 2000, Cl. No. 211.2)

Live load Reaction considering Impact = 0.08 x 972.13

= 77.77 kN

Live load Moment considering Impact along flow direction = 0.08 x 270.00

= 21.60 kN-m

Live load Moment considering Impact along traffic direction = 0.08 x 1122.8

= 89.82 kN-m

Longitudinal Forces

(A) Longitudinal force due to braking force

Total Load of Class 70 R wheeled vehicle = 1000 kN (As per IRC-6 Appendix 1))

Braking force is considered as 20% of total vehicle load

(As per IRC-6, 2000, cl.No 214.2)

Longitudinal force for calss 70-R wheeled load = 0.2 x 1000

= 200 kN

Moment at base of pier due to braking force

= 200 x 10.2 (Thickness of pier cap + Height of pier = 1.2 + 9 = 10.2m)

= 2040 kN-m

(B) Longitudinal force due to resistance force in bearings

It is possible that the frictional coefficients of the two bearings on the pier may happen to be different due to unequal efficiency of the bearings.

Assume the live load to be on left span = 0.25 %

Total resistance by left side bearings =

= 0.25 * (Dead Load of superstructure of one side span + Live Load reaction at left with one span disloged condition)

= 0.25 (4480 +891.58)

= 1342.89 kN

Assume the frictional coefficients of bearings = 0.225

Total resistance by right side bearings = 0.225 * Dead load of superstructure of One side span

= 0.225 x 4480

= 1008 kN

Unbalanced force at bearing = 1342.89 - 1008

= 334.89kN

Moment at base of pier due to resistance in bearing

= 334.89 x 10.2 (Thickness of pier cap + Height of pier = 1.2 + 9 = 10.2 m)

= 3415.92 kN-m

Wind Force

(A) Area of structure in elevation due to deck and handrails (to be computed from dimensions of superstructure of 40m span of MOST classification Fig 3.5) = 146 m^2 (As Per section 3.5, chapter 3)

Assuming the average height of exposed surface above the bed level to be 10 m.

So intensity of wind load = 0.91 kN/m^2 (As per IRC- 6, 2000, Table - 4)

Total wind force = 146 x 0.91

= 132.86 kN

(B) Wind force against moving load, considering Class A train

(Class A train has highest length, 20.4m, so it is considered for worst case)

Length of Class A train = 20.4m

Lateral wind force =3 kN/m (As Per IRC-6 cl. 212.4)

Wind force against moving load = 20.4 x 3

= 61.2 kN

(C) Total wind force as in (A) and (B) = 132.86 + 61.2 = 194.06 kN

(D) Minimum limiting force on deck at 4.5 kN/m (As per IRC-6 Cl.212.6)

= span * minimum limiting force in kN/m

= 40 x 4.5

= 180 kN

(E) Minimum limiting force at 2.4kN/m^2 on exposed surface

(As per IRC-6 Cl.212.5)

= Exposed area of superstructure * minimum limiting force in kN/m^2

= 146 x 2.4

= 350.4 kN

Since the force in (E) is the maximum, this will be adopted for design. This force will be assumed to act at the bearing level for the purpose of calculating the moment at the base of the pier

Moment at base of pier

= 350.4 x 10.2 (Thickness of pier cap + Height of pier = 1.2 + 9 = 10.2 m)

= 3574.08 kN-m

Water current Force

Intensity of pressure = 0.5 K V^2 kN/m^2 (As Per IRC-6, 2000,Cl. No 213.2)

Where K = a constant having the value for different shapes of piers.

If circular pier or pier with semicircular ends

then K =0.66 (From IRC-6, 2000, Cl.No. 213.2)

V = Maximum mean velocity of current

=3.6m/sec

Intensity of pressure = 0.5 x 0.66 x 3.6 x 3.6

= 4.3 kN/m^2

Force due to water current=Exposed area in flow direction*Intensity of pressure

Force due to water current = {1.8 + 2.7} / 2x 9 x 4.3 (Fig 4.4)

= 87.08 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base of pier = 87.08 x 5.4

= 470.21 kN-m

Water current with obliquity at 20°

Pressure parallel to pier = 4.3 x COS 20°

= 4 kN/m^2

Pressure perpendicular to pier =4.3 x SIN20°

= 1.5 kN/m^2

Force on pier along flow direction=Exposed area in flow direction* Intensity of pressure parallel to pier

= {1.8 + 2.7} / 2 x 9.00 x 4 (Fig 4.4)

= 81 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base of pier = 81 x 5.40

= 437.4 kN-m

Force on pier along traffic direction = Exposed area in traffic direction* Intensity of pressure perpendicular to pier

= [7.5 + {1.8 + 2.7} / 2]x 9.00 x 1.5

= 131.625 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base of pier =131.625 x 5.40

= 710.775 kN-m

Seismic Force

Horizontal Seismic Force in traffic direction

F_{eq} = Ah x G

$$Ah = \frac{(Z/2)\ (Sa/g)}{(R/I)}$$

Where Z = Zone factor as given in table 5 of IRC 6

Zone= 3, Z= 0.16

I = Importance factor

For important bridge = 1.5

Other bridges = 1.0

R = Response reduction factor = 2.5 (As per IRC- 6, Cl. No. 222)

Sa/g = Average response acceleration coefficient

= 2.5 (As per IRC- 6, Cl. No. 222)

G = Vertical Load (Self weight) of components

$$Ah = \frac{\{0.16 / 2 \times 2.5\}}{\{2.5 / 1.5\}}$$

$= 0.12$

Horizontal Seismic force (F_{eq}) = 0.12 x G

TABLE 4.3 SEISMIC FORCE AND MOMENT ON PIER

Components	**Vertical Load**	**Horizontal Seismic**	**Lever Arm (m)**	**Bending Moment**
	(Self-Weight)	**Force**	From base	**due to Horizontal**
	kN	kN	of Pier	**Seismic Force**
	G	0.12*G		kN-m
Superstructure	8960.00	1075.20	10.80	11612.16
Pier cap	1035.00	124.20	9.60	1192.32
Pier	4726.81	567.22	4.20	2382.31
Total		**1766.62**		**15186.79**

Table 4.3 gives Horizontal Seismic force and Moment for both span loaded conditions

One span dislodged condition

For one span disloged condition superstructure Load and moments becomes half (For load combination at construction stage As per IRC-6, 2000, Table 1)

Total Horizontal seismic Force = 1075.20/ 2+124.20+567.22

=1229.016 kN

Total B.M. at base of pier = 11612.16 / 2+ 1192.32+ 2382.31

= 9380.71 kN-m

4.9.2 Summary of Forces on pier

Summary of all calculated forces are shown in table 4.4

TABLE 4.4 SUMMARYOF FORCES ON PIER

SR NO	LOAD	VERTICAL LOAD kN	ALONG FLOW Transeverse Direction		ALONG TRAFFIC Longitudinal Direction	
			HT kN	MT kN-m	HL kN	ML kN-m
01	Dead load					
	a) One Span dislodged cond	10241.81				3584.00
	b) bothside spans	14721.81				
02	Buoyancy Force	622.48				
03	Live load					
	a) One Span disloged	891.58		710.00		1029.77
	b) Both Span Loaded	972.13		270.00		1122.81
04	Impact Force	77.77		89.82		21.60
05	Longitudinal Forces					
	a) Braking Effect				200.00	2040.00
	b) Bearing Rigidity Force				334.89	3415.93
06	Wind Force		350.40	3574.08		
07	Water Current					
	(with 20 obliqu.)		81.00	437.40	131.63	710.78
08	Seismic forces					
	a) One side span disloged				1229.02	9380.71
	b) Both side span loaded				1766.62	15186.79

Where,

HT = Horizontal Load in Transverse direction

HL = Horizontal Load in Longitudinal direction

MT = Moment in Transverse direction

ML = Moment in Longitudinal direction

4.9.3 Load Combinations for pier

Load combinations is done as per IRC-6, 2000, Table - 1

According to IRC-6, 2000, Load Combinations are done at construction condition and at service condition.

Construction Condition

(1) Dead Load + Wind Load + Water current + bearing rigidity + Buoyancy

P = 10241.81- 622.48 = 9619.32 kN

HT = 350.40 + 81.00 = 431.40 kN

HL = 334.89 + 131.63 = 466.52 kN

MT=3574.08+437.40=4011.48 kN-m

ML=3584.00+3415.93+710.78 = 7710.70 kN-m

(2) Dead Load + Water current force + Bearing rigidity force + Buoyancy + (0.5) Seismic Force

P = 10241.81 - 622.48= 9619.32 kN

HT = 81.00 kN

HL = 334.89 + 131.63 + (0.5) 1229.02= 1081.03 kN

MT = 437.40 kN-m

ML = 3584.00 + 710.78 + 3415.93 + (0.5) 9380.71= 12401.06 kN-m

Service Condition

(3) Dead Load + Live Load + Vehicle Impact Force + Water current Force + Braking Force + Bearing rigidity Force + Buoyancy Force

P = 14721.81 + 972.13 + 77.77 -622.48= 15149.22 kN

HT = 81.00 kN

HL = 200.00 + 334.89 + 131.63= 666.52 kN

MT = 270.00+ 437.40 + 89.82= 797.22 kN-m

ML=1122.81+2040.00+3415.93+710.78+21.60=7311.11kN-m

(4) Dead Load + Live Load + Vehicle Impact Force + Wind Force + Water current Force + Braking Force + Bearing rigidity Force + Buoyancy Force

P = 14721.81 + 972.13 + 77.77 - 622.48 = 15149.22 kN

HT = 350.40 + 81.00 = 431.40 kN

HL = 200.00 + 334.89 + 131.63= 666.52 kN

MT = 270.00 + 3574.08 + 437.40 + 89.82 = 4371.30kN-m

ML = 1122.81 + 2040.00 + 3415.93 + 710.78 + 21.60= 7311.11 kN-m

(5) Dead Load + (0.5) Live Load + (0.5) Impact Force + water current Force + (0.5)Braking Force+ (0.5) Bearing rigidity Force + Buoyancy Force + Seismic Force

P = 14721.81 + (0.5) 972.13 + (0.5) 77.77 - 622.48= 14624.27 kN

HT = 81.00 kN

HL = (0.5) 200.00 + (0.5) 334.89 + 131.63 + 1766.62 =2165.69 kN

MT = (0.5) 270.00 + (0.5) 89.82 + 437.40 = 617.31 kN-m

ML= (0.5)1122.81+ (0.5) 2040 + (0.5) 3415.93+710.78+15186.79+(0.5)21.60

= 19197.73 kN-m

4.9.4 Design Loads and Moments

Among all the load combinations, Load combination - 5 is governs the design for which

P = 14624.27 kN

MT = 617.31kN-m

ML = 19197.73 kN-m

Convert Pier with semicircular area in to Equivalent Rectangular area of width b and depth d.

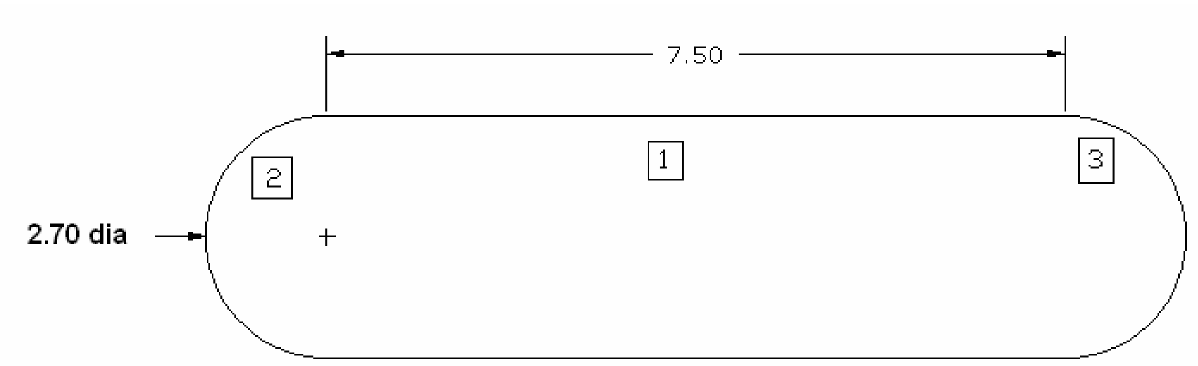


FIGURE 4.7 PLAN AT BASE OF PIER

A_1 = Area of portion $\boxed{1}$

A_2 = Area of portion $\boxed{2}$

A_3 = Area of portion $\boxed{3}$

$$\bar{X} = \frac{A_1 \mathbf{X}_1 + A_2 \mathbf{X}_2 + A_3 \mathbf{X}_3}{A_1 + A_2 + A_3}$$

$$= \frac{20.25 \times 5.1 + 2.86 \times 0.57 + 2.86 \times 9.42}{20.25 + 2.86 + 2.86}$$

$= 5.08$ m

$$\bar{Y} = \frac{A_1 Y_1 + A_2 Y_2 + A_3 Y_3}{A_1 + A_2 + A_3}$$

$$= \frac{20.25 \times 1.35 + 2.86 \times 1.35 + 2.86 \times 1.35}{20.25 + 2.86 + 2.86}$$

$=1.35$ m

$$Ixx = \frac{b\,d^3}{12} + A\,h^2 + \frac{\Pi\,d^4}{128} + A\,h^2 + \frac{\Pi\,d^4}{128} + A\,h^2$$

$= 12.30 + 1.30 + 1.30$

$= 14.91\ mm^4$

$$I = \frac{b\,d^3}{12} \qquad \text{Take } d = 2.7\text{m}$$

$$14.91 = \frac{b * d^3}{12}$$

$b = 9.09$ m

4.9.5 Design of reinforcement of Pier

Among the all load combinations, Load combination - 5 is governs the design

P = 14624.27 kN   b = 9.09 m

MT = 617.31 kN-m   D = 2.70 m   d'= 0.10 m

ML = 19197.73 kN-m   fck = 20N/mm^2

Pu = 21936.41 kN   fy = 415 N/mm^2

MT_u(ultimate moment in transverse direction)= 1.5 * 617.31 =925.96 kN-m

ML_u(ultimate moment in longitudinal direction) =1.5*19197.73= 28796.60 kN-m

Reinforcement is distributed equally on four sides.

As a first trial provide the reinforcement percentage, p = 0.30 which is minimum requirement of steel for pier (As per IRC-78,Cl.No. 710.3)

Moment capacity of the section about flow direction p/fck = 0.02 d'/D = 0.04 Pu / fck * b *D = 0.04

Reffering chart no. 43 (SP - 16)

Mu/ fck * b * D^2 = 0.04 Mux1 = 53012.88 > 28796.60 …..O.K

Moment capacity of the section about traffic direction

p/fck = 0.02 d'/D = 0.01 Pu / fck * b *D = 0.04

Reffering chart no. 32 (SP - 16)

Mu/ fck * b * D^2 = 0.04 Muy1 = 187400.53 > 925.97 …..O.K

Calculation of Puz:

Reffering chart 63 of SP-16 corrosponding to

p = 0.30 fy = 415 N/mm^2 fck = 20 N/mm^2

So, Puz/ Ag = 9.50 N/mm^2

= 9500.00 kN/m^2

so, Puz=9500.00 x 2.70 x 9.09 = 233169.56 kN

Pu/Puz = 0.09 ML/Mux1 = 0.54 MT/Muy1 = 0.02

Referring to Chart 64,

Corresponding to the above values of MT/Mux1 and Pu / Puz , the permissible values of Mux/Mux1 = 0.98 > 0.54

Hence the section is o.k.

So, Ast req = p * b * D /100

= 0.07 m^2

= 73632.49 mm^2

Provide 92 nos of 32 Φ

Ast provided = 80424.77 mm^2 > 73632.49 mm^2

Stirrups area = 0.04% (As per IRC -78,2000, Cl.No. 710.3.3)

Stirrups area = 0.04 * 9.09 * 2.7 / 100

= 0.009817 m^2

= 9817 mm^2

Provide 12mm dia. @200 mm c/c

Reinforcement details for pier are shown in fig4.8

4.9.6 Check for Stresses at the base of pier

B = 9.09 m

D = 2.70 m

$\bar{X}$ = 4.55 m

$\bar{Y}$ = 1.35 m

Area A = 2.70 x 9.09 = 24.54 mm^2

Iyy = $(D * b^3 / 12) = 9.09 \times 2.70^3 / 12.00$

$= 14.91\ mm^4$

Ixx = $(b * D^3 / 12) = 9.09 \times 2.70^3 / 12.00$

$= 169.02\ mm^4$

$$P_{max/min} = \left(\frac{V}{A}\right) + \text{ or } - \left(\frac{ML * \bar{X}}{Ixx}\right) + \text{ or } - \left(\frac{MT * \bar{Y}}{Iyy}\right)$$

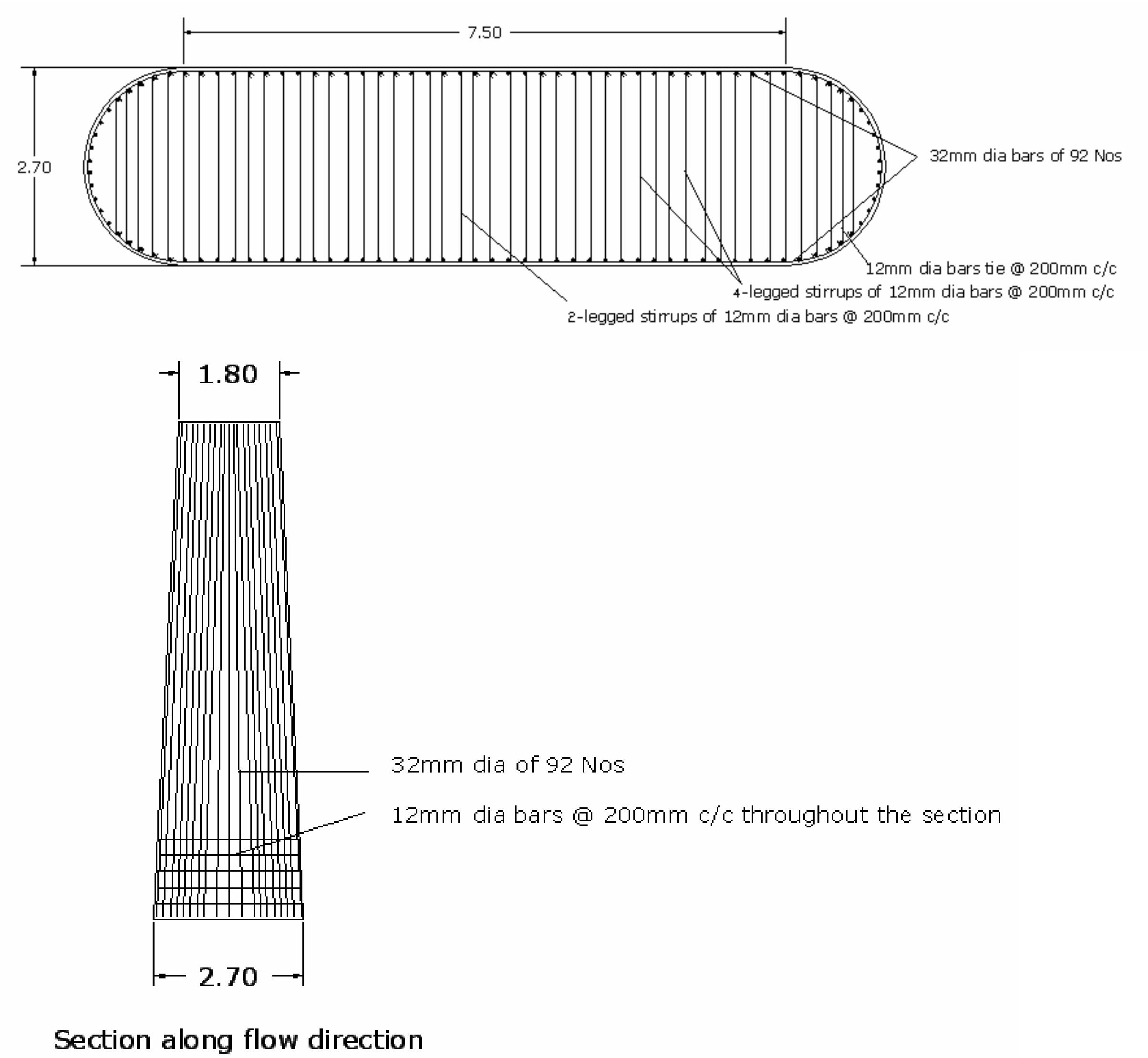
$$= \frac{14624.27}{24.54} (+ \text{ or } -) \frac{19197.73 \times 1.35}{169.02} (+ \text{ or } -) \frac{617.31 \times 4.55}{14.91}$$

$= 595.83 (+ \text{ or } -) 153.36 (+ \text{ or } -) 188.38$

$P_{max} = 937.98\ kN/m^2$

$P_{min} = 254.12\ kN/m^2$

Both stresses are in compression that means no tension at base of pier that means section is safe.



All dimensions are in meter

FIGURE 4.8 REINFORCEMENT DETAILS OF PIER

5. PILE FOUNDATION

5.1 General:

Pile foundations are part of a structure used to carry and transfer the load of the structure to the bearing ground located at some depth below ground surface. The main components of the foundation are pile cap and the piles. Piles are long and slender members which transfer the load to deeper soil or rock of high bearing capacity.

5.2 Function of pile

As with other types of foundations, the purpose of pile foundations is:

- to transmit a superstructure load and pier load to a solid ground below
- to resist vertical, lateral and uplift load

A structure can be founded on piles if the soil immediately beneath its base does not have adequate bearing capacity. In the cases of heavy constructions, it is likely that the bearing capacity of the shallow soil will not be satisfactory, and the construction should be built on pile foundations. Piles can also be used in normal ground conditions to resist horizontal loads. Piles are a convenient method of foundation for works over water, such as jetties or bridge piers. A cost estimate may indicate that a pile foundation may be cheaper than any other compared ground improvement costs.

5.3 Classification of Piles:

- Based on the use:
  1. End Bearing Pile
  2. Friction Pile
  3. Compaction Pile
  4. Tension Pile
  5. Sheet Pile
  6. Batter Pile

- **End bearing piles:** These piles transfer their load on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile (figure 5.1).
- **Friction piles:** These piles also transfer their load to the ground through skin friction. These types of pile foundations are commonly known as floating pile foundations. (figure 5.1).

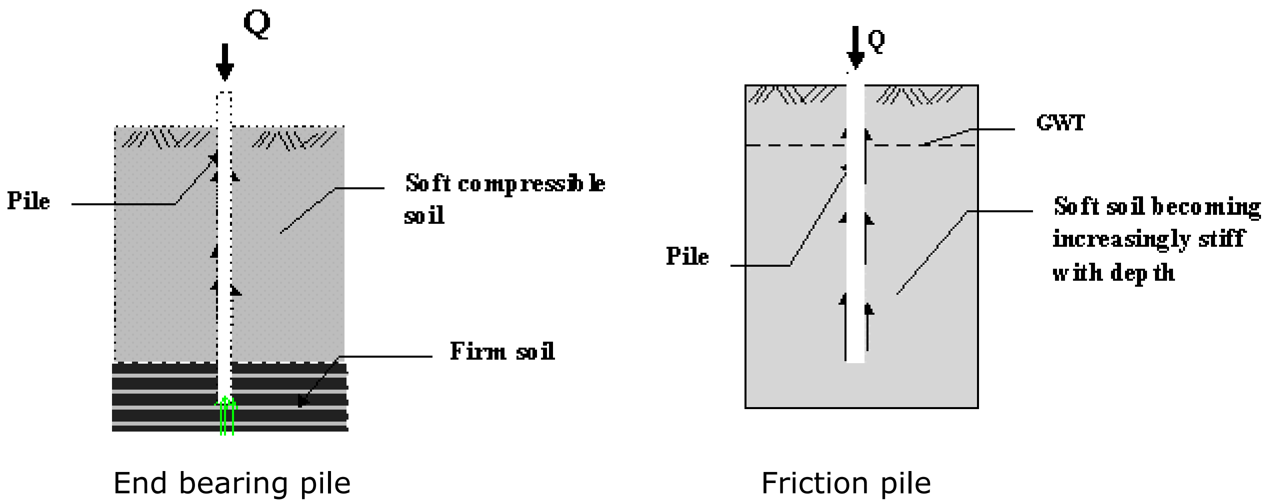


FIGURE 5.1 TYPES OF PILE BASE

Compaction piles are used to compact loose granular soil, thus increasing their bearing capacity. Tension or uplift piles anchors down structures subjected to uplift due to overturning moment or other cases. Anchor piles provide anchorage against horizontal pull from sheet piling or other pulling forces. Batter Piles are used to resist horizontal or inclined forces. Sheet piles are commonly used as bulkhead or as impervious cut off to reduce seepage and uplift in hydraulic structures.

- Based on Material of construction:
  1. Concrete Piles – Precast or Cast in situ.
  2. Steel Piles – H Piles, Pipe or sheet Pile
  3. Timber Piles
  4. Composite Piles

Precast concrete piles may be of square, hexagonal or octagonal shape, the former one being commonly used for their advantage of easy moulding and driving. Moreover, square piles provide more frictional surface which helps in taking more load. Hexagonal or octagonal piles, on the other hand, have advantage that they possess equal strength in flexure in all directions and the lateral reinforcement may be provided in the form of continuous spiral. Cast-in-situ piles are circular piles with variable size depending on the type and load carrying capacity. Timber piles are trunks of trees which are very tall and straight the branches being stripped off. Timber piles are cheaper than other varieties of piles but they lack in durability under certain conditions of service where variation of water level causing alternate drying and wetting of the piles is responsible for rapid decay of timber piles

5.4 Pile Spacing:

The spacing of piles should be considered in relation to the nature of the ground, their behavior in groups and the overall cost of the foundation. The spacing should be chosen with regard to the resulting heave or compaction and should be wide enough to enable the desired number of piles to be installed to the correct penetration without damage to any adjacent construction or to piles themselves. The recommended minimum spacing of piles is 3*d, where d is the diameter of circular piles or the length of the diagonal for square, hexagonal or octagonal piles. No limit has been fixed for the maximum spacing of the piles but it does not generally exceed 4*d.

5.5 Minimum diameter of pile

The minimum diameter of piles shall be as per IRC-78,cl. No. 709.1.7 which is mentioned in table 5.1

TABLE 5.1 MINIMUM DIAMETER OF PILE

	Bridge on Land	**River Bridge**
Driven cast-in-situ	0.5m	1.2m
Precast Piles	0.35m	1.0m
Bored Piles	1.0m	1.2m

5.6 Load transfer mechanism of Pile:

- Friction Piles: When a load is placed on the top of a friction pile driven in granular or cohesive soil, it tends to penetrate further. This tendency of downward movement of the pile is resisted by the skin friction between the pile surface and the soil. The magnitude of the skin friction per unit are of pile surface depends on the value of normal earth pressure p and the coefficient of friction between the soil and the pile surface; both of these values again depend on the nature of the pile surface and the nature of the soil.
- End Bearing Piles: End bearing piles are driven through very poor type of soil to rest on firm base such as compacted sand or gravel deposits or rock. Therefore, the friction developed between the pile surface and the soil is practically very small and the whole load is transmitted by the pile through bearing. These piles act as column and therefore, should be designed as column.

5.7 Load distribution in pile group:

When there is eccentricity about both axes, individual pile loads (P_{max}) may be determined by following formula,

$$\boxed{P_{max/min} = \frac{P}{n} \pm \frac{ML * e_x}{n * e_x} \pm \frac{MT * e_y}{n * e_y}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.1)$$

Where P = Total vertical load applied on pile group

n = Total Number of piles

ML = Total Longitudinal moment applied on pile group

MT = Total Transverse moment applied on pile group

e_x = Eccentricity with respect to centre of pile group measured along X-axis

e_y = Eccentricity with respect to centre of pile group measured along Y-axis

5.8 Load Carrying capacity of Pile based on pile soil interaction

Capacity of pile based on pile soil interaction that means whether Pile is located in soil or in rock, accordingly the capacity of pile is found out. For both condition the capacity of pile is found out by using following equations

5.8.1 Capacity of individual Pile in Rock:

The ultimate load carrying capacity can be calculated from

$$Q_a = R_e + R_{ef} \quad \text{...(5.2)}$$

$$Q_a = R_e + R_{ef} = k_{sp} * q_c * d_f * A_b + A_s * q_s \quad \text{.........................(5.3)}$$

Where, Q_a = Ultimate capacity of pile socketed in rock

R_e=Ultimate end bearing resistance

R_{ef}=Ultimate side socket shear

k_{sp}=An empirical co-efficient whose value ranges from 0.1 to 0.4

q_c=Average uniaxial compressive strength of rock at tip level

d_f =Depth factor

A_b=Cross-sectional area of base of pile

A_s=Surface area of socket.

q_s=Ultimate shear along the socket (value of qs may be taken as 50kg/cm^2 for normal rock and 20 kg/cm^2 for weathered rock)

5.8.2 Capacity of individual Pile in Soil:

Axial load carrying capacity of the pile is initially determined by calculating resistance from end bearing at toe/tip or wall friction/skin friction along pile surface or both. Based on the soil data, the ultimate load carrying capacity (Q_u) is given by:

$$Q_u = R_u + R_f \quad \text{.. (5.4)}$$

$$R_u = A_p * (0.5 * D * \gamma * N_\gamma + P_d * N_q) + A_p * N_c * C_p \quad \text{.............. (5.5)}$$

$$R_f = \sum_{i=1}^{n} KP_{di} (\tan\delta) A_{si} + \alpha C A_s \quad \text{.. (5.6)}$$

Where, A_p = Cross sectional are of base of pile

D = Pile diameter in cm

γ= Effective unit weight of soil at pile tip in kg/cm^3

N_γ & N_q = Bearing capacity factors based on angle of internal friction at pile Tip

N_c = Bearing capacity factor usually taken as 9

P_d= Effective overburden pressure at pile tip limited to 20 times diameter of pile for piles having length equal to more than 20 times diameter

C_p = Average cohesion at pile tip (from unconsolidated undrained test)

K = Coefficient of earth pressure

P_{dI} = Effective overburden pressure in kg/cm^2 along the embedment of pile for the I th layer where I varies from 1 to n

δ = Angle of wall friction between pile and soil in degrees. It may be taken equal to angle of internal friction of soil.

A_{sI} = Surface area of pile shaft in cm^2 in the I^{th} layer, where I varies from 1 to n

A_s = Surface are of pile shaft in cm^2

α= Reduction Factor

C = Average cohesion in kg/cm^2 throughout the embedded length of pile (From unconsolidated undrained test)

- While evaluating effective overburden pressure, total and submerged weight of soil shall be considered above and below water table respectively

5.8.3 Factor of Safety:

The minimum factor of safety on ultimate axial capacity computed on the basis of static formula shall be 2.5 for piles in soil. For piles in rock, factor of safety shall be 5 on the bearing component and 10 on socket side resistance component. (As per IRC-78, Cl. No. 709.3.2)

5.9 IRC Code Provisions for piles:

- The piles may be designed taking into consideration all the load effects and their structural capacity examined as a column. The reinforcement in pile should be provided for the full length of pile, as per the design requirements.

- However, the minimum area of longitudinal reinforcement shall be 0.4 percent of the area of the cross-section in all concrete piles.

- Lateral reinforcement shall be provided in the form of links or spirals with 8mm diameter steel, spacing not less than 150mm.

- Cover to main reinforcement shall not be less than 75mm

5.10 Pile load analysis

For the analysis of pile, forces mentioned in 5.10.1 are considered. All the below mentioned forces will act at the top of pile group or at the bottom of pile cap.

5.10.1 Forces on pile

The following forces are considered for the analysis of pile

- Dead load of superstructure, pier cap, pier, pile cap.
- Live load
- Impact effect
- Buoyancy Force
- Wind Force on moving load and on the superstructure
- Water current Force
- Longitudinal force due to braking effect of vehicles
- Seismic Force.

Moment at base of pile due to horizontal load at the top of pile can be calculated as per following equations. The following equations are mentioned in Amendment No. 3 of IS: 2911(part 1)

$$M = W*L \text{..........for free head pile}$$

$$M = W*L/2 \text{........for fixed head pile}$$

Where M = Moment at base of pile

L = Length of pile

W = Horizontal Load at the top of pile

5.10.2 Steps for Pile analysis:

Following steps should be followed for analysis of pile

- Calculate the forces as mentioned in 5.10.1
- Calculate actual loads and moments on individual pile using equation 5.1
- Calculate capacity of individual pile as mentioned in 5.2 or 5.4
- Calculate moments at the base of pile due to horizontal load at the top of pile
- Calculate moments at the base of pile due to water current force

5.11 Steps for reinforcement design of pile:

To design a pile following steps should be followed:

- For the design of a pile, the individual load on a pile should be calculated from equation-5.1.
- Then pile load carrying capacity, as per soil condition should be calculated as per section 5.7.1 and 5.7.2 , which should be more than the individual load on a pile
- The pile behaves as an ordinary column and should be designed as such a column subjected to biaxial bending.

5.12 Pile cap

5.12.1 General

A rigid pile cap in reinforced concrete should be provided to transfer the load from the pier to the piles as uniformly as possible under normal vertical loads. A minimum offset of 150mm should be provided beyond the outer faces of the outer-most piles in the group. There are two alternative theories on which pile caps can be assumed to transfer the loads from pier to pile foundation. They are (a) Truss analogy method, and (b) bending theory. Experiments shows that truss action can be predominant when (a_v/d′) ratio is less than 2 (fig. 5.2). When ratio of (a_v/d′) is 2 or more, bending action is more predominant than truss action. In bending theory the pile cap is designed as a normal beam for bending moment and shear. The reinforcement is evenly distributed over the section.

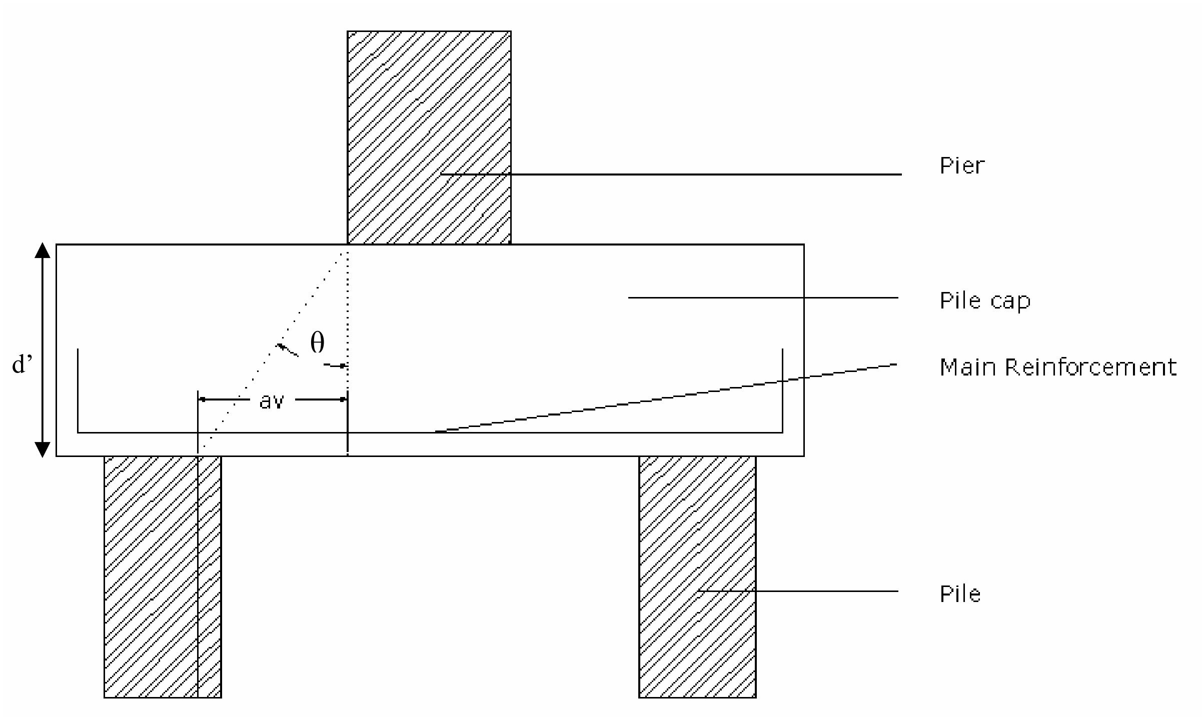


FIGURE 5.2 BEHAVIOR OF PILE CAP

Design by Truss Analogy Method:

The thickness of pile cap shall be so proportioned to act as stiff member. The minimum thickness of cap shall be 0.5 times the spacing of the pile where there are two rows of piles.

In truss analogy method pile cap area is divided into various strips in both Longitudinal and transverse directions considering number of pile and pile diameter as shown in fig. 5.3

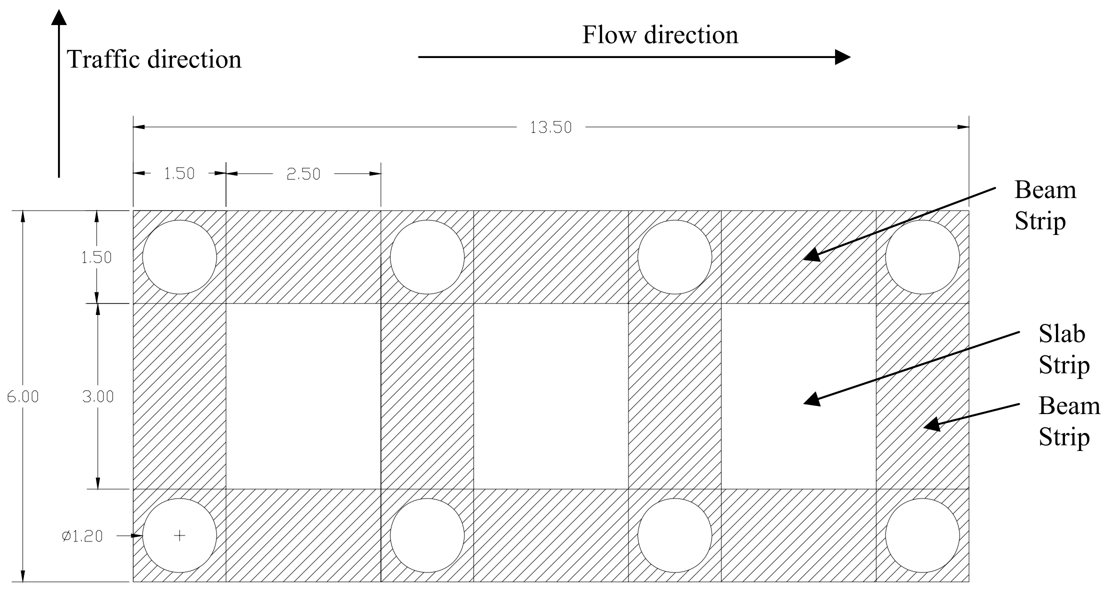


FIGURE 5.3 FORMATIONS OF STRIPS FOR PILE CAP

The truss should be in triangular form with a node at the centre of loaded area. The lower node of the truss lies at the intersection of the centre line of the piles with the tension reinforcement. The truss method is used with widely spaced piles (spacing exceeding three times the pile diameter).
Eighty percent of the total reinforcement shall be concentrated in strips linking the pile heads and the remainder reinforcement is uniformly distributed throughout the pile cap.
No check for shear is required for pile caps when designed by Truss Analogy method.

Design by Bending Theory:

When ratio of (a_v/d’) is 2 or more, bending action is more predominant than truss action. In bending theory the pile cap is designed as a normal beam for bending moment and shear. The bending moment at any section of the cap for a reinforced concrete column or wall shall be taken to be the moment of the forces over the entire area on one side of the section. The critical section for bending in the cap shall be taken at the face of the column or wall.
Out of theses two method, the pile cap is designed using truss analogy method and STTAD.Pro software to calculate the horizontal forces in pile cap strips

5.12.2 Steps for Analysis of pile cap in STAAD.Pro:

The analysis of pile cap by truss analogy method is done using by STAAD.Pro. The sequence of input to generate file in STAAD.Pro is mentioned below:

1. Node generation
2. Element generation
3. Define material property
4. Support generation
5. Load generation
6. Analysis of pile cap

STAAD.Pro input file for pile cap:

The input file that is generated in STAAD.Pro is having following format:

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 16-Apr-06

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

Node Generation

First define the plane in which geometry is to be created. For each node the location is specified with respect to global X, Y and Z axis. 8-nodes are defined at the top of pile and top joint of truss is defined at the base of pier.

Joint coordinates

The joint coordinate of the node 1 are 0 0 0 with respect to all axis. This means this is origin point of geometry. The joint coordinates are given as below:

1 0 0 0; 2 0 2 0; 3 -6 0 -2.25; 4 6 0 -2.25; 5 6 0 2.25; 6 -6 0 2.25;

7 -2 0 -2.25; 8 2 0 -2.25; 9 2 0 2.25; 10 -2 0 2.25;

Element generation

The element generated for connecting nodes are defined as a beam element.

Define Material Property

The material is defined with following property:

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 1e-005

DAMP 0.05

END DEFINE MATERIAL

Support generation

All supports are generated at the top of pile and top joint of truss are generated at the base of center pier. All supports are defined as pinned supports.

Load Generation

Load are generated at the top of pile cap, the vertical load at top of pile cap is given as preliminary load.

Analysis of Pile Cap

Analysis of Pile cap is done by using STAAD.Pro with considering pile cap as a space structure. Axial forces in beam and slab element are calculated using STAAD.Pro software.

5.12.3 Procedure for Design of pile cap

For the design of pile cap following steps should be followed:

1. Formation of strip in both longitudinal and transverse direction
2. From analysis of pile cap as space truss in STAAD.Pro, the axial force in strip in both directions is found out.
3. The strip is designed as a beam element subjected to axial force.
4. The middle strip is designed as a slab element.
5. Reinforcement details are calculated taking into account the axial force in each strip.

The strip is formatted as beam element with width equal to diameter of pile plus two times offset of the outer faces of the outer most piles in the group.

5.13 Analysis and Design of pile cap

A rigid pile cap in reinforced concrete should be provided to transfer the load from the pier to the piles as uniformly as possible under normal vertical loads.

Diameter of pile = 1.2 m

Total no. of pile = 8

No. of pile in flow direction = 4

No. of pile in traffic direction = 2

c/c spacing of pile in flow direction = 4m

c/c spacing of pile in traffic direction = 4.5m

Offset at edge of pile cap = 0.15 m

Width of pile cap in flow direction = (4-1) x c/c spacing of pile + diameter of pile + offset

= 3 * 4 + 1.2 + 2* 0.15 = 13.5 m

Width of pile cap in traffic direction = (2-1) x c/c spacing of pile + diameter of pile + offset

= 1 * 4.5 + 1.2 + 2* 0.15 = 6 m

As per IRC-21-2000, Cl.No. 307.2.5.2 The minimum thickness of cap shall be 0.5 times the spacing of the pile where there are two rows of piles. Here pile spacing is 4m c/c so 2m thick pile cap is provided.

5.13.1 Analysis for forces on Pile cap

Check for truss action

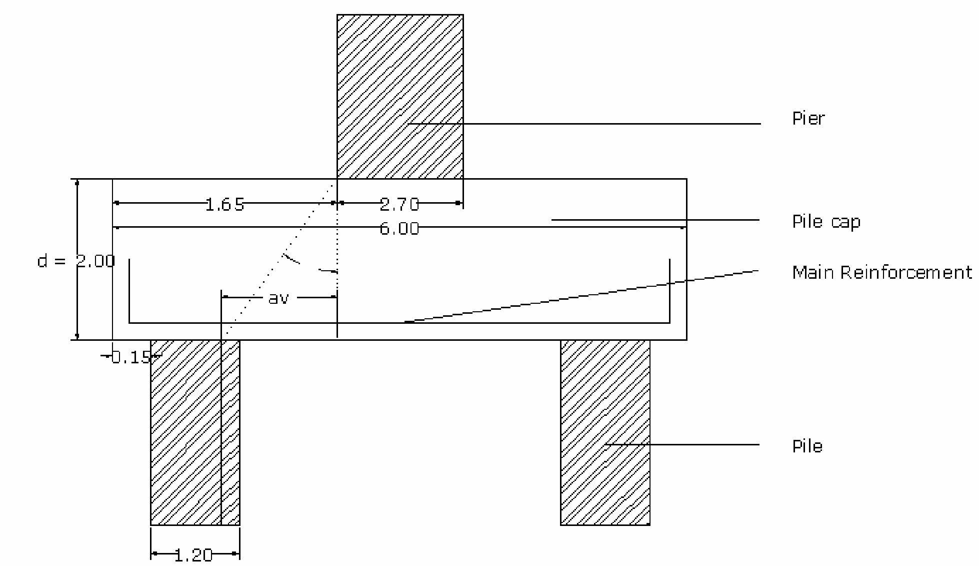


FIGURE 5.4 CHECK FOR av/d' RATIO

$a_v = 1.65 - 0.15 - 1.2 + 0.05 = 0.35$

d = depth of pile cap = 2.00 m

d' = effective depth of pile cap = 2.00 – 0.05 = 1.95 m

a_v /d' = 0.35 / 1.95 = 0.18 < 2

So, Analysis of pile cap, is done by truss analogy method considering pile cap as a space truss as shown in fig.5.6.

Dimensions of Pile cap=13.5 x 6 x 2m

Vertical Load from pier = 14624 kN

Width of strip in both direction = diameter of pile + 2(offset at the edge)

= 1.2 + 2 (0.15) = 1.5 m

Pile cap area is divided into various strips (Beam strip + Slab strip) in both the directions considering number of pile and pile diameter.

The strip are as shown in fig.5.5

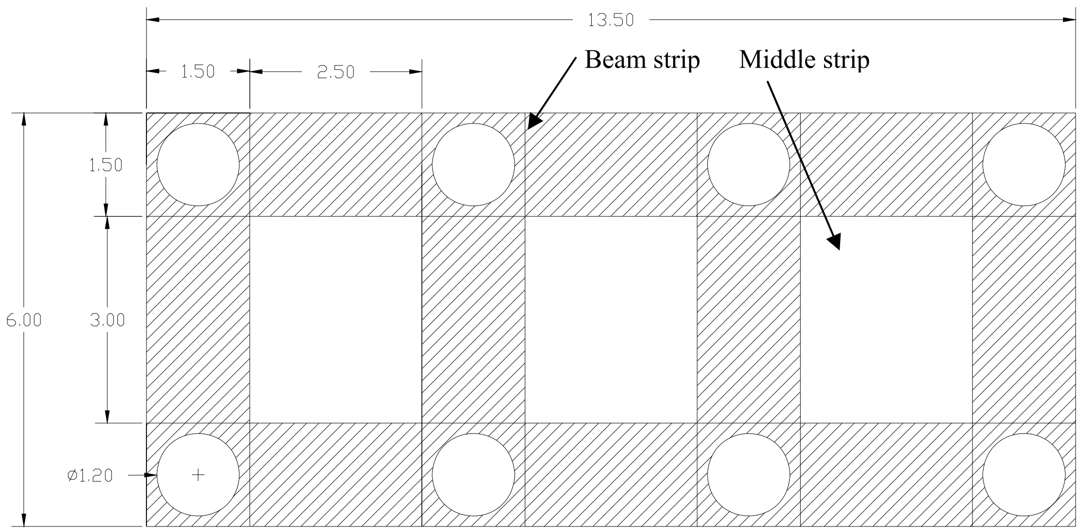


FIGURE 5.5 STRIPS FOR PILE CAP

The truss structure is shown in fig 5.6:

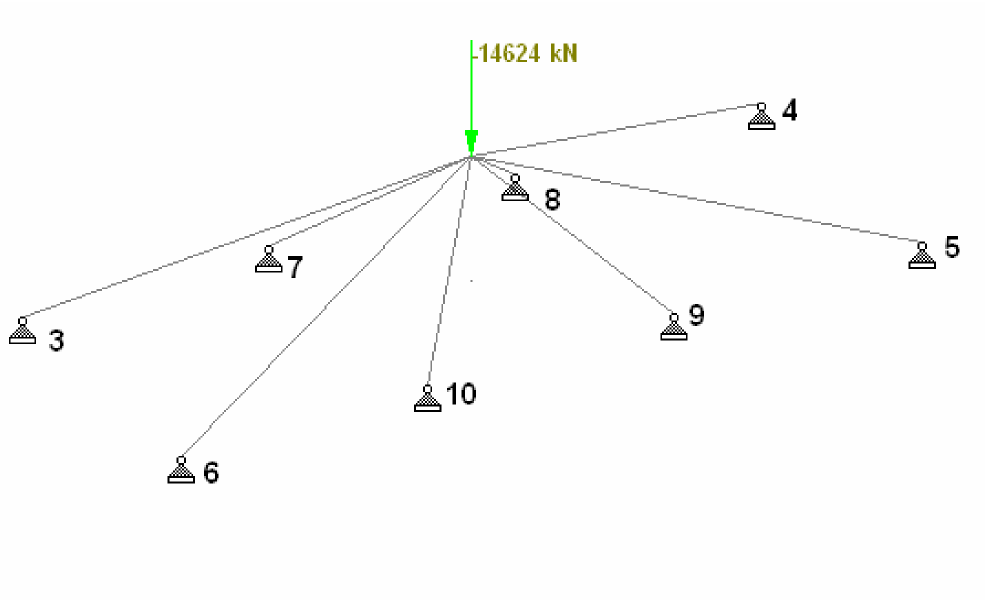


FIGURE 5.6 STADD MODEL OF PILE CAP

The result of STAAD.Pro analysis showing horizontal and vertical loads at each nodes are given in table 5.2:

TABLE 5.2 LOADS ON PILE CAP

	A	Horizontal	Horizontal	Vertical
Node	Load condition	Load kN	Load kN	Load kN
		In flow direction	In traffic direction	
3.0	Vertical load from pier	1451.9	544.5	496.0
4.0	Vertical load from pier	-1451.9	544.5	496.0
5.0	Vertical load from pier	-1451.9	-544.5	496.0
6.0	Vertical load from pier	1451.9	-544.5	496.0
7.0	Vertical load from pier	3084.8	3470.4	3160.0
8.0	Vertical load from pier	-3084.8	3470.4	3160.0
9.0	Vertical load from pier	-3084.8	-3470.4	3160.0
10.0	Vertical load from pier	3084.8	-3470.4	3160.0

5.13.2 Design of reinforcement pile cap

The design of reinforcement in various strips is carried out as under

Design of pile cap along flow direction

1.5m wide beam strip

From STAAD Analysis Axial Load on Strip of 1.5m width = 3084.83+1451.91

= 4536.73 kN

$$\text{Ast (req)} = \frac{1.5 \times 4536.73 \times 1000}{415 \times 0.87}$$

$= 18848.08\ mm^2$

Among that steel 80% steel is distributed in a 1.5m wide strip above pile(fig.5.5)

Area of steel required Ast = 0.8 x 18848.08

= 15078.46 mm^2

= 10052.30 mm^2/m

Provide 32mm dia bars @ 90mm c/c

Ast_{min} is as 0.2 % of cross-sectional area for strip portion.

Ast_{min} = 0.2 x 1900 x 1500 / 100

= 5700 mm^2 < 15078.46 mm^2

Provide 32mm dia bars @ 90mm c/c

Reinforcement details are shown in fig 5.7.

Middle Strip of width 3 m

Among total steel 20% steel is distributed in middle strip (fig5.5)

Area of steel required Ast = 0.2 x 18848.0

= 3769.62 mm^2

Ast min is provided as 0.12 % of cross-sectional area of middle strip portion.

Ast_{min} = 0.12 x 1900 x 3000 / 100

= 6840 mm^2 > 3769.62 mm^2

Provide 25mm dia bars @ 210mm c/c

Reinforcement detail is shown in fig 5.7 .

Design of pile cap along Traffic direction

1.5m wide beam strip

From STAAD Analysis Axial Load on Strip of 1.5m wide = 3470.42 kN

$$\text{Ast req} = \frac{1.5 \times 3470.42 \times 1000}{415 \times 0.87}$$

$= 14418.06\ mm^2$

Among that steel 80% steel is distributed in a 1.5m wide strip (fig5.5)

Ast = 0.8 x 14418.06

Ast = 11534.45 mm^2

= 7689.63 mm^2/m

Provide 32mm dia bars @ 110mm c/c

Ast min is provided as 0.2 % of cross-sectional area for strip portion.

Ast_{min} = 0.2 x 1900 x 1500 / 100

= 5700 mm^2 < 11534.45 mm^2

Provide 32mm dia bars @ 110mm c/c

Reinforcement detail is shown in fig 5.7 .

Middle Strip of width 7.5m

Among total steel 20% steel is distributed in middle strip

Ast = 0.2 x 14418.06

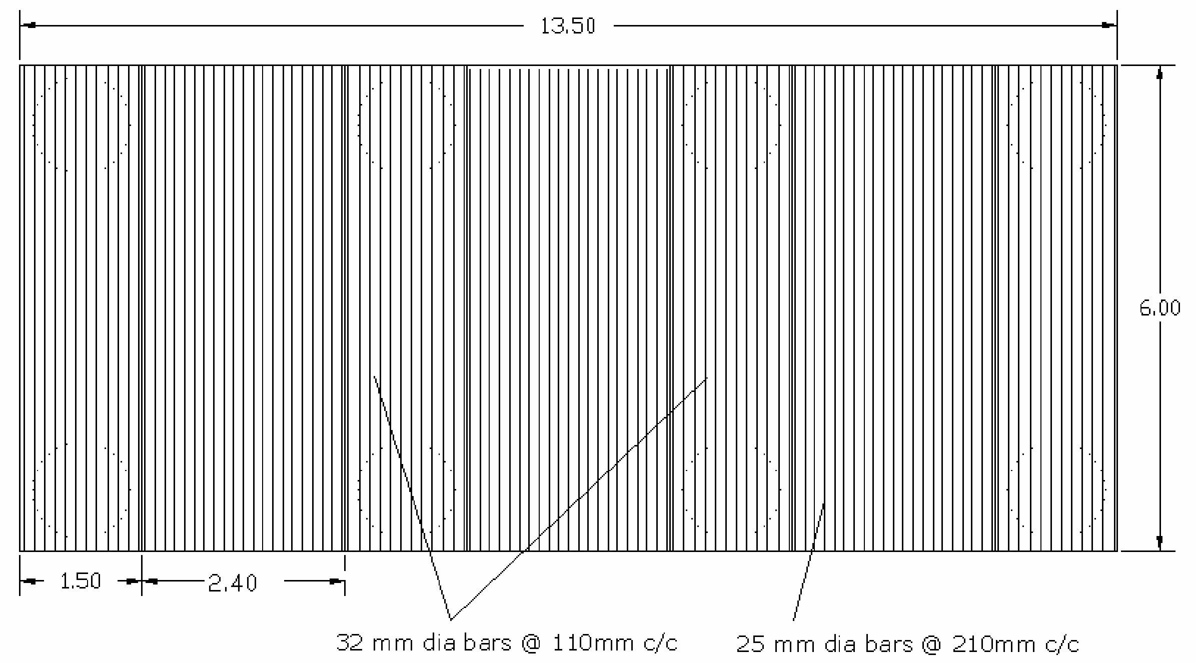
= 2883.61 mm^2

Ast_{min} = 0.12 x 1900 x 7500 /100

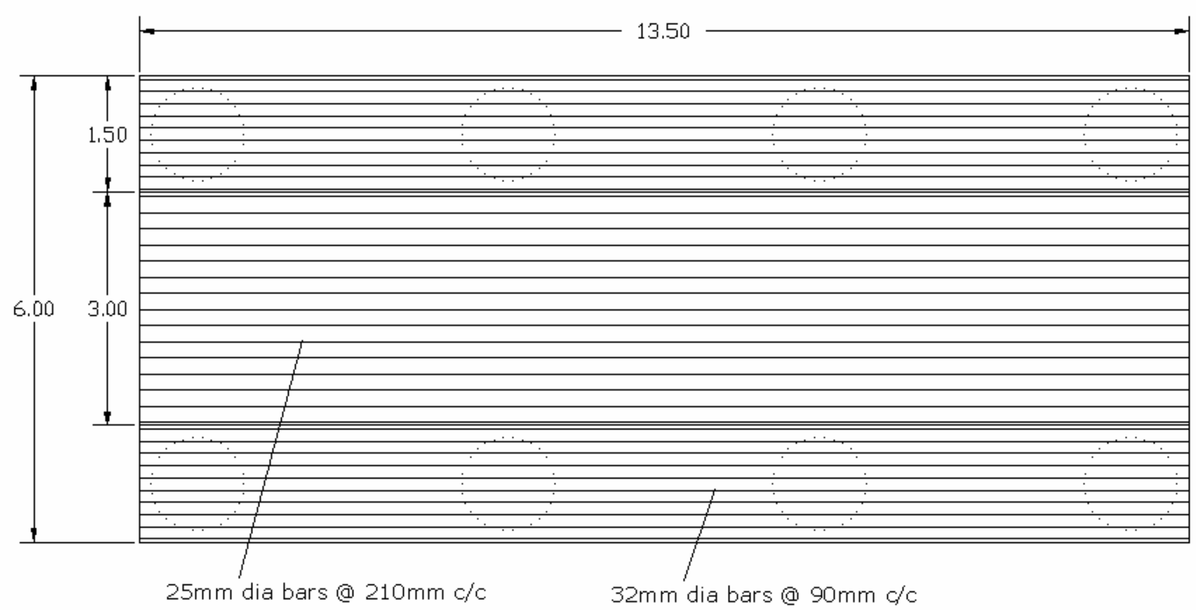
= 17100 mm^2 > 2883.61 mm^2

Provide 25mm dia bars @ 210mm c/c

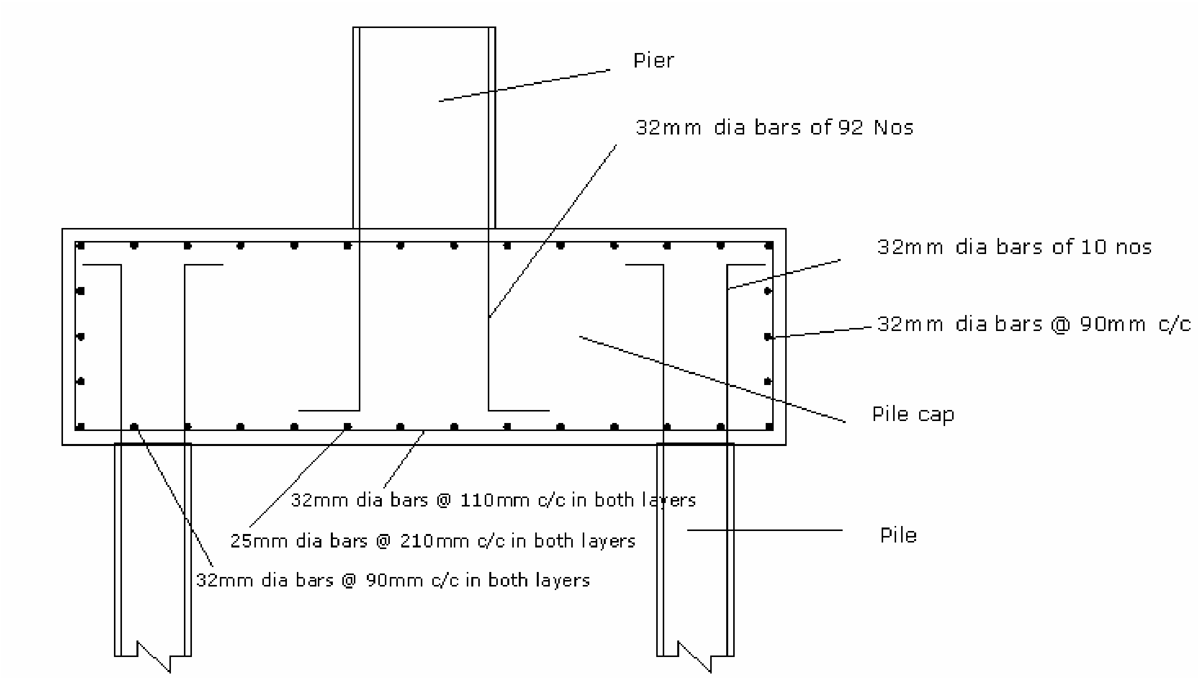
Reinforcement detail is shown in fig 5.7.



Pile cap along traffic direction



Pile cap along flow direction



Section of pile cap along traffic direction

FIGURE 5.7 REINFORCEMENT DETAIL OF PILE CAP

5.14 Analysis and Design of Pile

Data

Superstructure = simply supported Prestressed I girder

Span = 40 m

Pier = Wall type pier

Foundation = Pile Foundation

Types of Pile = End bearing Pile, cast-in-situ Pile

Diameter of pile = 1.2m

Length of up to Rock Level = 8m

Socket level is taken as 3 * Diameter of pile = 3 x 1.2 = 3.6 m

Total Length of pile = 8 +3.6 = 11.6 m

Total no. of pile = 8

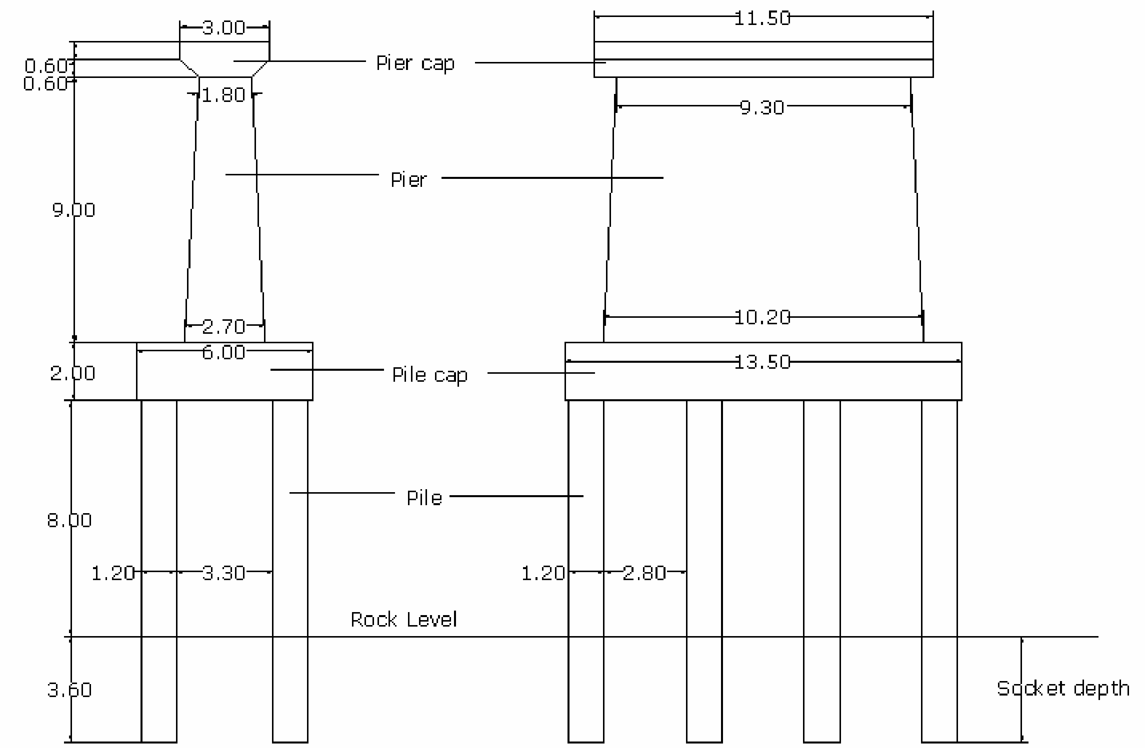
No. of pile in flow direction = 4

No. of pile in traffic direction = 2

c/c spacing of pile in flow direction = 4m

c/c spacing of pile in traffic direction = 4.5m

Dimensions of pile foundation are as shown in fig. 5.8



All dimensions are in meter

FIGURE 5.8 SECTIONS IN BOTH DIRECTIONS OF WHOLE PILE SYSTEM

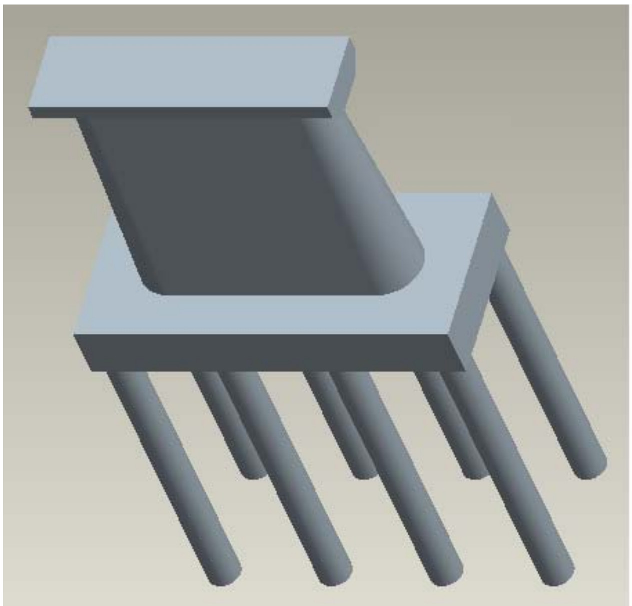


FIG 5.9 3-D VIEW OF WHOLE PILE SYSTEM

Dead load from each span = 4480 kN

Maximum mean velocity of water = 3.6 m/sec

Material for Pile = Cement concrete M 20 grade, Steel Fe415

Live load: IRC Class 70 R Wheeled vehicle

The size of pier cap = 11.5 x 3 x 1.2 m

The size of pile cap = 13.5 x 6 x2 m

5.14.1 Calculation of forces on pile

Dead Load

DL from Superstructure = 2 x 4480

= 8960.00 kN

Volume of pier cap = 11.5 x 3 x 1.2

= 41.4 m^3

Dead Load from pier cap = Volume of piercap * Density

= 41.4 x 25

= 1035.00 kN

Dead Load from pier

Area at base = 7.5 x 2.7 + 3.14 x 2.7 x 2.7 / 4

= 25.97 m^2

Area at top = 7.5 x 1.8 + 3.14 x 1.8 x 1.8 / 4

= 16.04 m^2

Average Area = 21.01 m^2

Total Volume = Average Area * Height

= 21.01 x 9

= 189.07 m^3

Dead Load of Pier = Volume of pier * Density

= 189.07 x 25

= 4726.81 kN

Volume of pile cap = 13.5x 6 x 2

= 162 m^3

Dead Load from pile cap = 162 x 25

= 3888.00 kN

Total direct load = Dead Load of Superstructure + Dead Load of Pier cap

+ Dead Load of Pier + Dead Load of Pile cap

= 8960 + 1035 + 4726.81+ 3888.00

=18609.81kN

Load for One span disloged condition = Half Dead Load of Superstructure

+ Dead Load of Pier cap + Dead Load of Pier + Dead Load of Pile cap

= 4480+ 1035 + 4726.81 +3888.00

= 14129.81 kN

Longitudinal moment for one span disloged condition

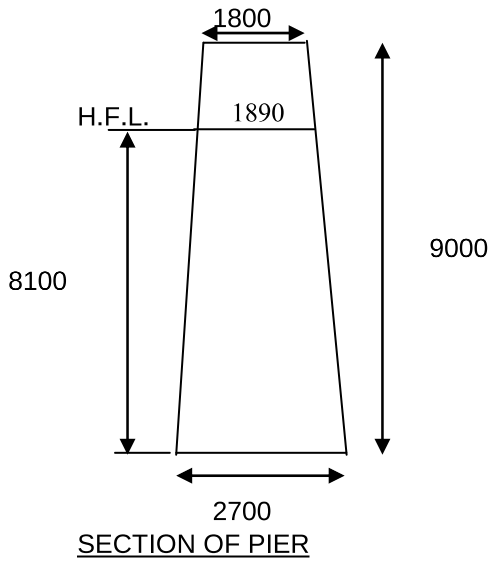
= Dead Load of superstructure of one span * eccentricity between bearing and center line of pier (fig.5.8)

= 4480.00 x 0.8

= 3584.00 kN-m

Effect of buoyancy

For Pier



All dimensions are in milimeter

FIGURE 5.10 SECTION OF PIER

Height of H.F.L. from top = 0.9 m

Width of pier at H.F.L. = 1.89 m

Submerged volume of pier (Height is from base to H.F.L)

= [7.5x {(1.89 + 2.7) / 2} x 8.1] + [3.14 x $\{2.7+ 1.89\}^2$ x 8.1/ 16]

= 172.91 m^3

Submerged Load = Volume of pier * Density

= 172.91 x 24

= 4149.88 kN

For Pile cap

Submerged volume of pile cap = 13.5 x 6 x 2

= 162 m^3

Submerged Load = Volume of pile cap * Density

= 162 x 24

= 3888 kN

Total Submerged Load = Submerged Load of pier +Submerged Load of pile cap

= 4149.88 +3888

= 8037.88 kN

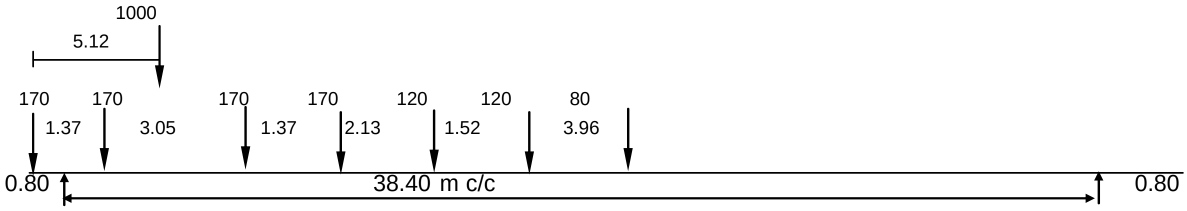
Buoyancy force is taken as 15% of submerged Load of pier and pile cap
(As per IRC-6, 2000 Cl. No. 213.5)

Buoyancy Force = 0.15 x 8037.88

=1205.68 kN

Live Load

A) 70 R (W) (a) (One Span disloged condition)



891.58kN

All loads are in kN and all distances are in meter

$$\text{Reaction @ left} = \frac{1000 \times (38.40 + 0.80 - 5.12)}{38.40}$$

$$= \frac{1000 \times 33.88}{38.40} = 891.58 \quad \text{kN}$$

Total live load Reaction = 891.58 kN

Longitudinal moment=Live Load reaction* eccentricity between bearing and center line of pier (fig. 5.11)

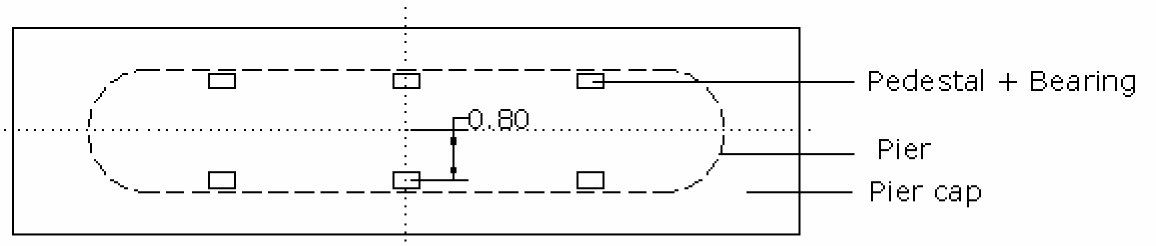


FIGURE 5.11 ECCENTRICCITY FOR LONGITUDINAL MOMENT

Longitudinal moment = 891.58 x 0.8

= 710.00 kNm

Maximum Transverse eccentricity=8.50/2 - 0.50 - 1.20 - 2.79 / 2

= 1.155 m (fig 5.12)

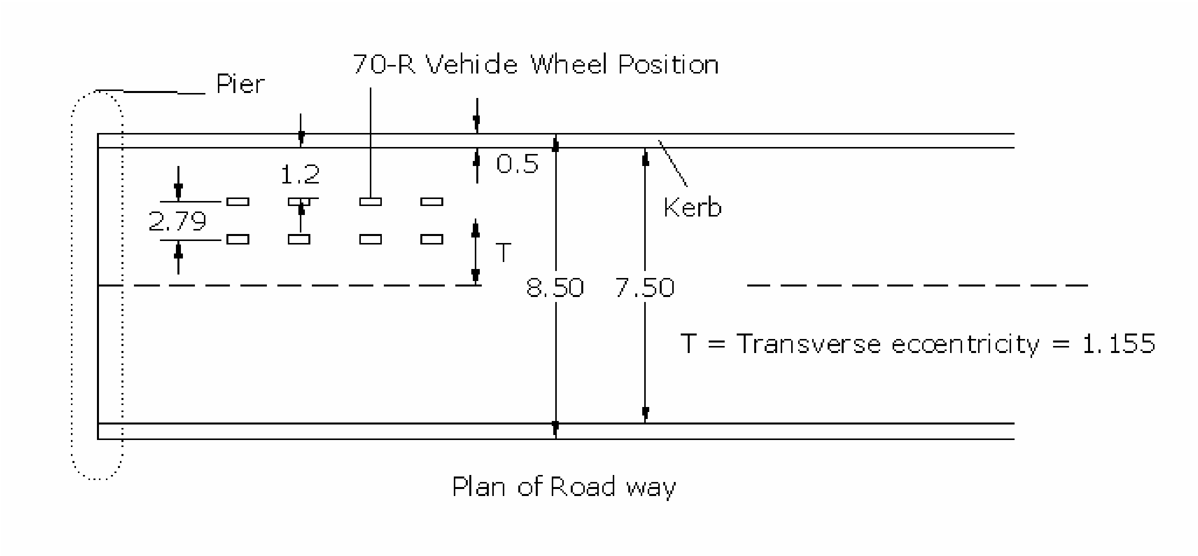


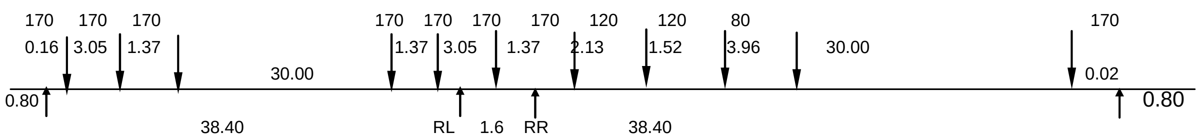
FIGURE 5.12 PLAN OF ROAD WAY FOR PILE

Transverse moment = Live Load reaction * Transverse eccentricity (fig.5.12)

= 891.58 x 1.155

= 1029.77 kN-m

70 R (W) (b) (Both span loaded condition)



All loads are in kN and all distances are in meter

RR = 536.04 kN

RL = 436.09 kN

Total Live load reaction = 972.13 kN

Longitudinal Moment = (536.04 - 436.09) + 170 x 0.8

= 270.00 kN-m

Maximum Transverse eccentricity = 8.50 / 2 - 0.50 - 1.20 - 2.79 / 2

= 1.155 m

Transverse Moment = Total live load reaction * transverse eccentricity

= 972.13 x 1.155

= 1122.81 kN-m

Impact Force

Impact Factor = 0.08 (As Per IRC - 6, 2000, Cl. No. 211.2)

Live load Reaction considering Impact = 0.08 x 972.13

= 77.77 kN

Live load Moment considering Impact along flow direction = 0.08 x 270.00

= 21.60 kN-m

Live load Moment considering Impact along traffic direction = 0.08 x 1122.8

= 89.82 kN-m

Longitudinal Forces

(A) Due to braking force

Total Load of Class 70 R wheeled vehicle = 1000 kN (As per IRC-6 Appendix 1))

Braking force is considered as 20% of total vehicle load

(As per IRC-6, 2000, cl.No 214.2)

Longitudinal force for class 70-R wheeled load = 0.2 x 1000

= 200 kN

Moment at base of pile cap due to braking force

=200 x 12.2

(Thickness of pier cap+Height of pier+Thickness of pile cap=1.2+9+2=12.2)

= 2440 kN-m

(B) Due to resistance in bearings to movement due to temperature

It is possible that the frictional coefficients of the two bearings on the pier may happen to be different due to unequal efficiency of the bearings.

Assume the live load to be on left span = 0.25 %

Total resistance by left side bearings =

= 0.25 * (Dead Load of superstructure of one side span + Live Load reaction at left with one span disloged condition)

= 0.25 (4480 +891.58)

= 1342.89 kN

Assume the frictional coefficients of bearings = 0.225

Total resistance by right side bearings = 0.225 * Dead load of superstructure of One side span

= 0.225 x 4480

= 1008 kN

Unbalanced force at bearing = 1342.89 - 1008

= 334.89kN

Moment at base due to bearing rigidity force

= 334.89 x 12.2

(Thickness of pier cap+Height of pier+Thickness of pile cap=1.2+9+2=12.2)

= 4085.71 kN-m

Wind Force

(A) Area of structure seen in elevation due to deck and handrails (to be computed from dimensions of superstructure of 40m span of MOST classification Fig. 3.5) = 146 m^2 (As Per section 3.5, chapter 3)

Assuming the average height of exposed surface above the bed level to be 10 m.

Intensity of wind load = 0.91kN/m^2 (As per IRC- 6, 2000, Table - 4)

Total wind force = 146 x 0.91

= 132.86 kN

(B) Wind force against moving load, considering Class A train

(Class A train has highest length, 20.4m, so it is considered for worst case)

Length of Class A train = 20.4m

Lateral wind force =3 kN/m (As Per IRC-6 cl. 212.4)

Wind force against moving load = 20.4 x 3

= 61.2 kN

(C) Total wind force as in (A) and (B) = 132.86 + 61.2 = 194.06 kN

(D) Minimum limiting force on deck at 4.5 kN/m (As per IRC-6 Cl.212.6)

= span * minimum limiting force in kN/m

= 40 x 4.5

= 180 kN

(E) Minimum limiting force at 2.4kN/m^2 on exposed surface

(As per IRC-6 Cl.212.5)

= Exposed area of superstructure * minimum limiting force in kN/m^2

= 146 x 2.4

= 350.4 kN

Since the force in (E) is the maximum, this will be adopted. This force will be assumed to act at the bearing level for the purpose of calculating the moment at the base of the pier

Moment at base of pile cap due to wind force

= 350.4 x12.2

(Thickness of pier cap+Height of pier+Thickness of pile cap=1.2+9+2=12.2)

= 4274.88 kN-m

Water current Force

For Pier

Intensity of pressure = 0.5 K V^2 kN/m^2 (As Per IRC-6, 2000,Cl. No 213.2)

Where K = a constant having the value for different shapes of piers.

If circular pier or pier with semicircular ends

then K =0.66 (From IRC-6, 2000, Cl.No. 213.2)

V = Maximum mean velocity of current

=3.6m/sec

Intensity of pressure = 0.5 x 0.66 x 3.6 x 3.6

= 4.3 kN/m^2

Force due to water current=Exposed area in flow direction*Intensity of pressure

Force due to water current = {1.8 + 2.7} / 2x 9 x 4.3 (see fig. 5.5)

= 87.08 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base = 87.075 x 7.4 (5.4m + thickness of pile cap(2m))

= 644.36 kN-m

Water current with obliquity at 20°

Pressure parallel to pier = 4.3 x COS20°

= 4 kN/m^2

Pressure perpendicular to pier =4.3 x SIN20°

= 1.5 kN/m^2

Force on pier along flow direction=Exposed area in flow direction* Intensity of pressure parallel to pier

= {1.8 + 2.7} / 2 x 9.00x 4 (see fig. 5.5)

= 81 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base = 81 x 7.40 (5.4m + thickness of pile cap(2m))

= 599.4 kN-m

Force on pier along traffic direction = Exposed area in traffic direction* Intensity of pressure perpendicular to pier

= 7.5 + {1.8 + 2.7 / 2} x 9.00 x 1.5 (see fig. 5.5)

= 131.625 kN

This force acts at 2 / 3 of height from base of pier to H.F.L. level

= 2 / 3 x 8.1

= 5.4 m above the base of pier

Moment at base =131.625x 7.40 (5.4m + thickness of pilecap(2m))

= 974.025 kN-m

For Pile cap

Intensity of pressure = 0.5 K V^2 kN/m^2 (As Per IRC-6, 2000,Cl. No 213.2)

Where K = a constant having the value for different shapes of Pile caps

If square ended pile caps

then K =1.5 (From IRC-6, 2000, Cl.No. 213.2)

V = Maximum mean velocity of current

=3.6m/sec

Intensity of pressure = 0.5 x 1.5 x 3.6 x 3.6

= 9.72 kN/m^2

Force due to water current

= 6 x 2 x 9.72 (size of Pile cap in that direction is 6 x2m)

= 116.64 kN

Force is act at half of the thickness of pile cap=2/ 2 (Thickness of pile cap is 2m)

= 1 m

Moment at base of pile cap = 116.64 x1

= 116.64 kN-m

Water current with obliquity at 20°

Pressure parallel to pier= 9.72 x COS 20°

= 9.13 kN/m^2

Pressure perpendicular to pier = 9.72 x SIN 20°

= 3.33 kN/m^2

Force on pilecap along flow direction

= 6 x 2 x 9.13 (size of Pile cap in that direction is 6 x2m)

= 109.60 kN

Force is act at half of the thickness of pilecap =2 /2 (Thickness of pile cap is 2m)

= 1 m

Moment at base of pile cap along flow direction

= 109.60 x 1

= 109.60 kN-m

Force on pier along traffic direction

= 13.5 x 2 x 3.33 (size of Pilecap in that direction is 13.5 x2m)

= 89.91

Force is act at half of the thickness of pilecap=2 / 2 (Thickness of pile cap is 2m)

= 1 m

Moment at base of pile cap along traffic direction

= 89.91 x 1

= 89.91kN-m

Total Force along flow direction = 81.00 + 109.60 = 191.00 kN

Total Force along traffic direction = 131.62 + 89.91 =221.60 kN

Total Moment along flow direction = 599.4 + 109.60 = 710.00 kN-m

Total Moment along traffic direction = 974.025 + 89.91 = 1064.00 kN-m

Seismic Force

Horizontal Seismic Force in traffic direction

F_{eq} = Ah x G

$$Ah = \frac{(Z/2)\,(Sa/g)}{(R/I)}$$

Where Z = Zone factor as given in table 5 of IRC 6

Zone= 3 , Z= 0.16

I = Importance factor

For important bridge = 1.5

Other bridges = 1.0

R = Response reduction factor = 2.5 (As per IRC- 6, Cl. No. 222)

Sa/g = Average response acceleration coefficient

= 2.5 (As per IRC- 6, Cl. No. 222)

G = Vertical Load (Self weight) of components

$$Ah = \frac{\{0.16 / 2 \times 2.5\}}{\{2.5 / 1.5\}}$$

= 0.12

F_{eq} = 0.12 x G

TABLE 5.3 SEISMIC FORCE AND MOMENT ON PILE

Components	Vertical Load (Self-Weight) kN G	Horizontal Seismic Force kN 0.12*G	Lever Arm (m) From base of Pier	Bending Moment due to Horizontal Seismic Force kN-m
Superstructure	8960.00	1075.20	10.80	11612.16
Pier cap	1035.00	124.20	9.60	1192.32
Pier	4726.81	567.22	4.20	2382.31
Pile cap	3888.00	466.56	1.00	466.56
Total		**2233.18**		**19186.58**

Table 5.3 gives Horizontal Seismic force and Moments for both span loaded conditions

One span dislodged condition

For one span disloged condition superstructure Load and moments becomes half (For load combination at construction stage As per IRC-6, 2000, Table 1)

Total Horizontal Force = 1075.20/ 2 +124.20 + 567.22 + 466.56

= 1695.57 kN

Total B.M. at base of Pile cap = 13762.56 / 2 + 1440.72 + 3516.74 + 466.56

= 12305.30 kN-m

5.14.2 Summary of Forces on pile

Summary of all calculated forces are shown in table 5.4

TABLE 5.4 SUMMARY OF FORCES ON PILE

SR NO	LOAD	VERTICAL LOAD kN	ALONG FLOW HT kN	ALONG FLOW MT kN-m	ALONG TRAFFIC HL kN	ALONG TRAFFIC ML kN-m
01	Dead load					
	a) One Span dislodged cond	14129.81				3584.00
	b) bothside spans	18609.81				
02	Buoyancy Force	1205.68				
03	Live load					
	a) One Span disloged	891.58		710.00		1029.77
	b) Both Span Loaded	972.13		270.0		1122.81
04	Impact Force	77.77		89.8		21.60
05	Longitudinal Forces					
	a) Braking Effect				200	2440.00
	b) Bearing Rigidity Force				333.88	4073.28
06	Wind Force		350.40	4274.88		
07	Water Current					
	(with 20 obliqu.)		191.00	710.00	221.60	1064.00
08	Seismic forces					
	a) One side span dislodged				1695.58	12305.30
	b) Both side span				2233.18	19186.58

Where,

HT = Horizontal Load in Transverse direction

HL = Horizontal Load in Longitudinal direction

MT = Moment in Transverse direction

ML = Moment in Longitudinal direction

5.14.3 Load Combinations for pile

Load combinations is done as per IRC-6, 2000, Table - 1

According to IRC-6, 2000, Load Combinations are done at construction condition and at service condition.

Construction Condition

(1) Dead Load + Wind Load + Water current + Bearing rigidity + Buoyancy

P = 14129.81-1205.68 =12924.12 kN

HT = 350.40 + 191.00 = 541.40 kN

HL = 334.89 + 221.60 = 556.49 kN

MT = 4274.88 + 710.00 = 4984.88 kN-m

ML =3584.00 + 4085.72+ 1064.00 =8721.72 kN-m

(2) Dead Load + Water current force + Bearing rigidity force + Buoyancy + (0.5) Seismic Force

P = 14129.81 - 1205.68 = 12924.12 kN

HT =191.00 kN

HL = 334.89 + 221.60 + (0.5) 1695.58 = 1404.28 kN

MT = 710.00 kN-m

ML = 3584.00 + 1064.00 + 4085.72 +(0.5) 12305.30 = 14873.37 kN-m

Service Condition

(3) Dead Load + Live Load + Vehicle Impact Force + Water current Force + Braking Force + Bearing rigidity Force + Buoyancy Force

P = 18609.81 + 972.13 + 77.77 - 1205.68 = 18454.02 kN

HT = 191.00 kN

HL = 200.00 + 334.89 + 221.60 = 756.49 kN

MT =270.00 + 710.00 + 89.82 = 1069.82 kN-m

ML = 1122.81 + 2440.00 + 4085.72 +1064.00 + 21.60 = 8734.13 kN-m

(4) Dead Load + Live Load + Vehicle Impact Force + Wind Force + Water current Force + Braking Force + Bearing rigidity Force + Buoyancy Force

P = 18609.81 + 972.13 + 77.77 - 1205.68 = 18454.02 kN

HT =350.40 + 191.00 =541.40 kN

HL =200.00 + 334.89+ 221.60 = 756.49 kN

MT = 270.00+ 4274.88 +710.00 + 89.82 = 5344.70 kN-m

ML =1122.81 + 2440.00 + 4085.72 + 1064.00 + 21.60 = 8734.13 kN-m

(5) Dead Load + (0.5) Live Load + (0.5) Impact Force + water current Force + (0.5) Braking Force + (0.5) Bearing rigidity Force + Buoyancy Force + Seismic Force

P= 18609.81 + (0.5) 972.13 + (0.5) 77.77 - 1205.68 = 17929.07 kN

HT = 191.00 kN

HL = (0.5) 200.00 + (0.5) 334.89 + 221.60 + 2233.18 =2722.22 kN

MT = (0.5) 270.00 + (0.5) 89.82 + 710.00 = 889.91 kN-m

ML = (0.5) 1122.81 + (0.5) 2440.00 + (0.5) 4085.72+1064.00
+19186.58+(0.5) 21.60 = 24085.65 kN-m

5.14.4 Load carrying capacity of a pile

Assume the diameter of pile = 1.2 m

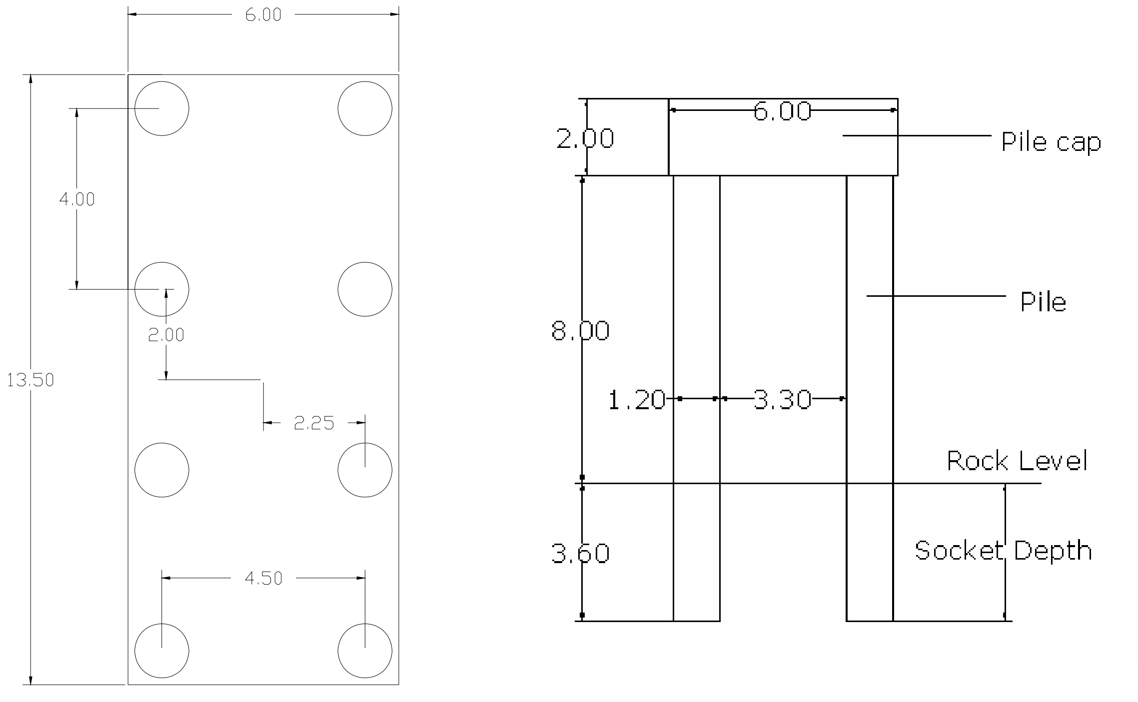
No. of Pile = 8

Types of pile = End Bearing piles, Cast-in-situ circular piles

Height of pile cap from rock level = 8 m (fig.5.10)

Socket level is taken as 3 * D = 3 x 1.2 = 3.6 m (fig.5.13)

Total Height of pile = 8 + 3.6 = 11.6 m (fig.5.13)



All dimensions are in meter

FIGURE 5.13 PILE ARRANGEMENT

Capacity of pile in rock is given by following equation

$$Q_a = R_e + R_{ef}$$

$$Q_a = k_{sp} * q_c * d_f * A_b + A_s * q_s$$

Where Q_a = Ultimate capacity of pile, socketed in rock

R_e = Ultimate end bearing

R_{ef} = Ultimate side socket shear

k_{sp} = An empirical co-efficient whose value ranges from 0.1 to 0.4 = 0.3

q_c = Average uniaxial compressive strength of rock at tip level

= 6411 kN/m^2

d f = Depth factor = (1 + 0.4 * (Length of socket / Dia of Socket)

Where Length of socket= 3.6 m

Diameter of socket = 1.2 m

d f = (1 + 0.4 * (3.6 / 1.2) = 2.2 m

A_b = Cross-sectional area of base of pile

= $(\Pi /4) * D^2 = 1.13\ m^2$

A_s = Surface area of socket.

= $\Pi (D) * L = 3.14 \times 1.2 \times 3.6 = 13.56\ m^2$

q_s = Ultimate shear along the socket, value of qs may be taken as 50 kg/cm^2 for normal rock and 20 kg/cm^2 for weathered rock

= 2000 kN/m^2 (For weathered rock)

$R_e = k_{sp} * q_c * d_f * A_b$

= 0.3 x 6411 x 2.2 x 1.13

= 4781.32 kN

Factor of safety for Re (Ultimate end bearing) is 5

(As Per IRC-78, Cl. No. 709.3.2)

R_e = 4781.32 / 5 = 956.26 kN

R_{ef} = A s * q s

= 13.56 x 2000 = 27129.6 kN

Factor of safety for Ref (Ultimate side socket shear) is 10

(As Per IRC-78, Cl. No. 709.3.2)

R_{ef} = 27129.6 / 10 = 2712.96 kN

Q_a = 956.26 + 2712.96 = 3669.22 kN

5.14.5 Loads and Moments at base of pile cap

Load combination 5 is governing case, so from load combination 5

P = 17929.07 kN

MT = 889.91 kN-m

ML = 24085.65 kN-m

From equation 5.1

$P_{max} = \left(\frac{P}{n}\right) + \left(\frac{ML * 2.25}{8 * 2.25^2}\right) + \left(\frac{MT * 6}{4 * 6^2 + 4 * 2^2}\right)$ n is no. of pile = 8

= 2066.13 + 1098.09 + 33.37

= 3197.60 kN

P_{min} = 2066.13 - 1098.09 - 33.37

= 934.67 kN

Self weight of Pile = $\frac{\pi}{4} * D^2 * L *$ Density

= (3.14/4) x 1.2 ^{2} x 11.6 x 24

= 314.86

Total Load = P_{max} + Self-weight of pile

= 3197.60 + 314.86

= 3512.46 kN < 3669.22 kN (ultimate capacity of pile Qa)……….O.K.

5.14.6 Design Moments for pile

Load combination 5 is governing, so from load combination 5

HL = 2722.22 kN

HT = 191.00 kN

Moment along Longitudinal direction

Horizontal Load per pile = 2722.22 / No. of pile No. of Pile is 8

=340.28 kN

Moment = $w * L/2$ = 340.28 x 8 / 2 (L is length of pile = 8m)

= 1361.11 kN-m

Moment due to water current

Intensity of pressure = 0.5 * K * V^2 (As Per IRC-6, 2000,Cl. No 213.2)

Where k = 0.66, for circular pile

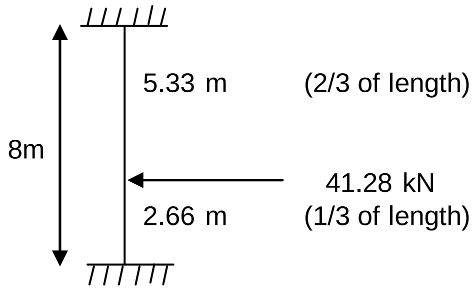
= 0.5 x 0.66 x 3.6 x 3.6

= 4.3 kN/m^2

Force due to water current = Intensity of pressure * Dia of pile * Length of pile

= 4.3 x 1.2 x 8

= 41.28 kN



Moment at fixity level = 41.28 x 2.66 x 5.33 ^{2} / 8 ^{2} 5.33 m (2/3 of length)

= 48.74 kN-m

Total Moment due to water current and horizontal force

= Moment due to water current + Moment due to horizontal force

= 1361.11 + 48.74

= 1409.85 kN-m

Moment Along Transverse direction

HT = 191.00 kN

Horizontal Load per = 191.00 / No. of pile No. of Pile is 8

= 23.88 kN

Moment = $w*L/2$ = 23.88 x 8 / 2 (L is length of pile = 8m)

= 95.5 kN-m

Moment due to water current

Moment due to water current in transverse direction is same as moment due to water current in longitudinal direction.

So, Moment due to water current = 48.74 kN-m

Total Moment due to water current and horizontal force

= Moment due to water current + Moment due to horizontal force

= 95.5 + 48.74

= 144.24 kN-m

Final $\boxed{\begin{array}{l} ML = 1409.85 kN\text{-}m \\ MT = 144.24 kN\text{-}m \end{array}}$ ………………..Design Moments

5.14.7 Design of reinforcement of Pile

P = 3512.46 kN D = 1.20 m

MT = 144.24 kN-m d' = 0.10 m

ML = 1409.85 kN-m f_{ck} = 25 N/mm^2

P_u = 5268.69 kN f_y = 415 N/mm^2

MT_u(ultimate moment in transverse direction) =1.5 * 144.24 =216.36 kN-m

ML_u(ultimate moment in longitudinal direction) =1.5*1409.85=2114.78 kN-m

M_u(ultimate resultant moment) = 2125.82 kN-m

As a first trial assume the reinforcement percentage, p = 0.40

Moment capacity of the section about flow direction

p/f_{ck} = 0.012 d'/D = 0.08 Pu / f_{ck} * D^2 = 0.146

Reffering chart no. 43 (SP - 16)

M_u/ f_{ck} * D^3 = 0.038 M_{ux1} = 1641.60 kN-m < 2125.82 kN-m

Increase percentage of steel

Take p = 0.70 p/f_{ck} = 0.028

d'/D = 0.08 P_u / f_{ck} * D^2 = 0.146

Reffering chart no 43 (SP - 16)

M_u/ f_{ck} * D^3 = 0.054 M_{ux1} = 2332.80 kN-m > 2125.82 kN-m

Calculation of Puz:

Reffering chart 63 of SP-16 corrosponding to

p = 0.60 fy = 415 N/mm^2 fck = 25 N/mm^2

so, P_{uz}/ Ag = 12.00 N/mm^2

= 12000.00 kN/m^2

so, P_{uz} = 12000.00 x 1.20 x 1.20

= 17280.00 kN

P_u/P_{uz} = 0.30 ML/M_{ux1} = 0.91 MT/M_{uy1} = 0.09

Referring to Chart 64,

Corresponding to the above values of MT/Mux1 and P_u/P_{uz} , the permissible values of M_{ux}/M_{ux1} = 0.95 > 0.91

Hence the section is o.k.

Reinforcement of one pile

So, Area of steel required = p * (Π/4) *D^2 /100

= 0.0079 m^2

= 7916.81 mm^2

Provide 10 nos of 32 mm dia bars

Ast Provided = 8042.48 mm^2 > 7916.81 mm^2

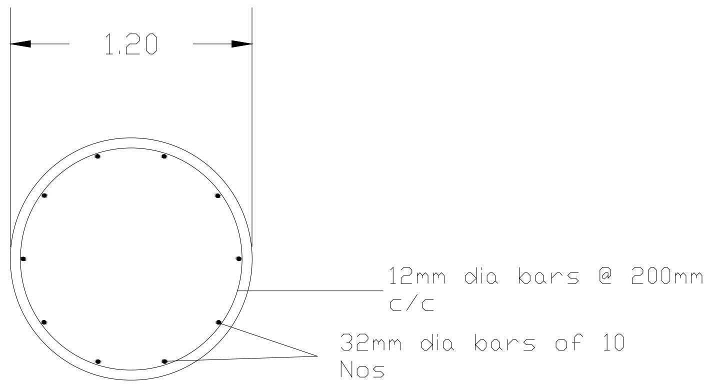
Stirrups area = 0.04% (As per IRC -78,2000, Cl.No. 710.3.3)

= 0.04 * (Π/4) *D^2 /100

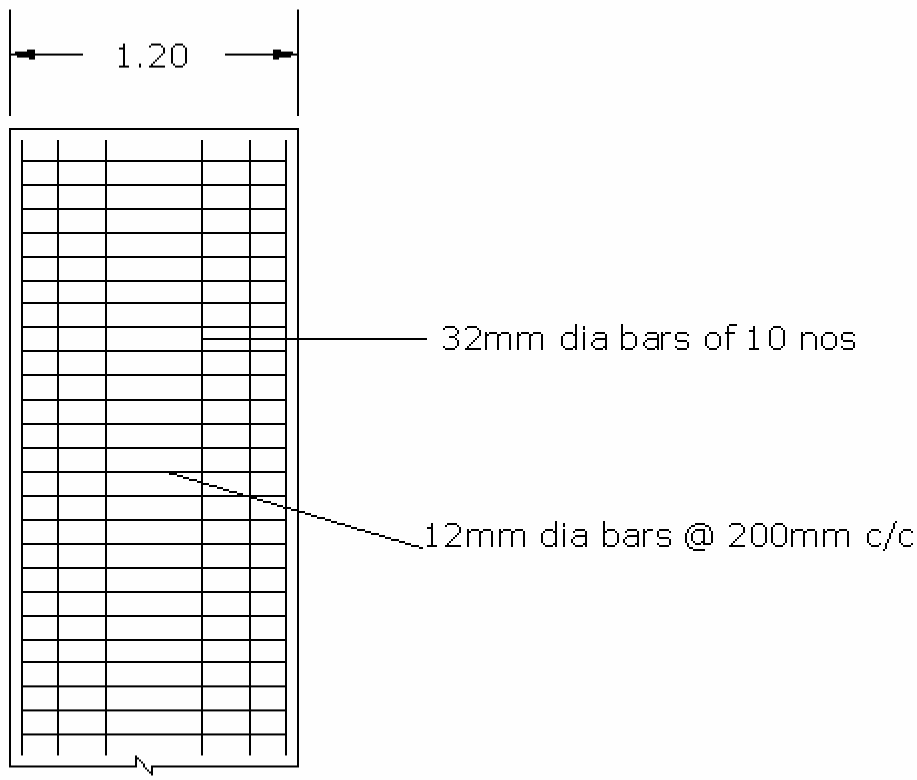
= 452 mm^2

Provide 12mm dia. @200 mm c/c

Reinforcement details of pile are shown in fig. 5.14



Plan of individual pile



Cross section of pile

FIGURE 5.14 REINFORCEMENT DETAILS OF ONE PILE

6. ABUTMENT COMPONENTS

6.1 General

Abutments are end supports to the superstructure of a bridge and they retain earth on their back side which serves as an approach to the bridge. In the case of river bridge, the abutment also protects the embankment from scour of the stream. Abutments are generally built using solid stone, brick masonry or concrete.

6.2 Types of Abutment:

Counterfort type Abutment:

Counterfort type abutments are closed type abutments having some counterforts connected by a face slab in front. The spacing of the counterforts is generally 2.5m to 3m. The stability of abutments is maintained by the self weight and the weight of the back fill materials in between the counterforts and above the foundation. Open raft or pile or well foundation is suitable for this sort of abutments.

Wall type Abutment:

This is a most common type of abutment and used for 4m to 10m height. In wall type abutment a wall with narrower width in traffic direction and higher width in flow direction are provided. The main force acting on wall type abutment is earth pressure acting on traffic direction.

6.3 Components of Abutment:

An abutment generally consists of the following components:

- Abutment wall
- Abutment cap
- Side wall of abutment
- Dirt wall

6.4 Abutment wall:

The height of abutment wall should be almost same as the height of pier. The width of abutment wall in traffic direction is narrower than the width of abutment wall in flow direction.

6.4.1 Forces on Abutment wall:

The forces considered on abutment wall are as mentioned below:

- Dead load of superstructure, abutment cap and dirt wall
- Self-weight of abutment wall
- Live Load
- Braking force
- Earth pressure
- Seismic force

Dead Load:
Dead Load on abutment wall includes dead load of abutment cap, dirt wall and superstructure. Self weight of abutment wall is also considered in dead load.

Live Load:
Live load can be calculated as explained in IRC-6 Cl. no.207. For worst condition, for two lanes, one lane of Class 70R should be considered.

Braking force:
The braking effect on a simply supported span or on continuous unit of spans or on any other type of bridge unit shall be assumed to have the following values:

(a) Single lane or two lane bridge:

20 percent of the first train load plus 10 percent of the load of the succeeding trains or part thereof, the train loads in one lane only being considered for the purposes. Where the entire first train is not on the full span, the braking force shall be taken as equal to 20 percent of the loads actually on the span

(b) More than two lane:

As in (a) above for the first two lanes plus 5 percent of the loads on the lanes in excess of two.

Earth Pressure:

Structures designed to retain earth fills shall be proportioned to withstand pressure calculated in accordance with any rational theory. Coulomb's theory shall be accepted subjected to modification that the centre of pressure exerted by the backfill, when considered dry, is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of that height. All abutments and sometimes piers shall be designed for a live load surcharge equivalent to 1.2m earth fill. (As per IRC-6, 2000, cl.no. 217.1)

Seismic force:

Bridges in seismic zones II and III need not be designed for seismic force provided both conditions are met, first one is span less than 15m and second is total bridge length is less than 60m. All ether bridge shall be designed for seismic force. For the purpose of determining the seismic force, the country is classified into four zones. The horizontal seismic forces to be resisted shall be computed as follows.

$F_{eq} = A_h$ * (Dead load).

Where F_{eq} = Seismic force to resisted.

A_h = horizontal seismic coefficient

$$= \left(\frac{Z * I * Sa}{2 * R * g} \right)$$

Z = zone factor as given in table 5 of IRC-6 -2000.

I = Importance factor, for important bridges…..1.5

For other bridges ………1.0

T = Fundamental period of the bridge member for Horizontal vibration

R = response reduction factor

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

6.4.2 Load Combinations:

For abutment wall following two conditions are taken for load combination (As per IRC-6, 2000, table no 1):

(1) Normal Condition

In normal condition following combination is considered

Dead Load + Live Load + Braking force + Earth pressure

(2) Seismic Condition

In seismic condition Live Load and Braking force are considered half

Dead Load + (0.5) Live Load + (0.5) Braking force + Earth pressure + Seismic force

When the ratio of moment due to normal condition and moment due to seismic condition is less than 1.5 the normal condition combination governs and if that ratio is more than 1.5 the seismic condition governs.

6.4.3 Design of Abutment wall:

Abutment wall is considered as column, so it should be designed as column with biaxial bending. The minimum reinforcement required for abutment wall is 0.3% of cross-section area at base.

6.5 Abutment Cap:

Abutment caps should be suitably designed and reinforced to take care of concentrated point loads dispersing in abutment. In case bearings are placed centrally over the abutment and the width of bearing is located within the abutment wall, the load form bearings will be considered to have been directly transferred to columns and the cap need not be designed for flexure. When the distance between the load/center line of bearing from the face of the support is equal to or less than the depth of the cap, the cap shall be designed as corbel.

6.5.1 Design of corbel portion:

A corbel is a short cantilever projection which supports a load bearing member. For the design of corbel portion the following step should be followed:

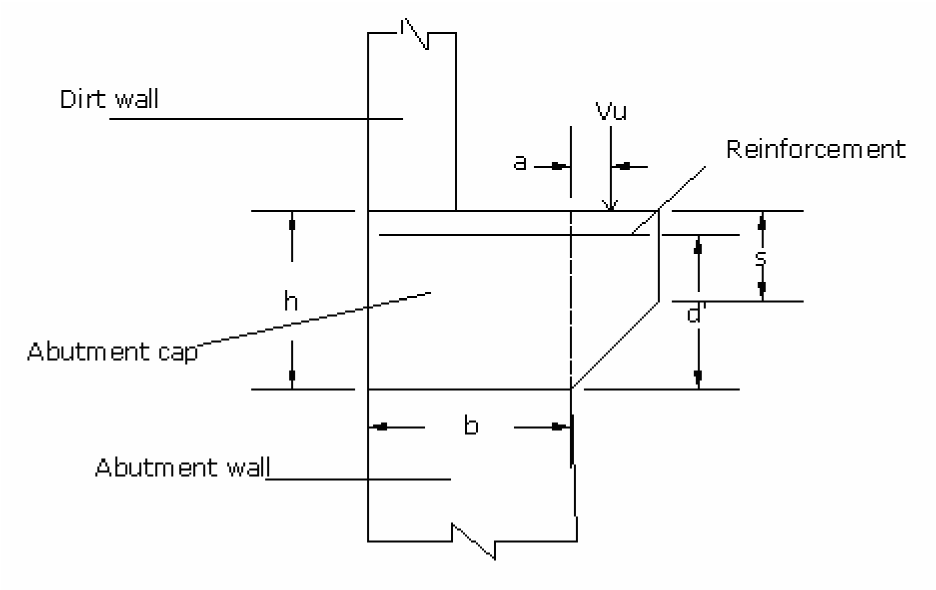


FIGURE 6.1 CORBEL PORTION OF ABUTMENT CAP

Step 1: Ensure a/d’ ≤ 1

Ensure s/d’ ≥ 0.5

Step 2: $\frac{Vu}{b*d} \leq 0.15*fc'$; otherwise revise section dimensions

Where, Vu = ultimate shear value

b = width of cantilever /corbel

fc’ = 28-day standard cylinder strength of concrete used.

d = 0.8 times effective depth (d = 0.8* d’)

s = Edge height of corbel

Step 3: Calculate shear friction reinforcement A_{vf}

$$A_{vf} = \frac{Vu}{0.85*f_{sy}*\mu}$$

Where f_{sy} = yield stress value of the reinforcement used

μ = 1.4 for concrete placed monolithically across interface

1.0 for concrete placed against hardened concrete but with roughened surface

0.7 for concrete anchored to structural steel

0.6 for concrete placed against hardened concrete but with surface not Roughened

Step 4: Calculate direct-tension reinforcement A_t

$$A_t = \frac{Hu}{0.85 * f_{sy}}$$

Where, Hu = 1.7 * actual horizontal force but Hu > 0.2 * Vu

Step 5: Calculate flexural-tension reinforcement A_f

$$A_f = \frac{[V_u * a + H_u * (h - d')]}{0.85 * f_{sy} * d}$$

Step 6: Compute total primary tensile reinforcement A_s

$$\left.\begin{aligned} A_s &\geq (A_f + A_t) \\ &\geq (\frac{2}{3} * A_{vf} + A_t) \\ &\geq (0.04 * fc' / f_{sy}) * b * d' \end{aligned}\right\} \text{Provide largest of these three magnitudes as } A_s$$

Step 7: Calculate total section area A_h of stirrups to be provided horizontally, one below other, below and next to A_s. A_h > 0.5* A_f and > 0.333* A_{vf} provide larger of the two. These stirrups shall be provided below A_s and within a depth of 2/3 d' below A_s

Step 8: Calculate vertical shear reinforcement A_v

$$A_v = \frac{0.5 * (V_u - V_c) * p}{f_{sy} * d}$$

Where, p = the pitch

Vc = $\tau_c * b * d$

The reinforcement detail of corbel portion of abutment is shown in fig 6.2

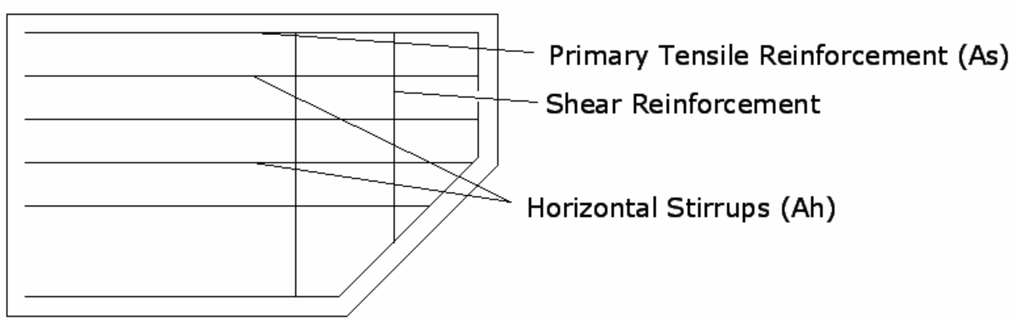


FIGURE 6.2 SKETCH OF REINFORCEMENT OF CORBEL PORTION

6.6 Side wall of Abutment:

The force coming on the side wall of abutment wall is Earth pressure. Earth pressure is calculated in accordance with any rational theory. Coulomb's theory shall be accepted subjected to modification that the centre of pressure exerted by the backfill, when considered dry, is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of that height. Side wall of abutment wall shall be designed for a live load surcharge equivalent to 1.2m earth fill. (As per IRC-6, 2000, cl.no. 217.1)

6.7 Dirt wall:

A dirt wall shall be provided to prevent the earth from approaches spilling on the bearings. Dirt wall is to be designed for self weight, Live load directly on dirt wall and braking force due to Live Load and Earth pressure.

6.8 Design of Abutment cap

Data

Depth of Abutment cap = 0.4 + 0.4 = 0.8 m

Width of Abutment cap at top = 1.3 m

Width of Abutment cap at bottom = 0.9 m

Width along flow direction = 8.58 m

Number of Bearings = Three

Size of Pedestal = 0.7 * 0.9 * 0.3m

Grade of concrete = M 30

Grade of steel = Fe415

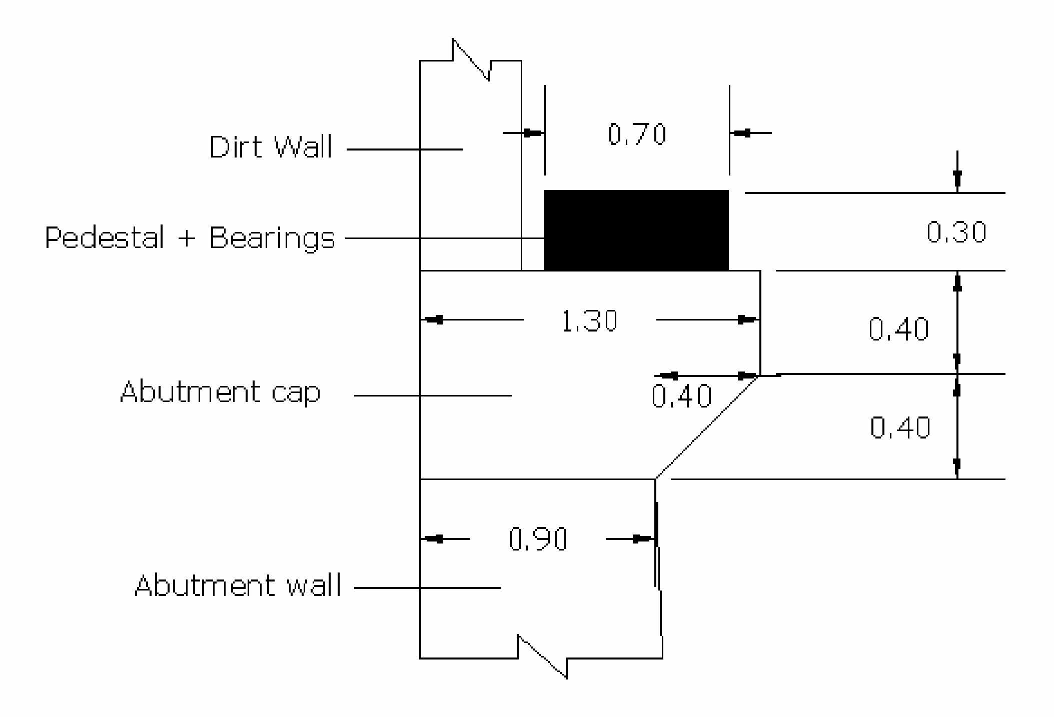


FIGURE 6.3 SECTION OF ABUTMENT CAP

Modification factor = 2.0 (As per IS-456, 2000, fig -4)

Span to effective depth ratio = 20 (As per IS-456, 2000, Cl. No. 23.2.1)

Span to effective depth ratio permissible = 2.0x 20 = 40

Depth required (d_{req}) = Width of abutment cap along flow direction / Span to effective depth ratio permissible

= 8.58 * 1000 / 40

= 214.5 mm < 800 mm (Depth Assumed) …..O.K

Adopting 0.12 % steel (As per IRC-21, 2000, Cl.No. 305.19)

Reinforcement required = 0.12 x 1000 x 800 / 100

= 960 mm^2 / m

Provide 16mm dia. Bars at 200mm c/c in both ways and in both layers.

Reinforcement details are shown in fig.6.5.

Design of Corbel Portion

The corbel portion is as shown in fig. 6.4

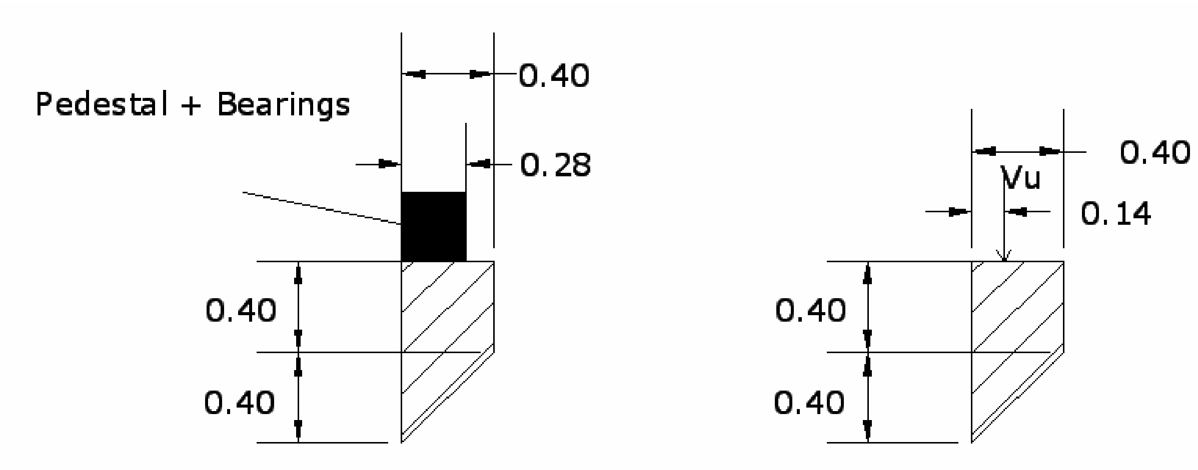


FIGURE 6.4 SECTION OF CORBEL PORTION

Loads on Corbel Portion

Dead Load on Corbel

Dead Load Vu = D.L. of girder + D.L. of pedestal + Self weight of corbel

Dead Load of girder = 4410 kN

Dead Load of girder per bearing = 4410 / No. of bearings

= 4410 / 3

= 1470 kN

Size of pedestal = 0.7 x 0.9 x 0.3m

Volume of Pedestal = 0.7 * 0.9 *0.3 = 0.189 m^3

Load of pedestal = Volume of Pedestal * Density

= 0.189 * 25

= 4.73 kN

Self weight of corbel = Volume of Corbel * Density of Concrete

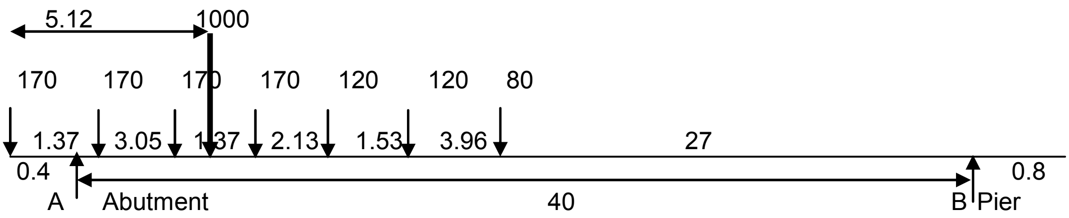
= 5.76 kN

Dead Load Vu = D.L. of girder + D.L. of pedestal + Self weight of corbel

= 1470 + 4.73 + 5.76

= 1480 kN

Live Load on corbel portion



RA = 882 kN

Live Load VL =882 kN

Total Load

Total Load = Dead Load +Live Load

= 1480 + 882

= 2362 kN

Total Load per meter on pedestal =Total Load/Width of pedestal in flow direction

= 2362 / 0.7

= 3374.97 kN/m

Load on corbel portion = Total load per meter on pedestal * Width of pedestal on corbel portion

= 3374.97 x 0.28

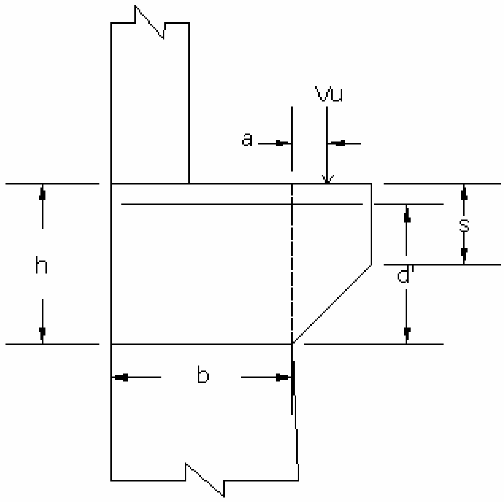
= 944.99 kN

Ultimate Load Vu = 944.99 * 1.5

= 1417.491 kN

Design of corbel

When the distance between the load/center line of bearing from the face of the support is equal to or less than the depth of the cap, the cap shall be designed as corbel. Here in this particular problem some part of bearing it self placed on cantilever projection. So it must be designed as Corbel portion.



a = 140 mm

h = 800 mm

d’ = 750 mm

b = 900 mm (Width of abutment)

s = 400 mm (edge height of corbel)

Vu = 1417 kN

fy = 415 N/mm^2

f'c = 30 N/mm^2

Step 1

Ensure a/ d’ should be less than 1 and s/d’ should be also less than 1

a / d' = 140 / 750 = 0.18 < 1 …….O.K

s / d' = 400 / 750 = 0.53 <1 …….O.K

Step 2

Ultimate Load Vu = 1417 kN = 1417000 N

d = 0.8 * d' = 0.8 * 750 = 600 mm

b = 900 mm

Vu / b * d = 1417000 / (600 * 900) = 2.62 N/mm^2

0.15 * f'c = 0.15 * 30 = 4.5 N/mm^2 > 2.62 N/mm^2 ……..O.K.

Step 3 Shear friction reinforcement

$$\text{Shear friction Reinforcement } (A_{vf}) = \frac{Vu}{0.85 * f_{sy} * \mu}$$

Where, μ = 1.4 for normal concrete

f_{sy} = 415 N/mm^2

A_{vf}= 1417000 / (0.85 * 415 * 1.4) = 2869.29 mm^2

Step 4 Direct - tension reinforcement

$$\text{Direct-tension Reinforcement}(A_t) = \frac{Hu}{0.85 * f_{sy}}$$

Where Hu = 0.2 * Vu= 0.2 * 1417 = 283.4 kN

f_{sy} = 415 N/mm^2

A_t = 283400 / (0.85*415) = 804 mm^2

Step 5 Flexural tension reinforcement

$$\text{Flexure tension Reinforcement}(A_f) = \frac{[V_u * a + H_u * (h - d')]}{0.85 * f_{sy} * d}$$

Where, Vu = 1417000 N

Hu = 283400 N

a = 140 mm

h = 800 mm

d' = 750 mm

d = 600 mm

f_{sy} = 415 N/mm^2

$$A_f = \frac{[1417000 * 140 + 283400 * (800 - 750)]}{0.85 * 415 * 600}$$

A_f= 1004.25 mm^2

Step 6 Total Primary tensile reinforcement (A_s)

Total primary tensile reinforcement should greater of

(1) $A_f + A_t$

(2) 2/3 A_{vf} + A_t

(3) (0.04 f' c / fy) b * d'

(1) $A_f + A_t$ = 804 + 10004.25 = 1808.25 mm^2

(2) 2/3 A_{vf} + A_t = (2/3) * 2869.23 + 804 = 2716.86 mm^2

(3) (0.04 f' c / fy) b * d' = (0.04 * 30 / 415) * 900 * 750 = 1951.80 mm^2

Adopt A_s = 2716.86 mm^2

Provide 6 nos of 28mm dia bars

As provided = 3696 mm^2

Step 7 Total section areas of stirrups to be provided horizontally (A_h)

Total section area of stirrups to be provided horizontally should be greater of

(1) 0.5 * A f

(2) 0.333 * A v f

(1) $0.5 * A_f = 0.5 * 1004.25 = 503\ mm^2$

(2) $0.333 * A_{vf} = 0.333 * 2869.29 = 956\ mm^2$

Adopt $A_h = 956\ mm^2$

Provide 4 nos of 20 mm dia bars

A_h provided = 1256 mm^2

Step 8 Shear reinforcement

$$\text{Shear Reinforcement } A_v = \frac{0.5*(V_u - V_c)*p}{f_{sy}*d}$$

Where pt = As * 100 / b *d

$$= \left(\frac{3696 \times 100}{1000 \times 600}\right) = 0.616$$

$\tau_c = 0.54$ (As per IS-456, 2000 , Table - 19)

$V_c = \tau_c * b * d = 0.54 * 900 * 600 = 291600$ N

Assume p = 250 mm

$$A_v = \frac{0.5*(1417000 - 291600)*250}{415*600}$$

$A_v = 564.95\ mm^2$

Provide 4legged stirrups of 16 mm dia bars @ 250mm c/c

A_v provided = 804 mm^2

Reinforcement details of abutment cap are shown in fig.6.5

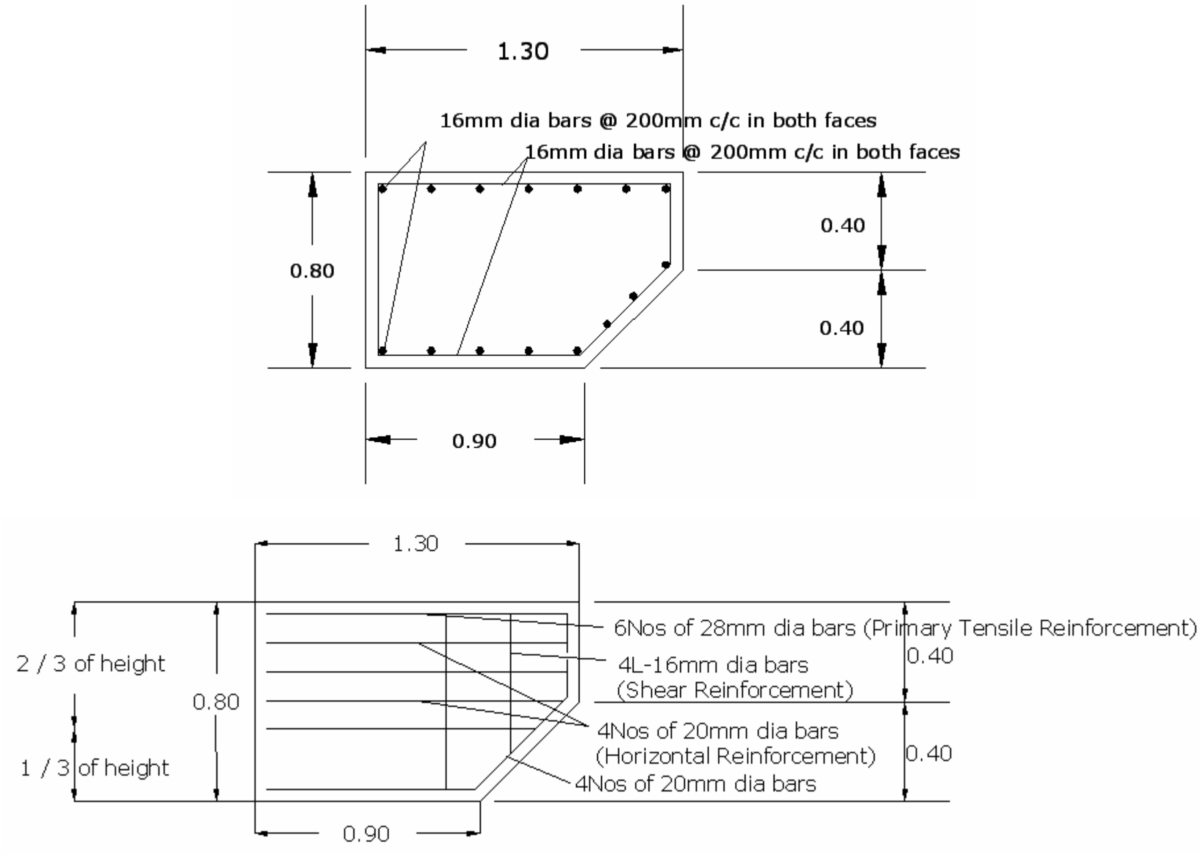


FIGURE 6.5 REINFORCEMENT DETAILS OF ABUTMENT CAP

6.9 Design of abutment wall

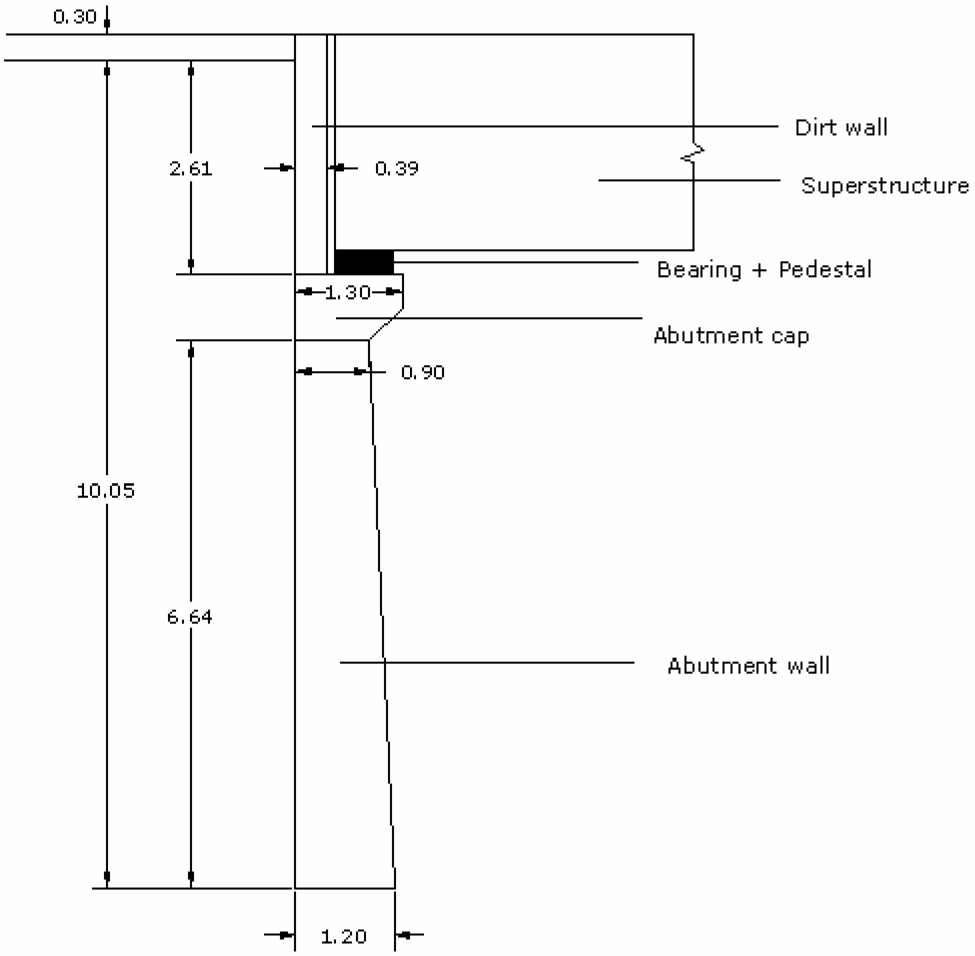
Design consists of determination of steel area for abutment wall which is R.C.C. solid wall type abutment having top width of 0.9 m and bottom width 1.2 m along traffic direction as shown in fig. 6.6

Data

Span of superstructure = 40 m

Width of abutment along flow direction =8.58 m

All other dimensions are as shown in fig.6.6



All dimensions are in meter

FIGURE 6.6 SECTION OF ABUTMENT WALL

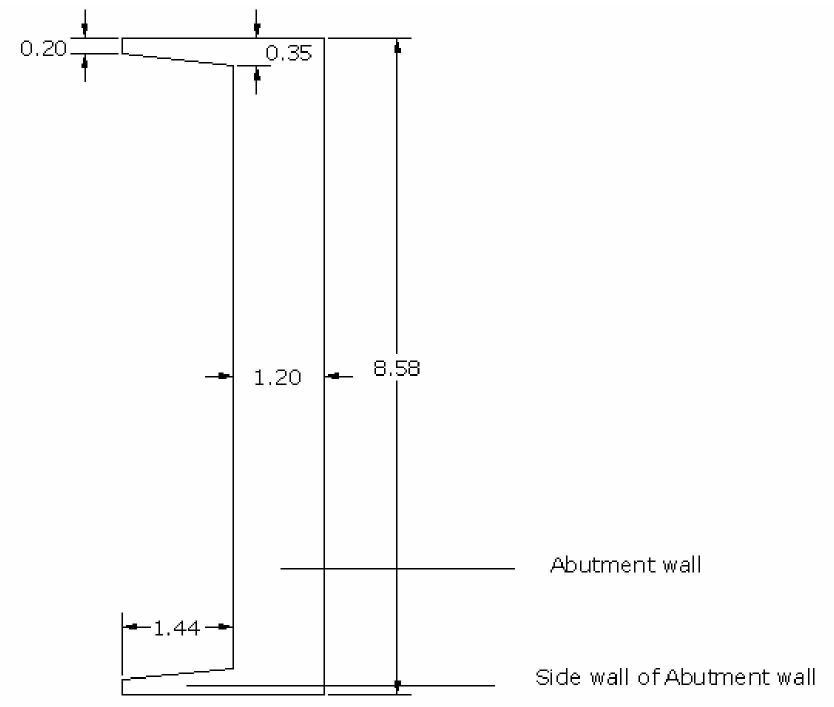


FIGURE 6.7 PLAN OF ABUTMENT WALL

Dead Load of superstructure

Dead load of superstructure for span 40 m = 8820 kN (Table 3.1)

Dead load on abutment =4410 kN (half the load on Abutment)

Eccentricity = 0.39 + 0.04 + 0.4 - 1.2 / 2

= 0.23 m (Eccentricity is calculated at center of base of Abutment)

Moment due to superstructure Mt = Dead Load * Eccentricity

= 4410 * 0.23

= 1014.3 kN-m

Dead Load of Substructure

Dirt wall

Height of dirt wall = 2.61 m

Thickness of dirt wall = 0.39 m

Volume of dirt wall = 2.61 * 0.39 * 8.58 = 8.73 m^3

Dead load of dirt wall = Volume of dirt wall * Density of concrete

= 8.73 * 24

= 209.60 kN

Eccentricity = 0.39 / 2 - 1.2 / 2 = -0.405 m

Moment due to dead load of dirt wall Mt = Dead Load * Eccentricity

= 209.60 * - 0.405

= -84.89 kN-m

Abutment cap

Rectangular portion

Height = 0.4 m

Width = 1.3 m

Volume of Abutment cap of rectangular portion = 0.4 * 1.3 * 8.58

= 4.46 m^3

Dead load of abutment cap of rectangular portion= Volume * Density of concrete

= 4.46 * 24

= 107.07 kN

Eccentricity = 1.3 / 2 - 1.2 / 2 = 0.05 m

Moment due to dead load of abutment cap (rectangular potion)

Mt = Dead Load * Eccentricity

= 107.07 * 0.05

= 5.35 kN-m

Trapezoidal portion

Height = 0.4 m

Width = (1.3 + 0.9) / 2 = 1.1 m

Volume of Abutment cap of trapezoidal portion = 0.4 * 1.1 * 8.58

= 3.77 m^3

Dead load of abutment cap of trapezoidal portion= Volume * Density of concrete

= 3.77 * 24

= 90.60 kN

Eccentricity = 1.1 / 2 - 1.2 / 2 = -0.05 m

Moment due to dead load of abutment cap (trapezoidal portion)

Mt = Dead Load * Eccentricity

=90.60 * -0.05

= -4.53 kN-m

Total Dead load = 107.07 + 90.60 = 197.68 kN

Total Moment due to dead load of abutment cap Mt = 5.35 – 4.53 = 0.82 kN-m

Abutment wall

Rectangular portion

Height = 6.64 m

Width = 0.9 m

Volume of Abutment Wall rectangular portion = 6.64 * 0.9 * 8.58

= 51.27 m^3

Dead load of abutment cap

=Volume of abutment wall of rectangular portion* Density of concrete

= 51.27 * 24

= 1230.57 kN

Eccentricity = 0.9 /2 - 1.2 / 2 = -0.15 m

Moment due to dead load of abutment wall rectangular potion

Mt = Dead Load * Eccentricity

= 1230.57 * -0.15

= -184.58 kN-m

Trapezoidal portion

Height = 6.64 m

Thickness = 0.15 m

Volume of Abutment Wall Trapezoidal portion = 6.64 * 0.15 * 8.58

= 8.54m^3

Dead load of abutment cap

= Volume of abutment wall of trapezoidal portion* Density of concrete

= 8.54 * 24

= 205.09kN

Eccentricity = 0.9 + (1.2-0.9) / 2 - 1.2 / 2 = 0.45 m

Moment due to dead load of abutment wall of trapezoidal portion

Mt = Dead Load * Eccentricity

= 205.09 * 0.45

= 92.29 kN-m

Approach slab

Height = 0.35 m

Thickness = 0.39 m

Volume of Approach slab = 0.35 * 0.39 * 8.58

= 1.17 m^3

Dead load of approach slab= Volume of approach slab* Density of concrete

= 1.17 * 24

= 28.10 kN

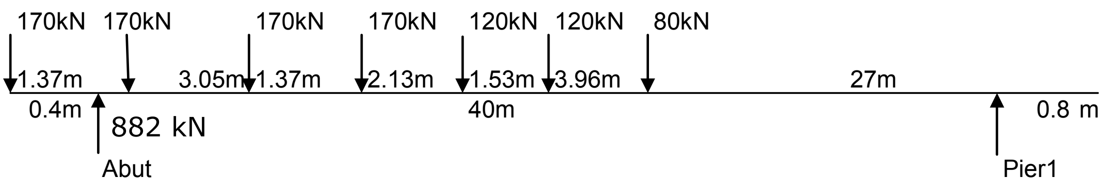
Eccentricity = 0.39 / 2 -1.2 / 2 = -0.405 m

Moment due to dead load of approach slab Mt = Dead Load * Eccentricity

=28.10 * -0.405

= -11.38 kN-m

Live Load on abutment wall



All loads are in kN and all distances are in meter

RA=1000(40+ 0.4 - 5.12)/ 40 = 882 kN

Eccentricity along traffic direction = 0.39 + 0.04 + 0.4 -1.2 / 2 = 0.23 m

Eccentricity across traffic direction = (7.5/2)-1.2-(2.79/2) = 1.155 m

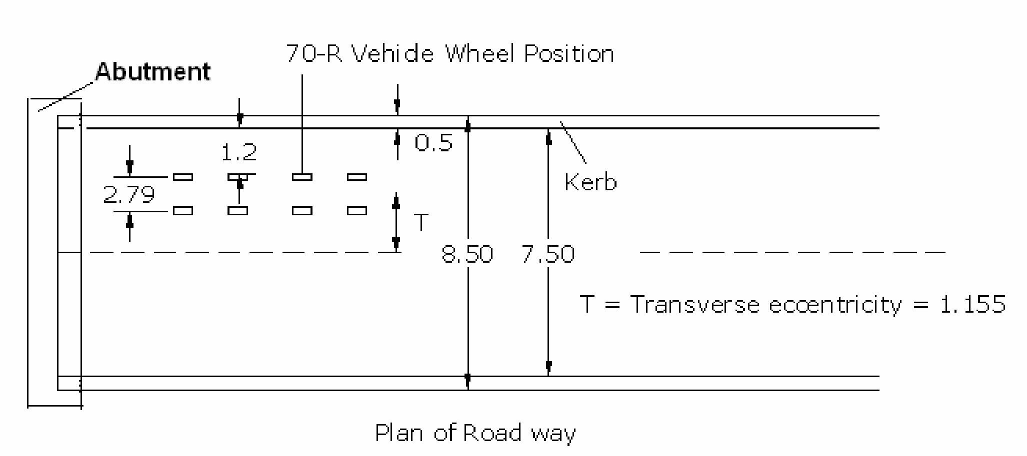


FIGURE 6.8 PLAN OF ROAD WAY

Where, Width of carriageway = 7.5m

Width of vehicle = 2.79m

Min. Edge of carriageway at end from where vehicle is provided = 1.2 m

Moment due to Live load along traffice direction = 882 x 0.23

= 202.86 kN-m

Moment due to Live load across traffic direction = 882 x 1.155

= 1018.71 kN-m

Braking Force

Total load of Class70-R wheeled vehicle = 1000 kN (As per IRC-6 Appendix 1))

Braking force is considered as 20% of Total vehicle load and it will act at bearing level (As per IRC-6, 2000, cl.No 214.2)

Longitudinal force for class 70-R wheeled load = 0.2 x 1000

= 200 kN

Eccentricity = 0.3 + 0.4 + 0.4 + 6.64 = 7.74m

Where, Height of Bearing + Pedestal = 0.3m

Height of Abutment cap = 0.4 + 0.4 =0.8m

Height of Abutment = 6.64m

Moment due to braking force Mt = Load * Eccentricity

= 200 * 7.74

= 1548 kN-m

Earth Pressure

Following soil parameters are considered of backfill material

$C = 0\ kN/m^2$

$\Phi = 30°$

$\delta = 20°$

Unit weight of soil $\gamma = 17\ kN/m^3$

Co-efficient of active earth pressure is given by:

$$k_a = \frac{\cos^2\Phi}{\left[1 + (\sin(\Phi+\delta)*\sin\Phi / \cos\delta)^{1/2}\right]^2}$$

$= 0.279$

Earth pressure intensity at road level

(As per IRC-78, 2000, Cl. No. 710.4.4 "All abutments shall be designed for a live load surcharge equivalent to 1.2 m height of earth fill)

$P_a = k_a * \gamma * q$ kN/m

Where Width of abutment wall = 8.58 m

width of side wall = 0.35m

q = height of earth fill * Width of Abutment – width of side wall of abutment

ka = 0.279

γ= 17 kN/m^3

P_a = 0.279 x 17 x 1.2 x (8.58 - 2 * 0.35)

P_a= Earth pressure intensity at road level = 44.91kN/m

Earth pressure intensity at base of abutment

P_a = k_a * γ * q kN/m

Where width of abutment wall = 8.58 m

width of side wall = 0.35m

q = height of earth fill * Width of Abutment – width of side wall of abutment

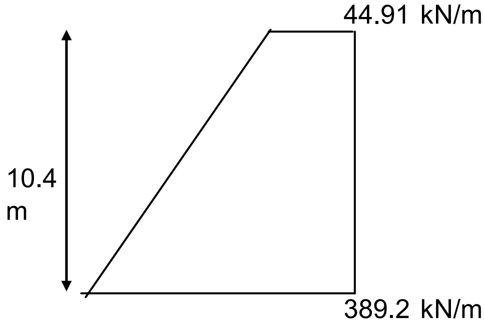
ka = 0.279

γ= 17 kN/m^3

P_a = Earth pressure intensity at base of abutment

= 0.279 x 17 x 10.4 x (8.58 - 2 * 0.35)

= 389.23 kN/m Where 10.4 is height from road level to base of abutment



Earth Pressure intensity Diagram

Now Load = Pressure * Height

Load due to earth pressure at road level

F_1 = 44.91 x 10.40 = 467.08 kN

Load due to earth pressure at base of abutment

F_2 =389.23 x 10.40 = 2024.01 kN

Eccentricity of load due to earth pressure at road level

E_1 = 10.40 / 2 = 5.20 m

Eccentricity of load due to earth pressure at base of abutment

E_2 = 0.42 x10.40 =4.37 m

Moment due to earth pressure at road level M_1 = Load * Eccentricity

= 5.20 x 467.08

= 2428.81 kN-m

Moment due to earth pressure at base of abutment M_2 = Load * Eccentricity

= 4.37 x 2024.01

= 8840.88 kN-m

Seismic force

Horizontal Seismic Force in traffic direction

F_{eq} = Ah x G

$$Ah = \frac{(Z/2)\ (Sa/g)}{(R/I)}$$

Where Z = Zone factor as given in table 5 of IRC 6

Zone= 3, Z= 0.16

I = Importance factor

For important bridge = 1.5

Other bridges = 1.0

R = Response reduction factor = 2.5 (As per IRC- 6, Cl. No. 222)

Sa/g = Average response acceleration coefficient

= 2.5 (As per IRC- 6, Cl. No. 222)

G = Vertical Load (Self weight) of components

$$Ah = \frac{\{0.16 / 2 \times 2.5\}}{\{2.5 / 1.5\}}$$

= 0.12

F_{eq} = 0.12 x G

TABLE 6.1 SEISMIC FORCE AND MOMENT ON ABUTMENT WALL

Sr No	Components	Dead Load(G)	Ah * G	Lever Arm	Moments	
		kN	kN	m	kN-m	
1	Super-structure	4410.00	529.20	8.77	4641.08	
2	Dirt wall	209.61	25.15	8.75	219.96	
3	Abutment cap(rect)	107.08	12.85	7.24	93.03	
4	Abutment cap(trap)	90.60	10.87	6.84	74.37	
5	Abutment wall(rect)	1230.58	147.67	3.32	490.26	
6	Abutment wall(trap)	205.10	24.61	2.21	54.47	
7	Approach slab	28.11	3.37	10.23	34.49	
8	Earth filling	2024.01	242.88	4.37	1060.91	
	Total		996.61	kN	6668.57	kN-m

Loads and moments at base of abutment wall(Normal condition) are shown in table 6.2

TABLE 6.2 LOADS AND MOMENTS AT BASE OF ABUTMENT WALL (NORMAL CONDITION)

Sr No	Description	Loads	L.A.	Moments(kN-m)		
		kN	m	MT	ML	
1	Dead load					
	Superstructure	4410.00	0.23		1014.30	
	Dirtwall	209.61	-0.41		-84.89	
	Abutment cap(rect)	107.08	0.05		5.35	
	Abutment cap(trap)	90.60	-0.05		-4.53	
	Abutment wall(rect)	1230.58	-0.15		-184.59	
	Abutment wall(trap)	205.10	0.45		92.29	
	Approach slab	28.11	-0.41		-11.38	
2	Live load	882.00	0.23	1018.71	202.86	
3	Braking force	200.00			1548.00	
4	Earth pressure					
	F1 = 467.08		5.20		2428.81	
	F2 = 2024.01		4.37		8840.88	
	Total	7363.07	kN	1018.71	13847.11	kN-m

Loads and moments at base of abutment wall (Seismic condition) are shown in table 6.3

In seismic condition live load and braking force is to be considered half (As per IRC-6 -2000 Load combinations, pg no 8)

TABLE 6.3 LOADS AND MOMENTS AT BASE OF ABUTMENT WALL (SEISMIC CONDITION)

Sr No	Description	Loads	L.A.	Moments(kN-m)	
		kN	m	MT	ML
1	Dead load				
	Superstructure	4410.00	0.23		1014.30
	Dirtwall	209.61	-0.41		-84.89
	Abutment cap(rect)	107.08	0.05		5.35
	Abutment cap(trap)	90.60	-0.05		-4.53
	Abutment wall(rect)	1230.58	-0.15		-184.59
	Abutment wall(trap)	205.10	0.45		92.29
	Approach slab	28.11	-0.41		-11.38
2	Live load	441.00	0.23	509.36	101.43
3	Braking force	100.00			774.00
4	Earth pressure				
	F1 = 467.08		5.20		2428.81
	F2 = 2024.01		4.37		8840.88
5	Seismic force				
	= 996.61				6668.57
	Total	6822.07	kN	509.36	19640.25 kN-m

Ratio of moment due to seismic condition and normal condition will decide which case will govern

(If Ratio is greater than 1.5 then seismic case govern and if ratio is less than 1.5 then normal case governs As per IRC-6-2000, Load combinations, pg no. 8)

Ratio = 19640.24 / 13847.10 = 1.42

So, Normal case governs

Design of Abutment Wall

Abutment wall design mainly consists of reinforcement calculations

Design abutment as biaxial column with loads and moments are as follows

P = 7363.07 kN   b = 1.20 m f_y=415 N/mm^2

ML = 13847.11kN-m   D = 8.58 m f_{ck}= 20 N/mm^2

MT = 1018.71kN-m   d'= 0.10 m

P_u(ultimate load) = 11044.61 kN

ML_u(ultimate moment in longitudinal direction) =20770.66 kN-m

MT_u(ultimate moment in traffic direction) = 1528.07 kN-m

Reinforcement is distributed equally on two sides.

As a first trial assume the reinforcement percentage, p = 0.30 which is minimum requirement of steel for pier (As per IRC-78,Cl.No. 710.3)

Uniaxial moment capacity of the section about flow direction

p/f_{ck} = 0.02   d'/D = 0.01 P_u / f_{ck} * b *D = 0.05

Reffering chart no. 31. (SP - 16)

M_u/ f_{ck} * b * D^2 = 0.05 Mux_1 =79505.71> 13847.11 kN-m   …..O.K

Uniaxial moment capacity of the section about traffic direction

p/fck =0.02   d'/D= 0.08 P_u / f_{ck} * b *D = 0.05

Reffering chart no. 32 (SP - 16)

M_u/ f_{ck} * b * D^2 = 0.04 Muy_1 = 10378.37> 1018.71 kN-m   …..O.K

Calculation of Puz:

Reffering chart 63 of SP-16 corrosponding to

p = 0.30   fy = 415.00 N/mm^2 f_{ck} = 20N/mm^2

so, Puz/ Ag = 9.50 N/mm^2 = 9500 kN/m^2

so, Puz =9500 x 1.20 x 8.58 = 97812 kN

Pu/Puz= 0.11   ML/Mux_1= 0.26   MT/Muy_1 = 0.15

Reffering to Chart 64,

Corrosponding to the above values of MT/Mux_1 and Pu/Puz , the permissible values of Mux/Mux_1 = 0.93 > 0.26

Hence the section is o.k.

Area of steel required Ast $_{req}$ = p * b * D /100 = 0.030888 m^2 = 30888 mm^2

Provide 66 nos of 25 Φ

Ast provided =32397.67 mm^2 > 30888 mm^2

Stirrups area = 0.04% (As per IRC -78,2000, Cl.No. 710.3.3)

Stirrups area = 0.04 * 8.58 * 1.2 / 100

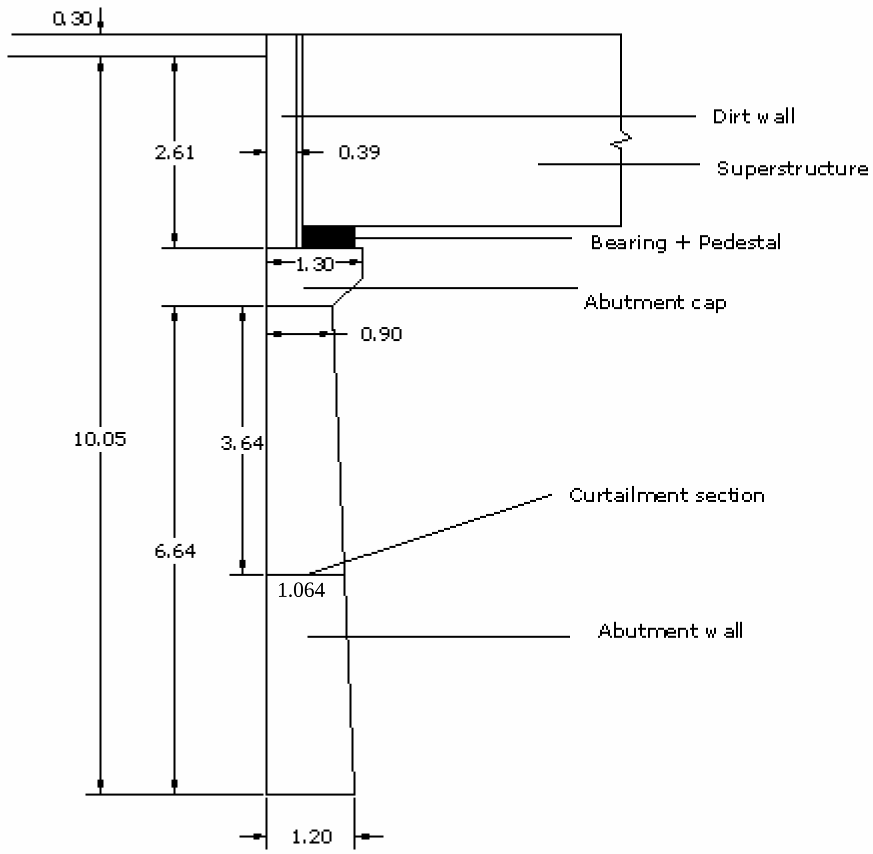
= 4118 mm^2

Provide 12mm dia. @200 mm c/c

Reinforcement details of abutment wall are shown in fig.6.11

Curtailment section

Curtailment of reinforcement is given at 3.64m below top of the abutment



All dimensions are in meter

FIGURE 6.9 SECTION OF ABUTMENT WALL AT CURTAILMENT SECTION

Dead Load of superstructure

Dead load of superstructure for span 40 m = 8820 kN (Table 3.1)

Dead load on abutment =4410 kN (half the load on Abutment)

Eccentricity = 0.39 + 0.04 + 0.4 – 1.064 / 2

= 0.298 m (Eccentricity is calculated at center of base of Abutment)

Moment due to superstructure Mt = Dead Load * Eccentricity

= 4410 * 0.298 = 1314.18 kN-m

Dead Load of Substructure

Dirt wall

Height of dirt wall = 2.61 m

Thickness of dirt wall = 0.39 m

Volume of dirt wall = 2.61 * 0.39 * 8.58 = 8.73 m^3

Dead load of dirt wall = Volume of dirt wall * Density of concrete

= 8.73 * 24 = 209.60 kN

Eccentricity = 0.39 / 2 - 1.064 / 2 = -0.337 m

Moment due to dead load of dirt wall Mt = Dead Load * Eccentricity

= 209.60 * -0.337

= -70.63 kN-m

Abutment cap

Rectangular portion

Height = 0.4 m

Thickness =1.3 m

Volume of Abutment cap of rectangular portion = 0.4 * 1.3 * 8.58

= 4.46 m^3

Dead load of abutment cap of rectangular portion= Volume * Density of concrete

= 4.46 * 24

= 107.07 kN

Eccentricity = 1.3 / 2 - 1.064 / 2 = 0.118 m

Moment due to dead load of abutment cap (rectangular potion)

Mt = Dead Load * Eccentricity

= 107.07 * 0.118

= 12.63 kN-m

Trapezoidal portion

Height = 0.4 m

Thickness = 1.1 m

Volume of Abutment cap of trapezoidal portion = 0.4 * 1.1 * 8.58

= 3.77 m^3

Dead load of abutment cap of trapezoidal portion

= Volume of abutment cap* Density of concrete

= 3.77 * 24

= 90.60 kN

Eccentricity = 0.568 m

Moment due to dead load of abutment cap (trapezoidal portion)

Mt = Dead Load * Eccentricity

=90.60 * 0.568

= 51.46 kN-m

Total Load = 107.07 + 90.60 = 197.68 kN

Total Moment due to abutment cap Mt = 12.63 + 51.46 = 64.09 kN-m

Abutment wall

Rectangular portion

Height = 3.64 m

Thickness = 0.9 m

Volume of Abutment Wall rectangular portion = 3.64 * 0.9 * 8.58

= 28.10 m^3

Dead load =Volume of abutment wall of rectangular portion* Density of concrete

= 28.10 * 24

= 674.6 kN

Eccentricity = 0.9 /2 - 1.064 / 2 = -0.08 m

Moment due to dead load of abutment wall (rectangular portion)

Mt = Dead Load * Eccentricity

=674.6 * -0.08

= -55.31 kN-m

Trapezoidal portion

Height =3.64m

Thickness=0.082m

Volume of Abutment Wall Trapezoidal portion = 3.64 * 0.082 * 8.58

= 2.56 m^3

Dead load = Volume of abutment wall of trapezoidal portion* Density of concrete

= 2.56 * 24

= 61.46 kN

Eccentricity = 0.9 + (1.064 - 0.9) / 2 - 1.064 / 2 =0.45 m

Moment due to dead load of abutment wall of trapezoidal portion Mt

= Dead Load * Eccentricity

= 27.65 kN-m

Approach slab

Height = 0.35 m

Thickness = 0.39 m

Volume of Approach slab = 0.35 * 0.39 * 8.58 = 1.17 m^3

Dead load = Volume of approach slab* Density of concrete

= 1.17 * 24

= 28.10 kN

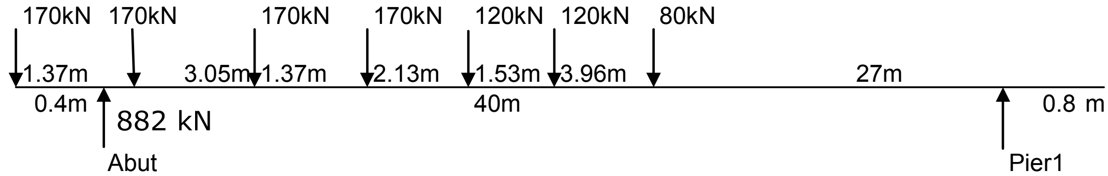
Eccentricity = 0.39 / 2 -1.064 / 2 = -0.337 m

Moment due to approach slab Mt = Dead Load * Eccentricity

= 28.10 * -0.337

= -9.47 kN-m

Live Load



All loads are in kN and all distances are in meter

RA=1000(40+ 0.4 - 5.12)/ 40 = 882 kN

Eccentricity along traffic direction = 0.298 m

Eccentricity across traffic direction (fig.6.10) = (7.5/2)-1.2-(2.79/2) = 1.155 m

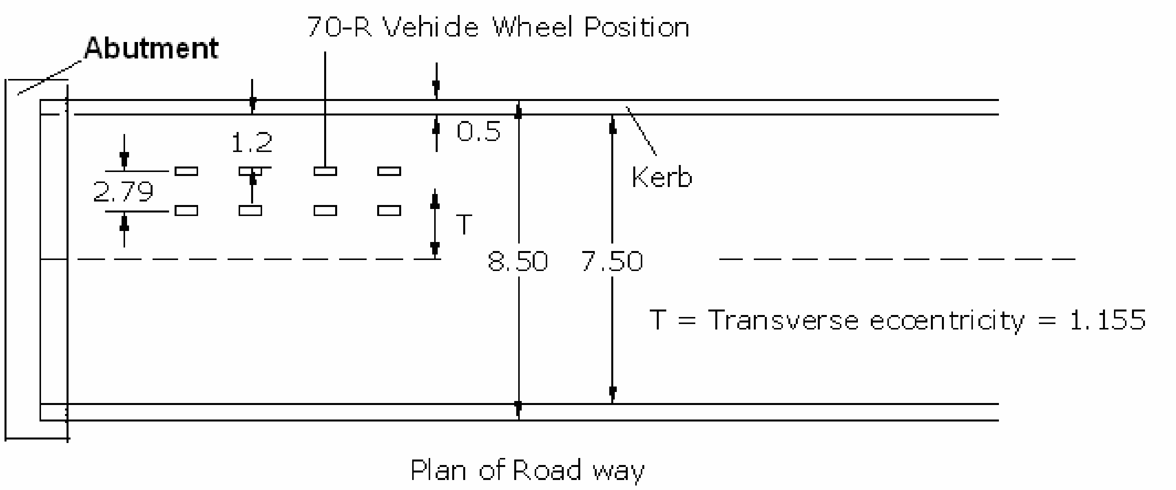


FIGURE 6.10 PLAN OF ROAD WAY FOR CURTAILMENT SECTION

Where, Width of carriageway = 7.5m

Width of vehicle = 2.79m

Min. Edge of carriageway at end from where we can provide vehicle = 1.2 m

Moment due to Live load along traffic direction = 882 x 0.23

= 262.83 kN-m

Moment due to Live load across traffic direction = 882 x 1.155

= 1018.71 kN-m

Braking Force

Total load of Class70-R wheeled vehicle = 1000 kN (As per IRC-6 Appendix 1))

Braking force is considered as 20% of Total vehicle load and it will act at bearing level (As per IRC-6, 2000, cl.No 214.2)

Longitudinal force for class 70-R wheeled load = 0.2 x 1000

= 200 kN

Eccentricity = 0.3 + 0.4 + 0.4 +3.64

= 4.74 m

Where, Height of Bearing + Pedestal = 0.3m

Height of Abutment cap = 0.4 + 0.4 =0.8m

Height of Abutment at curtailment level = 3.64 m

Moment due to braking force Mt = Braking force * Eccentricity

= 200 * 4.74

= 948 kN-m

Earth Pressure

Following soil parameters are considered of backfill material

$C = 0\ kN/m^2$

$\Phi = 30°$

$\delta = 20°$

Unit weight of soil $\gamma = 17\ kN/m^3$

Co-efficient of active earth pressure is given by:

$$k_a = \frac{\cos^2\Phi}{\left[1 + (\sin(\Phi+\delta)*\sin\Phi / \cos\delta)^{1/2}\right]^2}$$

= 0.279

Earth pressure intensity at road level

(As per IRC-78, 2000, Cl. No. 710.4.4 "All abutments shall be designed for a live load surcharge equivalent to 1.2 m height of earth fill)

$P_a = k_a * \gamma * q$ kN/m

Where Width of abutment wall = 8.58 m

width of side wall = 0.35m

q = height of earth fill * Width of Abutment – width of side wall of abutment

ka = 0.279

γ= 17 kN/m^3

P_a = 0.279 x 17 x 1.2 x (8.58 - 2 * 0.35)

P_a= Earth pressure intensity at road level = 44.91kN/m

Earth pressure intensity at base of abutment

$P_a = k_a * \gamma * q$ kN/m

Where width of abutment wall = 8.58 m

width of side wall = 0.35m

q = height of earth fill * Width of Abutment – width of side wall of abutment

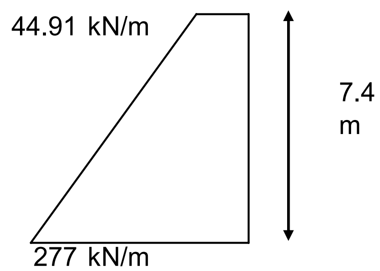
ka = 0.279

γ= 17 kN/m^3

P_a = Earth pressure intensity at base of abutment

= 0.279 x 17 x 7.4 x (8.58 - 2 * 0.35)

= 276.95kN/m Where 7.4 is height from road level to curtailment section



Earth Pressure intensity Diagram

Now Load = Pressure * Height

Load due to earth pressure at road level

F_1 = 332.34 kN

Load due to earth pressure at base of abutment

F_2 = 1024.73 kN

Eccentricity of load due to earth pressure at road level

E_1 =3.7 m

Eccentricity of load due to earth pressure at base of abutment

E_2 = 3.108 m

Moment due to earth pressure at road level M_1= Load * Eccentricity

=332.34 * 3.7

= 1229.67 kN-m

Moment due to earth pressure at base of abutment M_2 = Load * Eccentricity

=1024.73 * 3.108

= 3184.86kN-m

Seismic force

Horizontal Seismic Force in traffic direction

F_{eq} = Ah x G

$$Ah = \frac{(Z/2)\ (Sa/g)}{(R/I)}$$

Where Z = Zone factor as given in table 5 of IRC 6

Zone= 3, Z= 0.16

I = Importance factor

For important bridge = 1.5

Other bridges = 1.0

R = Response reduction factor = 2.5 (As per IRC- 6, Cl. No. 222)

Sa/g = Average response acceleration coefficient

= 2.5 (As per IRC- 6, Cl. No. 222)

G = Vertical Load (Self weight) of components

$$Ah = \frac{\{0.16 / 2 \times 2.5\}}{\{2.5 / 1.5\}}$$

= 0.12

F_{eq} = 0.12 x G

TABLE 6.4 SEISMIC FORCE AND MOMENT ON ABUTMENT WALL AT CURTAILMENT SECTION

Sr No	Components	Dead Load(G)	Ah * G	L.A.	Moments	
		kN	kN	m	kN-m	
1	Super-structure	4410	529.2	5.77	3053.48	
2	Dirt wall	209.61	25.153	5.75	144.50	
3	Abutment cap(rect)	107.08	12.849	4.24	54.48	
4	Abutment cap(trap)	90.60	10.873	3.84	41.75	
5	Abutment wall(rect)	674.59	80.951	1.82	147.33	
6	Abutment wall(trap)	61.46	7.3756	1.21	8.95	
7	Approach slab	28.11	3.373	7.23	24.37	
8	Earth filling	1024.73	122.97	3.11	382.18	
	Total		792.74	kN	3857.05	kN-m

Loads and moments at curtailment section of abutment wall are shown in table 6.5 (Normal condition)

TABLE 6.5 LOADS AND MOMENTS AT CURTAILMENT SECTION ABUTMENT WALL (NORMAL CONDITION)

Sr No	Description	Loads	L.A.	Moments(kN-m)		
		kN	m	MT	ML	
1	Dead load					
	Superstructure	4410.00	0.30		1314.18	
	Dirtwall	209.61	-0.34		-70.64	
	Abutment cap(rect)	107.08	0.12		12.64	
	Abutment cap(trap)	90.60	0.57		51.46	
	Abutment wall(rect)	674.59	-0.08		-55.32	
	Abutment wall(trap)	61.46	0.45		27.66	
	Approach slab	28.11	-0.34		-9.47	
2	Live load	882.00	0.30	262.84	262.84	
3	Braking force	200.00			948.00	
4	Earth pressure					
	F1 = 332.34		3.70		1229.68	
	F2 = 1024.7		3.11		3184.86	
	Total	6663.45	kN	262.84	6895.88	kN-m

Loads and moments at curtailment section of abutment are shown in table 6.6 (Seismic condition)

In seismic condition live load and braking force is to be considered half (As per IRC-6 -2000 Load combinations, pg no 8)

TABLE 6.6 LOADS AND MOMENTS AT CURTAILMENT SECTION ABUTMENT WALL (SEISMIC CONDITION)

Sr No	Description	Loads	L.A.	Moments(kN-m)		
		kN	m	ML	MT	
1	Dead load					
	Superstructure	4410.00	0.30		1314.18	
	Dirtwall	209.61	-0.34		-70.64	
	Abutment cap(rect)	107.08	0.12		12.64	
	Abutment cap(trap)	90.60	0.57		51.46	
	Abutment wall(rect)	674.59	-0.08		-55.32	
	Abutment wall(trap)	61.46	0.45		27.66	
	Approach slab	28.11	-0.34		-9.47	
2	Live load	441.00	0.30	262.84	131.42	
3	Braking force	100.00			474.00	
4	Earth pressure					
	F1 = 332.34		3.70		1229.68	
	F2 = 1024.7		3.11		3184.86	
5	Seismic force					
	= 792.74				3857.05	
	Total	6122.45	kN	262.84	10147.52	kN-m

Ratio of moment due to seismic condition and normal condition will decide which case will governs

(If Ratio is greater than 1.5 then seismic case govern and if ratio is less than 1.5 then normal case governs As per IRC-6-2000, Load combinations, pg no. 8)

Ratio = 10147.51/ 6895.88 = 1.47

So, Normal case governs

Design of Abutment wall at curtailment section

Abutment wall design mainly consists of reinforcement calculation and reinforcement details.

Design abutment as biaxial column with loads and moments are as follows

P = 6663.45 kN b = 1.064 m fy=415 N/mm^2

ML = 6895.88 kN-m D=8.58 m fck=20 N/mm^2

MT = 262.83 kN-m d'= 0.1m

P_u(ultimate load) = 9995.18

ML_u(ultimate moment in longitudinal direction) = 10343.82 kN-m

MT_u(ultimate moment in traffic direction) = 394.25 kN-m

Reinforcement is distributed equall on two sides.

As a first trial assume the reinforcement percentage, p = 0.3

Uniaxial moment capacity of the section about flow direction

p/f_{ck} = 0.015 d'/D = 0.01 P_u / f_{ck} * b *D = 0.054

Reffering chart no.31 (SP - 16)

M_u/ f_{ck} * b * D^2 = 0.044 Mux_1 = 68928.50 >6895.88 kN-m …..O.K

Uniaxial moment capacity of the section about traffic direction

p/f_{ck} = 0.015 d'/D = 0.093 P_u / f_{ck} * b *D = 0.054

Reffering chart no. 32 (SP - 16)

Mu/ fck * b * D^2= 0.041 Muy_1 = 7964.97 > 262.83 kN-m …..O.K

Calculation of Puz:

Reffering chart 63 of SP-16 corrosponding to

P = 0.3 f_y = 415 N/mm^2 fck = 20 N/mm^2

So, Puz/ Ag = 9.5 N/mm^2 = 9500 kN/m^2

So, Puz = 86726.64kN

Pu/Puz = 0.115 ML/Mux_1 = 0.150 MT/Muy_1= 0.049

Reffering to Chart 64,

Corrosponding to the above values of MT/Mux1 and Pu/Puz , the permissible values of Mux/Mux_1 = 0.86 > 0.1500

Hence the section is o.k.

So, Ast_{req} = p * b * D /100 = 0.02738736m^2 = 27387.36 mm^2

Provide 56 nos of 25 Φ

Ast provided = 27488.93 mm^2 > 27387.36 mm^2

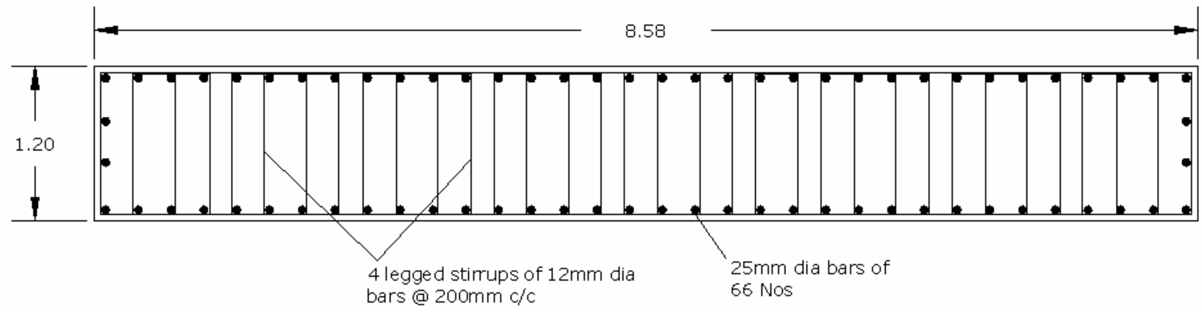
Stirrups area = 0.04% (As per IRC -78,2000, Cl.No. 710.3.3)

Stirrups area = 0.04 * 8.58 * 1.064 / 100

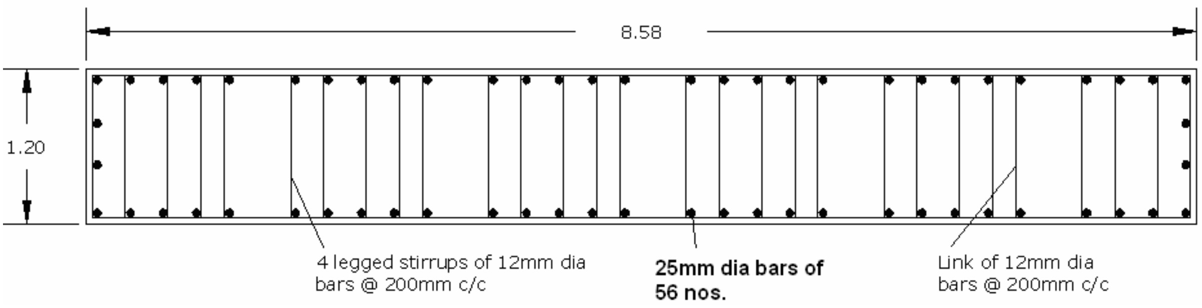
= 3651 mm^2

Provide 12mm dia. @200 mm c/c

Reinforcement details of abutment wall sections are shown in fig.6.11



Reinforcement detail at the base of Abutment wall



Reinforcement detail at the curtailment section of Abutment Wall

FIGURE 6.11 REINFORCEMENT DETAILS OF ABUTMENT WALL

6.10 Design of Dirt wall

Dirt wall is to be designed for self weight, Earth Pressure, Live load acting directly on dirt wall and braking force due to Live Load.

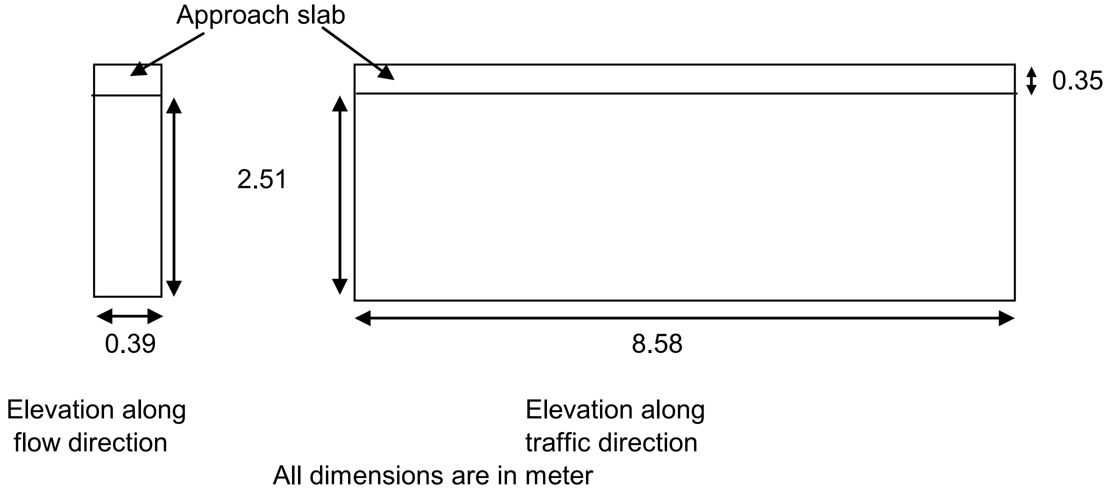


FIGURE 6.12 ELEVATION OF DIRT WALL

Design Loads

Self weight of dirt wall

Self weight = 0.39 x 2.51 x 24 + 0.35 x 24 = 31.89 kN/m

Live Load (70-R Wheeled vehicle)

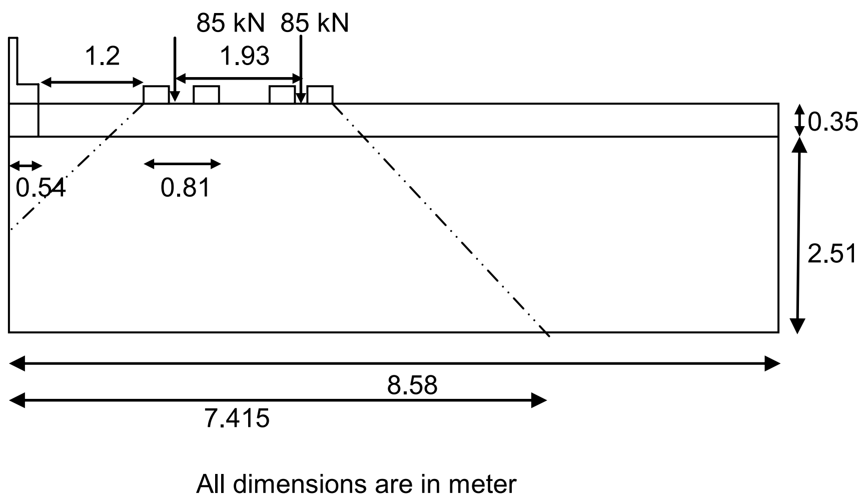


FIGURE 6.13 DISPERSION WIDTH OF LIVE LOAD FOR DIRT WALL

Dispersion width=0.54+1.2+ (0.81/ 2) +1.93+0.075+0.35+ (0.81/ 2) +2.51

= 7.415 m Where wearing coat thickness = 0.075 m

Live load /m width = 170 / 7.415 = 22.92 kN/m

Braking Force

Total load of Class70-R wheeled vehicle = 1000 kN (As per IRC-6 Appendix 1))

Braking force is considered as 20% of Total vehicle load

(As per IRC-6, 2000, cl.No 214.2)

Longitudinal force for class 70-R wheeled load = 0.2 x 1000

= 200 kN

Braking force /m width = Braking force / Dispersion width

= 200 / 7.415 = 26.97 kN/m

Moment due to braking force = 26.97x (1.2+0.075+0.35+2.51)

= 111.53 kN/m

Earth pressure

$C = 0\ kN/m^2$

$\Phi = 30°$

$\delta = 20°$

Ka = 0.279

Unit weight of soil $\gamma = 17\ kN/m^3$

Earth pressure intensity at road level

$Pa = ka * \gamma * q$

$= 0.279 \times 17 \times 1.2 + 0.279 \times 0.35 \times 24$

$= 8.04\ kN/m^2$

Earth pressure intensity at top of abutment cap

$Pa = ka * \gamma * q$

$= 8.04 + 0.279 \times 17 \times 2.51$

$= 8.04 + 11.92$

$= 19.96\ kN/m^2$

Moment at bottom of dirt wall due to earth pressure

$M = (8.04 \times 2.51^2 / 2) + (1 / 2 \times 11.92 \times 2.51 \times 0.42 \times 2.51)$

= 41.11 kN/m

Total Moment on dirt wall

= Moment due to braking force +Moment due to earth pressure

= 111.53 + 41.11

=152.64 kN/m

Total Load on dirt wall = 31.89+ 22.92 = 54.82 kN/m

Now, Dirt wall designed as uniaxial column

P = 54.82 kN b = 0.39 m fck =25 N/mm^2

P_u = 82.23 kN D = 1 m fy =415 N/mm^2

M = 152.64 kN/m d'=0.05 m

M_u = 228.97 kN/m

d'/D = 0.05 =0.05 will be used

Pu/fck * b * D= 0.0084 Mu/fck * b * D^2 =0.0235

Reffering to chart 31 of SP-16

p/fck=0.015 so, p = 0.015*fck = 0.375

Ast req =0.001463 m^2 =1462.5 mm^2

Provide 16mm dia bar of 8 nos on both side on 1 meter strip

Ast provided =1608 mm^2 > 1462.5 mm^2

Stirrups area = 0.04% (As per IRC -78,2000, Cl.No. 710.3.3)

Stirrups area = 0.04 * 0.39 * 1.0 / 100

= 156 mm^2

Provide 12mm dia. @220 mm c/c

Reinforcement details of dirt wall are shown in fig.6.14

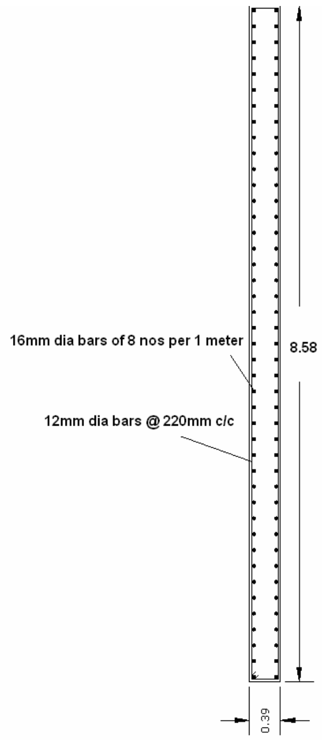
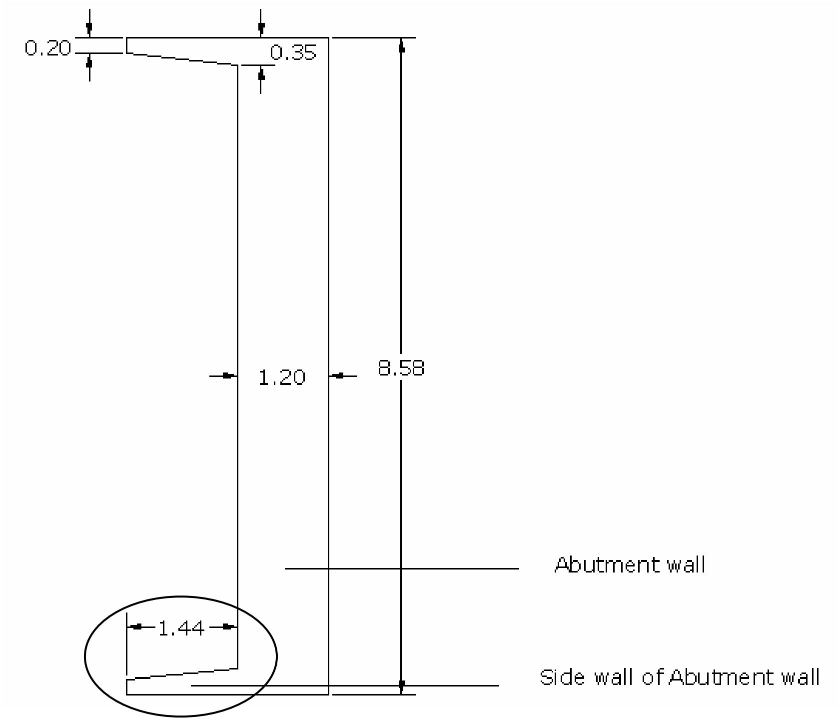
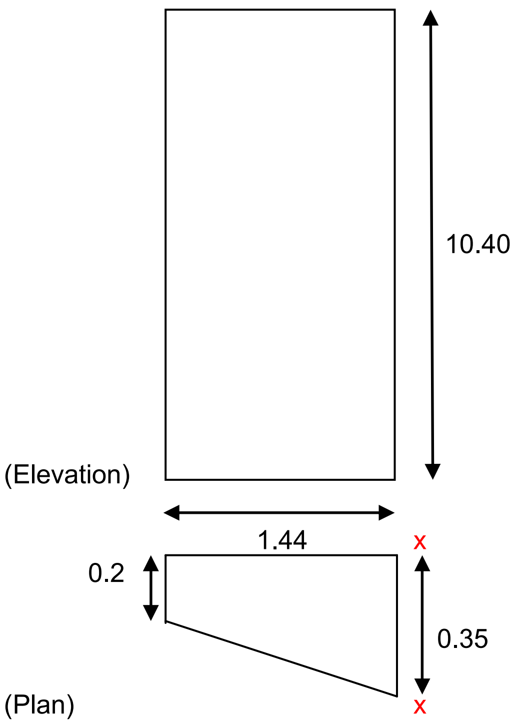


FIGURE 6.14 REINFORCEMENT DETAILS OF DIRT WALL

6.11 Design of side wall of Abutment



Plan of side wall with abutment wall



All dimensions are in meter

FIGURE 6.15 ELEVATION AND PLAN OF SIDE WALL OF ABUTMENT WALL

C = 0kN/m^2

Φ = 30°

δ= 20°

Ka =0.279

Unit weight of soil γ=17kN/m^3

f_{ck} = 30 N/mm^2

f_y = 415 N/mm^2

(A) Earth pressure intensity at road level

Pa = ka * γ * q

= 0.279 x 17 x 1.2

= 5.69 kN/m^2

(B) Earth pressure intensity at base of abutment

Pa = ka * γ * q

= 0.279 x 17 x 10.40

= 49.39 kN/m^2

Total pressure = 55.09 kN/m^2

Moment at x-x (fig.6.11)

= 55.09 x 1.44^2 / 2

= 57.12kN-m/m

Ultimate moment M_u = 1.5 * 57.12 = 85.68 kN-m/m

For M 30 Grade concrete

Depth required = M_u / 0.136 * f_{ck} * b

= 144.91 mm

Depth provided= 0.35 - 0.05 - 0.008

(Cover) (Half the dia of reinforcement)

= 292 mm > 144.91 mm …… o.k

$$Pt = 50\left(\frac{1-\sqrt{1-\dfrac{4.6*Mu}{fck*b*d^2}}}{fy/fck}\right)$$

= 2.78E-07 ≅ 0

Provide 0.2% steel as a main vertical reinforcement

Ast= p * b * d /100

= 0.2 x 1.44 x 0.275 / 100

=792 mm^2

Provide 12mm diameter bars of 12 nos equally distributed

Reinforcement details of side wall are shown in fig.6.16

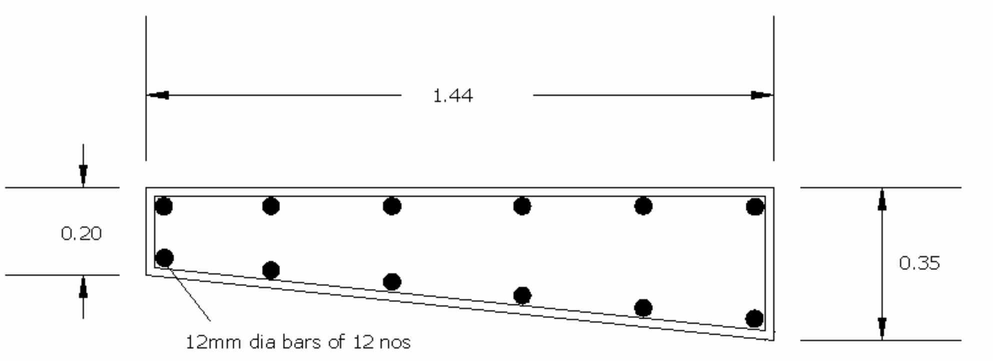


FIGURE 6.16 REINFORCEMENT DETAILS OF SIDE WALL OF ABUTMENT WALL

7.1 Introduction

Well foundations have been used in India for hundreds of years for providing deep foundations below the water level for bridges and aqueducts. A well foundation is similar to an open caisson foundation. At locations where the depth of water is greater than 5m to 6m and the velocity of water is high, wells can be fabricated on the river bank and then floated to the final position and grounded. Great care is to be exercised while grounding a well to ensure that it's position is correct.

7.2 Different shapes of well foundation:

Different shapes of wells which are commonly used are shown in fig 7.2

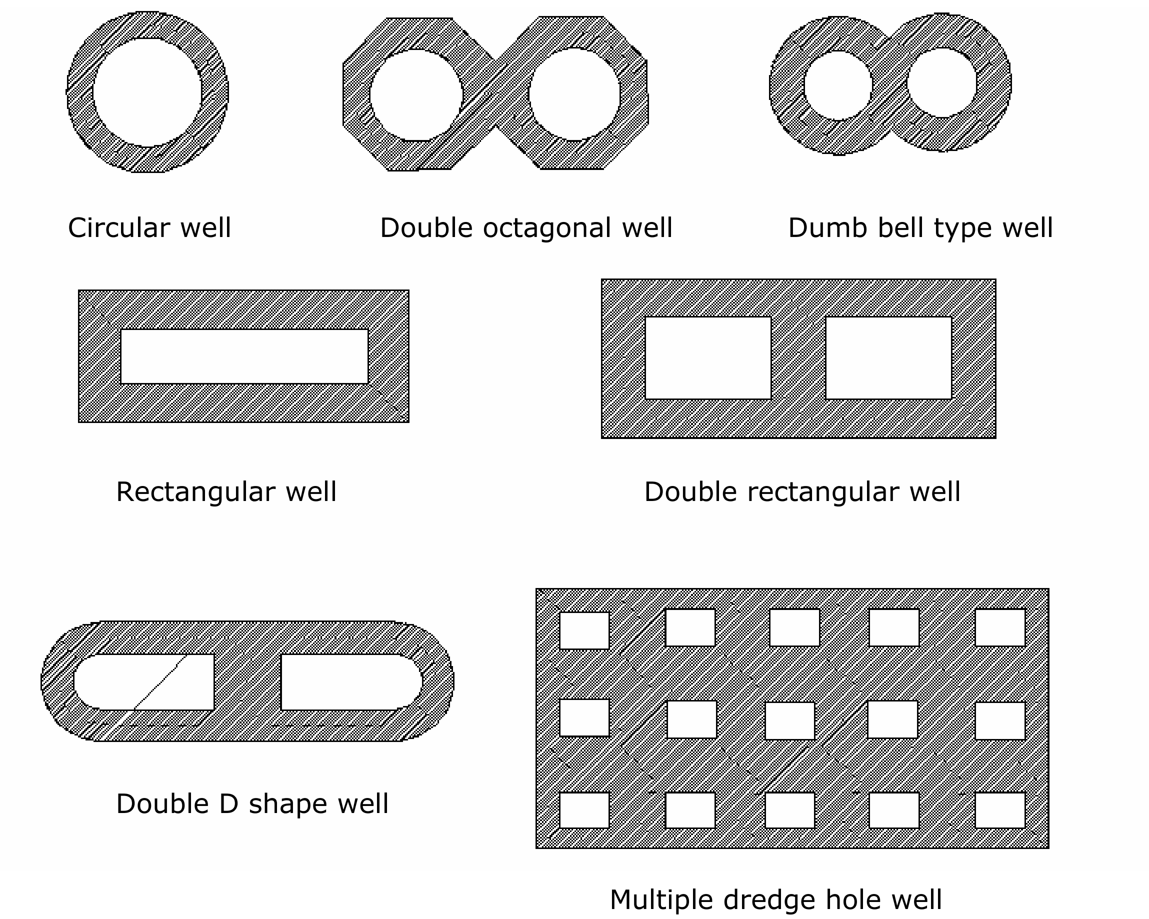


FIGURE 7.1 DIFFERENT SHAPES OF WELL

- **Circular well :**

  The most commonly used shape is circular, as it has high structural strength and is convenient in sinking. The chances of tilting are also minimum in this shape. The shape is quite suitable for piers of the single line railway bridges and the double lane road bridges. However, when the piers or abutments are excessively long, the circular shape becomes uneconomical. The maximum diameter of circular wells is generally limited to 10m

- **Double octagonal well:**

  Double octagonal wells are better than the double D wells in many respects. The square corners are eliminated and bending stresses are considerably reduced. However, they offer greater resistance than double D wells against sinking on account of increases surface area. However, the construction of double octagonal wells is more difficult.

- **Dumb bell type well:**

  A dumb bell shaped well is very similar to double D well except that the dredge holes are circular in shape. It has thus the advantages of twin circular wells in terms of easy accessibility to the dredging equipment and also has the advantage of double D wells.

- **Rectangular well and Double rectangular well:**

  Rectangular wells are generally used for bridge foundation having depths up to 7-8m. For large foundations, double-rectangular wells are used.

- **Double D shape well:**

  Double d well is common for the piers and abutments of bridges which are too long to be accommodated on circular wells. The wells of this shape can also be sunk easily.

- **Multiple dredge hole well:**

  For piers and abutments of very large sizes, wells with multiple dredge holes are used. Wells of this type have been used for the towers of Howrah Bridge.

7.3 Forces on well foundation:

The following forces are considered for analysis and design of well foundation

- Dead Load
- Live Load
- Impact force
- Seismic force
- Water current force
- Earth pressure

7.4 Components of well foundation:

The main components of well foundations are as under:

(a) Steining

(b) Well curb

(c) Bottom plug.

(d) Sand filling

(e) Top plug

(f) Well cap

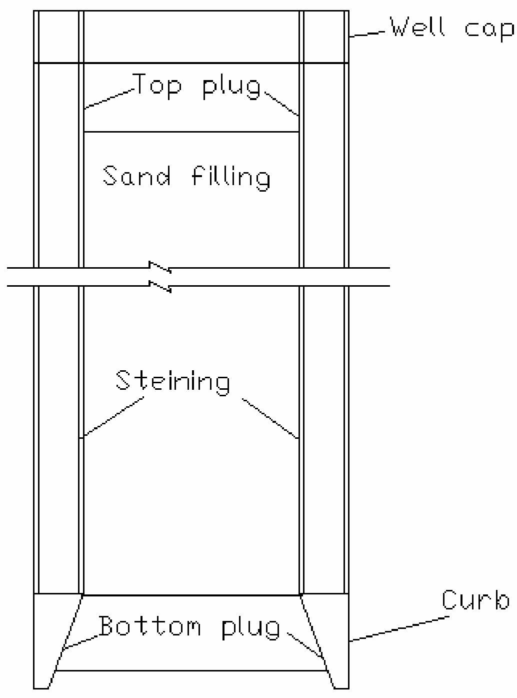


FIGURE 7.2 COMPONENTS OF WELL FOUNDATION

7.4.1 Well cap

A well cap is needed to transfer the loads and moments from the pier or abutment to the well. The shape of the well cap is normally kept the same as of the well with a possible overhang all round of about 150mm or more. If two or three wells support the pier or abutment the well cap will be extended to cover all the wells.

The cap is designed as a slab resting over the top of well.

7.4.2 Well steining

The well Steining is the main body of the well. The thickness of Steining is fixed based on the following considerations:

- It should be possible to sink the well without excessive kentledge.
- The well should not get damaged during sinking.

The thickness of well steining is calculated by using followed equation:

$$t = K\,(B/8 + H/100) \quad \text{(Eqn. 7.1)}$$

Where t = thickness of well steining

K =a constant (= 1.0 for sandy soils; 1.1 for soft clay and 1.25 for hard clay and boulders)

In a circular well inner soil and outer soil pressure will induce hoop compressive stresses which can be worked out using the following formulae applicable to thick shells:

Stress along the inner face, $f_1 = \dfrac{2 * p_1 * r_2^{\,2}}{r_2^{\,2} - r_1^{\,2}}$ …………………… (Eqn. 7.2)

Stress along the outer face, $f_2 = p_1 * \dfrac{r_2^{\,2} + r_1^{\,2}}{r_2^{\,2} - r_1^{\,2}}$ …………………… (Eqn. 7.3)

Where p_1 = Net pressure outside the well

r_1 = Internal diameter of well

r_2 = External diameter of well

Values of $f_1$and f_2 should not exceed the permissible compressive stress of concrete

It can be shown that in a heavy well, section of zero shear force will be at a distance X from scour level which is given by:

$$x = \left(\frac{2 * F * H}{\gamma_b (k_p - k_a) * B} \right)^{1/2} \quad \text{.............................. (Eqn. 7.4)}$$

The maximum moment in steining will be $M_{max} = M_0 + \frac{2}{3} * H * x$ .... (Eqn. 7.5)

Where, γ_b =submerged density of soil

M_{max} = Maximum value of bending moment

M_0 = Moment at scour level

K_p, k_a = passive and active earth pressure coefficients

H = Resultant horizontal force at scour level

F= Factor of safety usually taken as 2.0

B = Diameter of well

7.4.3 Well curb

The well curb is a R.C.C. member which should have a shape offering the minimum resistance during sinking, and should be strong enough to be able to transmit superimposed loads from the steining to the bottom plug. The curb should invariably be reinforced concrete of mix not less than M20 with minimum reinforcement of 72 kg per cu.m. The outer faces of the curb should be protected with suitable steel plates of thickness not less than 6 mm up to half the height of the well curb on the outside and on the inner face not less than 10 mm thick up to top of well curb.
Balwant Rao and Muthuswami have suggested tow conditions to be considered for the design of curb.

A. Design of curb for sinking:

During the process of sinking when the well is dredged, the curb cuts through the soil by the dead weight of the steining including kentledge, if any. The stresses in the curb thus need to be considered.

Total hoop tension = $0.75 * N * \left(\frac{\sin\theta - \mu\cos\theta}{\mu\sin\theta + \cos\theta} \right) * d$ .... (Eqn. 7.6)

Where, N = weight of steining in kN per m run

θ = Angle in degrees of beveling face with the horizontal

μ = Coefficient of friction between soil and concrete of curb

d = Average diameter of steining

B. Curb resting on bottom plug:

Under conditions when the cutting edge is not able to move downwards due to reactions developed at the curb and bottom plug interface neglecting the effect of skin friction – reaction can be resolved into horizontal and vertical components. Assuming parabolic arch within the thickness of the bottom plug, the weight of material filled in the well and that of the bottom plug will be transmitted directly to the bed through it. Hoop tension is given by equation 7.7

$$\text{Hoop tension (H)} = \frac{q*d^3}{16*r} \quad \text{(Eqn. 7.7)}$$

Where, q = ratio of total weight on the base to area of plug

r = vertical height of imaginary inverted arch

$$\text{Hoop compression (C)} = (p_1 + p_2)\frac{b*d}{4} \quad \text{(Eqn. 7.8)}$$

Where, p_1 = Active earth pressure at depth D_f

= $\gamma_b*k_a*D_f$

p_2 = Active earth pressure at depth D_f – b

= $\gamma_b*k_a*(D_f - b)$

k_a = Coefficient of active earth pressure,

γ = submerged unit weight of soil.

Thus net hoop tension = (H – C)

7.4.4 Bottom plug

Bottom plug of a well usually consists of M15 cement concrete. The bottom plug may be in the shape of a bulb. The advantage of this is that it will produce arch action, reduce hoop tension in curb and give greater bearing area. It is essential that the bottom plug should be capable of receiving the load from the steining and transmitting it to the base in which case thickness of the bottom plug should be at least equal to half the diameter of dredge hole. Based on the theory of elasticity, the thickness of the seal t for circular well is given by:

$t^2 = 1.18*r^2*q/f_c$.................................(Eqn. 7.9)

Where, q = ratio of total weight on the base to area of plug

r = radius of well steining

f_c = Flexure strength of concrete seal

7.5 Lateral Stability of well:

Lateral stability of well is most important check for well foundations. The stability of well under the action of lateral loads depends on the resistance of the oil on its sides and base. Lateral stability of well can be checked by Elastic analysis and Ultimate resistance approach. For the lateral stability of well foundation the following conditions should be satisfied.

$$\boxed{H > \frac{M}{r}(1+\mu*\mu') - \mu*W}$$ ....(Eqn. 7.10)

$$\boxed{H < \frac{M}{r}(1-\mu*\mu') + \mu*W}$$ ....(Eqn. 7.11)

Where, $r = \frac{D}{2} * \frac{I}{m*I_v}$

$I = I_B + m*I_v(1+2*\mu'*\alpha)$

$I_v = L*D^3/12$

α = 0.318 * B / D

B = Diameter of well

D = Grip length of well

I_B = Moment of inertia at base

W = Vertical force

H = Horizontal force

M = Moment

m = a constant = 1.0

μ’ = tan δ

μ = tan Φ

$$\boxed{m * \frac{M}{I} < \lambda'(k_p - k_a)} \quad \text{.............................(Eqn. 7.12)}$$

Where , k_p = coefficient of passive earth pressure

K_a = coefficient of active earth pressure

$$\boxed{\frac{W}{A} < \frac{q_u}{2}} \quad \text{.............................(Eqn. 7.13)}$$

Where, A = Area of well at base

q_u = ultimate bearing capacity of soil

$$\boxed{M < 0.7\ (M_b + M_s + M_f)} \quad \text{........................(Eqn. 7.14)}$$

Where, M = Total Moment on well at scour level

M_b = Ultimate moment of resistance of the base section

= C * W * B * tan Φ

M_s = Ultimate moment of resistance on the well sides due to passive resistance

= 0.1 * γ' * D^3 * (k_p - K_a) * L

M_f = Ultimate moment of resistance on the well sides due to frictional resistance

= 0.1 * γ' * (k_p - K_a) * D^2 * B^2 * sinδ

C = a constant depends upon aspect ratio and shape of well

γ' = submerged unit weight of soil

δ = Angel of wall friction in degree

Φ = Angel of internal friction of soil.

7.6 IRC-code provision for well foundation:

- For plain concrete wells, vertical reinforcement in the steining should not be less than 0.12% of gross section area. This should be equally distributed on both faces of the steining.
- In case where the well steining is designed as a reinforced concrete element, it should be considered as a column section subjected to combined axial load and bending. However, the amount of vertical reinforcement provided in the steining should not be less than 0.2% of the actual cross section area of the steining.
- The transverse reinforcement should not be less than 0.04% of the volume per unit length of the steining.

7.7 Analysis and Design of Well Foundation

Data

High Flood Level = 103.50 m

Bed Level = 100.00 m

Scour Level = 92.30 m

Base Level = 82.30 m

External diameter = 9.00 m

Total Vertical Load = 7363 kN (From Table 6.2 of chapter of Abutment)

Total Horizontal Load at bed level = 2492 kN (From Table 6.2)

Moment on well at scour level = 13467 kN-m (From Table 6.2)

Seismic coefficient = 0.12 (From Calculation of seismic force, section 6.10)

γ_{sat} = Saturated density of soil = 20kN/m^3

Φ = Angle of internal friction of soil in degree = 30°

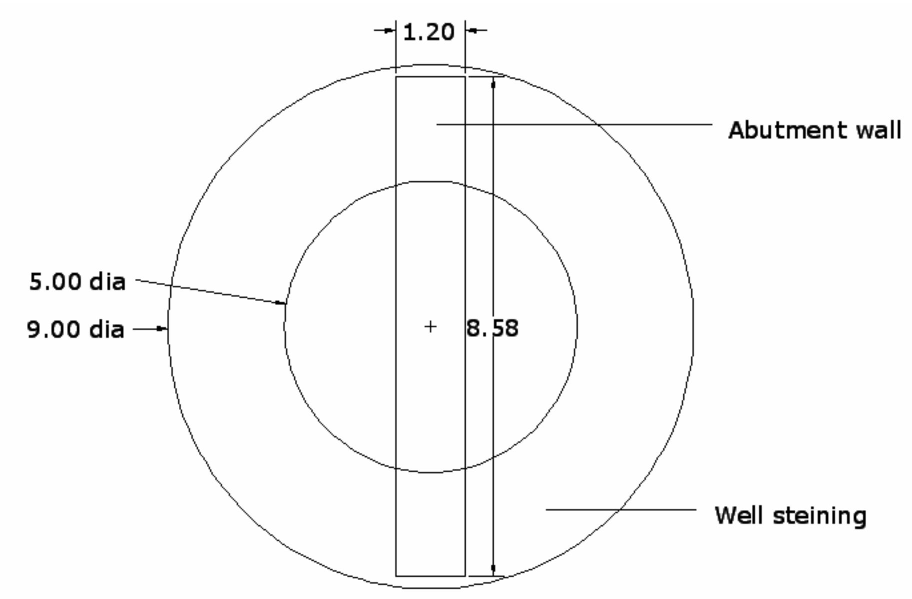
K = a constant = 1.25 (For hard clay)

δ = Angle of Wall friction in degree = 20°

Kp = Co-efficient of passive earth pressure = 6.105

Ka = Co-efficient of active earth pressure = 0.297

qu = Ultimate bearing capacity of the soil below base of well = 1800 kN/m^2



All dimensions are in meter

FIGURE 7.3 PLAN OF WELL FOUNDATION

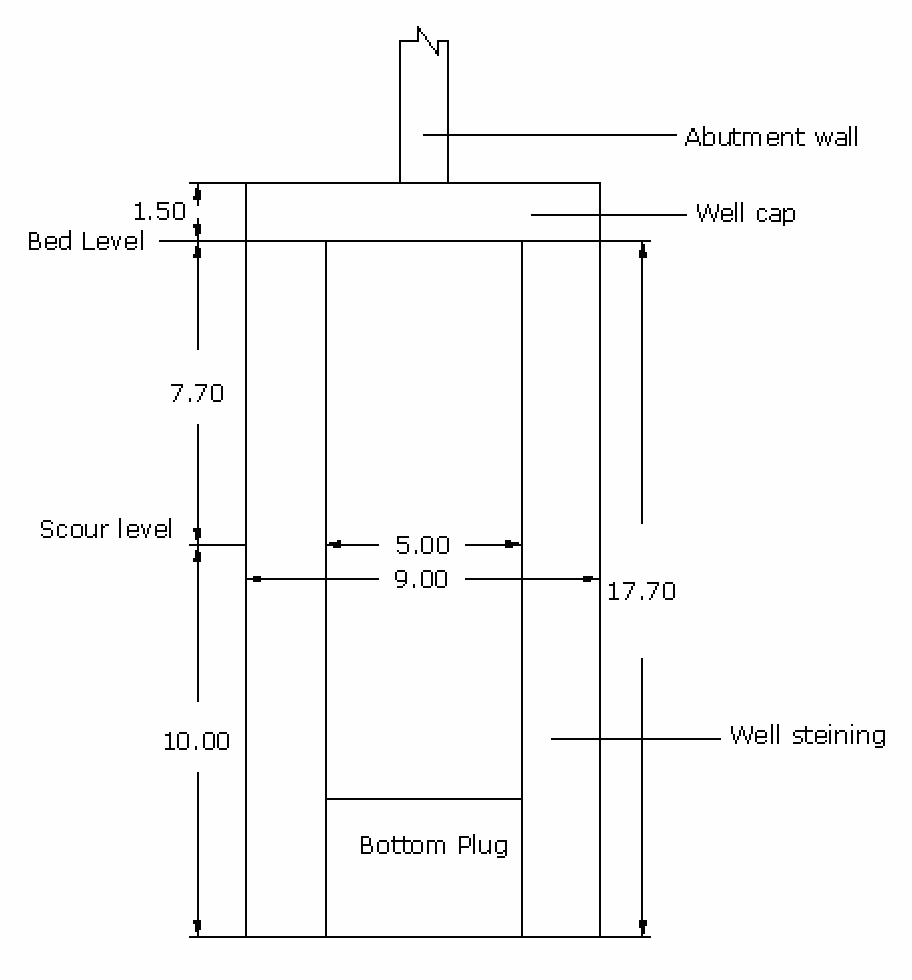


FIGURE 7.4 CROSS-SECTION OF WELL FOUNDATION

Let us assume that the low water level is at bed level.

Thickness of Steining is given by equation (Using equation 7.1)

t= K (B/8 + H/100)

Where B =Outer Diameter = 9.00 m

H = Height of Steining = 17.70 m

K = a constant = 1.25 (For hard clay)

Thickness of steining t = 1.25 x (9.00 / 8 + 17.7 / 100)

= 1.62 m Say 2 m

Internal Diameter = Outer diameter of steining – 2* thickness of steining

= 9.00 – 2 x 2 = 5 m

7.7.1 Design of Well Cap

assume that the thickness of the well cap is1500 mm

Weight of well cap = Volume of well cap * Density

$= (\Pi / 4) \times 9.00^2 \times 1.5 \times 24$

$= 2290.22$ kN

Buoyancy force is taken as 15% of submerged weight

(As per IRC-6, 2000 Cl. No. 213.5)

Buoyancy Force = $0.15 \times (\Pi / 4) \times 9.00^2 \times 1.5 \times 10$

$= 143.13$ kN

Area of cross-section = $(\Pi / 4) \times (\text{Outer diameter}^2 - \text{Internal Diameter}^2)$

$= (\Pi / 4) \times (9.00^2 - 5^2)$

$= 43.98\ m^2$

Moment of Inertia (IB) = $(\Pi / 4) \times (\text{Outer diameter}^4 - \text{Internal Diameter}^4)$

$= (\Pi / 64) \times (9.00^4 - 5^4)$

$= 291.38\ m^4$

Section Modules = I / y =291.38 / 4.5 = 64.75 m^3

Total Vertical Load, neglecting buoyancy

= Load from Abutment + Self-weight of well cap

= 7363 + 2290.22 = 9653.22 kN

The maximum and minimum pressure are given by

$$q_{\max/\min} = \frac{P}{A} \pm \frac{M}{Z}$$

$$q_{max} = \left(\frac{9653.22}{43.98}\right) + \left(\frac{13467}{64.8}\right)$$

$= 219.48 + 207.97$

$= 427.46\ kN/m^2$

$$q_{min} = \left(\frac{9653.22}{43.98}\right) - \left(\frac{13467}{64.8}\right)$$

$= 219.48 - 207.97$

$= 12\ kN/m^2$

Average $q_a = \{(q_{max} + q_{min}) / 2\} = 219.48\ kN/m^2$

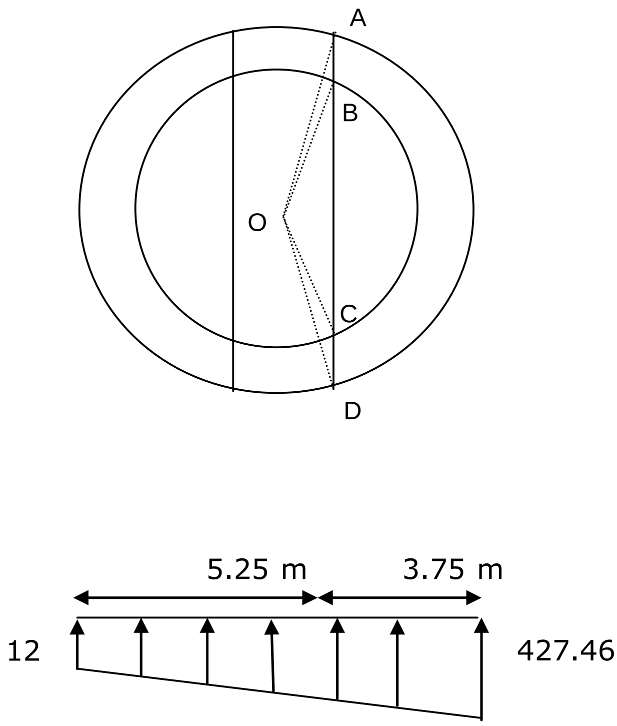


FIGURE 7.5 PRESSURE DISTRIBUTION ON WELL CAP

Pressure at the face of abutment

$$= 12 + \frac{(427.46 - 12)}{9.00} \times 5.25$$

$= 254.14\ kN/m^2$

Intensity of pressure due to self-weight

qc = Self-weight of well cap / area of well

$= 2290.22 / \{ (\Pi/4) \times 9.00^2 \} = 36\ kN/m^2$

In figure 7.5 , Let the angel AOD be θ_1 and BOC be θ_2

$\cos(\theta_1/2) = 0.75 / 4.5$ $\theta_1 = 160.81°$

$\cos(\theta_2/2) = 0.75 / 2.5$ $\theta_2 = 145.08°$

Chord AD = $2 \times 4.5 \times \sin(\theta_1/2) = 8.87$ m

Chord BC = $2 \times 2.5 \times \sin(\theta_2/2) = 4.76$ m

Area of outer segment,

$A_1 = 0.5 \times R^2 (\theta 1 - \sin\theta 1)$

$= 0.5 \times 4.50^2 \times (160.81° - 0.329)$

$= 25.09\ m^2$

Area of inner segment,

$A_2 = 0.5 \times R^2 (\theta 2 - \sin\theta 2)$

$= 0.5 \times 2.50^2 \times (145.08° - 0.572)$

$= 6.12 \text{ m}^2$

The distance of the centroids of these areas are obtained as under,

$Z_1 = 4 * R * \sin^3 (\theta_1/2) / \{3 (\theta_1 - \sin\theta_1)\}$

$= 4 \times 4.50 \times 0.98^3 / \{3 \times (2.80 - 0.329)$

$= 2.32 \text{ m}$

$Z_2 = 4 * R * \sin^3 (\theta_2/2) / \{3 (\theta_2 – \sin\theta_2)\}$

$= 4 \times 2.50 \times 0.954^3 / \{3 \times (2.533 - 0.572)$

$= 1.48 \text{ m}$

Moment about face AD= $q_a[A_1(Z_1\text{-}0.75)\text{-}A_2(Z_2\text{-}0.75)] - q_c[A_1(Z_1\text{-}0.75)]$

Where, q_a= Average pressure = 219.48 kN/m^2

qc = Intensity of pressure due to self-weight = 36.00 kN/m^2

A_1= Area of outer segment = 25.09 m^2

A_2= Area of inner segment = 6.12 m^2

Z_1 = distance of centroids from area of outer segment = 2.32 m

Z_2 = distance of centroids from area of inner segment = 1.48 m

Moment about face AD, M = 6246.96 kN-m

Maximum Shear Force = $q_a (A_1\text{-}A_2) – q_c * A_1$

= 219.48 x (25.09 - 6.12) – 36 x 25.09

= 3260.29 kN

Width of section AD = 8.47 m

Thickness of well cap d = $\left(\dfrac{6247*1.5*10^6}{0.138*20*8470}\right)^{1/2}$

= 633.11 mm

Provided thickness is 1500mm .As the method of designing the well is approximate, let the total thickness of well cap remain as 1500mm

Effective thickness of well cap = 1450 mm

$$Mu = 0.87 * f_y * A_{st} * d\left(1 - (A_{st} * f_y)/b * d * f_{ck}\right)$$

Where Mu(ultimate moment) = 6246.96 x 1.5 = 9370.45 kN-m

f_y =415 kN/m^2

f_{ck} = 20 kN/m^2

d = Effective thickness of well cap = 1450 mm

b = Width of section AD = 8470 mm

Solving above equation found out value of A_{st}

Area of steel required A_{st} = 18475 mm^2

= 18475 / 8.47 mm^2/m

= 2181.22 mm^2/m

= 2200 mm^2/m

Provide 20mm dia bars @ 130mm c/c in both ways and at both layers at top and bottom

Reinforcement details of well cap are shown in fig 7.6

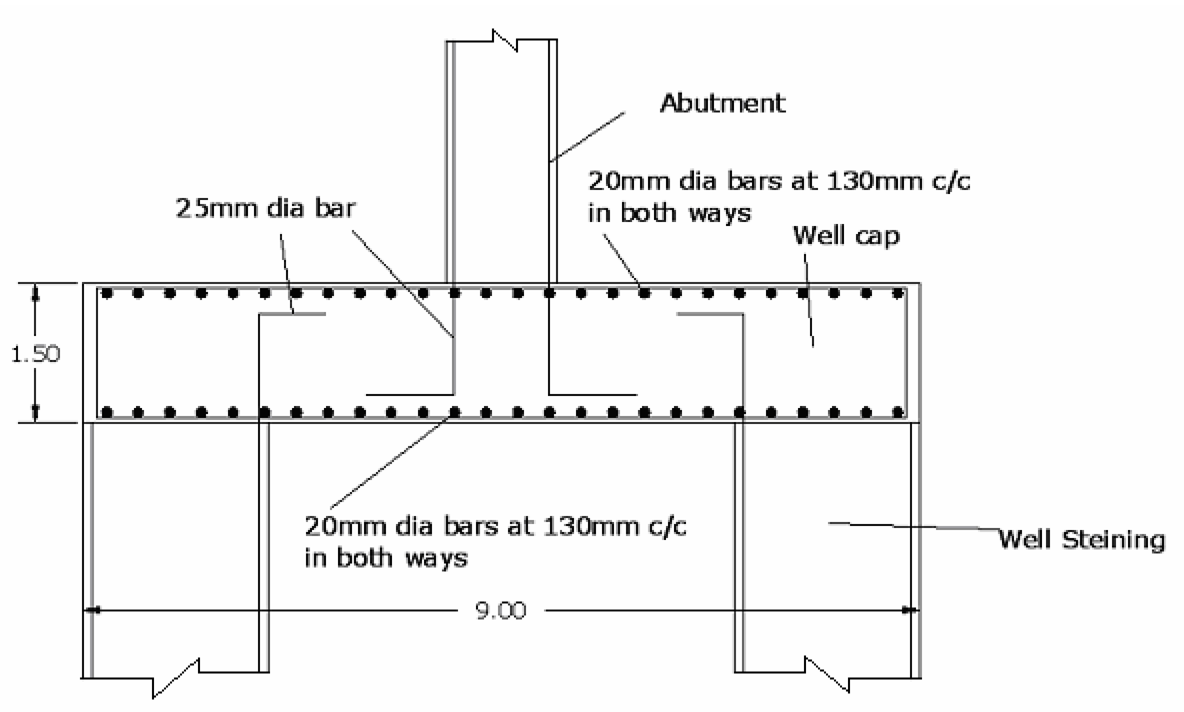


FIGURE 7.6 REINFORCEMENT DETAILS OF WELL CAP

7.7.2 Design of Steining

Total Vertical Dead Load

= Load at the base of Abutment wall + Weight of Well cap

= 7363 + 2290.22 = 9653.22 kN

Horizontal force = 2492 kN (From Table 6.2 of chapter of Abutment)

Moment at scour level =13467 kN-m (From Table 6.2 of chapter of Abutment)

Weight of Steining up to scour level

height of steining from base of well cap to scour level =100–1.5 - 92.30 = 6.2m

Weight of steining up to scour level

= (Π/4) x (9.00^2 - 5.00^2) x 6.2 x 24 = 6544.56 kN

Seismic force on steining = Co-efficient of seismic force * Weight of steining up to scour level

= 0.12 x 6544.56 = 785.34 kN

Total Horizontal force at scour level

= Horizontal Load at base of Abutment + Horizontal Load due to seismic force

= 2492 + 785.34

= 3277.34 kN

Moment at scour level (M)

= Moment at base of Abutment wall + Moment due to horizontal force at base of Abutment wall + Moment due to seismic force

= 13467 + 2492 x 7.70 + 785.34 x 7.7 / 2

= 16212.33 kN-m

The depth of the point of zero shear below the scour level is given by

(Using equation 7.4)

$$x = \left(\frac{2 * F * Q}{\gamma' * (Kp - Ka) * L} \right)^{1/2}$$

Where, F = 2

Q = Total Horizontal force at scour level =3277.34 kN

γ' = Density of submerged soil = 10 kN/m^3

Kp = 6.105 (From Data)

Ka = 0.297 (From Data)

L = External Diameter of well = 9.00m

$$x = \left(\frac{2 \times 2 \times 3277.34}{10 \times (6.105 - 0.297) \times 9.00} \right)^{1/2}$$

= 2.43 m

Now, Maximum Moment is given by: (Using equation 7.5)

M_{max} = M + 2/3 Q * x

= 16212.33 + (2/3) 3277.34 x 2.43

= 21521.63 kN-m

assume a tilt of 1 in 60.

Eccentricity due to tilt at the top = Total Height / 60

= 17.70 / 60

= 0.295 m

Eccentricity at scour level = 0.295 x 10.0 / 17.7 = 0.167m

Moment due to tilt

= Total vertical load and weight of steining up to scour level *Eccentricity

= (9653.22 +6544.56) x 0.167

= 2699.63 kN-m

assume a shift of 1%

Total Shift = 0.01 * Height of steining

=0.01 x 17.7

= 0.177 m

Moment due to shift

= Total vertical load and weight of steining up to scour level *Shift

= (9653.22 + 6544.56) x 0.177

= 2867.01 kN-m

Total Moment = M_{max} + Moment due to tilt + Moment due to Shift

= 21521.63 + 2699.63 + 2867.01

= 27088.27 kN-m

Weight of steining up to the point of zero shear

= 6544.56 x (6.2 +2.43) / 6.2

= 9889.33 kN

Total vertical weight = Load at the base of Abutment wall + Weight of Well cap + Weight of steining up to the point of zero shear

= 7363 + 2290.22 + 9889.33

= 19542.55 kN

The maximum and minimum stresses are given by

$$q_{\max/\min} = \frac{P}{A} \pm \frac{M * y}{I}$$

$$q_{max} = \left(\frac{19542.55}{43.98}\right) + \left(\frac{27088.27}{291.38}\right) \times 4.5$$

= 862.66 kN/m²

$$q_{min} = \left(\frac{19542.55}{43.98}\right) - \left(\frac{27088.27}{291.38}\right) \times 4.5$$

= 25.98 kN/m²

fcc = 0.45 x f_{ck} = 0.45* 20 =9 N/m² = 9000 kN/m² > 862.66 kN/m² (q_{max})

Hence provide nominal reinforcement

Assuming that the open dredging method has been used in sinking the well, then maximum pressure intensity acting on the well from outside is given by,

= γ * H * ka

= 10 x 17.70 x 0.297

= 52.56 kN/m²

Maximum hoop stress (Using equation 7.3)

$$= 52.569 \times \left(\frac{4.5^{2} + 2.5^{2}}{4.5^{2} - 2.5^{2}}\right)$$

=99.50 kN/m² = 0.10 N/m² < 20 N/m² ....Safe

Minimum area of vertical reinforcement is 0.2%,

A_{st} = 0.2 x 43.98 / 100

= 0.08796 m^2

= 87964.59 mm^2

Provide 25mm dia bars of 180 nos

Use 110 bars of 25mm dia on the outer face and 70 bars on inner face

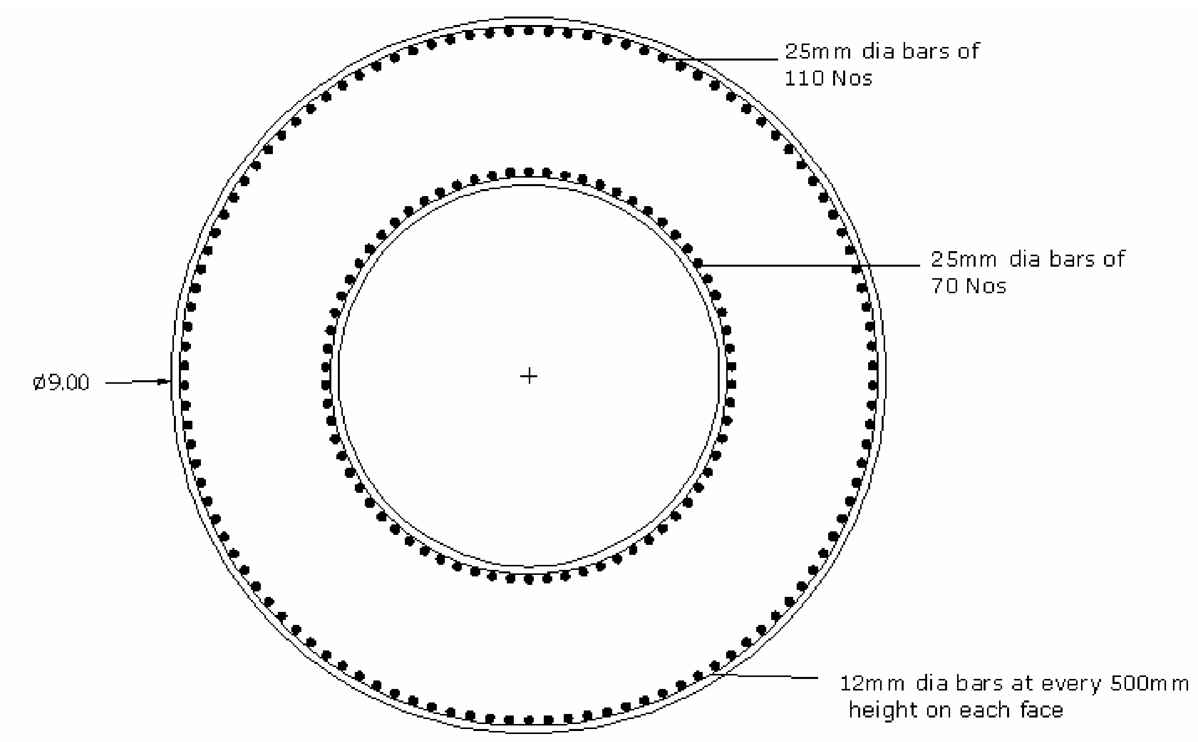
Reinforcement details of well steining are shown in fig.7.7

Transeverse reinforcement = 0.04 x 43.98 / 100

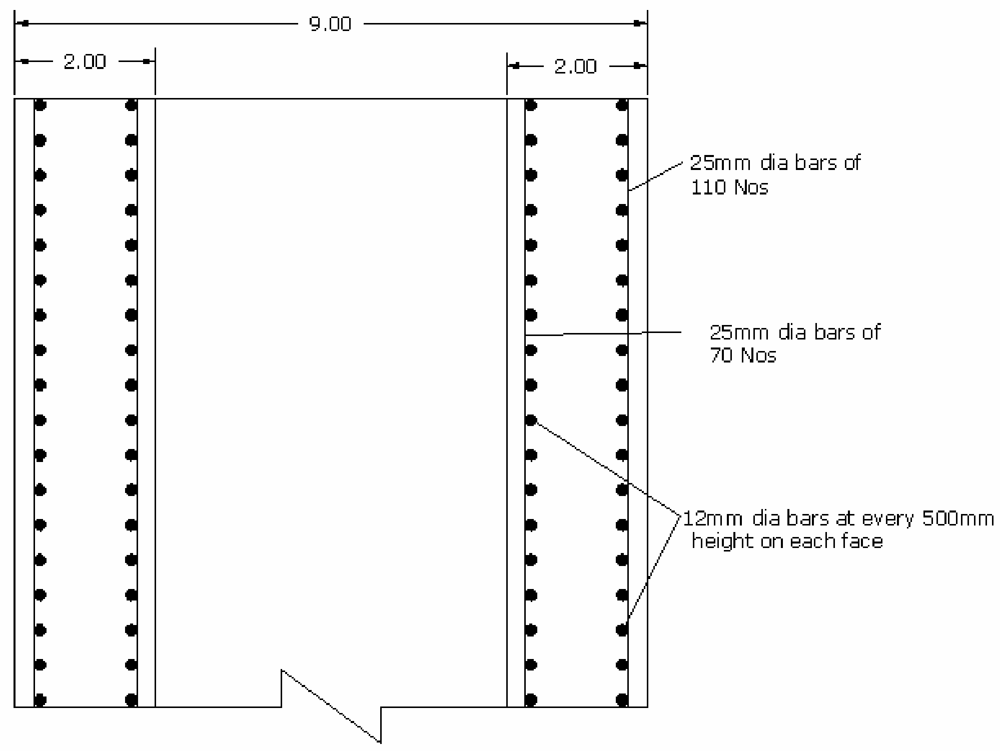
= 17592.92 mm2

Provide 12mm dia bars at every 500mm height on each face.

Reinforcement details of well steining are shown in fig.7.7



Plan of well steining



Cross section of well steining

FIGURE 7.7 REINFORCEMENT DETAILS OF WELL STEINING

7.7.3 Design of well curb

(a) Stress during sinking

Weight of steining per m run

= (Π/4) (9.00^2 - 5.00^2) x 1.0 x 25

= 1099.55 kN/m

Assuming angle of face with vertical as 30° and μ = 0.4

Total Hoop tension (Using equation 7.6)

$$= 0.75 * N * \left(\frac{\sin\theta - \mu\cos\theta}{\mu\sin\theta + \cos\theta}\right) * d$$

Where, N = weight of steining per m run= 1100 kN/m

θ =30°

μ =0.4

d = average diameter = (9.0+ 5.0) / 2 = 7 m

Total Hoop tension =831.71 kN

(b) Stresses in curb when resting on bottom plug

Net vertical weight = Load from abutment and above + weight of well cap + selfweight of well

= 7363 + 2290.22 + 18683.67

= 28336.90 kN

q = Net vertical weight / Area at base

= 28336.90 / (Π/4) * 9.00^2 = 445.42 kN/m^2

Assuming height of imaginary arch r = 5m

Hoop tension (Using equation 7.7)

= q * d^3/16 * r = 2850.73 kN

Assuming the height of well curb = 2.5m

Hoop compression (Using equation 7.8)

$$= (p_1 + p_2)\frac{b*d}{4}$$

Where, p_1 = Active earth pressure at depth D_f = γ_b*k_a*D_f

p_2 = Active earth pressure at depth D_f – b = γ_b*k_a*(D_f – b)

γb = 10 kN/m^3

k_a = 0.297

D_f = 92.3 - 82.3 = 10.0 m

b = height of well curb = 2.5 m

d = effective diameter of well= 8 m

Hoop compression =259.87 kN

Net hoop tension = Hoop tension – Hoop compression

= 2850.73 - 259.87 = 2590.86 kN

Design hoop tension = 1.5 x 2590.86 = 3886.29 kN

Hoop reinforcement = 3886.29 x 1000 / 415

= 9364.56 mm^2

Minimum reinforcement = 0.72 kN/m^3 = 72 kg/ m^3

Reinforcement per meter (Assuing 1 meter strip) = 72 kg/ m^2

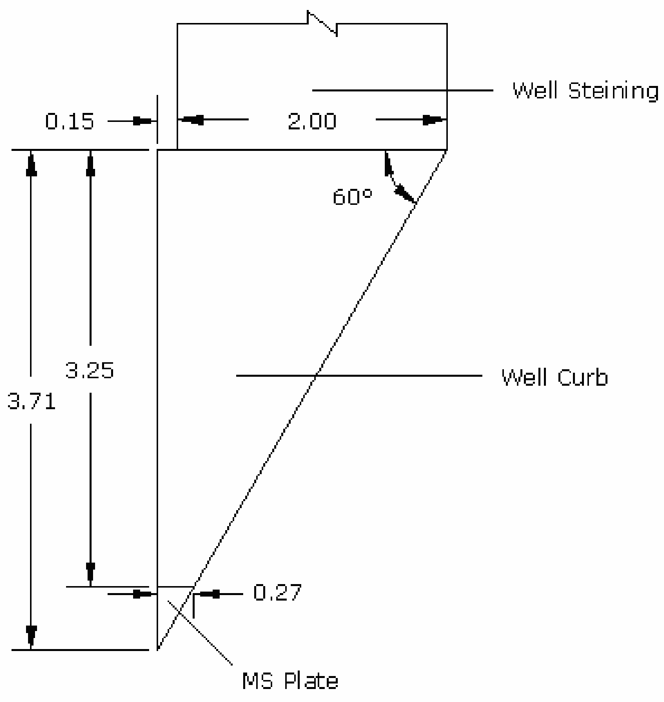


FIGURE 7.8 CROSS-SECTION OF WELL CURB

Area of Well curb section = 3.93 m^2

Reinforcement per meter in well curb section = 72 * 3.93 kg

= 283 kg

Weight per meter in kg of 25mm dia bar = 3.854 kg

Total Number of bar required = 283 / 3.854 = 74 Nos.

Perimeter of Well curb section = 9m

Spacing of bar = Perimeter / Total No. of bar

= 9000 / 74

= 121.6 mm

Provide 25mm dia bars at 120mm c/c throughout the section

Provide 8mm dia rings at 200mm c/c throughout the section

Reinforcement details of well curb are shown in fig.5.9

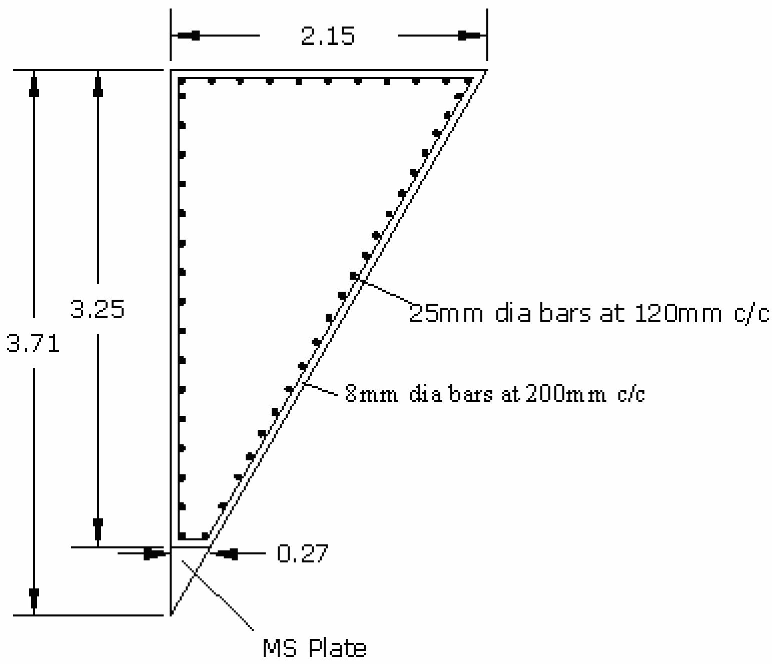


FIGURE 7.9 REINFORCEMENT DETAILS OF WELL CURB

7.7.4 Design of bottom plug

The thickness of bottom plug is given by (Using equation 7.9)

$$t^2 = 1.18 * r^2 * q / fc$$

Where, q = ratio of total weight on the base to area of plug = 445.42 kN/m^2

r = radius of well steining =4.5 m

f_c = Flexure strength of concrete seal

Thickness of bottom plug = 1.18 x 4.5 ^{2} x 445.42 / 4500

t = 1.53 m

Provided thickness is 3.5m hence safe.

7.7.5 Check for Lateral Stability of the well Foundation

Total weight of the steining = 6544.57 x16.20/ 6.2

= 17100.32 kN

Weight of water in the well

= (Π/4) x 5 ^{2} x 6.2 x10 =1021.02 kN

Weight of bottom plug (2.5m thick)

= (Π/4) x5 ^{2} x 2.5 x 24

= 1178.09 kN

Weight of sand fills up to scour level

= (Π/4) x 5 ^{2} x 7.7 x 20

= 3023.78 kN

Total weight at the base

= Load at the base of Abutment wall + Weight of Well cap +Weight of the steining + Weight of water in the well + Weight of bottom plug + Weight of sand fills up to scour level.

= 9653.22 + 17100.32 + 1021.02 + 1178.09 + 3023.78

= 31976.44 kN

Buoyancy on well

= (П/4) x 9.00^2 x 17.7x 10

= 11260.25 kN

Net downward force = Total Weight at the base – Buoyancy force

= 31976.44 - 11260.25 = 20716.18 kN

Weight of water =1021.02 kN

Horizontal seismic force = Co-efficient of seismic force * Weight of water

= 0.12 x 1021.02 = 122.52 kN

Moment of seismic force about scour level

= 122.52 x (1.75 + 10.05 / 2)

= 830.08 kN

Total horizontal force at scour level = Weight of water + Seismic force

= 1021.02 + 122.52 = 1143.54 kN

Total Moment at scour level

= Moment at scour level + Moment due to seismic force

= 16212.33 + 830.09 = 17042.42 kN

Moment at base = Moment at scour level + Moment due to horizontal force

+ Moment due to tilt + Moment due to Shift

= 17042.42 + 1143.54 x 10.0 + 2699.63 + 2867.00

= 34044.45 kN-m

Elastic Analysis

The forces and moments acting at the base are as under,

Vertical Force = 20716.18 kN

Horizontal force = 1143.54 kN

Moment = 34044.45 kN-m

In this case, the grip length, D = 10.00 m

L = 0.9 x 9.00 = 8.1 m

$Iv = L * D^3 / 12 = 8.1 \times 10.0^3 \times /12 = 675.00\ m^4$

I = IB + m Iv (1+ 2μ'ɑ)

Where,ɑ = 0.318 * B / D = 0.318 x 9.00 / 10.0 = 0.29

m = 1.0,

I = 291.38 +1.0 x 675.00 (1 + 2 tan20 x 0.29)

$= 1107.01\ m^4$

$$r = \left(\frac{D * I}{2*m*Iv} \right)$$

= 10.0 x 1107.0 / (2 x 1.0 x 675.00)

= 8.20 m

Now Using equation 7.10

H > M/r (1 + μ*μ’) - μ* W

Where,M = 34044.45 kN-m

r = 8.20 m

μ = tan 30°

μ' =tan 20°

W = 20716.18 kN

M/r (1 + μ*μ’) - μ* W= -6936.33 kN < H = 1143.54 kN……………………….Satisfied

As H is 1143.54 kN , which is greater than -6936.33 kN , the above condition is satisfied.

Using equation 7.11

H < M/r (1 - μ*μ') + μ*W

M/r (1 - μ*μ') + μ*W = 15239.8 kN > H = 1671.33kN………………………. Satisfied

As H is 1143.54 kN , which is less than 15239.8 kN , the above condition is satisfied.

Taking submerged unit weight (Using equation 7.12)

m * M / I < γ' * (Kp - Ka)

m * M / I =1.00 x 34044.45 / 1107.01 = 30.8

γ' * (Kp - Ka) = 58.08………………………. Satisfied

$P = M/r = 34044.45 / 8.20 = 4152$

$$Pt = \left(\frac{(W - \mu' P)}{A}\right) + \left(\frac{M*B}{2 * I}\right)$$

$$= \left(\frac{21187.42 - 0.364 \times 4152}{63.61}\right) + \left(\frac{34044.45 \times 9.00}{2 \times 1107.01}\right)$$

$= 301.88 + 138.4$

$= 440.27\ kN/m^2$

$$Pt = \left(\frac{(W - \mu' P)}{A}\right) - \left(\frac{M*B}{2 * I}\right)$$

$= 301.88 - 138.4$

$= 163.49\ kN/m^2$……………………….   Satisfied

Ultimate resistance

Using equation 7.13

$$\mathbf{W / A < q_u/2}$$

$W/A = 20716.18 / 63.62 = 325.6$

$q_u/2 = 1800 / 2 = 900$………………………. Satisfied

Using equation 7.14

$$\mathbf{M < 0.7(Mb + Ms + Mf)}$$

$Mb = C * W * B * \tan\Phi$

Where, C =0.28

$Mb = 0.28 \times 20716.18 \times 9.00 \times \tan 30 = 30140.44\ kN\text{-}m$

$Ms = 0.1 * \gamma' * D^3 * (Kp\text{-}Ka) * L$

$= 0.1 \times 10 \times 10.00^3 \times (6.105 - 0.297) \times 8.1$

$= 47044.8\ kN\text{-}m$

$Mf = 0.1 * \gamma' * (Kp\text{-}Ka) * B^2 * D^2 * \sin\delta$

$= 0.1 \times 10 \times (6.105 - 0.297) \times 10.0^2 \times 9.00^2 \times \sin 20$

$= 16090.26\ kN\text{-}m$

$(Mb + Ms + Mf) = 30140.44 + 47044.8 + 16090.26$

$= 93275.51$ kN-m

$0.7(Mb + Ms + Mf) = 0.7 \times 93275.51$

$= 60629.15$ kN-m > 39350.60 kN-m (Moment M)

8.1 Introduction

This is a study which is related to particular pier and pile foundation design. To design pier and pile foundation the magnitude of design loads and moments acting on pier and pile foundation should be known, so the attempt is made to find out magnitude of various forces acting on pier and pile foundation and load combinations (as per IRC-6, 2000) to find magnitude of design load and moments on pier and pile foundation. For that the program is developed in Visual Basic-6. The forms of program and source files are given in Appendix-1 & 2

8.2 Parametric study for pier

The parametric study shows the graph of design loads and moments acting on pier verses the span of the superstructure in meter. This parametric study shows that as the span of superstructure increases, the design loads and moments also increases. The design of pier is carried out considering the magnitude of P, MT, and ML. In this parametric study the loads acting on the pier are dead load of pier cap, superstructure dead load (As per MOST) and self-weight of pier, Live load(70-R Wheeled vehicle), Wind force, Water current force, longitudinal forces due to braking effect and bearing rigidity, seismic force, Impact force due to live load and buoyancy force. The moments in transverse direction and longitudinal direction are calculated with magnitude and location of above all forces in respective directions and therefore

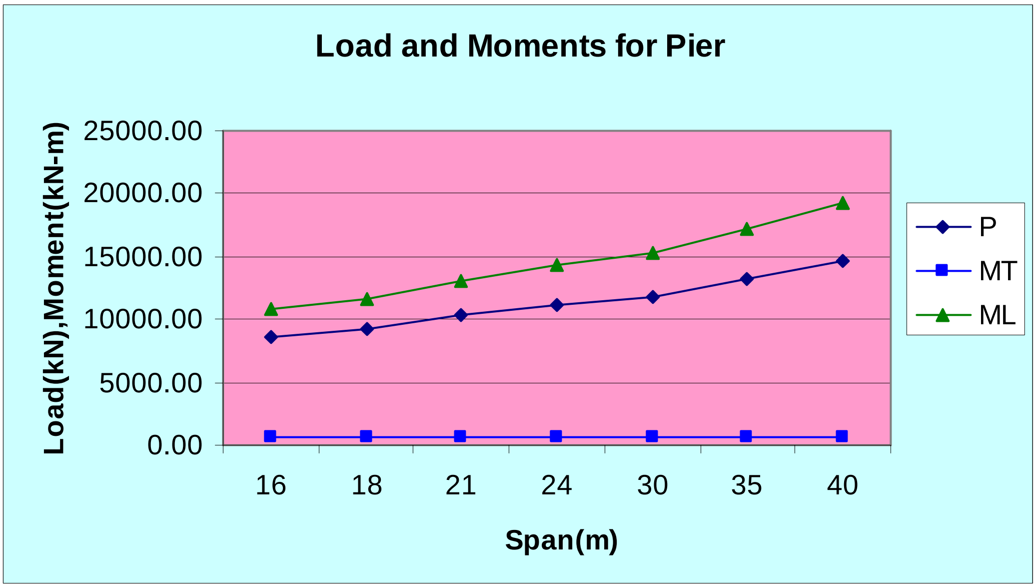
MT = Moment in transverse direction

= Horizontal force in transverse direction * lever arm from base of pier in transverse direction

ML = Moment in longitudinal direction

= Horizontal force in longitudinal direction * lever arm from base of pier in longitudinal direction

The span of superstructure is ranging from 15m to 40m and all the dimensions of pier remains same as manual calculated for span 40m.



Where P = Vertical Load on pier

MT = Moment in Transverse direction

ML = Moment in Longitudinal direction

8.3 Parametric study for Pile

The parametric study shows the graph of design loads and moments acting on pile verses the span of the superstructure in meter. This parametric study shows that as the span of superstructure increases, the design loads and moments also increases. The design of pile foundation is carried out considering the magnitude of P, MT, and ML. In this parametric study the loads acting on the pile are dead load of pier cap, superstructure dead load (As per MOST), dead load of pier and dead load of pile cap, Live load (70-R Wheeled vehicle), Wind force, Water current force, longitudinal forces due to braking effect and bearing rigidity, seismic force, Impact force due to live load and buoyancy force. The moments in transverse direction and longitudinal direction are calculated with magnitude and location of above all forces in respective directions and therefore

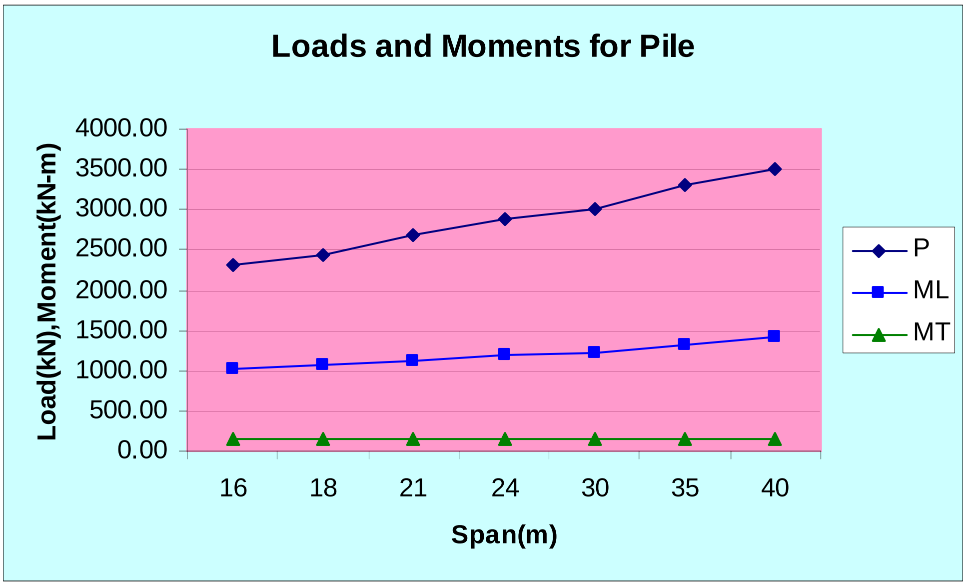
MT = Moment in transverse direction

= Horizontal force in transverse direction * lever arm from base of pile cap in transverse direction

ML = Moment in longitudinal direction

= Horizontal force in longitudinal direction * lever arm from base of pile cap in longitudinal direction

The span of superstructure ranging between 15m to 40m and all the dimensions of pile is remain same as manual calculated for span 40m.



Where, P = Vertical Load on Pile

MT = Moment in Transverse direction

ML = Moment in Longitudinal direction

9 CONCLUSION AND FUTURESCOPE OF WORK

9.1 Conclusion

Aim of the study is to analyze and design the substructure units of the bridge and its foundation. As per IRC-6, 2000, the following loads are acting on the substructure unit:

- Dead Load of superstructure for span of 40m (As per MOST)
- Live Load of vehicle (70-R Wheeled Vehicle)
- Impact effect
- Buoyancy force
- Effect of wind on moving load and on the superstructure
- Forces due to water current
- Longitudinal forces due to braking effect of vehicles and due to bearing rigidity.
- Seismic force.

All these loads are calculated as per relevant IRC-codes. The excel spreadsheets are prepared for the calculation of all these loads.

In this study substructure units like pier cap, pier, abutment wall, dirt wall etc are analyzed and design. Also a pile foundation for intermediate span with pile cap and well foundation for end span with well cap is analyzed and design as per their relevant IRC-codes. For this the standard sections of the superstructure given by Ministry of Surface Transport is considered. The span considered is 40m.

Considering IRC-6, 2000, the load combinations for all the above loads are considered under service condition and construction condition. The governing load combination is then considered for the design of substructure units and foundation.

The design of subst5ructure unit like pier and abutment are designed as column subjected to biaxial loading using IRC-78 and SP-16 provisions.

For intermediate span, pile foundation is designed using circular cast-in-situ piles in a group (8 piles in a group). The load carrying capacity of individual end bearing pile in rock is checked with the actual load acting on a pile and it is found to be o.k. The pile cap analysis is carried out using truss analogy method and STAAD.Pro software. Piles are designed as end bearing column under biaxial bending using IRC-78 and SP-16.

For the end span, well foundation is analyzed and designed using IRC-78. Stability of well foundation is checked using elastic approach and ultimate resistance approach and it was found o.k.

The parametric study is carried out using Visual Basic program to find design loads and moments for pier as well as for pile for a superstructure span ranging from 15m to 40m. This design loads and moments are very much useful to design pie and pile foundation.

9.2 Future scope of work

- Prepare a program in C or in C^{++} to analysis and design of pier and abutment.
- Prepare a program in C or in C^{++} to analysis and design of pile foundation.
- Prepare a program in C or in C^{++} to analysis and design of well foundation.
- Analysis and design of friction pile for intermediate supporting system.

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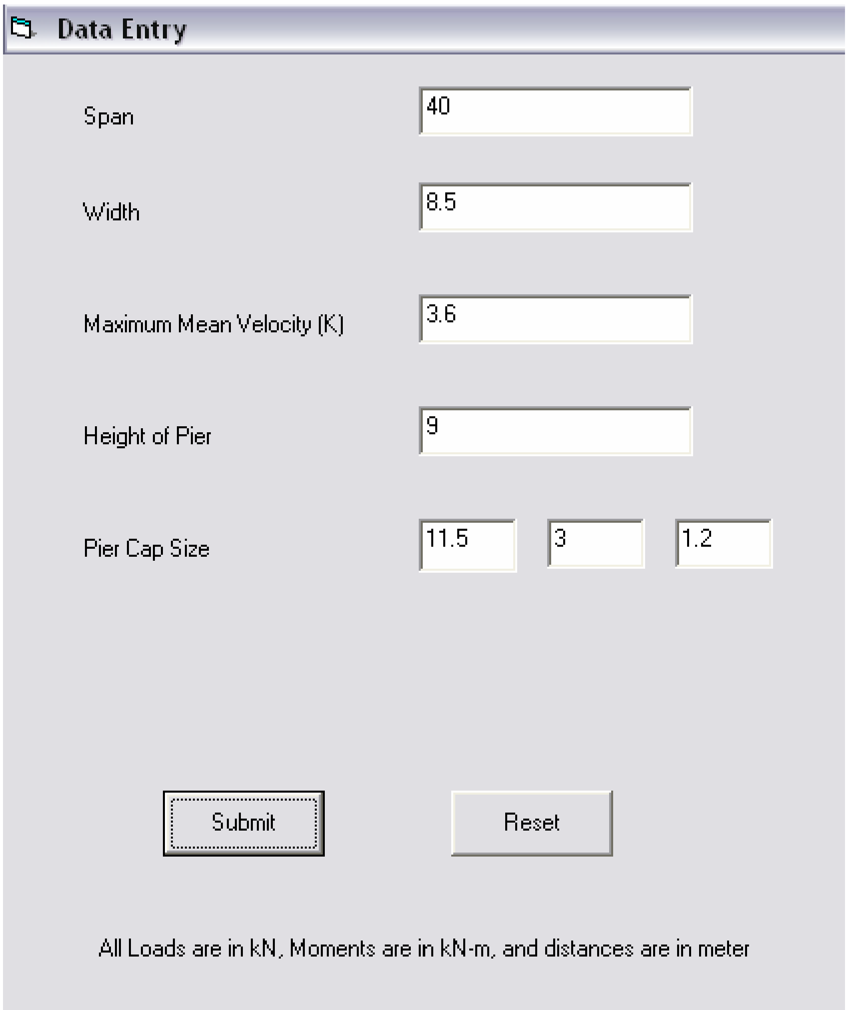
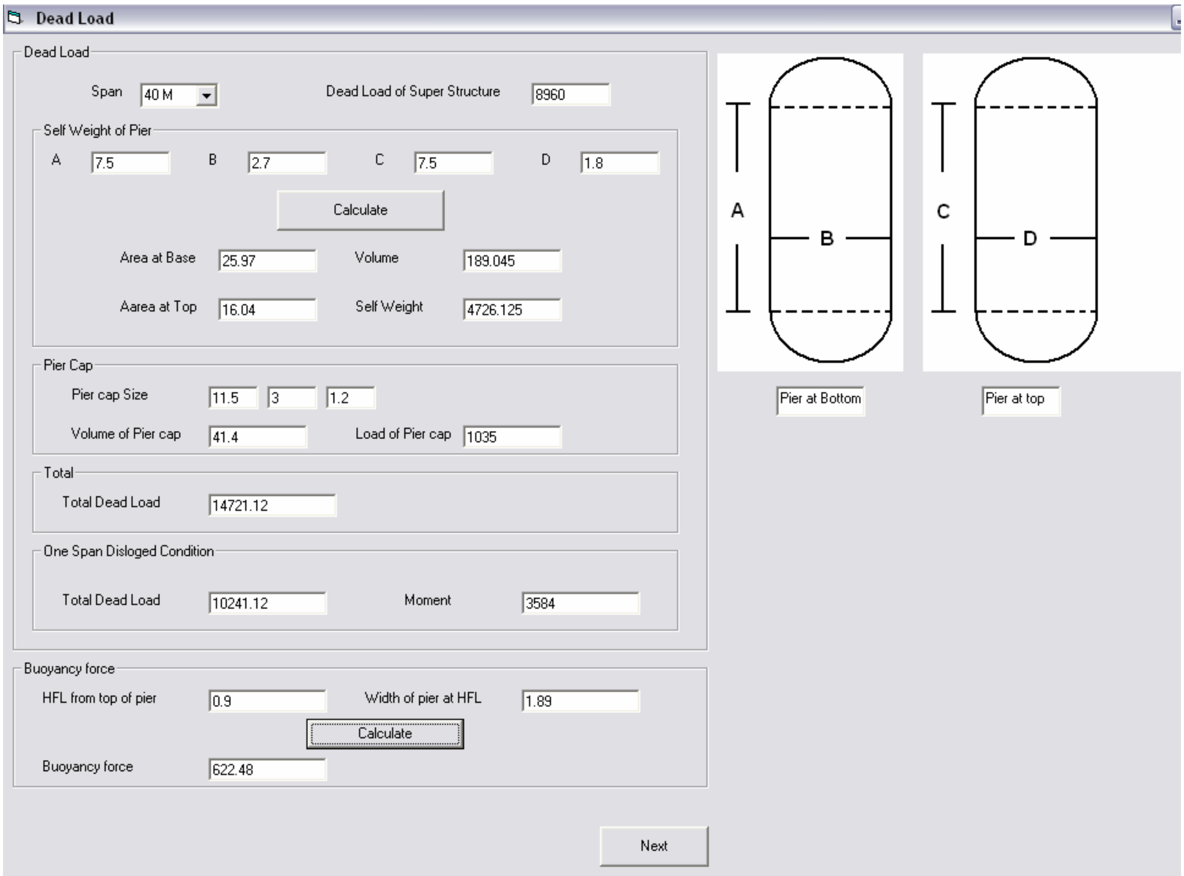
Appendix 1

For parametric study of pier the program is developed in Visual Basic-6. This program gives design load and moments on pier. This program's forms and source files are given in this appendix.

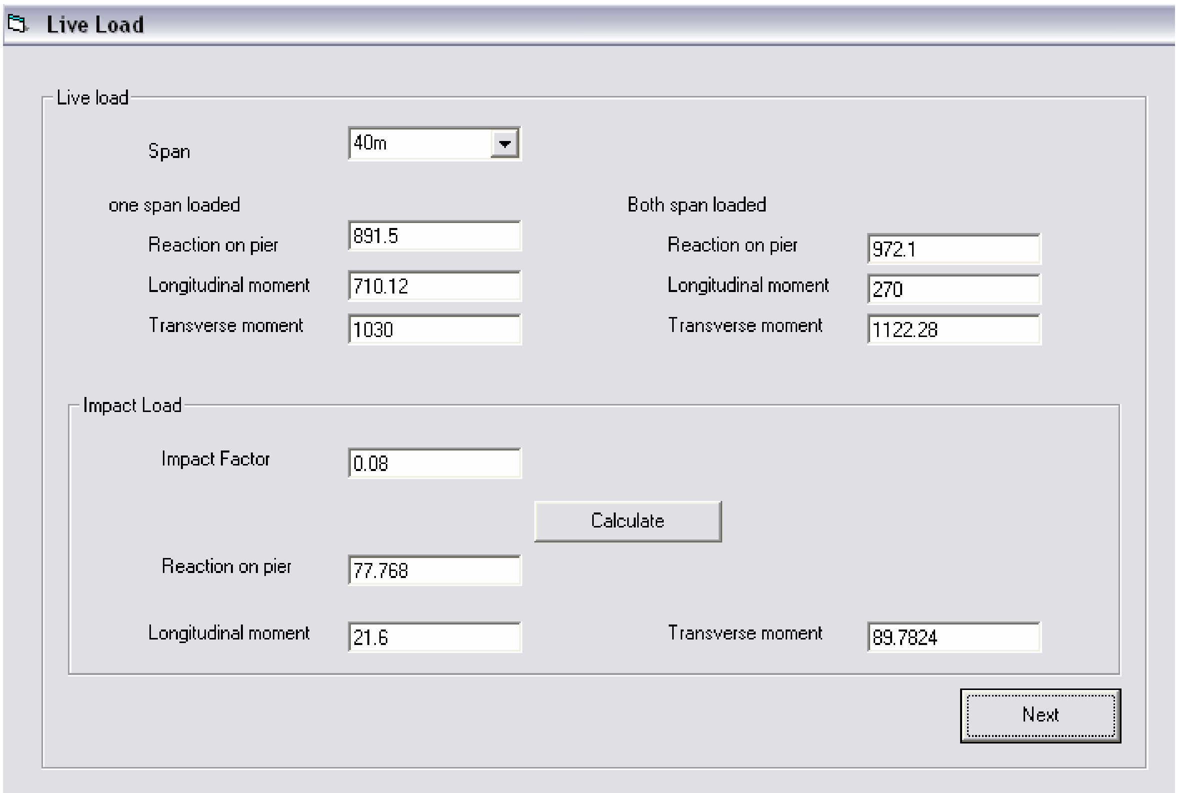
Form 1



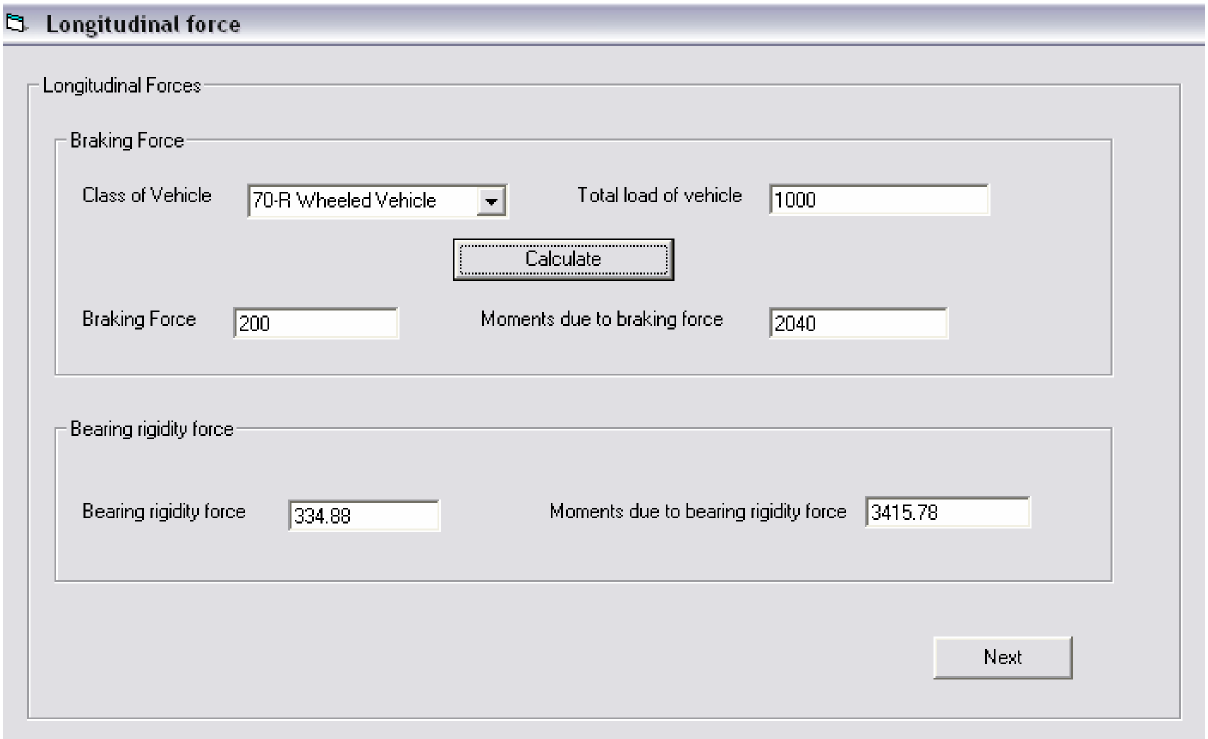
Form 2


Dead Load
Dead Load
Span 40 M
Dead Load of Super Structure 8960
Self Weight of Pier
A 7.5
B 2.7
C 7.5
D 1.8
Calculate
Area at Base 25.97
Volume 189.045
Aarea at Top 16.04
Self Weight 4726.125
Pier Cap
Pier cap Size 11.5 3 1.2
Volume of Pier cap 41.4
Load of Pier cap 1035
Total
Total Dead Load 14721.12
One Span Disloged Condition
Total Dead Load 10241.12
Moment 3584
Buoyancy force
HFL from top of pier 0.9
Width of pier at HFL 1.89
Calculate
Buoyancy force 622.48
Next
A
B
C
D
Pier at Bottom
Pier at top

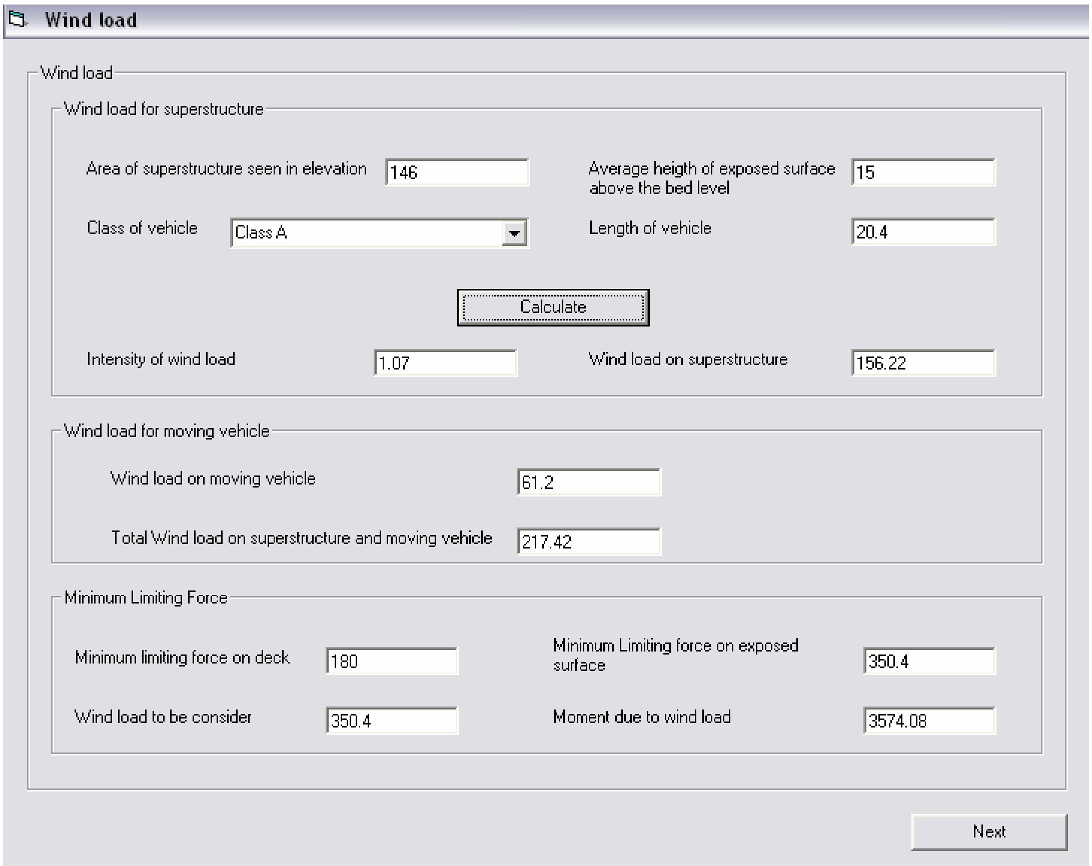
Form 3


Live Load
Live load
Span 40m
one span loaded
Reaction on pier 891.5
Longitudinal moment 710.12
Transverse moment 1030
Both span loaded
Reaction on pier 972.1
Longitudinal moment 270
Transverse moment 1122.28
Impact Load
Impact Factor 0.08
Calculate
Reaction on pier 77.768
Longitudinal moment 21.6
Transverse moment 89.7824
Next

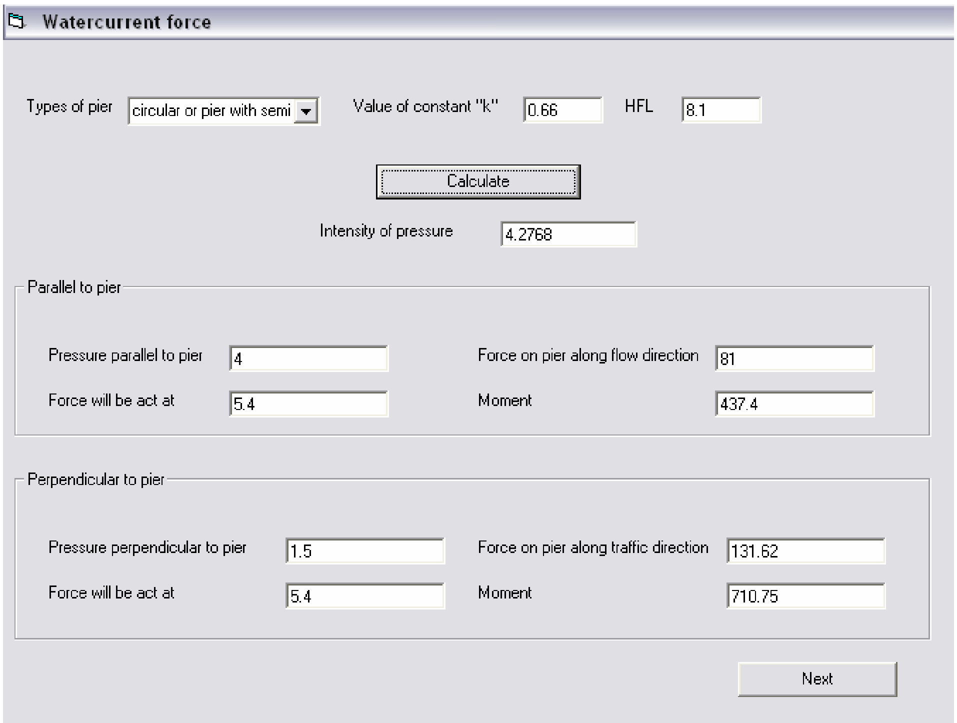
Form 4


Longitudinal force
Longitudinal Forces
Braking Force
Class of Vehicle
70-R Wheeled Vehicle
Total load of vehicle
1000
Calculate
Braking Force
200
Moments due to braking force
2040
Bearing rigidity force
Bearing rigidity force
334.88
Moments due to bearing rigidity force
3415.78
Next

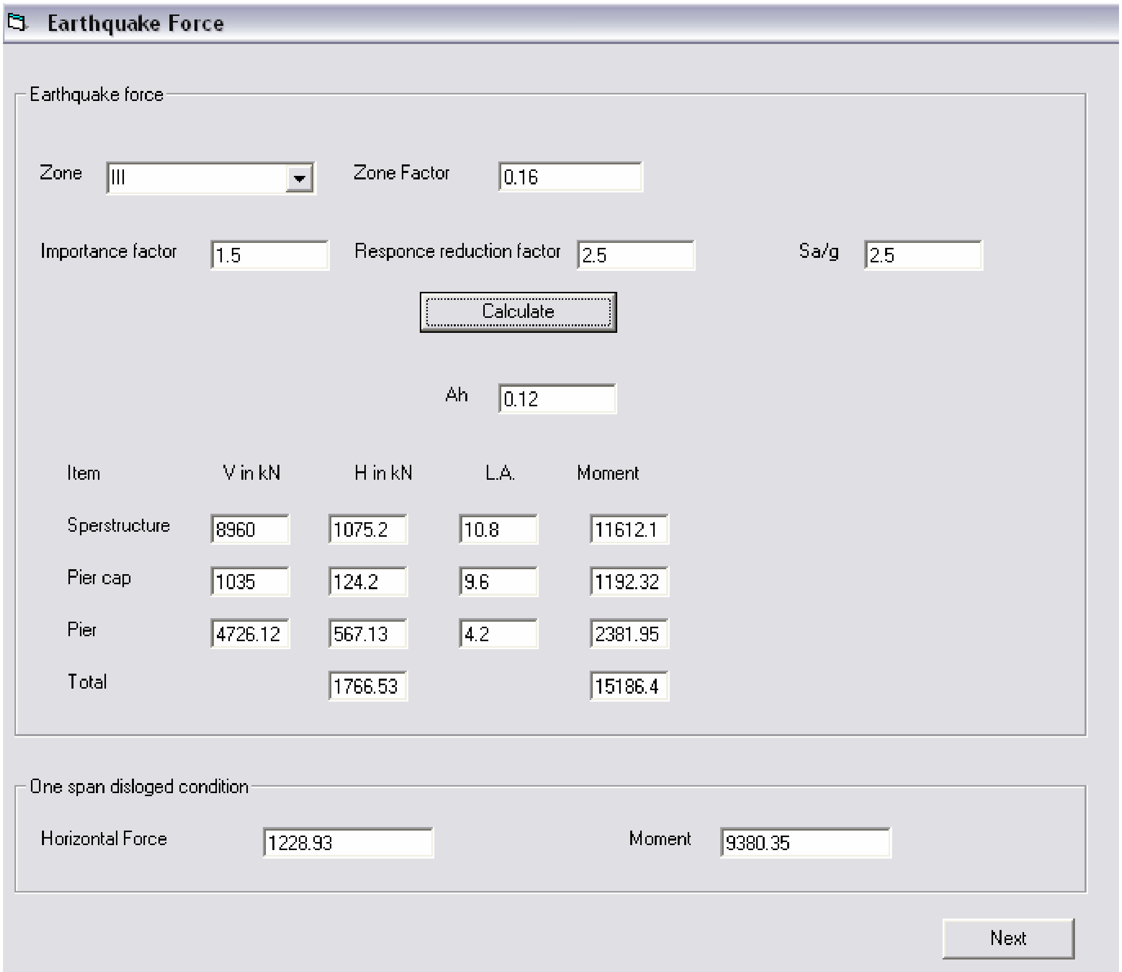
Form 5


Wind load
Wind load
Wind load for superstructure
Area of superstructure seen in elevation
146
Average heigth of exposed surface above the bed level
15
Class of vehicle
Class A
Length of vehicle
20.4
Calculate
Intensity of wind load
1.07
Wind load on superstructure
156.22
Wind load for moving vehicle
Wind load on moving vehicle
61.2
Total Wind load on superstructure and moving vehicle
217.42
Minimum Limiting Force
Minimum limiting force on deck
180
Minimum Limiting force on exposed surface
350.4
Wind load to be consider
350.4
Moment due to wind load
3574.08
Next

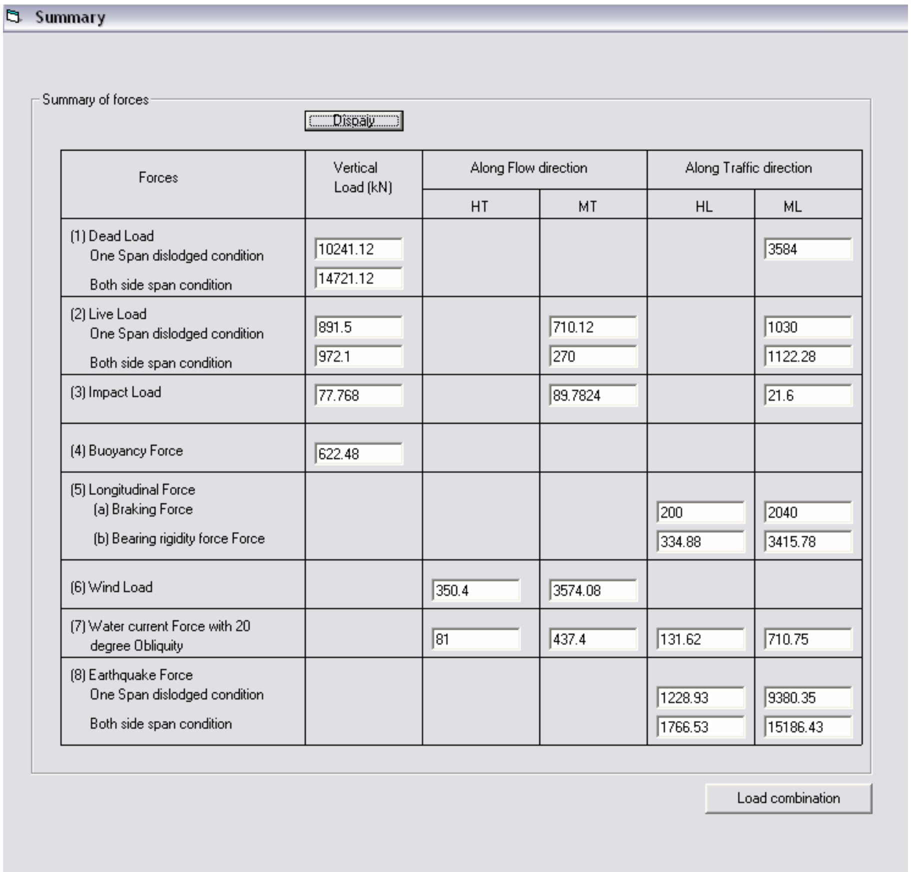
Form 6


Watercurrent force
Types of pier
circular or pier with semi
Value of constant "k"
0.66
HFL
8.1
Calculate
Intensity of pressure
4.2768
Parallel to pier
Pressure parallel to pier
4
Force on pier along flow direction
81
Force will be act at
5.4
Moment
437.4
Perpendicular to pier
Pressure perpendicular to pier
1.5
Force on pier along traffic direction
131.62
Force will be act at
5.4
Moment
710.75
Next

Form 7


Earthquake Force
Earthquake force
Zone
III
Zone Factor
0.16
Importance factor
1.5
Responce reduction factor
2.5
Sa/g
2.5
Calculate
Ah
0.12
Item
V in kN
H in kN
L.A.
Moment
Sperstructure
8960
1075.2
10.8
11612.1
Pier cap
1035
124.2
9.6
1192.32
Pier
4726.12
567.13
4.2
2381.95
Total
1766.53
15186.4
One span disloged condition
Horizontal Force
1228.93
Moment
9380.35
Next

Form 8



Summary

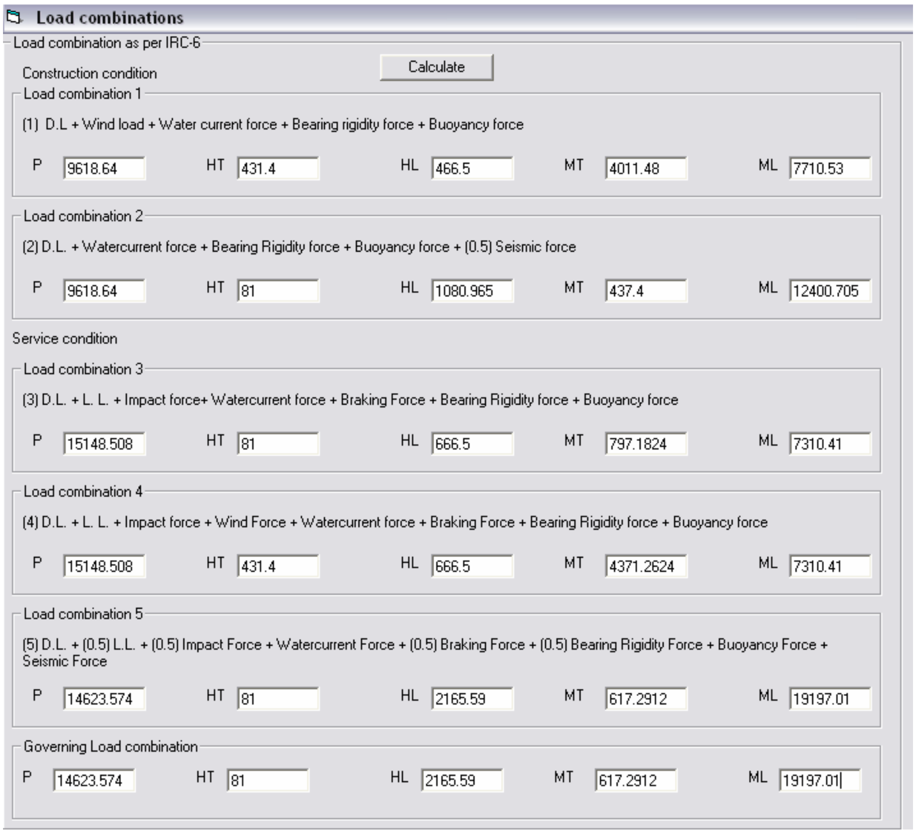
Summary of forces

Display

Forces	Vertical Load (kN)	Along Flow direction		Along Traffic direction	
		HT	MT	HL	ML
(1) Dead Load One Span dislodged condition Both side span condition	10241.12 14721.12				3584
(2) Live Load One Span dislodged condition Both side span condition	891.5 972.1		710.12 270		1030 1122.28
(3) Impact Load	77.768		89.7824		21.6
(4) Buoyancy Force	622.48				
(5) Longitudinal Force (a) Braking Force (b) Bearing rigidity force Force				200 334.88	2040 3415.78
(6) Wind Load		350.4	3574.08		
(7) Water current Force with 20 degree Obliquity		81	437.4	131.62	710.75
(8) Earthquake Force One Span dislodged condition Both side span condition				1228.93 1766.53	9380.35 15186.43

Load combination

Form 9



Load combinations

Load combination as per IRC-6

Calculate

Construction condition

Load combination 1

(1) D.L + Wind load + Water current force + Bearing rigidity force + Buoyancy force

P 9618.64 HT 431.4 HL 466.5 MT 4011.48 ML 7710.53

Load combination 2

(2) D.L. + Watercurrent force + Bearing Rigidity force + Buoyancy force + (0.5) Seismic force

P 9618.64 HT 81 HL 1080.965 MT 437.4 ML 12400.705

Service condition

Load combination 3

(3) D.L. + L. L. + Impact force+ Watercurrent force + Braking Force + Bearing Rigidity force + Buoyancy force

P 15148.508 HT 81 HL 666.5 MT 797.1824 ML 7310.41

Load combination 4

(4) D.L. + L. L. + Impact force + Wind Force + Watercurrent force + Braking Force + Bearing Rigidity force + Buoyancy force

P 15148.508 HT 431.4 HL 666.5 MT 4371.2624 ML 7310.41

Load combination 5

(5) D.L. + (0.5) L.L. + (0.5) Impact Force + Watercurrent Force + (0.5) Braking Force + (0.5) Bearing Rigidity Force + Buoyancy Force + Seismic Force

P 14623.574 HT 81 HL 2165.59 MT 617.2912 ML 19197.01

Governing Load combination

P 14623.574 HT 81 HL 2165.59 MT 617.2912 ML 19197.01

Form 1

```
Option Explicit
Dim span1 As Double
Dim widht As Double
Dim K As Double
Dim Hp As Double
Dim Pcz1 As Double
Dim Pcz2 As Double
Dim Pcz3 As Double
Private Sub Command1_Click()
   span1 = CDbl(Text1.Text)
   widht = CDbl(Text2.Text)
   K = CDbl(Text3.Text)
   Hp = CDbl(Text4.Text)
   Pcz1 = CDbl(Text5.Text)
   Pcz2 = CDbl(Text6.Text)
   Pcz3 = CDbl(Text7.Text)
   Form1.Show vbModal
End Sub
Private Sub Command2_Click()
   Text1.Text = ""
   Text2.Text = ""
   Text3.Text = ""
   Text4.Text = ""
   Text5.Text = ""
   Text6.Text = ""
   Text7.Text = ""
End Sub
```

Form 2

```
Private Sub MyMethod()
   Select Case Combo1.ListIndex
      Case 0
         Text1.Text = "2974"
      Case 1
         Text1.Text = "3536"
```

```
        Case 2
            Text1.Text = "4632"
        Case 3
            Text1.Text = "5546"
        Case 4
            Text1.Text = "6100"
        Case 5
            Text1.Text = "7503"
        Case 6
            Text1.Text = "8960"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
    Text10.Text = Round((Val((CDbl(Text6.Text) * CDbl(Text7.Text))) +
Val(((CDbl(Text7.Text) * CDbl(Text7.Text)) * 3.14 / 4))), 2)
    Text11.Text = Round(((CDbl(Text8.Text) * CDbl(Text9.Text)) +
((CDbl(Text9.Text) * CDbl(Text9.Text)) * 3.14 / 4)), 2)
    Text12.Text = ((CDbl(Text11.Text) + CDbl(Text10.Text)) / 2) *
CDbl(DataEntry.Text4.Text)
    Text13.Text = (CDbl(Text12.Text)) * 25
    Text18.Text = CDbl(Text14.Text) * CDbl(Text15.Text) * CDbl(Text16.Text)
    Text19.Text = Round(CDbl(Text18.Text) * 25, 2)
    Text17.Text = Round((Val(CDbl(Text19.Text)) + Val(CDbl(Text13.Text)) +
Val(CDbl(Text1.Text))), 2)
    Text4.Text = Round((CDbl(Text1.Text) / 2) + CDbl(Text13.Text) +
CDbl(Text19.Text), 2)
```

```
    Text5.Text = Round((CDbl(Text1.Text) / 2) * 0.8, 2)
End Sub
Private Sub Command2_Click()
Form2.Show vbModal
End Sub
Private Sub Command3_Click()
Text26.Text = Round((Val((((((CDbl(Text6.Text) * ((CDbl(Text27.Text) +
CDbl(Text7.Text)) / 2)) + ((((CDbl(Text27.Text) + CDbl(Text7.Text)) / 2) *
(CDbl(Text27.Text) + CDbl(Text7.Text)) / 2) * 3.14 / 4)) *
(CDbl(DataEntry.Text4.Text) - CDbl(Text25.Text))) * 24))))) * 0.15, 2)
End Sub
Private Sub Form_Load()
    Text1.Text = "2974 kN"
    Combo1.ListIndex = 0
     Text14.Text = DataEntry.Text5.Text
     Text15.Text = DataEntry.Text6.Text
     Text16.Text = DataEntry.Text7.Text
End Sub
```

Form 3

```
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text1.Text = "705.7"
            Text2.Text = "815.1"
            Text3.Text = "280.5"
            Text4.Text = "955.7"
            Text5.Text = "705.7"
            Text12.Text = "827.5"
        Case 1
            Text1.Text = "742.5"
            Text2.Text = "857.5"
            Text3.Text = "285.4"
            Text4.Text = "966.6"
            Text5.Text = "742.5"
```

```
    Text12.Text = "849"
Case 2
    Text1.Text = "783.1"
    Text2.Text = "904.5"
    Text3.Text = "282.9"
    Text4.Text = "973.4"
    Text5.Text = "783.1"
    Text12.Text = "855.1"
Case 3
    Text1.Text = "812.7"
    Text2.Text = "938.6"
    Text3.Text = "294.8"
    Text4.Text = "992.5"
    Text5.Text = "812.7"
    Text12.Text = "890.2"
Case 4
    Text1.Text = "852.8"
    Text2.Text = "984.9"
    Text3.Text = "299.9"
    Text4.Text = "1018.3"
    Text5.Text = "852.8"
    Text12.Text = "913.3"
Case 5
    Text1.Text = "875.1"
    Text2.Text = "1010.7"
    Text3.Text = "303.2"
    Text4.Text = "1033.3"
    Text5.Text = "875.1"
    Text12.Text = "926.8"
Case 6
    Text1.Text = "710.12"
    Text2.Text = "1030"
    Text3.Text = "270"
    Text4.Text = "1122.28"
    Text5.Text = "891.5"
```

```
            Text12.Text = "972.1"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Form3.Show vbModal
End Sub
Private Sub Command2_Click()
Text7.Text = (CDbl(Text6.Text)) * (CDbl(Text12.Text))
Text10.Text = (CDbl(Text6.Text)) * (CDbl(Text3.Text))
Text11.Text = (CDbl(Text6.Text)) * (CDbl(Text4.Text))
End Sub
```

Form 4

```
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text1.Text = "1000"
        Case 1
            Text1.Text = "554"
        Case 2
            Text1.Text = "332"
        Case 3
            Text1.Text = "400"
        Case 4
            Text1.Text = "700"
```

```
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Text2.Text = Round((CDbl(Text1.Text)) * 0.2, 2)
Text3.Text = Round((CDbl(Text2.Text)) * (CDbl(DataEntry.Text4.Text) +
CDbl(DataEntry.Text7.Text)), 2)
Text4.Text = Round(0.25 * ((CDbl(Form1.Text1.Text) / 2) +
(CDbl(Form2.Text5.Text))) - 0.225 * (CDbl(Form1.Text1.Text) / 2), 2)
Text5.Text = Round(CDbl(Text4.Text) * (CDbl(DataEntry.Text4.Text) +
CDbl(DataEntry.Text7.Text)), 2)
End Sub
Private Sub Command2_Click()
Form4.Show vbModal
End Sub
```

Form 5

```
Dim a(110, 0) As Double
Private Sub Command1_Click()
   Text3.Text = a(Val(Text2.Text), 0)
   Text4.Text = Round(CDbl(Text1.Text) * CDbl(Text3.Text), 2)
   Text6.Text = Round(CDbl(Text5.Text) * 3, 2)
   Text7.Text = Round(Val(CDbl(Text4.Text)) + Val(CDbl(Text6.Text)), 2)
   Text8.Text = Round(CDbl(DataEntry.Text1.Text) * 4.5, 2)
   Text9.Text = Round(CDbl(Text1.Text) * 2.4, 2)
   If Val(Text4) > Val(Text8) Then
      If Val(Text4) > Val(Text9) Then
        Text10 = Text4
```

```
     End If
   ElseIf Val(Text9) > Val(Text8) Then
      Text10 = Text9
   Else
      Text10 = Text8
   End If
   Text11.Text = Round(CDbl(Text10.Text) * (CDbl(DataEntry.Text4.Text) +
CDbl(DataEntry.Text7.Text)), 2)
End Sub
Private Sub Command2_Click()
Form5.Show vbModal
End Sub
Private Sub Form_Load()
a(0, 0) = 0.4
a(1, 0) = 0.46
a(2, 0) = 0.52
a(3, 0) = 0.575
a(4, 0) = 0.63
a(5, 0) = 0.68
a(6, 0) = 0.73
a(7, 0) = 0.775
a(8, 0) = 0.82
a(9, 0) = 0.865
a(10, 0) = 0.91
a(11, 0) = 0.942
a(12, 0) = 0.974
a(13, 0) = 1
a(14, 0) = 1.03
a(15, 0) = 1.07
a(16, 0) = 1.09
a(17, 0) = 1.11
a(18, 0) = 1.14
a(19, 0) = 1.16
a(20, 0) = 1.19
a(21, 0) = 1.21
```

a(22, 0) = 1.23

a(23, 0) = 1.25

a(24, 0) = 1.27

a(25, 0) = 1.3

a(26, 0) = 1.322

a(27, 0) = 1.344

a(28, 0) = 1.366

a(29, 0) = 1.388

a(30, 0) = 1.41

a(31, 0) = 1.426

a(32, 0) = 1.442

a(33, 0) = 1.458

a(34, 0) = 1.474

a(35, 0) = 1.49

a(36, 0) = 1.506

a(37, 0) = 1.522

a(38, 0) = 1.538

a(39, 0) = 1.554

a(40, 0) = 1.57

a(41, 0) = 1.584

a(42, 0) = 1.598

a(43, 0) = 1.612

a(44, 0) = 1.626

a(45, 0) = 1.64

a(46, 0) = 1.654

a(47, 0) = 1.668

a(48, 0) = 1.682

a(49, 0) = 1.696

a(50, 0) = 1.71

a(51, 0) = 1.722

a(52, 0) = 1.734

a(53, 0) = 1.746

a(54, 0) = 1.758

a(55, 0) = 1.77

a(56, 0) = 1.782

a(57, 0) = 1.794

a(58, 0) = 1.806

a(59, 0) = 1.818

a(60, 0) = 1.83

a(61, 0) = 1.84

a(62, 0) = 1.85

a(63, 0) = 1.86

a(64, 0) = 1.87

a(65, 0) = 1.88

a(66, 0) = 1.89

a(67, 0) = 1.9

a(68, 0) = 1.91

a(69, 0) = 1.92

a(70, 0) = 1.93

a(71, 0) = 1.939

a(72, 0) = 1.948

a(73, 0) = 1.957

a(74, 0) = 1.966

a(75, 0) = 1.975

a(76, 0) = 1.984

a(77, 0) = 1.993

a(78, 0) = 2.002

a(79, 0) = 2.011

a(80, 0) = 2.02

a(81, 0) = 2.028

a(82, 0) = 2.036

a(83, 0) = 2.044

a(84, 0) = 2.052

a(85, 0) = 2.06

a(86, 0) = 2.068

a(87, 0) = 2.076

a(88, 0) = 2.084

a(89, 0) = 2.092

a(90, 0) = 2.1

a(91, 0) = 2.107

```
a(92, 0) = 2.114
a(93, 0) = 2.121
a(94, 0) = 2.128
a(95, 0) = 2.135
a(96, 0) = 2.142
a(97, 0) = 2.149
a(98, 0) = 2.156
a(99, 0) = 2.163
a(100, 0) = 2.17
a(101, 0) = 2.177
a(102, 0) = 2.184
a(103, 0) = 2.191
a(103, 0) = 2.198
a(104, 0) = 2.205
a(105, 0) = 2.212
a(106, 0) = 2.219
a(107, 0) = 2.226
a(108, 0) = 2.233
a(109, 0) = 2.24
a(110, 0) = 2.24
End Sub
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
  Select Case Combo1.ListIndex
    Case 0
      Text5.Text = "13.4"
    Case 1
      Text5.Text = "20.4"
    Case 2
      Text5.Text = "20.4"
    Case 3
      Text5.Text = "3.2"
    Case 4
      Text5.Text = "7.2"
```

```
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
```

Form 6

```
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text1.Text = "1.5"
        Case 1
            Text1.Text = "0.66"
        Case 2
            Text1.Text = "0.50"
        Case 3
            Text1.Text = "0.6"
        Case 4
            Text1.Text = "0.8"
        Case 5
            Text1.Text = "0.45"
        Case 6
            Text1.Text = "0.5"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
```

```
Private Sub Combo1_KeyPress(KeyAscii As Integer)
   MyMethod
End Sub
Private Sub Combo1_Scroll()
   MyMethod
End Sub
Private Sub Command1_Click()
Text2.Text = 0.5 * (CDbl(Text1.Text)) * (CDbl(DataEntry.Text3.Text)) *
(CDbl(DataEntry.Text3.Text))
Text3.Text = Round((CDbl(Text2.Text)) * (Cos(20 * 3.14 / 180)), 0)
Text4.Text = Round((CDbl(Text3.Text)) * (CDbl(DataEntry.Text4.Text)) *
(((CDbl(Form1.Text7.Text)) + (CDbl(Form1.Text9.Text))) / 2), 2)
Text5.Text = Round((2 / 3) * (CDbl(Text7.Text)), 2)
Text6.Text = Round((CDbl(Text5.Text)) * (CDbl(Text4.Text)), 2)
Text8.Text = Round((CDbl(Text2.Text)) * (Sin(20 * 3.14 / 180)), 1)
Text9.Text = Round((CDbl(Text8.Text)) * (CDbl(DataEntry.Text4.Text)) *
(CDbl(Form1.Text6.Text) + (((CDbl(Form1.Text7.Text)) +
(CDbl(Form1.Text9.Text))) / 2)), 2)
Text10.Text = Round((2 / 3) * (CDbl(Text7.Text)), 2)
Text11.Text = Round((CDbl(Text9.Text)) * (CDbl(Text10.Text)), 2)
End Sub
Private Sub Command2_Click()
Form6.Show vbModal
End Sub
```

Form 7

```
Private Sub MyMethod()
   Select Case Combo1.ListIndex
      Case 0
         Text1.Text = "0.1"
      Case 1
         Text1.Text = "0.16"
      Case 2
         Text1.Text = "0.24"
      Case 3
         Text1.Text = "0.36"
```

```
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub

Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Text5.Text = Round(((CDbl(Text1.Text)) / 2) * (CDbl(Text2.Text)) *
((CDbl(Text3.Text)) / (CDbl(Text4.Text))), 2)
Text6.Text = Round((CDbl(Form1.Text1.Text)), 2)
Text7.Text = Round((CDbl(Form1.Text19.Text)), 2)
Text8.Text = Round((CDbl(Form1.Text13.Text)), 2)
Text9.Text = Round((CDbl(Text5.Text)) * (CDbl(Text6.Text)), 2)
Text10.Text = Round((CDbl(Text5.Text)) * (CDbl(Text7.Text)), 2)
Text11.Text = Round((CDbl(Text5.Text)) * (CDbl(Text8.Text)), 2)
Text18.Text = Round(((CDbl(Text9.Text)) + (CDbl(Text10.Text)) +
(CDbl(Text11.Text))), 2)
Text12.Text = Round((CDbl(DataEntry.Text4.Text) + 1.8), 2)
Text13.Text = Round(((CDbl(DataEntry.Text4.Text)) +
((CDbl(DataEntry.Text7.Text)) / 2)), 2)
Text14.Text = 4.2
Text15.Text = Round((CDbl(Text9.Text)) * (CDbl(Text12.Text)), 2)
Text16.Text = Round((CDbl(Text10.Text)) * (CDbl(Text13.Text)), 2)
Text17.Text = Round((CDbl(Text11.Text)) * (CDbl(Text14.Text)), 2)
Text19.Text = Round(((CDbl(Text15.Text)) + (CDbl(Text16.Text)) +
(CDbl(Text17.Text))), 2)
Text20.Text = Round(((CDbl(Text18.Text)) - ((CDbl(Text9.Text)) / 2)), 2)
Text21.Text = Round((((CDbl(Text15.Text)) / 2) + (CDbl(Text16.Text)) +
(CDbl(Text17.Text))), 2)
```

```
End Sub
Private Sub Command2_Click()
Form7.Show vbModal
End Sub
```

Form 8

```
Private Sub Command1_Click()
Text11.Text = (CDbl(Form1.Text4.Text))
Text53.Text = (CDbl(Form1.Text5.Text))
Text12.Text = (CDbl(Form1.Text17.Text))
Text4.Text = (CDbl(Form2.Text5.Text))
Text3.Text = (CDbl(Form2.Text12.Text))
Text19.Text = (CDbl(Form2.Text2.Text))
Text20.Text = (CDbl(Form2.Text4.Text))
Text43.Text = (CDbl(Form2.Text1.Text))
Text44.Text = (CDbl(Form2.Text3.Text))
Text9.Text = (CDbl(Form2.Text7.Text))
Text27.Text = (CDbl(Form2.Text10.Text))
Text51.Text = (CDbl(Form2.Text11.Text))
Text38.Text = (CDbl(Form3.Text2.Text))
Text50.Text = (CDbl(Form3.Text3.Text))
Text37.Text = (CDbl(Form3.Text4.Text))
Text49.Text = (CDbl(Form3.Text5.Text))
Text64.Text = (CDbl(Form4.Text10.Text))
Text63.Text = (CDbl(Form4.Text11.Text))
Text18.Text = (CDbl(Form5.Text4.Text))
Text24.Text = (CDbl(Form5.Text6.Text))
Text36.Text = (CDbl(Form5.Text9.Text))
Text48.Text = (CDbl(Form5.Text11.Text))
Text35.Text = (CDbl(Form6.Text20.Text))
Text47.Text = (CDbl(Form6.Text21.Text))
Text34.Text = (CDbl(Form6.Text18.Text))
Text46.Text = (CDbl(Form6.Text19.Text))
Text2.Text = (CDbl(Form1.Text26.Text))
End Sub
Private Sub Command2_Click()
```

Form8.Show vbModal

Form 9

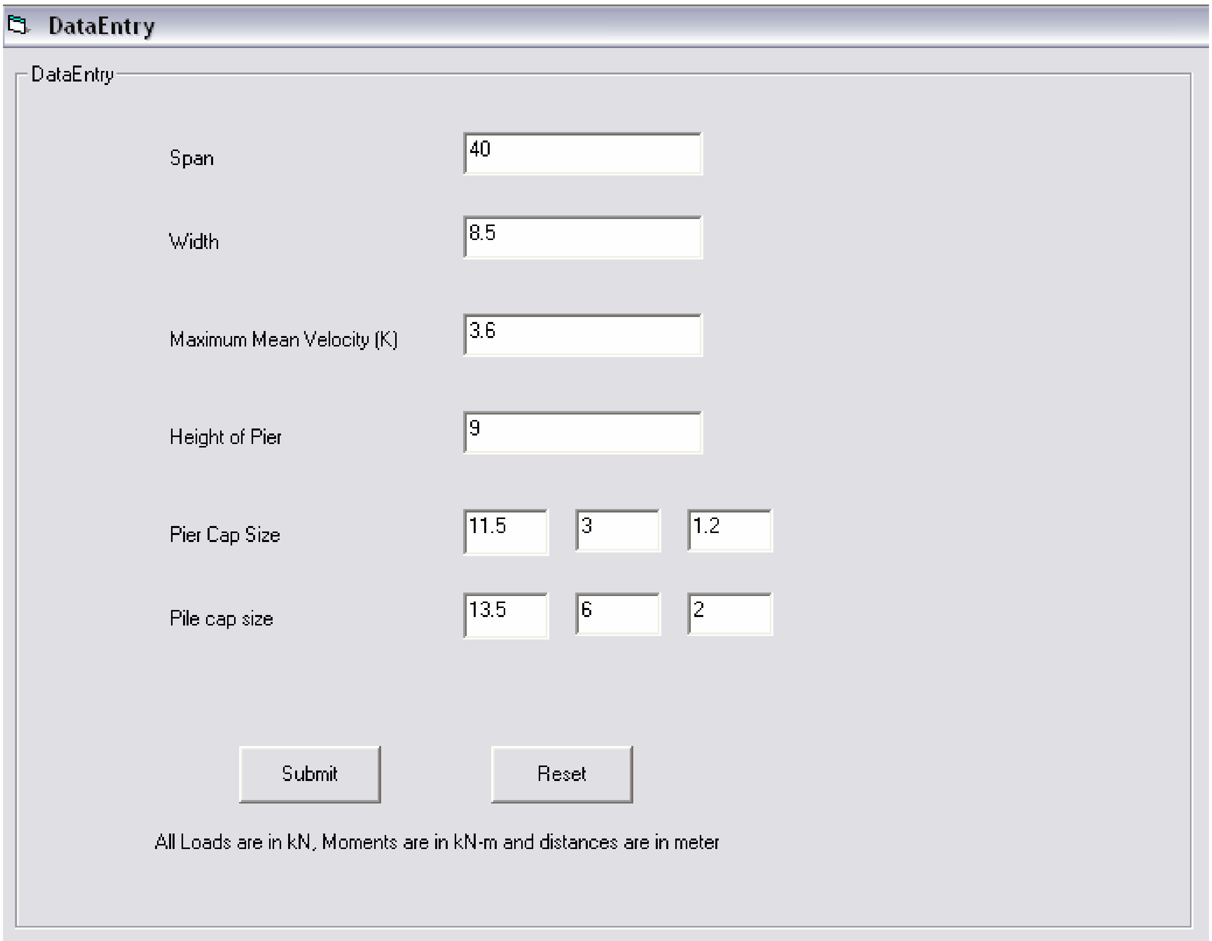
```
Private Sub Command1_Click()
'Load combination 1
Text1.Text = Val(CDbl(Form7.Text11.Text)) - Val(CDbl(Form7.Text2.Text))
Text7.Text = Val(CDbl(Form7.Text18.Text)) + Val(CDbl(Form7.Text64.Text))
Text6.Text = Val(CDbl(Form7.Text36.Text)) + Val(CDbl(Form7.Text37.Text))
Text5.Text = Val(CDbl(Form7.Text63.Text)) + Val(CDbl(Form7.Text24.Text))
Text4.Text = Val(CDbl(Form7.Text53.Text)) + Val(CDbl(Form7.Text48.Text)) +
Val(CDbl(Form7.Text49.Text))
'Load combination 2
Text2.Text = Val(CDbl(Form7.Text11.Text)) - Val(CDbl(Form7.Text2.Text))
Text10.Text = Val(CDbl(Form7.Text18.Text))
Text9.Text = Val(CDbl(Form7.Text36.Text)) + Val(CDbl(Form7.Text37.Text)) +
Val(0.5 * (CDbl(Form7.Text35.Text)))
Text8.Text = (CDbl(Form7.Text24.Text))
Text3.Text = Val(CDbl(Form7.Text53.Text)) + Val(CDbl(Form7.Text48.Text)) +
Val(CDbl(Form7.Text49.Text)) + Val(0.5 * (CDbl(Form7.Text47.Text)))
'Load combination 3
Text15.Text = Val(CDbl(Form7.Text12.Text)) + Val(CDbl(Form7.Text3.Text)) +
Val(CDbl(Form7.Text9.Text)) - Val(CDbl(Form7.Text2.Text))
Text11.Text = Val(CDbl(Form7.Text18.Text))
Text12.Text = Val(CDbl(Form7.Text36.Text)) + Val(CDbl(Form7.Text38.Text)) +
Val(CDbl(Form7.Text37.Text))
Text13.Text = Val(CDbl(Form7.Text44.Text)) + Val(CDbl(Form7.Text51.Text)) +
Val(CDbl(Form7.Text24.Text))
Text14.Text = Val(CDbl(Form7.Text20.Text)) + Val(CDbl(Form7.Text27.Text)) +
Val(CDbl(Form7.Text48.Text)) + Val(CDbl(Form7.Text50.Text)) +
Val(CDbl(Form7.Text49.Text))
'Load combination 4
Text16.Text = Val(CDbl(Form7.Text12.Text)) + Val(CDbl(Form7.Text3.Text)) +
Val(CDbl(Form7.Text9.Text)) - Val(CDbl(Form7.Text2.Text))
Text20.Text = Val(CDbl(Form7.Text18.Text)) + Val(CDbl(Form7.Text64.Text))
Text19.Text = Val(CDbl(Form7.Text36.Text)) + Val(CDbl(Form7.Text38.Text)) +
Val(CDbl(Form7.Text37.Text))
```

```
Text18.Text = Val(CDbl(Form7.Text44.Text)) + Val(CDbl(Form7.Text51.Text)) +
Val(CDbl(Form7.Text24.Text)) + Val(CDbl(Form7.Text63.Text))
Text17.Text = Val(CDbl(Form7.Text20.Text)) + Val(CDbl(Form7.Text27.Text)) +
Val(CDbl(Form7.Text48.Text)) + Val(CDbl(Form7.Text50.Text)) +
Val(CDbl(Form7.Text49.Text))
'Load combination 5
Text25.Text = Val(CDbl(Form7.Text12.Text)) + Val(0.5 *
CDbl(Form7.Text3.Text)) + Val(0.5 * CDbl(Form7.Text9.Text)) -
Val(CDbl(Form7.Text2.Text))
Text21.Text = Val(CDbl(Form7.Text18.Text))
Text22.Text = Val(CDbl(Form7.Text36.Text)) + Val(0.5 *
CDbl(Form7.Text38.Text)) + Val(0.5 * CDbl(Form7.Text37.Text)) +
Val(CDbl(Form7.Text34.Text))
Text23.Text = Val(0.5 * CDbl(Form7.Text44.Text)) + Val(0.5 *
CDbl(Form7.Text51.Text)) + Val(CDbl(Form7.Text24.Text))
Text24.Text = Val(0.5 * CDbl(Form7.Text20.Text)) + Val(0.5 *
CDbl(Form7.Text27.Text)) + Val(CDbl(Form7.Text48.Text)) + Val(0.5 *
CDbl(Form7.Text50.Text)) + Val(0.5 * CDbl(Form7.Text49.Text)) +
Val(CDbl(Form7.Text46.Text))
End Sub
Private Sub Command2_Click()
Form9.Show vbModal
End Sub
```

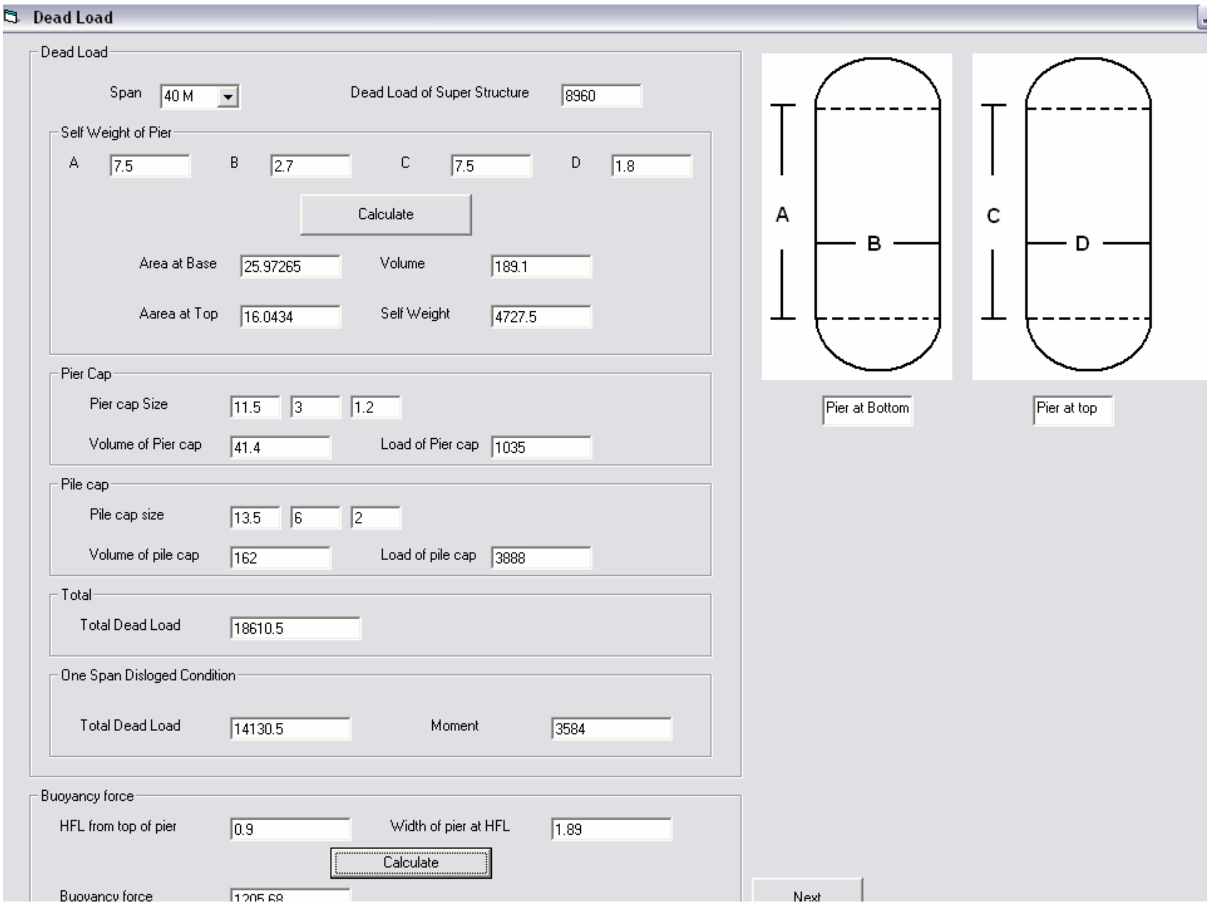
Appendix 2

For parametric study of pile the program is developed in Visual Basic-6. This program gives design load and moments on pile. This program's forms and source files are given in this appendix.

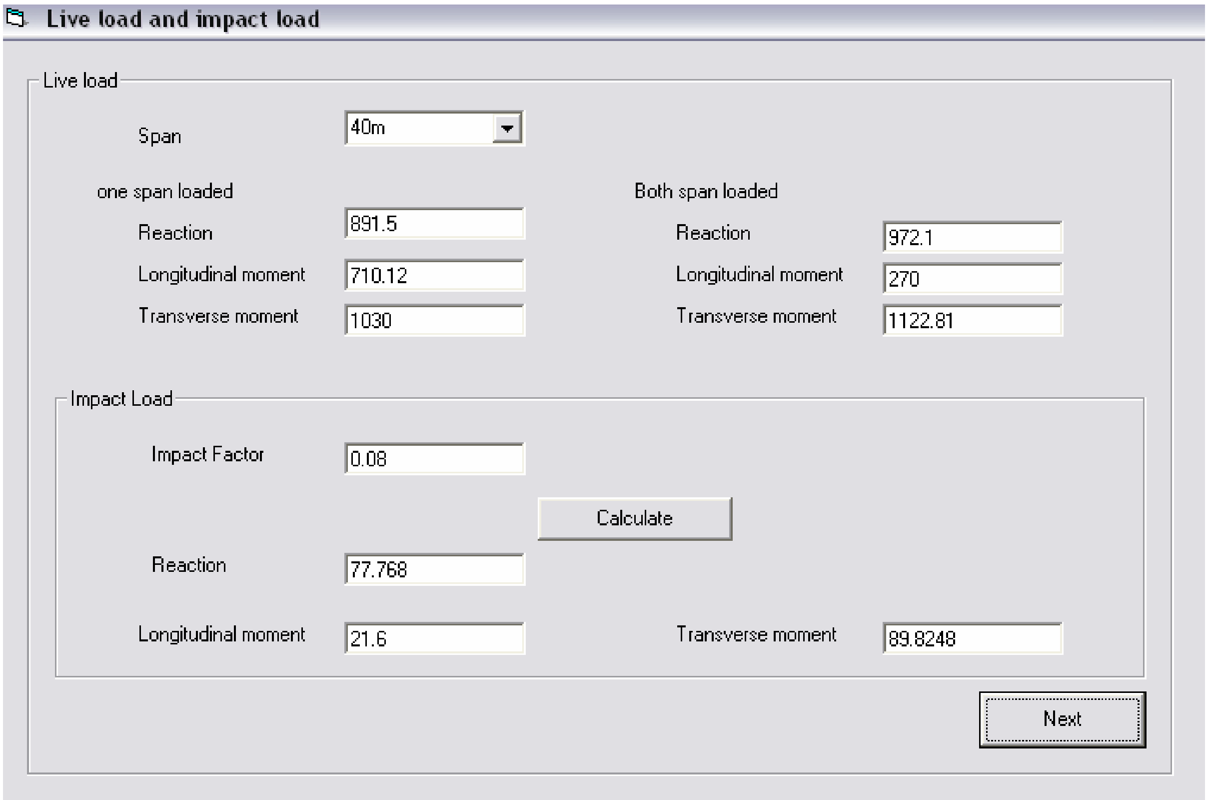
Form 1



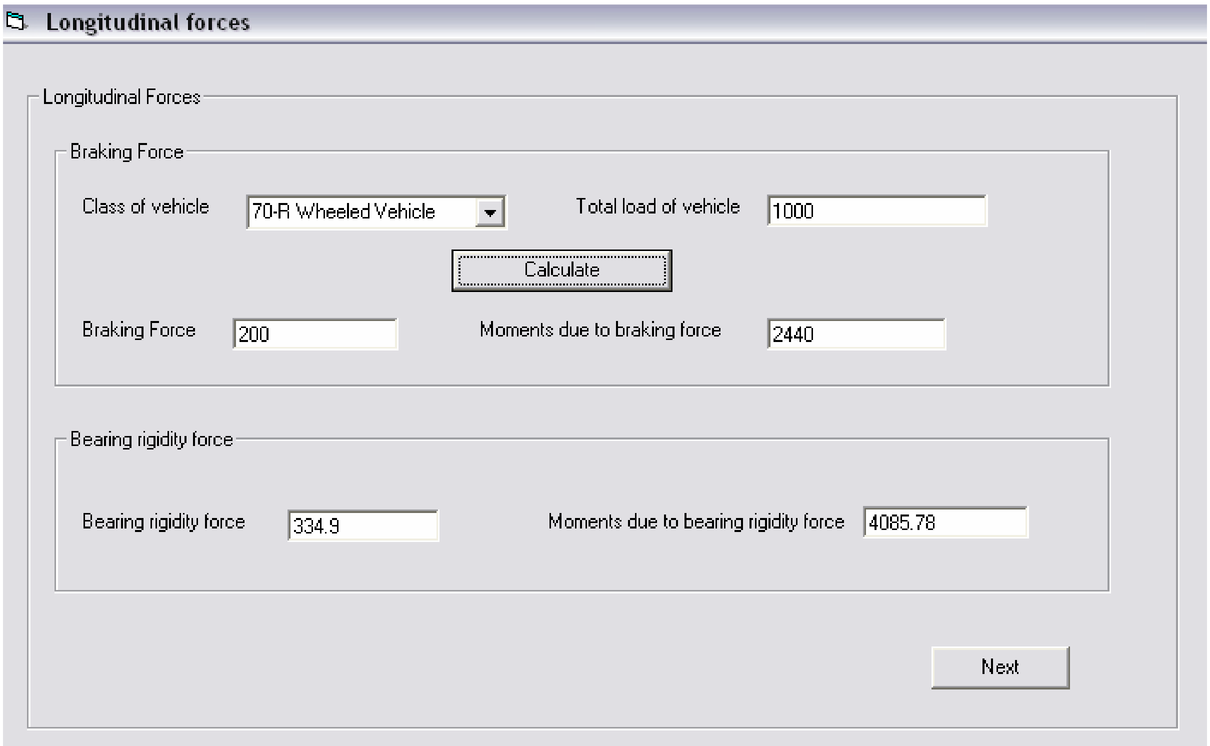
Form 2


Dead Load
Dead Load
Span
40 M
Dead Load of Super Structure
8960
Self Weight of Pier
A
7.5
B
2.7
C
7.5
D
1.8
Calculate
Area at Base
25.97265
Volume
189.1
Aarea at Top
16.0434
Self Weight
4727.5
Pier Cap
Pier cap Size
11.5
3
1.2
Volume of Pier cap
41.4
Load of Pier cap
1035
Pile cap
Pile cap size
13.5
6
2
Volume of pile cap
162
Load of pile cap
3888
Total
Total Dead Load
18610.5
One Span Disloged Condition
Total Dead Load
14130.5
Moment
3584
Buoyancy force
HFL from top of pier
0.9
Width of pier at HFL
1.89
Calculate
Buoyancy force
A
B
C
D
Pier at Bottom
Pier at top
Next

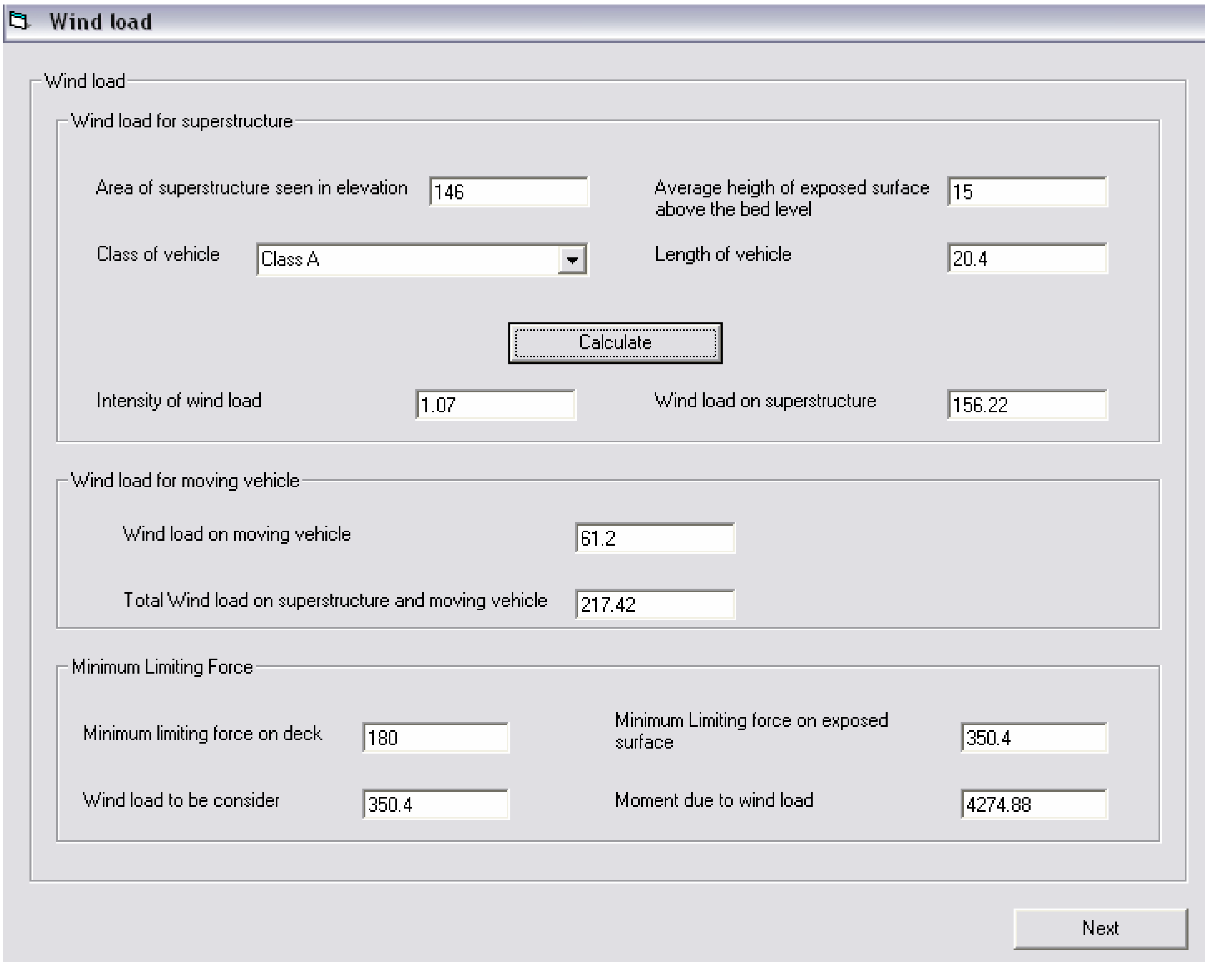
Form 3


Live load and impact load
Live load
Span
40m
one span loaded
Reaction
891.5
Longitudinal moment
710.12
Transverse moment
1030
Both span loaded
Reaction
972.1
Longitudinal moment
270
Transverse moment
1122.81
Impact Load
Impact Factor
0.08
Calculate
Reaction
77.768
Longitudinal moment
21.6
Transverse moment
89.8248
Next

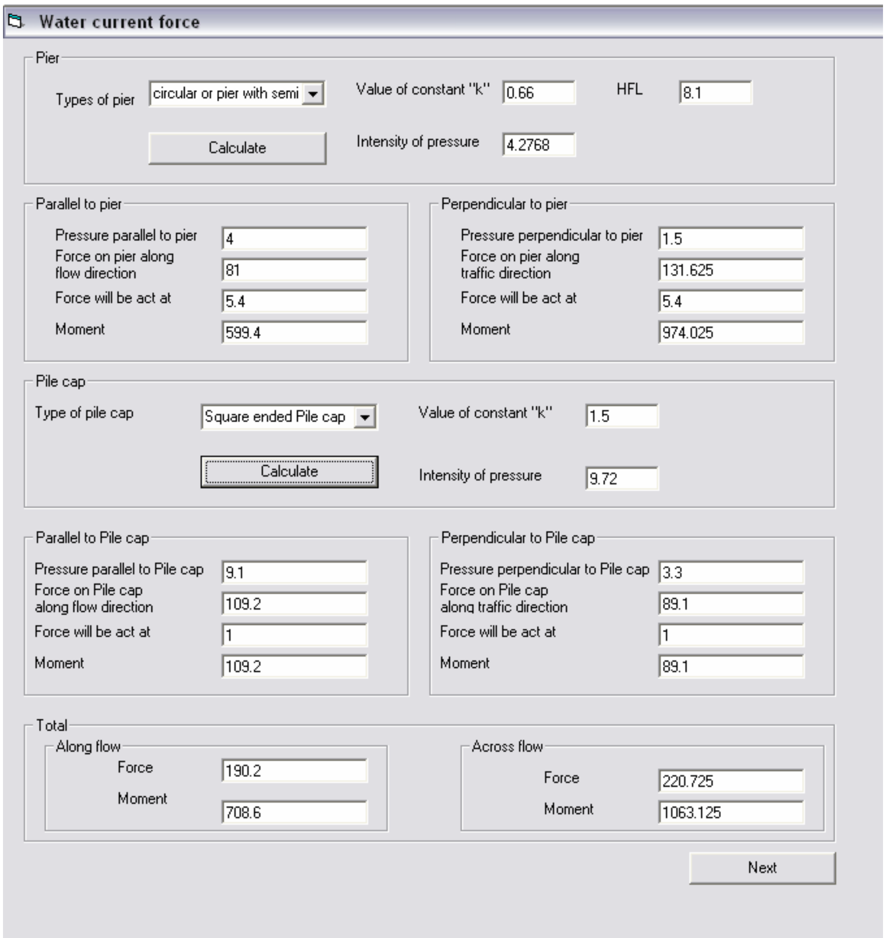
Form 4


Longitudinal forces
Longitudinal Forces
Braking Force
Class of vehicle
70-R Wheeled Vehicle
Total load of vehicle
1000
Calculate
Braking Force
200
Moments due to braking force
2440
Bearing rigidity force
Bearing rigidity force
334.9
Moments due to bearing rigidity force
4085.78
Next

Form 5


Wind load
Wind load
Wind load for superstructure
Area of superstructure seen in elevation
146
Average heigth of exposed surface above the bed level
15
Class of vehicle
Class A
Length of vehicle
20.4
Calculate
Intensity of wind load
1.07
Wind load on superstructure
156.22
Wind load for moving vehicle
Wind load on moving vehicle
61.2
Total Wind load on superstructure and moving vehicle
217.42
Minimum Limiting Force
Minimum limiting force on deck
180
Minimum Limiting force on exposed surface
350.4
Wind load to be consider
350.4
Moment due to wind load
4274.88
Next

Form 6



Water current force

Pier

Types of pier: circular or pier with semi | Value of constant "k": 0.66 | HFL: 8.1

Calculate | Intensity of pressure: 4.2768

Parallel to pier		Perpendicular to pier	
Pressure parallel to pier	4	Pressure perpendicular to pier	1.5
Force on pier along flow direction	81	Force on pier along traffic direction	131.625
Force will be act at	5.4	Force will be act at	5.4
Moment	599.4	Moment	974.025

Pile cap

Type of pile cap: Square ended Pile cap | Value of constant "k": 1.5

Calculate | Intensity of pressure: 9.72

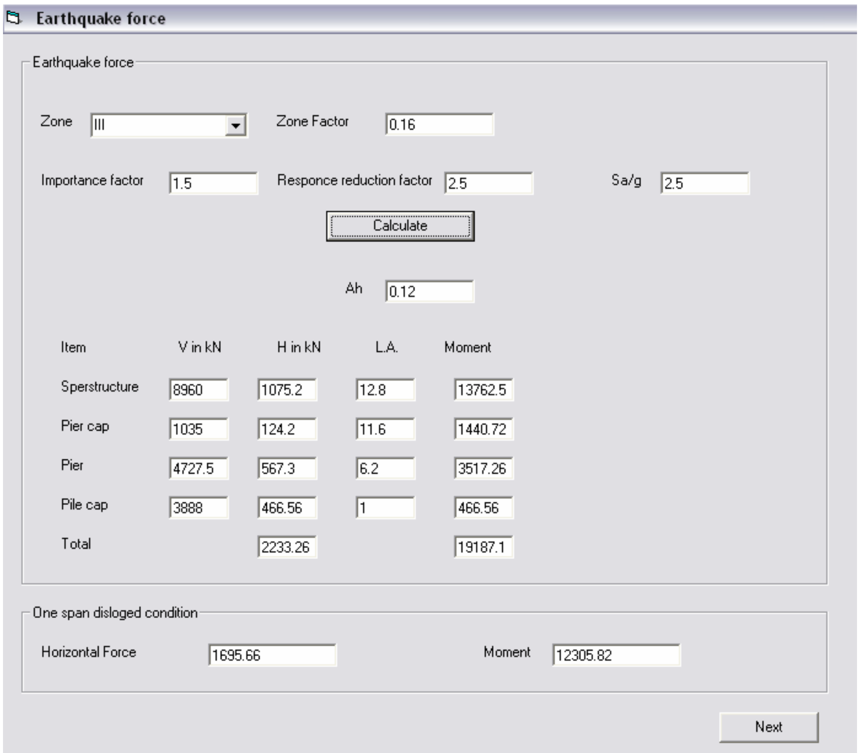
Parallel to Pile cap		Perpendicular to Pile cap	
Pressure parallel to Pile cap	9.1	Pressure perpendicular to Pile cap	3.3
Force on Pile cap along flow direction	109.2	Force on Pile cap along traffic direction	89.1
Force will be act at	1	Force will be act at	1
Moment	109.2	Moment	89.1

Total

Along flow		Across flow	
Force	190.2	Force	220.725
Moment	708.6	Moment	1063.125

Next

Form 7



Earthquake force

Earthquake force

Zone: III | Zone Factor: 0.16

Importance factor: 1.5 | Responce reduction factor: 2.5 | Sa/g: 2.5

Calculate

Ah: 0.12

Item	V in kN	H in kN	L.A.	Moment
Sperstructure	8960	1075.2	12.8	13762.5
Pier cap	1035	124.2	11.6	1440.72
Pier	4727.5	567.3	6.2	3517.26
Pile cap	3888	466.56	1	466.56
Total		2233.26		19187.1

One span disloged condition

Horizontal Force: 1695.66 | Moment: 12305.82

Next

Form 8

Summary

Summary of forces

Dispaly

Forces	Vertical Load (kN)	Along Flow direction		Along Traffic direction	
		HT	MT	HL	ML
(1) Dead Load					
One Span dislodged condition	14130.5				3584
Both side span condition	18610.5				
(2) Live Load					
One Span dislodged condition	891.5		710.12		1030
Both side span condition	972.1		270		1122.81
(3) Impact Load	77.768		89.8248		21.6
(4) Buoyancy Force	1205.68				
(5) Longitudinal Force					
(a) Braking Force				200	2440
(b) Bearing rigidity force Force				334.9	4085.78
(6) Wind Load		350.4	4274.88		
(7) Water current Force with 20 degree Obliquity		190.2	708.6	220.725	1063.125
(8) Earthquake Force					
One Span dislodged condition				1695.66	12305.82
Both side span condition				2233.26	19187.1

Load Combinations

Form 9

Load combination as per IRC-6

Construction condition

Calculate

Load combination 1

(1) D.L + Wind load + Water current force + Bearing rigidity force + Buoyancy force

P 12924.82 HT 540.6 HL 555.62 MT 4983.48 ML 8732.91

Load combination 2

(2) D.L. + Watercurrent force + Bearing Rigidity force + Buoyancy force + (0.5) Seismic force

P 12924.82 HT 190.2 HL 1403.46 MT 708.6 ML 14885.82

Service condition

Load combination 3

(3) D.L. + L. L. + Impact force+ Watercurrent force + Braking Force + Bearing Rigidity force + Buoyancy force

P 18454.69 HT 190.2 HL 755.62 MT 1068.42 ML 8733.32

Load combination 4

(4) D.L. + L. L. + Impact force + Wind Force + Watercurrent force + Braking Force + Bearing Rigidity force + Buoyancy force

P 18454.69 HT 540.6 HL 755.62 MT 5343.3 ML 8733.32

Load combination 5

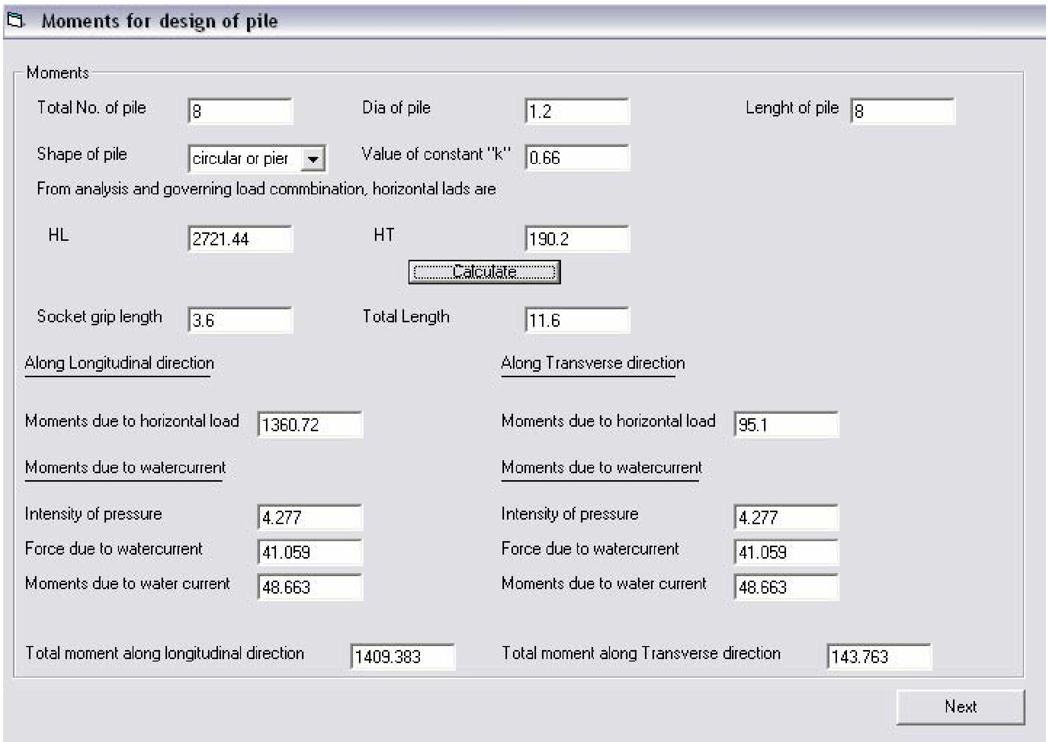
(5) D.L. + (0.5) L.L. + (0.5) Impact Force + Watercurrent Force + (0.5) Braking Force + (0.5) Bearing Rigidity Force + Buoyancy Force + Seismic Force

P 17929.75 HT 190.2 HL 2721.44 MT 888.51 ML 24085.32

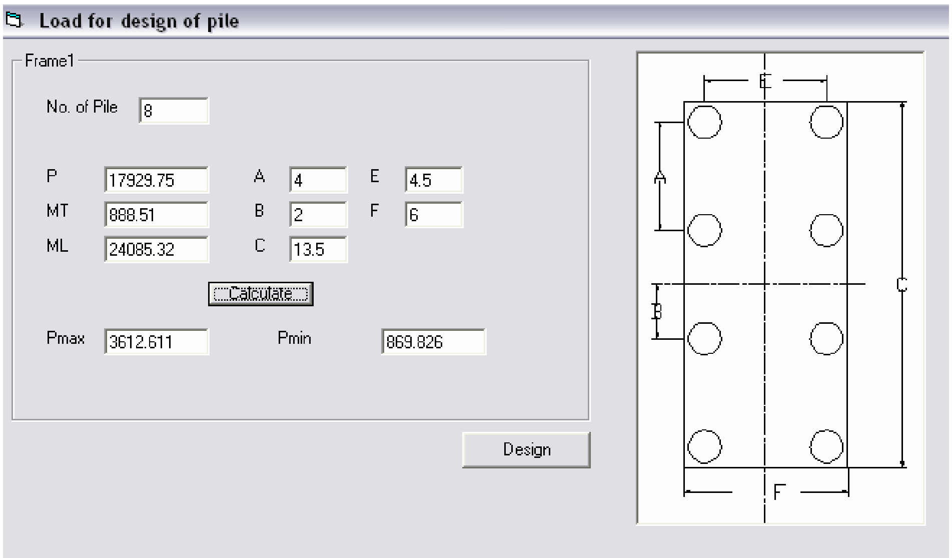
Governing Load combination

P 17929.75 HT 190.2 HL 2721.44 MT 888.51 ML 24085.32

Form 10



Form 11



Form 1

```
Option Explicit
Dim span1 As Double
Dim widht As Double
Dim K As Double
Dim Hp As Double
Dim Pcz1 As Double
Dim Pcz2 As Double
```

```
Dim Pcz3 As Double
Dim Picz1 As Double
Dim Picz2 As Double
Dim Picz3 As Double
Private Sub Command1_Click()
   span1 = CDbl(Text1.Text)
   widht = CDbl(Text2.Text)
   K = CDbl(Text3.Text)
   Hp = CDbl(Text4.Text)
   Pcz1 = CDbl(Text5.Text)
   Pcz2 = CDbl(Text6.Text)
   Pcz3 = CDbl(Text7.Text)
   Picz1 = CDbl(Text8.Text)
   Picz2 = CDbl(Text9.Text)
   Picz3 = CDbl(Text10.Text)
   Form2.Show vbModal
End Sub
Private Sub Command2_Click()
   Text1.Text = ""
   Text2.Text = ""
   Text3.Text = ""
   Text4.Text = ""
   Text5.Text = ""
   Text6.Text = ""
   Text7.Text = ""
   Text8.Text = ""
   Text9.Text = ""
   Text10.Text = ""
End Sub
```

Form 2

```
Private Sub MyMethod()
   Select Case Combo1.ListIndex
      Case 0
         Text1.Text = "2974"
      Case 1
```

```
            Text1.Text = "3536"
        Case 2
            Text1.Text = "4632"
        Case 3
            Text1.Text = "5546"
        Case 4
            Text1.Text = "6100"
        Case 5
            Text1.Text = "7503"
        Case 6
            Text1.Text = "8960"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
    Text10.Text = (Val((CDbl(Text6.Text) * CDbl(Text7.Text))) +
Val(((CDbl(Text7.Text) * CDbl(Text7.Text)) * 3.14 / 4)))
    Text11.Text = ((CDbl(Text8.Text) * CDbl(Text9.Text)) + ((CDbl(Text9.Text)
* CDbl(Text9.Text)) * 3.14 / 4))
    Text12.Text = Round(((CDbl(Text11.Text) + CDbl(Text10.Text)) / 2) *
CDbl(Form1.Text4.Text), 1)
    Text13.Text = (CDbl(Text12.Text)) * 25
    Text18.Text = CDbl(Text14.Text) * CDbl(Text15.Text) * CDbl(Text16.Text)
    Text19.Text = CDbl(Text18.Text) * 25
    Text5.Text = (CDbl(Text1.Text) / 2) * 0.8
    Text21.Text = CDbl(Text22.Text) * CDbl(Text23.Text) * CDbl(Text24.Text)
    Text20.Text = (CDbl(Text21.Text)) * 24
```

```
   Text4.Text = (CDbl(Text1.Text) / 2) + CDbl(Text13.Text) +
CDbl(Text19.Text) + CDbl(Text20.Text)
   Text17.Text = (Val(CDbl(Text19.Text)) + Val(CDbl(Text13.Text)) +
Val(CDbl(Text1.Text)) + Val(CDbl(Text20.Text)))
End Sub
Private Sub Command2_Click()
Form3.Show vbModal
End Sub
Private Sub Command3_Click()
Text26.Text = Round((Val((((((CDbl(Text6.Text) * ((CDbl(Text27.Text) +
CDbl(Text7.Text)) / 2)) + ((((CDbl(Text27.Text) + CDbl(Text7.Text)) / 2) *
(CDbl(Text27.Text) + CDbl(Text7.Text)) / 2) * 3.14 / 4)) *
(CDbl(Form1.Text4.Text) - CDbl(Text25.Text))) * 24)))) +
Val(CDbl(Text20.Text))) * 0.15, 2)
End Sub
Private Sub Form_Load()
   Text1.Text = "2974 kN"
   Combo1.ListIndex = 0
    Text14.Text = Form1.Text5.Text
    Text15.Text = Form1.Text6.Text
    Text16.Text = Form1.Text7.Text
    Text22.Text = Form1.Text8.Text
    Text23.Text = Form1.Text10.Text
    Text24.Text = Form1.Text9.Text
End Sub
```

Form 3

```
Private Sub MyMethod()
   Select Case Combo1.ListIndex
      Case 0
         Text1.Text = "705.7"
         Text2.Text = "815.1"
         Text3.Text = "280.5"
         Text4.Text = "955.7"
         Text5.Text = "705.7"
         Text12.Text = "827.5"
```

```
Case 1
    Text1.Text = "742.5"
    Text2.Text = "857.5"
    Text3.Text = "285.4"
    Text4.Text = "966.6"
    Text5.Text = "742.5"
    Text12.Text = "849"
Case 2
    Text1.Text = "783.1"
    Text2.Text = "904.5"
    Text3.Text = "282.9"
    Text4.Text = "973.4"
    Text5.Text = "783.1"
    Text12.Text = "855.1"
Case 3
    Text1.Text = "812.7"
    Text2.Text = "938.6"
    Text3.Text = "294.8"
    Text4.Text = "992.5"
    Text5.Text = "812.7"
    Text12.Text = "890.2"
Case 4
    Text1.Text = "852.8"
    Text2.Text = "984.9"
    Text3.Text = "299.9"
    Text4.Text = "1018.3"
    Text5.Text = "852.8"
    Text12.Text = "913.3"
Case 5
    Text1.Text = "875.1"
    Text2.Text = "1010.7"
    Text3.Text = "303.2"
    Text4.Text = "1033.3"
    Text5.Text = "875.1"
    Text12.Text = "926.8"
```

```
        Case 6
            Text1.Text = "710.12"
            Text2.Text = "1030"
            Text3.Text = "270"
            Text4.Text = "1122.81"
            Text5.Text = "891.5"
            Text12.Text = "972.1"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Form4.Show vbModal
End Sub
Private Sub Command2_Click()
Text7.Text = (CDbl(Text6.Text)) * (CDbl(Text12.Text))
Text10.Text = (CDbl(Text6.Text)) * (CDbl(Text3.Text))
Text11.Text = (CDbl(Text6.Text)) * (CDbl(Text4.Text))
End Sub
```

Form 4

```
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text1.Text = "1000"
        Case 1
            Text1.Text = "554"
```

```
        Case 2
           Text1.Text = "332"
        Case 3
           Text1.Text = "400"
        Case 4
           Text1.Text = "700"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Text2.Text = (CDbl(Text1.Text)) * 0.2
Text3.Text = (CDbl(Text2.Text)) * (CDbl(Form1.Text4.Text) +
CDbl(Form1.Text9.Text) + CDbl(Form1.Text7.Text))
Text4.Text = Round(0.25 * ((CDbl(Form2.Text1.Text) / 2) +
(CDbl(Form3.Text5.Text))) - 0.225 * (CDbl(Form2.Text1.Text) / 2), 1)
Text5.Text = CDbl(Text4.Text) * (CDbl(Form1.Text4.Text) +
CDbl(Form1.Text9.Text) + CDbl(Form1.Text7.Text))
End Sub
Private Sub Command2_Click()
Form5.Show vbModal
End Sub
```

Form 5

```
Dim a(110, 0) As Double
Private Sub Command1_Click()
   Text3.Text = a(Val(Text2.Text), 0)
   Text4.Text = CDbl(Text1.Text) * CDbl(Text3.Text)
   Text6.Text = CDbl(Text5.Text) * 3
```

```
  Text7.Text = Val(CDbl(Text4.Text)) + Val(CDbl(Text6.Text))
  Text8.Text = CDbl(Form1.Text1.Text) * 4.5
  Text9.Text = CDbl(Text1.Text) * 2.4
  If Val(Text4) > Val(Text8) Then
    If Val(Text4) > Val(Text9) Then
     Text10 = Text4
    End If
   ElseIf Val(Text9) > Val(Text8) Then
     Text10 = Text9
   Else
     Text10 = Text8
   End If
   Text11.Text = (CDbl(Text10.Text) * (Val(CDbl(Form1.Text4.Text) +
Val(CDbl(Form1.Text9.Text)) + Val(CDbl(Form1.Text7.Text)))))
End Sub
Private Sub Command2_Click()
Form6.Show vbModal
End Sub
Private Sub Form_Load()
a(0, 0) = 0.4
a(1, 0) = 0.46
a(2, 0) = 0.52
a(3, 0) = 0.575
a(4, 0) = 0.63
a(5, 0) = 0.68
a(6, 0) = 0.73
a(7, 0) = 0.775
a(8, 0) = 0.82
a(9, 0) = 0.865
a(10, 0) = 0.91
a(11, 0) = 0.942
a(12, 0) = 0.974
a(13, 0) = 1
a(14, 0) = 1.03
a(15, 0) = 1.07
```

a(16, 0) = 1.09

a(17, 0) = 1.11

a(18, 0) = 1.14

a(19, 0) = 1.16

a(20, 0) = 1.19

a(21, 0) = 1.21

a(22, 0) = 1.23

a(23, 0) = 1.25

a(24, 0) = 1.27

a(25, 0) = 1.3

a(26, 0) = 1.322

a(27, 0) = 1.344

a(28, 0) = 1.366

a(29, 0) = 1.388

a(30, 0) = 1.41

a(31, 0) = 1.426

a(32, 0) = 1.442

a(33, 0) = 1.458

a(34, 0) = 1.474

a(35, 0) = 1.49

a(36, 0) = 1.506

a(37, 0) = 1.522

a(38, 0) = 1.538

a(39, 0) = 1.554

a(40, 0) = 1.57

a(41, 0) = 1.584

a(42, 0) = 1.598

a(43, 0) = 1.612

a(44, 0) = 1.626

a(45, 0) = 1.64

a(46, 0) = 1.654

a(47, 0) = 1.668

a(48, 0) = 1.682

a(49, 0) = 1.696

a(50, 0) = 1.71

a(51, 0) = 1.722

a(52, 0) = 1.734

a(53, 0) = 1.746

a(54, 0) = 1.758

a(55, 0) = 1.77

a(56, 0) = 1.782

a(57, 0) = 1.794

a(58, 0) = 1.806

a(59, 0) = 1.818

a(60, 0) = 1.83

a(61, 0) = 1.84

a(62, 0) = 1.85

a(63, 0) = 1.86

a(64, 0) = 1.87

a(65, 0) = 1.88

a(66, 0) = 1.89

a(67, 0) = 1.9

a(68, 0) = 1.91

a(69, 0) = 1.92

a(70, 0) = 1.93

a(71, 0) = 1.939

a(72, 0) = 1.948

a(73, 0) = 1.957

a(74, 0) = 1.966

a(75, 0) = 1.975

a(76, 0) = 1.984

a(77, 0) = 1.993

a(78, 0) = 2.002

a(79, 0) = 2.011

a(80, 0) = 2.02

a(81, 0) = 2.028

a(82, 0) = 2.036

a(83, 0) = 2.044

a(84, 0) = 2.052

a(85, 0) = 2.06

```
a(86, 0) = 2.068
a(87, 0) = 2.076
a(88, 0) = 2.084
a(89, 0) = 2.092
a(90, 0) = 2.1
a(91, 0) = 2.107
a(92, 0) = 2.114
a(93, 0) = 2.121
a(94, 0) = 2.128
a(95, 0) = 2.135
a(96, 0) = 2.142
a(97, 0) = 2.149
a(98, 0) = 2.156
a(99, 0) = 2.163
a(100, 0) = 2.17
a(101, 0) = 2.177
a(102, 0) = 2.184
a(103, 0) = 2.191
a(103, 0) = 2.198
a(104, 0) = 2.205
a(105, 0) = 2.212
a(106, 0) = 2.219
a(107, 0) = 2.226
a(108, 0) = 2.233
a(109, 0) = 2.24
a(110, 0) = 2.24
End Sub
Private Sub List1_Click()

End Sub
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text5.Text = "13.4"
        Case 1
```

```
            Text5.Text = "20.4"
        Case 2
            Text5.Text = "20.4"
        Case 3
            Text5.Text = "3.2"
        Case 4
            Text5.Text = "7.2"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
```

Form 6

```
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
    Select Case Combo1.ListIndex
        Case 0
            Text1.Text = "1.5"
        Case 1
            Text1.Text = "0.66"
        Case 2
            Text1.Text = "0.50"
        Case 3
            Text1.Text = "0.6"
        Case 4
            Text1.Text = "0.8"
        Case 5
            Text1.Text = "0.45"
```

```
        Case 6
            Text1.Text = "0.5"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Text2.Text = 0.5 * (CDbl(Text1.Text)) * (CDbl(Form1.Text3.Text)) *
(CDbl(Form1.Text3.Text))
Text3.Text = Round((CDbl(Text2.Text)) * (Cos(20 * 3.14 / 180)), 1)
Text4.Text = (CDbl(Text3.Text)) * (CDbl(Form1.Text4.Text)) *
(((CDbl(Form2.Text7.Text)) + (CDbl(Form2.Text9.Text))) / 2)
Text5.Text = (2 / 3) * (CDbl(Text7.Text))
Text6.Text = (CDbl(Text4.Text)) * ((Val(CDbl(Text5.Text))) +
(Val(CDbl(Form1.Text9.Text))))
Text8.Text = Round((CDbl(Text2.Text)) * (Sin(20 * 3.14 / 180)), 1)
Text9.Text = (CDbl(Text8.Text)) * (CDbl(Form1.Text4.Text)) *
(CDbl(Form2.Text6.Text) + (((CDbl(Form2.Text7.Text)) +
(CDbl(Form2.Text9.Text))) / 2))
Text10.Text = (2 / 3) * (CDbl(Text7.Text))
Text11.Text = (CDbl(Text9.Text)) * ((Val(CDbl(Text10.Text))) +
(Val(CDbl(Form1.Text9.Text))))
End Sub
Private Sub List2_Click()
End Sub
Private Sub MyMethod1()
    Select Case Combo2.ListIndex
        Case 0
```

```
            Text12.Text = "1.5"
        Case 1
            Text12.Text = "0.66"
        Case 2
            Text12.Text = "0.50"
        Case 3
            Text12.Text = "0.6"
        Case 4
            Text12.Text = "0.8"
        Case 5
            Text12.Text = "0.45"
        Case 6
            Text12.Text = "0.5"
    End Select
End Sub
Private Sub Combo2_Click()
    MyMethod1
End Sub
Private Sub Combo2_KeyPress(KeyAscii As Integer)
    MyMethod1
End Sub
Private Sub Combo2_Scroll()
    MyMethod1
End Sub
Private Sub Command3_Click()
Text13.Text = 0.5 * (CDbl(Text12.Text)) * (CDbl(Form1.Text3.Text)) *
(CDbl(Form1.Text3.Text))
Text14.Text = Round((CDbl(Text13.Text)) * (Cos(20 * 3.14 / 180)), 1)
Text15.Text = (CDbl(Form1.Text9.Text)) * (CDbl(Form1.Text10.Text)) *
(CDbl(Text14.Text))
Text16.Text = ((CDbl(Form1.Text9.Text)) / 2)
Text17.Text = (CDbl(Text16.Text)) * (CDbl(Text15.Text))
Text18.Text = Round((CDbl(Text13.Text)) * (Sin(20 * 3.14 / 180)), 1)
Text19.Text = (CDbl(Text18.Text)) * (CDbl(Form1.Text9.Text)) *
(CDbl(Form1.Text8.Text))
```

```
Text20.Text = ((CDbl(Form1.Text9.Text)) / 2)
Text21.Text = (CDbl(Text20.Text)) * (CDbl(Text19.Text))
Text22.Text = Val(CDbl(Text4.Text)) + Val(CDbl(Text15.Text))
Text23.Text = Val(CDbl(Text6.Text)) + Val(CDbl(Text17.Text))
Text24.Text = Val(CDbl(Text9.Text)) + Val(CDbl(Text19.Text))
Text25.Text = Val(CDbl(Text11.Text)) + Val(CDbl(Text21.Text))
End Sub
Private Sub Command2_Click()
Form7.Show vbModal
End Sub
```

Form 7

```
Private Sub MyMethod()
   Select Case Combo1.ListIndex
      Case 0
         Text1.Text = "0.1"
      Case 1
         Text1.Text = "0.16"
      Case 2
         Text1.Text = "0.24"
      Case 3
         Text1.Text = "0.36"
   End Select
End Sub
Private Sub Combo1_Click()
   MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
   MyMethod
End Sub
Private Sub Combo1_Scroll()
   MyMethod
End Sub
Private Sub Command1_Click()
Text5.Text = ((CDbl(Text1.Text)) / 2) * (CDbl(Text2.Text)) *
((CDbl(Text3.Text)) / (CDbl(Text4.Text)))
```

```
Text6.Text = (CDbl(Form2.Text1.Text))
Text7.Text = (CDbl(Form2.Text19.Text))
Text8.Text = (CDbl(Form2.Text13.Text))
Text22.Text = (CDbl(Form2.Text20.Text))
Text9.Text = (CDbl(Text5.Text)) * (CDbl(Text6.Text))
Text10.Text = (CDbl(Text5.Text)) * (CDbl(Text7.Text))
Text11.Text = (CDbl(Text5.Text)) * (CDbl(Text8.Text))
Text23.Text = (CDbl(Text5.Text)) * (CDbl(Text22.Text))
Text18.Text = Round(((CDbl(Text9.Text)) + (CDbl(Text10.Text)) +
(CDbl(Text11.Text))) + (CDbl(Text23.Text)), 2)
Text12.Text = (CDbl(Form1.Text4.Text) + 1.8) + (CDbl(Form1.Text9.Text))
Text13.Text = ((CDbl(Form1.Text4.Text)) + ((CDbl(Form1.Text7.Text)) / 2)) +
(CDbl(Form1.Text9.Text))
Text14.Text = 4.2 + (CDbl(Form1.Text9.Text))
Text24.Text = ((CDbl(Form1.Text9.Text)) / 2)
Text15.Text = (CDbl(Text9.Text)) * (CDbl(Text12.Text))
Text16.Text = (CDbl(Text10.Text)) * (CDbl(Text13.Text))
Text17.Text = (CDbl(Text11.Text)) * (CDbl(Text14.Text))
Text25.Text = (CDbl(Text23.Text)) * (CDbl(Text24.Text))
Text19.Text = Round(((CDbl(Text15.Text)) + (CDbl(Text16.Text)) +
(CDbl(Text17.Text)) + (CDbl(Text25.Text))), 2)
Text20.Text = Round(((CDbl(Text18.Text)) - ((CDbl(Text9.Text)) / 2)), 2)
Text21.Text = Round((((CDbl(Text15.Text)) / 2) + (CDbl(Text16.Text)) +
(CDbl(Text17.Text)) + (CDbl(Text25.Text))), 2)
End Sub
Private Sub Command2_Click()
Form8.Show vbModal
End Sub
```

Form 8

```
Private Sub Command1_Click()
'Dead Load
Text11.Text = (CDbl(Form2.Text4.Text))
Text53.Text = (CDbl(Form2.Text5.Text))
Text12.Text = (CDbl(Form2.Text17.Text))
'Live Load
```

```
Text4.Text = (CDbl(Form3.Text5.Text))
Text3.Text = (CDbl(Form3.Text12.Text))
Text19.Text = (CDbl(Form3.Text2.Text))
Text20.Text = (CDbl(Form3.Text4.Text))
Text43.Text = (CDbl(Form3.Text1.Text))
Text44.Text = (CDbl(Form3.Text3.Text))
'Impact Force
Text9.Text = (CDbl(Form3.Text7.Text))
Text27.Text = (CDbl(Form3.Text10.Text))
Text51.Text = (CDbl(Form3.Text11.Text))
'Longitudinal Force
Text38.Text = (CDbl(Form4.Text2.Text))
Text50.Text = (CDbl(Form4.Text3.Text))
Text37.Text = (CDbl(Form4.Text4.Text))
Text49.Text = (CDbl(Form4.Text5.Text))
'wind load
Text64.Text = (CDbl(Form5.Text10.Text))
Text63.Text = (CDbl(Form5.Text11.Text))
'Watercurrent force
Text18.Text = Form6.Text22.Text
Text24.Text = (CDbl(Form6.Text23.Text))
Text36.Text = (CDbl(Form6.Text24.Text))
Text48.Text = (CDbl(Form6.Text25.Text))
'Earthquake force
Text35.Text = (CDbl(Form7.Text20.Text))
Text47.Text = (CDbl(Form7.Text21.Text))
Text34.Text = (CDbl(Form7.Text18.Text))
Text46.Text = (CDbl(Form7.Text19.Text))
'Buoyancy force
Text2.Text = (CDbl(Form2.Text26.Text))
End Sub
Private Sub Command2_Click()
Form9.Show vbModal
End Sub
```

Form 9

```
Private Sub Command1_Click()
'Load combination 1
Text1.Text = Round(Val(CDbl(Form8.Text11.Text)) -
Val(CDbl(Form8.Text2.Text)), 2)
Text7.Text = Round(Val(CDbl(Form8.Text18.Text)) +
Val(CDbl(Form8.Text64.Text)), 2)
Text6.Text = Round(Val(CDbl(Form8.Text36.Text)) +
Val(CDbl(Form8.Text37.Text)), 2)
Text5.Text = Round(Val(CDbl(Form8.Text63.Text)) +
Val(CDbl(Form8.Text24.Text)), 2)
Text4.Text = Round(Val(CDbl(Form8.Text53.Text)) +
Val(CDbl(Form8.Text48.Text)) + Val(CDbl(Form8.Text49.Text)), 2)
'Load combination 2
Text2.Text = Round(Val(CDbl(Form8.Text11.Text)) -
Val(CDbl(Form8.Text2.Text)), 2)
Text10.Text = Round(Val(CDbl(Form8.Text18.Text)), 2)
Text9.Text = Round(Val(CDbl(Form8.Text36.Text)) +
Val(CDbl(Form8.Text37.Text)) + Val(0.5 * (CDbl(Form8.Text35.Text))), 2)
Text8.Text = Round((CDbl(Form8.Text24.Text)), 2)
Text3.Text = Round(Val(CDbl(Form8.Text53.Text)) +
Val(CDbl(Form8.Text48.Text)) + Val(CDbl(Form8.Text49.Text)) + Val(0.5 *
(CDbl(Form8.Text47.Text))), 2)
'Load combination 3
Text15.Text = Round(Val(CDbl(Form8.Text12.Text)) +
Val(CDbl(Form8.Text3.Text)) + Val(CDbl(Form8.Text9.Text)) -
Val(CDbl(Form8.Text2.Text)), 2)
Text11.Text = Round(Val(CDbl(Form8.Text18.Text)), 2)
Text12.Text = Round(Val(CDbl(Form8.Text36.Text)) +
Val(CDbl(Form8.Text38.Text)) + Val(CDbl(Form8.Text37.Text)), 2)
Text13.Text = Round(Val(CDbl(Form8.Text44.Text)) +
Val(CDbl(Form8.Text51.Text)) + Val(CDbl(Form8.Text24.Text)), 2)
Text14.Text = Round(Val(CDbl(Form8.Text20.Text)) +
Val(CDbl(Form8.Text27.Text)) + Val(CDbl(Form8.Text48.Text)) +
Val(CDbl(Form8.Text50.Text)) + Val(CDbl(Form8.Text49.Text)), 2)
'Load combination 4
```

```
Text16.Text = Round(Val(CDbl(Form8.Text12.Text)) +
Val(CDbl(Form8.Text3.Text)) + Val(CDbl(Form8.Text9.Text)) -
Val(CDbl(Form8.Text2.Text)), 2)
Text20.Text = Round(Val(CDbl(Form8.Text18.Text)) +
Val(CDbl(Form8.Text64.Text)), 2)
Text19.Text = Round(Val(CDbl(Form8.Text36.Text)) +
Val(CDbl(Form8.Text38.Text)) + Val(CDbl(Form8.Text37.Text)), 2)
Text18.Text = Round(Val(CDbl(Form8.Text44.Text)) +
Val(CDbl(Form8.Text51.Text)) + Val(CDbl(Form8.Text24.Text)) +
Val(CDbl(Form8.Text63.Text)), 2)
Text17.Text = Round(Val(CDbl(Form8.Text20.Text)) +
Val(CDbl(Form8.Text27.Text)) + Val(CDbl(Form8.Text48.Text)) +
Val(CDbl(Form8.Text50.Text)) + Val(CDbl(Form8.Text49.Text)), 2)
'Load combination 5
Text25.Text = Round(Val(CDbl(Form8.Text12.Text)) + Val(0.5 *
CDbl(Form8.Text3.Text)) + Val(0.5 * CDbl(Form8.Text9.Text)) -
Val(CDbl(Form8.Text2.Text)), 2)
Text21.Text = Round(Val(CDbl(Form8.Text18.Text)), 2)
Text22.Text = Round(Val(CDbl(Form8.Text36.Text)) + Val(0.5 *
CDbl(Form8.Text38.Text)) + Val(0.5 * CDbl(Form8.Text37.Text)) +
Val(CDbl(Form8.Text34.Text)), 2)
Text23.Text = Round(Val(0.5 * CDbl(Form8.Text44.Text)) + Val(0.5 *
CDbl(Form8.Text51.Text)) + Val(CDbl(Form8.Text24.Text)), 2)
Text24.Text = Round(Val(0.5 * CDbl(Form8.Text20.Text)) + Val(0.5 *
CDbl(Form8.Text27.Text)) + Val(CDbl(Form8.Text48.Text)) + Val(0.5 *
CDbl(Form8.Text50.Text)) + Val(0.5 * CDbl(Form8.Text49.Text)) +
Val(CDbl(Form8.Text46.Text)), 2)
End Sub
Private Sub Command2_Click()
Form10.Show vbModal
End Sub
```

Form 10

```
Private Sub List1_Click()
End Sub
Private Sub MyMethod()
```

```
    Select Case Combo1.ListIndex
        Case 0
            Text5.Text = "1.5"
        Case 1
            Text5.Text = "0.66"
        Case 2
            Text5.Text = "0.50"
        Case 3
            Text5.Text = "0.6"
        Case 4
            Text5.Text = "0.8"
        Case 5
            Text5.Text = "0.45"
        Case 6
            Text5.Text = "0.5"
    End Select
End Sub
Private Sub Combo1_Click()
    MyMethod
End Sub
Private Sub Combo1_KeyPress(KeyAscii As Integer)
    MyMethod
End Sub
Private Sub Combo1_Scroll()
    MyMethod
End Sub
Private Sub Command1_Click()
Text7.Text = Round(CDbl(Text9.Text) * 3, 3)
Text6.Text = Round(CDbl(Text7.Text) + CDbl(Text8.Text), 3)
Text4.Text = Round((CDbl(Text3.Text) / CDbl(Text1.Text)) * CDbl(Text8.Text) /
2, 3)
Text10.Text = Round(0.5 * CDbl(Text5.Text) * (CDbl(Form1.Text3.Text)) *
(CDbl(Form1.Text3.Text)), 3)
Text11.Text = Round(CDbl(Text10.Text) * CDbl(Text9.Text) * CDbl(Text8.Text),
3)
```

```
Text12.Text = Round((CDbl(Text11.Text) * (CDbl(Text8.Text) / 3) *
((CDbl(Text8.Text) * 2 / 3) * CDbl(Text8.Text) * 2 / 3)) / (CDbl(Text8.Text) *
CDbl(Text8.Text)), 3)
Text13.Text = Round(Val(CDbl(Text12.Text)) + Val(CDbl(Text4.Text)), 3)
Text14.Text = Round((CDbl(Text2.Text) / CDbl(Text1.Text)) * CDbl(Text8.Text)
/ 2, 3)
Text15.Text = Round(0.5 * CDbl(Text5.Text) * (CDbl(Form1.Text3.Text)) *
(CDbl(Form1.Text3.Text)), 3)
Text16.Text = Round(CDbl(Text10.Text) * CDbl(Text9.Text) * CDbl(Text8.Text),
3)
Text17.Text = Round((CDbl(Text11.Text) * (CDbl(Text8.Text) / 3) *
((CDbl(Text8.Text) * 2 / 3) * CDbl(Text8.Text) * 2 / 3)) / (CDbl(Text8.Text) *
CDbl(Text8.Text)), 3)
Text18.Text = Round(Val(CDbl(Text17.Text)) + Val(CDbl(Text14.Text)), 3)
End Sub
Private Sub Form_Load()
Text3.Text = Form9.Text29.Text
Text2.Text = Form9.Text30.Text
End Sub
Private Sub Command2_Click()
Form11.Show vbModal
End Sub
```

Form 11

```
Private Sub Command1_Click()
Text11.Text = Round((CDbl(Text1.Text) / CDbl(Text12.Text)) +
((CDbl(Text3.Text) * CDbl(Text7.Text) / 2) / (CDbl(Text12.Text) *
(CDbl(Text7.Text) / 2) * (CDbl(Text7.Text) / 2))) + (((CDbl(Text2.Text) *
((CDbl(Text4.Text) + (CDbl(Text10.Text))) / ((CDbl(Text12.Text) / 2) *
(CDbl(Text4.Text) + CDbl(Text10.Text)) * (CDbl(Text4.Text) +
CDbl(Text10.Text)) + ((CDbl(Text12.Text) / 2) * CDbl(Text10.Text) *
CDbl(Text10.Text))))))), 3)
Text5.Text = Round((CDbl(Text1.Text) / CDbl(Text12.Text)) -
((CDbl(Text3.Text) * CDbl(Text7.Text) / 2) / (CDbl(Text12.Text) *
(CDbl(Text7.Text) / 2) * (CDbl(Text7.Text) / 2))) - (((CDbl(Text2.Text) *
((CDbl(Text4.Text) + (CDbl(Text10.Text))) / ((CDbl(Text12.Text) / 2) *
```

```
(CDbl(Text4.Text) + CDbl(Text10.Text)) * (CDbl(Text4.Text) +
CDbl(Text10.Text)) + ((CDbl(Text12.Text) / 2) * CDbl(Text10.Text) *
CDbl(Text10.Text))))))), 3)
End Sub
Private Sub Command2_Click()
Form12.Show vbModal
End Sub
Private Sub Form_Load()
Text1.Text = Form9.Text26.Text
Text2.Text = Form9.Text28.Text
Text3.Text = Form9.Text27.Text
End Sub
```