DESIGN AND DETAILING OF EARTHQUAKE RESISTANT RC BRIDGE STRUCTURE

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Abstract : Bridges must be strong and bear a great deal of weight, or force, to keep us safe as we travel across them. Since Nepal is earthquake zone area. As well as here, the bridge to be design is carried in rural and hilly areas. So, the project entitled as "Design and Detailing of Earthquake resistance RC BRIDGE STRUCTURE". Initially the testing of soil is done and based on obtained result the design of bridge is done manually of substructure and superstructures. The design includes seismic loads due to earthquake, standard vehicle load and other type of loads. The main aim of this project is to utilize our academic knowledge and to design project in terms of safety, economy, stability and efficiency to get optimum results.

I. INTRODUCTION

1.1 GENERAL

A Bridge is a structure providing passage over an obstacle without closing the way beneath. The required passage may be for a road, a railway, pedestrians, a canal or a pipeline. The obstacle to be crossed may be a river, a road, railway or a valley. Bridges range in length from a few meters to several kilometers. They are among the largest structures built by man. The demands on design and on materials are very high. A bridge must be strong enough to support its own weight as well as the weight of the people and vehicles that use it. The structure also must resist various natural occurrences, including earthquakes, strong winds, and changes in temperature. Most bridges have a concrete, steel, or wood framework and an asphalt or concrete road way on which people and vehicles travel. The T-beam Bridge is by far the Most commonly adopted type in the span range of 10 to 25M. The structure is so named because the main longitudinal girders are designed as T-beams integral with part of the deck slab, which is cast monolithically with the girders.

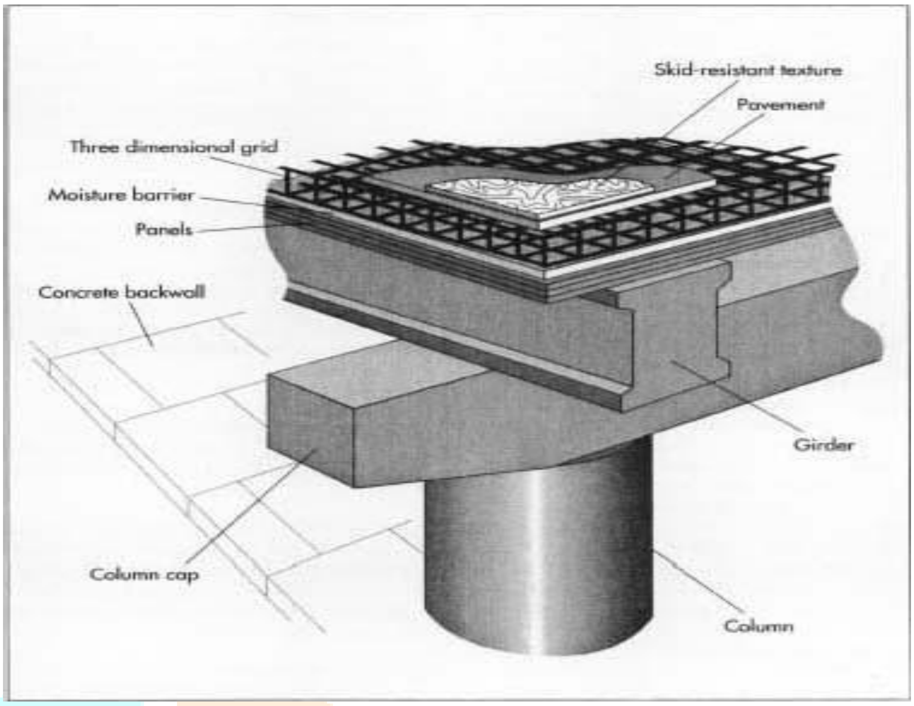


FIGURE 1: CUTWAY VIEW OF TYPICAL CONCRETE BRIDGE.

1.1.1 OBJECTIVE OF THE THESIS

The objectives of the thesis is to design a RC Bridge which is earthquake resistance. The design base involved here is manual calculation which is precise and economical. The use of this procedure helps proper design of any RC Bridges. This thesis aims at gaining the needed parameters from different soil test and site conditions which are then formulated by Indian Road Congress IRC.

1.1.2 MAIN COMPONENTS OF A BRIDGE:

The Superstructure consists of the following components:

- Deck slab
- Cantilever slab portion
- Footpaths, if provided, kerb and handrails or crash barriers
- Longitudinal girders, considered in design to be of T-section
- Cross beams or diaphragms, intermediate and end ones
- Wearing coat

The Substructure consists of these Structures:

- Abutments at the extreme ends of the bridge
- Piers at intermediate supports in case of multiple span bridges.
- Bearings and pedestals for the decking.
- Foundations for both abutments and piers may be of the type open, well, pile, etc.

1.1.3 TYPES OF BRIDGES

a. Girder Bridge
b. Truss Bridge
c. Arch Bridge
d. Cantilever Bridge
e. Suspension Bridge
f. Cable-stayed Bridge
g. Movable Bridge
h. Slab Bridge

1.2 PARAMETERS GOVERNING CHOICE OF SUPERSTRUCTURE:

The basic function of a bridge superstructure is to permit uninterrupted smooth passage of traffic over it and to transmit the loads and to transmit the load and forces to the substructure safely through the bearings. Although it is difficult to stipulate the aesthetic requirements, itshould, however, be ensured that the type of superstructure adopted is simple, pleasing to the eye, and blends with the environment. No hard and fast rules can be laid regarding the economy in cost. The designer should, however, be able to evolve the most economical type of superstructure based on his judgment and experience given the particular conditions prevailing at the particular site at the particular time. The following factors are to be considered while selecting the type of abridge superstructure

i. The nature of river or streams
ii. Nature of foundation / founding strata available
iii. The amount and type of traffic
iv. Whether used for navigation purposes
v. Climatic conditions
vi. Hydraulic data
vii. Type of available construction material
viii. Labour available
ix. The available facilities for erections
x. Maintenance provisional
xi. The availability of funds
xii. Time available for construction
xiii. Strategic consideration
xiv. Economic consideration
xv. Aesthetic consideration

1.3 THREE TYPES OF LOADS CONSIDERED IN BRIDGE CONSTRUCTION

When building a bridge, engineers need to consider the weight and environment, or load types the bridge will encounter over a long period of time. These factors determine what material should be used to build the bridge as well as the type of structure that will best withstand the loads. Also known as forces, the type of loads considered in bridge construction is vital to its integrity.

1.3.1 DEAD LOAD

The dead load of a bridge is the bridge itself -- all the parts and materials that are used in the construction of the bridge. This includes the foundation, beams, cement, cables, steel or anything else that comprises the parts of the bridge. It's called a dead load because it doesn't move. It may breathe with the seasons or sway with the wind, but those movements are almost imperceptible.

1.3.2 LIVE LOAD

A live load is the moving weight the bridge will hold, such as traffic. It is based on traffic patterns that include the number of cars, trucks and other vehicles that will travel across it at any given time. Certain variables, such as snow, may be calculated into the total live weight for a more accurate estimate. The heaviest possible weight in the most extreme conditions is also a factor despite the rarity of such an occurrence.

1.3.3 DYNAMIC LOAD

Dynamic loads are outside forces that cannot be accurately measured such as wind, vibration and extreme weather. These factors need to be considered in the construction of a bridge to build "breathing" room into the structure. This breathing room allows the bridge to move or adjust to the dynamic loads without collapsing or permanently shifting. As solid as a bridge may seem, it still has the ability to sway when a strong wind is present.

1.3.4 OTHER LOADS

When building a bridge, there are other types of loads that need to be considered that are specific to the terrain in which the foundation will be laid. Environmental factors and weather patterns are also considered when calculating load-bearing needs. The load expectation of a bridge will determine the best design for strength and to ensure its longevity, whether the bridge is to span over large bodies of water or between rising mountaintops.

1.4 GENERAL GUIDELINES FOR ANALYSIS AND DESIGN OF A BRIDGE STRUCTURE

Procedure for preparation of General Arrangement Drawing of a Bridge:

I. First of all the required formation level is found out. On knowing this the permissible structural depth is established. This is done after taking into account the following two things :(i)Minimum vertical clearance required taking into account the difference between the affluxed high flood level and the soffit of the deck.(ii) Thickness of wearing coat required below the formation level.

II. Considering the depth of foundations, the height of deck above the bed level and low water level, average depth of water during construction season, the type of bridge, span lengths, type of foundations, cross section of the deck, method of construction and loading sequence.

III. Trial cross sections of the deck, sizes of various elements of the substructure and superstructure are decided upon and drawn to arrive at the preliminary general arrangement of the bridge. Various trials lead to a structural form with optimum placements of its load masses. Relative proportions and sizes of certain members as well as their shapes are decided upon and drawn to a certain scale on this drawing. The type of bearing to be used along with their locations depending upon the support system is also established. The main basis of the general arrangement drawing of a bridge structure is a quick preliminary analysis and design of the member sections. This is essential for forming the basis of the detailed to be carried later on depending upon the requirements of the project.

1.5 ABOUT THIS PROJECT

This report describes detailed explanatory notes on detailed design of the Bridge.

Particulars	Required information / number / range / value(s)
Geographical Location:	
Classification of the Road	Feeder Road
Type of the Road Surface	Existing track
Terrain/Geology	Terai
Information on structure:	
Total length of the Bridge	50m
Span arrangement	2×25m(c/c of bearing)
Total width of the Bridge	11.0m
Width of:	
Carriageway:	7.5m
Footpath(s):	1.5m both side
Kerbs	
Type of superstructure:	RCC
Type of Bearings:	Neoprene
Type of abutments:	Rectangular RCC with cantilever return wall
Type of piers:	
Type and depth of foundations:	Pile foundation
Design Data:	
Live load:	IRC Class A /AA Loading
Net bearing capacity of soil	
Design discharge	1190.40 m^3/sec
Linear waterway	163.885 m

CHAPTER 2

1. LITERATURE REVIEW

2.1 INTRODUCTION

Many researchers had done project on bridges to analyses the nature of bridges and affects on it due to earthquake.

Charles Abdunar (1993) described a method which directly measure the stress in either concrete or masonry bridges. In this method a small slot is cut in the plane normal to the desired stress direction, and a very thin flat jack is then placed in this slot and used to restore the initial displacement field. The amount of hydraulic pressure required to do so provide a value for the
absolute compressive stress normal to the slot. The advantage of the method is that it is a direct measurement technique and the elastic properties of the material are not required.

Tadros M K et al. (1979) discusses about the long term deflections of segmental bridges. A step by step computer method was proposed for determining the deflection sand stress distribution due to creep and shrinkage of concrete and relaxation of pre-stressed steel. The computer program accounts for the presence of the non pre-stressed steel, difference in ages of the concrete segments, the multiple stages in which the external loads and pre-stressing are applied, and the changes in geometry and support condition as construction progresses. Deflection of a particular node with and without considering creep, shrinkage and relaxation was evaluated. Graphs are plotted for the vertical deflection of bridge at various construction stages considering the effects of long term deflections.

Around 2550 BC, Imhotep, the first documented engineer, built a famous stepped pyramid for King Djoser located at Saqqara Necropolis. With simple tools and mathematics he created a monument that stands to this day. His greatest contribution to engineering was his discovery of the art of building with shaped stones. Those who followed him carried engineering to remarkable heights using skill and imagination. Ancient historic civil engineering constructions include the Qanat water management system (the oldest older than 3000 years and longer than 71 km,) the Parthenon by Iktinos in Ancient Greece (447-438 BC), the Appian Way by Roman engineers (c. 312 BC), the Great Wall of China by General Meng T'ien under orders from Ch'in Emperor Shih Huang Ti (c. 220 BC) and the stupas constructed in ancient Sri Lanka like the Jetavanaramaya and the extensive irrigation works in Anuradhapura. The Romans developed civil structures throughout their empire, including especially aqueducts, insulae, harbours, bridges, dams and roads.

Saad El-Azazy, Ph.D., P.E. • study on seismic bridge design improvements through research implementation. He studied the affects of earthquake on bridge and he found the solution to address the problem by implementing the different codes. Research findings were incorporated into the seismic design code of bridges resulting in improvements in seismic performance.

Ritesh Sharma, D. K. Sharma discussed about the Review on Bridge Construction Technology.
With the advancement and recent development in bridge construction technologies now engineers have several options to select bridge from different types as discussed and also which fulfills different parameters viz. economy, safety, stability and aesthetic view of sub-structure. Introduction and different types of bridges considered in this review and the selection of different type of bridges in construction technologies in civil engineering.

Mohamed SOBAIH And Adel GABR. The purpose of this paper is to develop a methodology for the evaluation of the seismic vulnerability of existing reinforced concrete highway bridges. This evaluation methodology aims to express quantitatively what is known only qualitatively such that it can deal with the complicated dynamic characteristics of the highway structure and quantify its seismic risk. Finally they evaluate the seismic vulnerability and risk levels of existing reinforced concrete highway bridges, a new methodology that has been developed and implemented into a computer program.

CHAPTER 3

3.1 DESIGN AND DETAILING OF RCC BRIDGE

3.2 DESIGN OF BRIDGE ABUTMENT

3.2.1 DESIGN DATA

1.1) Materials and Properties

Concrete	M25
Reinforcement	Fe 500

Basic Permissible Stresses of Concrete as per IRC: 21-2000

Permissible direct compressive stress (σco)	6.25MPa
Permissible flexural compressive stress (σc)	8.33MPa
Basic permissible tensile tress (σt)	0.61MPa
Maximum Permissible shear Stress (τmax)	1.90MPa

Basic Permissible Stresses of Reinforcing Bars as per IRC: 21-2000

Permissible Flexural Tensile stress (σst)	240.00 MPa
Permissible Direct Compressive stress (σco)	205.00 MPa

Self-weight of materials as per IRC: 6-2000

Concrete (Cement-Reinforced)	25.00KN/m3
Macadam (Binder Premix)	22.00KN/m3
Water	10.00KN/m3
Backfill	16.00KN/m3

Design of Data:

Modular Ratio (m)	11.20
Neutral axis depth factor, n = ($m\sigma c$)/($m\sigma c+\sigma st$)	0.28
Lever arm factor, j = (1-n/3)	0.91
Moment of resistance coefficient, R = ½ x n x j x σc	1.06Mpa

Left Bank foundation on Rock, Right Bank foundation on soil

Maximum Seismic Coefficient for Seismic Zone V according to IS: 1893-1984

0.169

3.2.2 DIMENSION PARAMETERS

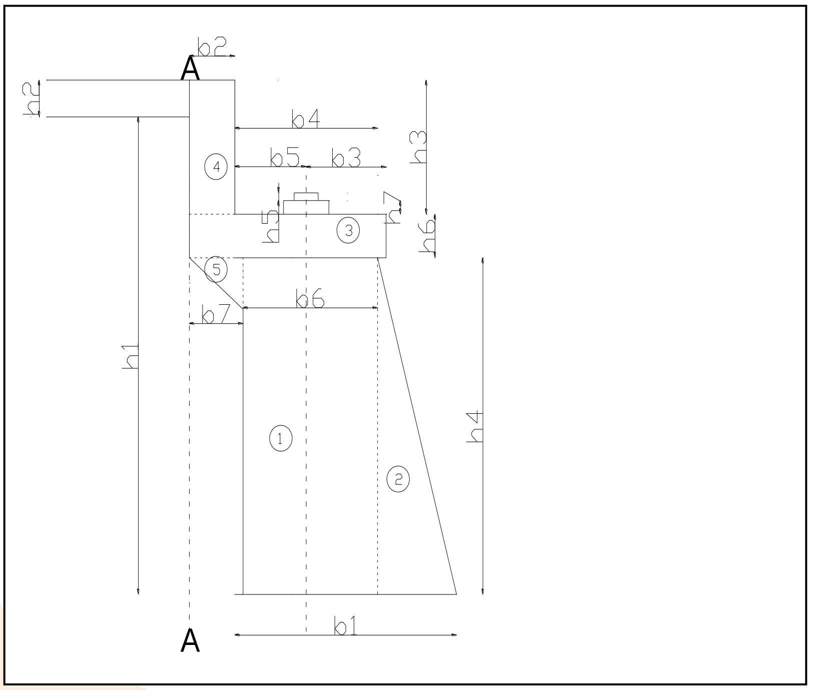
Effective Span of Bridge	25.00m
Number of spans	2.00Nos.
Width of Expansion Joint	0.05m
Total Span of Bridge	54.25m
Angle of internal friction of backfill	29.00degree
Return wall thickness	0.30m

Approach slab

Length, L	3.50m
Width, B	11.00m
Depth, D	0.30m

Size of bearing

Length	1.20m
Width	1.20m
Thickness	0.05m
High Flood Level, HFL	149.81m
Lowest Bed Level, LBL	144.25 m
Bottom level of abutment	147.72 m
Clearance above HFL	1.50m
Top level of abutment	151.06 m
Depth of superstructure	2.50m



3.2.3 CALCULATION OF WEIGHT AND C.G. OF THE ABUTMENT STEM

1 CALCULATION OF WEIGHT AND CG OF ABUTMENT

Elements	Area, Ai (m^2)	Weight (KN)	Xi (m) from back A-A	Yi (m) from bottom	AiXi	AiYi
P_1	2.84	781.27	1.2	1.42	3.41	4.04
P_2	0.28	78.13	1.77	0.95	0.50	0.27
P_3	1.05	288.75	0.9	3.09	0.95	3.25
P_4	1.10	302.50	0.20	4.72	0.22	5.19
P_5	0.25	67.38	0.47	2.61	0.11	0.64
	5.28	1518.03			5.19	13.38

C.G. from back of abutment A-A, x = 0.98m

From bottom of abutment, y = 2.54m

e = 0.4+0.61-0.98 = 0.03m

which is <1.2/6= 0.2, Hence there will be no tension force.

Moment about back face =425.05 KN-m

Moment about C.G. of Abut = 0KN-m

3.2.4 CALCULATION OF LOADS AND MOMENTS

Due to Dead Load

Dead load from superstructure = 2540.00KN

Weight of bearings, expansion joint etc. = 100.00 KN

Total dead load from superstructure on each abutment, PDL= 2640.00KN

Distance of bearing center from front of abutment= 0.79m

Eccentricity of DL & LL acting through bearing, e = 0.03m

Moment due to DL of superstructure about back of abutment =787.40 KN-m

Due to Pedestrian Loading

Intensity of pedestrian loading = 3.22KN/m^2

Load on one abutment = 60.42KN

Due to Utility Loading

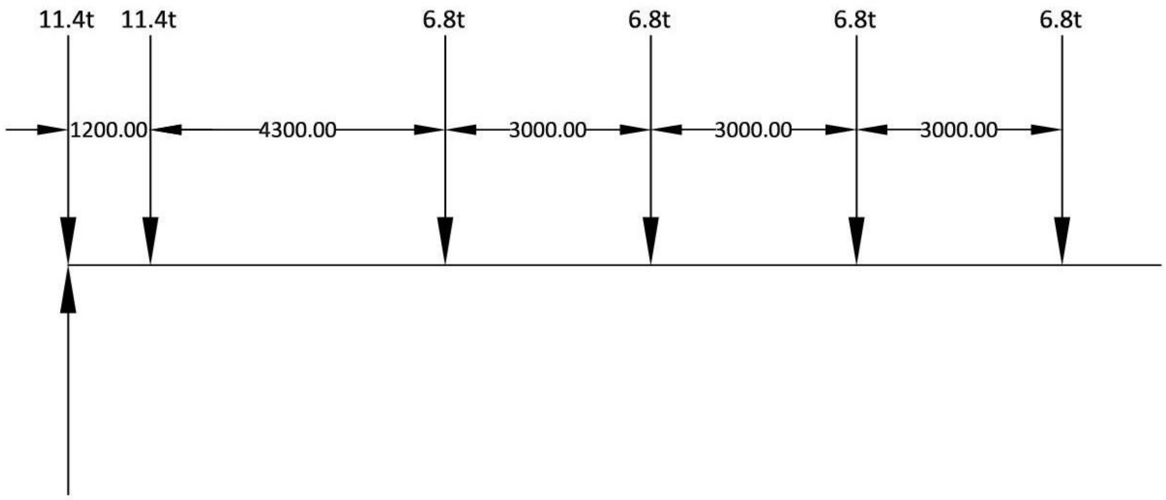
Intensity of Utility loading = 1.00KN/m

Load on one abutment = 12.50KN

Due to Live Load

Live load from superstructure:

Due to IRC Class A wheel load, in longitudinal direction,



Max. LL on abutment from right side

(11.4x25+11.4x23.8+6.8x19.5+6.8x16.5+6.8x13.5+6.8x10.5)/25

= 38572.80Kg = 385.728KN

Impact factor = 4.5/(6+25)

= 1.15

Max LL including Impact for two identical vehicles, P_{LL} = 883.44 KN

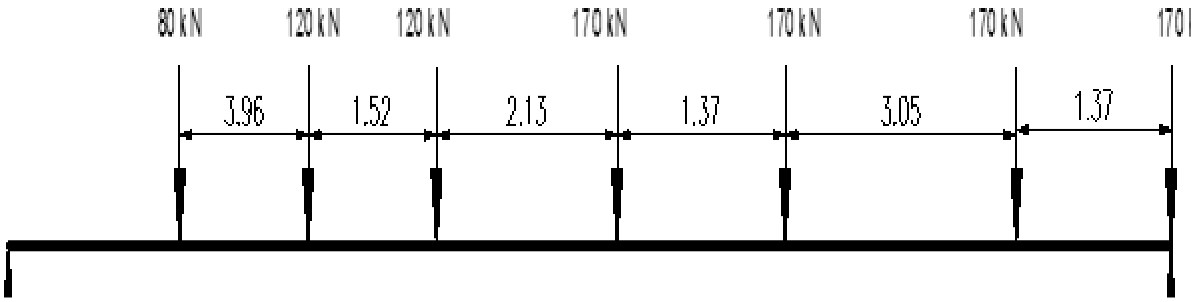
Moment due to eccentric load = 943.86*(0.4+0.61-0.7)

= 292.60 KN-m

Due to Live Load

Live load from superstructure:

Due to IRC Class 70R wheel load, in longitudinal direction,



Max. LL on abutment from right side
(12x25+12x23.48+17x21.35+17x19.98+17x16.93+17x15.56)/25
= 73468Kg
= 734.68KN
Impact factor = 1.09
Max LL including Impact for one 70R identical vehicles, PLL = 800.80KN
Moment due to eccentric load= 861.22*(0.4+0.61-0.7)
=266.98KNm

<u>Due to Earth Pressure</u>
Horizontal force due to earth pressure = 0.5 × ϒs × h12 × tan2(45^0-Φ/2) × t
= 0.5*16*5.8*5.8*tan(30.5)*tan(30.5)*11
= 1027.15KN
Which acts at a distance from abutment base = 0.42xh1
= 2.44m
Magnitude of surcharge, q = 1.2 × ϒs
= 19.2 KN/m²
Horizontal force due to surcharge = qxh1xtan²(45-Φ/2)xt
= 425.03 KN
Which acts at a distance h1/2 from abutment base = 2.90m
Moment due to earth pressure about abutment base = 1027.15x2.436+425.03x2.9 KN-m
= 3734.73KN-m

<u>Due to Temperature Variation</u>
Maximum temperature variation, T = 40.00ºC
Coefficient of thermal expansion, a = 1.2E-05/m/ºC
Strain due to concrete shrinkage = 0.0002
Total Strain due to shrinkage = 6.68E-04
Hori. Deformation of deck due to temp. & shrinkage affecting one abutment
= 54.36mm

Shear Modulus of bearing material, G = 1.00Mpa
Depth of bearing = 50mm
Strain in bearing = 1.55
Hori. force due to strain in long. Direction at bearing level
= 128.087KN
This force acts at the bearing level, i.e. at a distance from abutment base
= 2.89m
Moment due to temperature variation = 370.300KN.m

<u>Due to Braking Effect</u>

Effect due to class 70R loading = 0.2x1000	200KN
This acts at distance above deck level	1.2m
And at a distance from abutment base	7.30m
Moment due to braking about abutment base	1460.00KN-m

Due to Seismic force

Description	Total Load (KN)	Seismic Load (KN)	Lever arm (m)	Moment (KN-m)
Superstructure DL	2652.50	447.61	4.59	2054.97
Abutment shaft P_1	781.27	131.84	1.42	187.28
Abutment shaft P_2	78.13	131.84	0.95	124.85
Abutment shaft P_3	288.75	131.84	3.09	407.52
Abutment shaft P_4	302.50	51.05	4.72	240.74
Approach slab	288.75	48.73	5.95	289.92
Backfill	0.00	0.00	2.90	0.00
Total	4391.90	942.90		3305.28

3.2.5 SUMMARY OF LOADS AND MOMENTS

The summary of loads and moments are tabulated below. The transverse forces and moments are not considered, since it would not be critical due to high moment of inertia of abutment. Therefore stresses are checked in longitudinal direction only.

Description	Vertical load (KN)	Horizontal force (KN)	Moment (KN-m)
Superstructure DL	2652.50		787.40
LL including Impact	943.86		292.60
Self wt. of abutment	1518.03		0.00
Braking effect		200	1460.00
Temperature Variation		128.09	370.30
Earth Pressure		1027.15	3734.73
Sub Total	5114.39	1355.24	6645.03
Seismic Force		942.90	3305.28
Total	5114.39	2298.14	9950.31

As per standard design practice, design of abutment would be carried out for case A, and checked for case B, as given below.

Case	A: (N+T)	B: (N+T+S)
Vertical load (KN)	5114.39	5114.39
Horizontal force (KN)	1355.24	2298.14
Moment (KN-m)	6645.03	9950.31

Design of abutment stem section

Abutment section will be designed for case A and the section adequacy will be checked for case B.

Design vertical load 5114.39KN

Design moment 6645.03KN-m

Depth of section required=M/(Rxb)

= 755.86 mm

< 1200-75-12.5 = 1112.5 mm, OK

Clear cover to reinforcement= 75mm

Effective depth provided= 1112.5 mm

Area of steel required for tension, Ast= M/(σst x j x d)

= 27449.7mm²

Thus provide 72 Nos. Φ 25.00 mm bars @ c/c spacing of 150 mm

Giving an Steel Area of= 35342.92mm²

At the front side of the abutment,

Provide 72 Nos. of Φ 20.00 mm bars @ c/c spacing of 150 mm

Giving an Steel Area of= 22619.47mm²

Design horizontal force = 1355.24KN

Shear stress, τv= 0.11N/mm²

Percentage area of tension steel, 100Ast/bd = 0.29%

Shear strength increment factor = 1+5P/(Ag x fck) = 1.00

< 1.5, OK

Design shear strength of concrete, τc = 0.28N/mm²

> 0.11, OK

Check for shear at the bottom of the abutment cap.

Total horizontal force at the bottom of the cap = 328.09 KN

Depth of abutment = 1200mm

Effective depth, d = 1093mm

Corresponding shear stress, τv= 0.03N/mm²

< 0.28, OK

Tension reinforcement required at different level along the height of abutments.

At 1.42m above base of abutment

Description	Load (KN)	Lever Arm (m)	Moment (KN-m)
Superstructure DL	2652.50	0.03	0.00
Live load (LL)	943.86	0.03	28.32
Braking load	200.00	5.88	1175.90
Temperature load	128.09	1.92	245.99
Earth Pressure	1027.15	1.84	1889.34
Surcharge	242.33	2.19	530.65
Total			3870.19

Overall depth of abutment = 1100.00mm

Effective depth of abutment = 1009.00mm

Area of tension reinforcement req. = M/(σst x j x d)

17627.17mm²

After Curtailment

Thus provide 37 Nos. of Φ 25.00 mm bars @ c/c spacing of 300 mm

Giving an Steel Area of= 18162.33mm²

Actual point of curtailment is 2.57 m from base of abutment.

No curtailment is proposed in this case.

Giving an Steel Area of= 35342.917mm²

Rear side Reinforcement

Provide 16 mm f bars @ 150 mm c/c at rear side of the abutment.

Check for stresses for case B

Bending Moment = 9950.31KN-m

Stress in steel = 2.79Mpa

< 360 Mpa, OK

Horizontal force = 2298.14KN

Corresponding shear stress = 0.19N/mm²

< 0.42 N/mm², OK

3.3 DESIGN OF ABUTMENT CAP:

Vertical reaction due to Dead load, PDL= 2652.50KN

Live load including impact, PLL= 943.86 KN

Total vertical reaction= 3596.36KN

Assume Cap thickness= 500.00 mm

Providing min. area of steel @1%, area of steel required /m = 5000.00mm²

Thus Provide, on top and bottom, Φ 20.00 mm bars @100.00 mm c/c

Giving an Steel Area of= 6283.19mm²

Check for bearing stresses in cap:

Grade of concerete = M30

The allowable bearing pressure with nearly uniform distribution on the loaded area of a footing or base under a bearing or column shall be given by following equation,

$C = C0(A_1/A_2)^{1/2}$

Permissible direct compressive stress in concrete, C_0= 7.50MPa

Dispersed largest concentric area similar to A2, A1= 2.73m²

Loaded area, A2= 0.42m²

Therefore, C = 15.0MPa

Actual compressive stress= 4.28MPa

< 15 MPa, OK

Check for bearing stresses in pedestal:

The allowable bearing pressure with nearly uniform distribution on the loaded area of a footing or base under a bearing or column shall be given by following equation,

Length of pedestal = 0.56m

Width of pedestal = 0.75m

Thickness of pedestal = 0.20m

C = C0(A1/A2)1/2

Permissible direct compressive stress in concrete, C_0= 7.50MPa

Dispersed largest concentric area similar to A_2, A_1= 2.56m²

Loaded area, A_2= 1.44m²

Therefore, C= 10.00MPa

Actual compressive stress= 1.25MPa

< 10 MPa, OK

3.4 DESIGN OF BACKWALL:

Horizontal force due to earth pressure = $0.5\gamma s\times(h3- h2)^2\times\tan^2(45o-\Phi/2)$

= 16.66KN/m

Which acts at a distance from backwall base = 1.029m

Magnitude of surcharge, q = 19.2KN/m²

Horizontal force due to surcharge = q×(h3-h2)×tan2(45o-Φ/2)
= 16.32KN/m
Which acts at a distance from backwall base= 1.225m

Self weight of backwall /m width= 27.5KN/m
Live load on back wall considering class A loading= 57KN
This acts at a distance from backwall toe= 0.200m

Moment due to earth pressure about backwall base= 37.14KN-m/m
Moment due to self weight and LL= 16.90KN-m/m
Total Moment= 54.04KN-m/m

Effective depth of backwall= 342mm

Area of steel required, Ast= 726.14 mm²

Thus Provide Φ 16.00 mm bars @150.00 mm c/c
Giving an Steel Area of= 1340.41mm²
And, as distribution bar, provide Φ 12.00 mm bars @200.00 mm c/

3.5 DESIGN OF APPROACH SLAB:

The approach slab is resting over the abutment and the other end supported by the soil underneath. It should be designed on the basis of elastic base theory, which is complicated. Hence the dimensions and reinforcement is provided as per the standard design practice.

thickness= 0.3 m
Self weight /m width= 7.50KN/m
Weight of pavement= 1.65KN/m
Total udl = 9.15KN/m

Maximum live load= 2×114 KN at 1.2mspacing
Max moment in longitudinal Direction (from SAP analysis)= 20.10KN-m/m

Effective depth of slab required= 137.88 mm
Effective depth provided= 254.00 mm
Area of steel required= 363.67mm²

Thus, provide, on top and bottom, Φ 12.00 mm bars @150.00 mm c/c
Giving an Steel Area of= 753.98 mm²

Max moment transverse Direction (from SAP analysis)= 15.90Kg-m
Effective depth of slab required= 122.63 mm
Effective depth provided= 242.00 mm

Area of steel required= 301.94 mm²

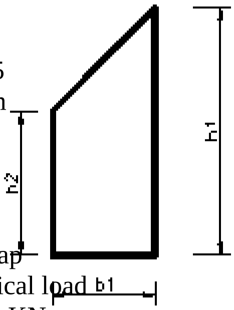
Thus, provide, on top and bottom, Φ 12.00 mm bars @150.00 mm c/c

Giving an Steel Area of= 753.98 mm²
And, as distribution bar, provide Φ 10.00 mm bars @150.00 mm c/c

Maximum shear force= 31.00 kN/m
Shear stress= 0.13 N/mm²
Percentage area of steel= 0.31 %
Permissible shear stress= 0.25 N/mm²
> 0.13 N/mm², OK

3.6 DESIGN OF RCC STOPPERS:

Grade of stopper = M25
Base width of sttoper, b1 = 0.8m
Height above abutment cap, h2 = 0.45m
Height above abutment cap, h1 = 1.22m
Length of sttoper, l = 1.40m



Horizontal force acting on the stopper above pier cap
= 10% of vertical load
= 265.25 KN

Friction resistance= 397.88 KN

However for additional factor of safety, assuming an extra lateral force in excess of frictional resistance taking aR = 0.20 in worst case.

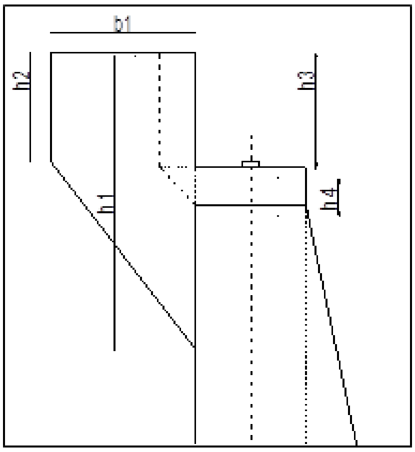
Therefore, Fs= 530.50 KN
Difference 132.63 KN
Moment at the base of stopper = 161.80 KN-m
Effective depth required = 330.61 mm
Effective depth provided =742.00 mm
Area of steel required = 1002.13mm²
Thus Provide Φ 16.00 mm bars @100.00 mm c/c
Giving an Steel Area of= 2814.87mm²

Max. Shear stress= 0.13N/mm²
Percentage area of steel provided= 0.27%
Permissible shear stress= 0.24N/mm²
> 0.13 N/mm², OK

Thus provide nominal shear reinforcement, Φ 10.00 mm bars @100.00 mm c/c

3.7 DESIGN OF RETURN WALL:

Return wall will be monolithic with back wall. They are joined together as shown. The load acting on the wing walls would be earth pressure and is designed to withstand a live load equivalent to surcharge of 1.2 m height of earth fill according to IRC:78-2000.



h1 = 4.0m
h2 = 2.00m
h3 = 2.75m
h4 = 0.50m
b1 = 3.50m
Thickness of return wall assumed= 350.00 mm
Average value of earth pressure
$=0.5\gamma s\times(h1/2+h2/2+1.2)^2\times\tan^2(45o-\Phi/2)$
= 49.21KN
Acting at a distance from wing wall bottom= 1.51m
Moment due to earth pressure= 180.69 KN-m/m
Depth required= 238.26 mm
Effective depth provided= 292.00 mm
Area of steel required= 2843.76mm²

Thus Provide Φ 16.00 mm bars @150.00 mm c/c
Giving an Steel Area of= 4035.13mm²
And, as distribution bar, provide Φ 10.00 mm bars @150.00 mm c/c in vertical direction

In the fillet joint, provide Φ 10.00 mm bars @200.00 mm c/c as nominal reinforcement

The wing wall would be properly anchored to the abutment, back wall and abutment cap.

3.8 DESIGN OF PILES AND PILE CAP

3.8.1 Materials and Properties

Concrete	M25
Reinforcement	Fe 500

3.8.2 Load and moment at the base of pile cap

Total vertical load excluding pile cap, ΣW=	5114.386KN
Pile cap load, ΣW=	3547.500KN
Vertical soil load at back to be taken by pile cap, ΣW=	3776.960KN
Vertical soil load at front to be taken by pile cap, ΣW=	651.200KN
Total vertical load, ΣW=	13090.05KN
Total horizontal load, ΣH=	2298.14KN

i) Let Bored pile of 800mm diameter and 15.45m effective length below scour depth should be used.

D = 800.00 mm= 0.80m
L = 15.45m

From Geo-technical investigation,
Q_a= 985.30 KN (From geo-tech report)
> 654.5022858KN ok

Let Initial Nos of pile= 20.00Nos.
20.00Nos.
Provide 20 piles in a grid pattern
Now for 100% efficiency of pile group , spacing of piles
= (1.57×B×m×n-2×B)/(m+n-2)
where,
Nos. of rows in pile group, m= 5.00
Nos. of columns in pile group, n= 4.00
Pile diameter, B= 800.00 mm
Thus spacing= 3360.00mm
Provide spacing of= 2400.00mmc/c
=(3-4)d
Considering an imaginary footing located at the ground surface
Keeping clear cover for the pile cap = 200 mm
Plan dimension of imaginary footing= 11 x 8.6= 94.6 m²

Extreme loads on individual pile
maximum load on individual pile= v/n+V.e.xi/Sxi2+V.e.yi/Syi2
minimum load on individual pile= v/n+V.e.xi/Sxi2+V.e.yi/Syi2
where,

v=	n=	M_y(long.)
13090.05	20.00	5718.46

Square of Distance of the piles= SXi2 = 2*(5*(2.4/2)^2+5*(2.4+2.4/2)^2) 144m²
Square of Distance of the piles=SYi2= 2*(4*(2.4)^2+4*(2.4+2.4)^2) 230.4m²
xi= 2.4/2+2.4 = 3.6
yi= 2.4+2.4 = 4.8
ex= 0.4+0.61-0.98= 0.03
ey= = 0Placed Symmetrical w.r.t Y-axis

V max = 797.46 KN < 985.3 Ok
V min = 644.68 KN > 0 Ok

3.8.3 Design of piles

Effective leng of piles, L1+Lf= 9.20
Lateral Dimension, B= 0.80m

Le/B= 11.501
< 12, SHORT COLUMN

3.8.4 Depth of Fixity:

The maximum scour level is at 142.67 m
The soil strata is Silty sand with mix Coarse Sand & Clay .
Modulus of Horizontal subgrade reaction (nh) = 4.500MN/m^3(from IS 2911, SEC 2)
Modulus of Elasticity(E) =25000MN/m^2
Moment of Inertia(I) = 0.020m^4

Stiffness Factor,T = 2.57m

Assuming Scour has taken place,
Cantilever Length of pile above scour level, L1= 3.55m
Therefore, L1/T = 1.383
From Fig 4 of IS 2911, Sec 2, Lf/T = 2.20 (For Fixed Head Piles)
Thus, depth to the point of Fixity, Lf = 5.65m
Reduction Factor corresponding to L1/T,(Fig 5) = 0.86
Horizontal load on each pile = 114.91 KN
Moment (M) = 454.6KN-m

Design moment =454.61 KN-m
Design Load = 114.91 KN

Effective dia. Of pile, = 630mm
Minimum area of steel= 0.80%
Maximum area of steel= 6.00%
Assume percentage area of steel= 1.00%
Gross area of concrete, Ag= 0.503m^2
Net area of concrete, Ac= 0.498m^2
X-sectional area of steel, As= 5027.00mm^2
Equivalent Area of Concrete = 577088.67mm^2
Equivalent Moment of Inertia = 24048421828mm^4
Section modulus, W= 60121054.57mm^3

Therefore, direct stress, σco,cal= 0.13N/mm^2
Bending compressive stress, σc,cal= 5.04N/mm^2

i) Combined stress = (σco,cal/σco)+(σc,cal/σc)
= 0.63
0.63 < 1, OK

Required Reinforcement (Ast) = 5027.00mm^2
Hence provide bars of 18 numbers of 20mm dia bars
Provided reinforcement (Ast) = 5654.87mm^2

3.8.5 Lateral Ties

Minimum volume of lateral reinforcement per m length of pile
= 0.2% of vol. of pile
= 1005309.65mm^3
Assume Φ 10.00 mm lateral ties
Volume of 10 mm dia. tie= 197392.09mm^3

No. of ties per meter of pile= 6.00 Nos.
Hence spacing for lateral reinforcement= 166.67 mm
< 500 mm
< 36 x 20 = 720 mm
< 48 x 10 = 480 mm

Hence provide lateral tie of 10 mm dia. @ 150 mm c/c

3.8.6 Design of pile cap

Pile cap is 11m x 8.6m

Bending moment at the face of the abutment wall= 1262.65KN-mper m width of pile cap

From working stress method:

d= Sqrt(M/(Rxb))

1092.777mm

Adopt effective thickness of pile cap as d = 1412.5 mm

Total thickness of pile cap, D= 1500mm

Factored Punching shear force= 19635.07KN

Punching shear stress= 0.63N/mm²

< 1.11 N/mm², OK

Reinforcement

From Minimum reinforcement criteria,(0.15%) Ast= 2118.75mm²

From Moment Criteria, Ast= M/(σst x j x d)

= 4108.1 mm²

Provide Φ 25.00 mm bars

Spacing of bar= 120.00 mm

Thus provide 25 mm dia @ 120 mm c/c in both directions of pile cap at bottom.

Check for one- way shear

Upward pressure due to total load = 207.559KN/m²

S.F. per m width at section critical section= 433.28 KN

Shear stress, τv= 0.31N/mm²

From code table-19,

Minimun τv= 0.18N/mm²

Design for Stirrups

Percentage of Steel = 0.29%

Shear Strength of Fe 500 concrete for 0.29 % steel, τc'= kτc

k= 1.00

τc= 0.24N/mm²

So, τc'= 0.24N/mm²

So, Shear Reinforcement is required

3.8.7 Design of Stirrups

V_{us}=(V-τc*b*d)= 90.504 KN

Assume Φ 16.00 mm lateral ties 1 legged(Open ties)

Spacing of Stirrups = 1365.02mm

Thus provide 16mm dia open ties @ 1000 mm spacing c/c.

No. of Pile in shorter Strip= 4

width of the strip= 2400.00mm

Max. shear force= 797.464KN

Shear stress, τv= 0.235N/mm²

Shear Strength of Concrete = 0.24N/mm²

Nominal shear reinforcement required

CHAPTER 4

2. *DESIGN OF BRIDGE PIER*

4.1 DESIGN DATA

4.4.1 Materials and Properties

Concrete	M25
Reinforcement	Fe 500

<u>Basic Permissible Stresses of Concrete as per IRC : 21-2000</u>

Permissible direct compressive stress(σco)	6.25MPa
Permissible flexural compressive stress(σc)	8.33MPa
Basic permissible tensile Stress(σt)	0.61MPa
Maximum Permissible shear stress(τmax)	1.90MPa

<u>Basic Permissible Stresses of Reinforcing Bars as per IRC : 21-2000</u>

Permissible Flexural Tensile stress(σst)	240.00MPa
Permissible Direct Compressive stress(σco)	205.00MPa

<u>Self weight of materials as per IRC : 6-2000</u>

Concrete (Cement-Reinforced)	2500.00Kg/m³
Macadam (Binder Premix)	2200.00Kg/m³
Water	1000Kg/m³
Backfill	1600Kg/m³

<u>Design of Data:</u>

Modular Ratio (m)	11.20
Neutral axis depth factor, n = (mσc)/(mσc+σst)	0.28
Lever arm factor, j = (1-n/3)	0.91
Moment of resistance coefficient, R = ½ x n x j x σc	1.06MPa

Left Bank foundation on Rock

Right Bank foundation on soil

Maximum Seismic Coefficient for Seismic Zone V according to IS: 1893-1984	0.169

4.1.2 Dimension Parameters

Effective Span of Bridge	25.00m
Total Span of Bridge	54.25m
High Flood Level, HFL	149.81m
Lowest Bed Level, LBL	144.25m

Size of bearing

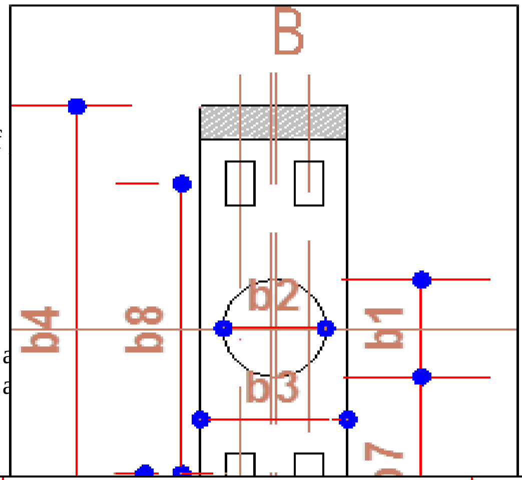
Length	1.20m
Width	1.20m

Expansion Joint	0.05m
Depth of Superstructure	2.50m
Depth of Bearing & Pad	0.25m
Ave. Velocity of water current	2.00m/s
Pier shape, circular or with semi-circular ends, K	0.66
RL of max scour lev.	142.668m
Free board	1.5m
RL of Pier Cap required	151.060m
RL of Pier Cap provided	151.060m
RL of foundation top	143.970m
Ht. of HFL from base of pier	5.84m
Ht. upto deck from GL	9.81m
C/C distance between girder	5.00m

b1=
b2=
b3=
length of

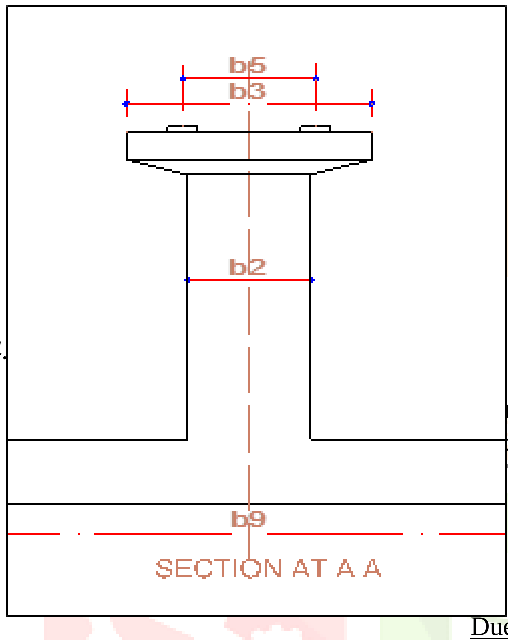
b5=
b6=
b7=
b8=
h1=

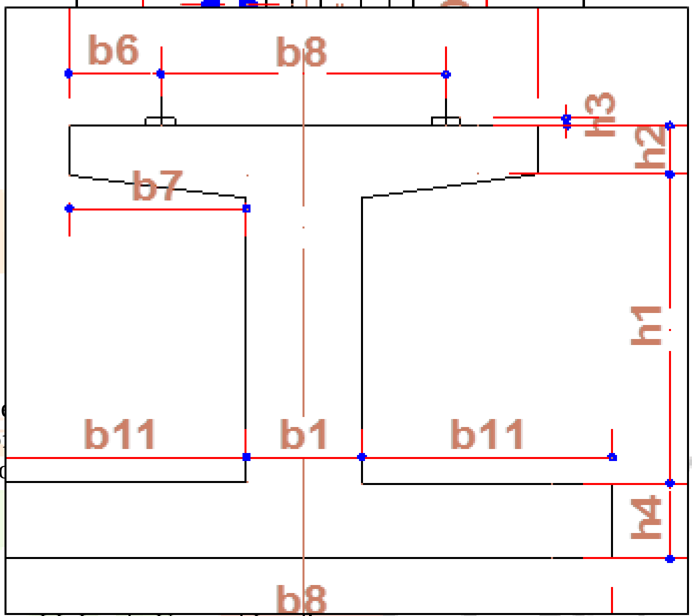
h3=



h4= 1.50

4.

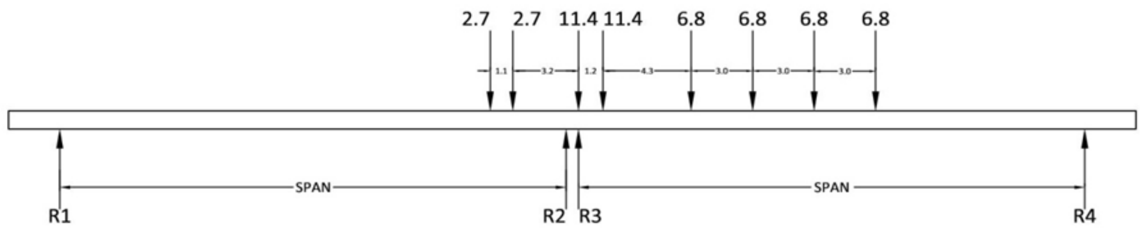




<u>Due to LL Load - Unequal Loading</u>

Class A wheel loading in longitudinal direction

For maximum load on pier, the arrangement of IRC class A loading will be as shown below:



Max. LL on pier from right side =(11.4x26.2+11.4x25+6.8x20.7+6.8x17.7+6.8x14.7+6.8x11.7)/25

=40972.80Kg

Impact factor= 1.15

Max LL including Impact(double lane)= 93840.93Kg

LL / m length of pier= 39932.31Kg

LL from left side= (2.7x20.72+2.7x21.82)/25

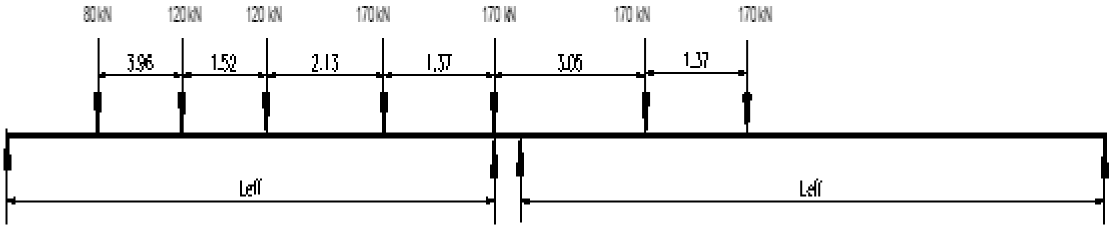
4594.32Kg

LL including Impact (double lane)= 10522.47Kg

Total live load including impact for double lane = 110405.07Kg

Class 70R wheel loading in longitudinal direction

For maximum load on pier, the arrangement of IRC class 70R loading will be as shown below:



Max. LL on pier from right side = (17x25+17x23.63+12x21.5+12x19.98+8x16.02)/25
= 58105.20Kg

Impact factor= 1.09
Max LL including Impact= 63334.67Kg
LL from left side= (17x23.17+17x21.8)/25)
30579.60Kg
LL including Impact= 33331.76Kg
Total live load including impact = 102708.10

In Transverse Direction

Max. LL including impact (for class A)= 104363.40Kg
Max. LL including impact (For 70R)= 96666.43Kg
Max. LL including impact= 104363.40Kg
Eccentricity (Forcals A)= 0.95m
Eccentricity (For 70R)= 1.15m
Moment due to LL= 111166.40Kg-m

Due to Longitudinal Forces

Due to tractive effort or braking= 20000.00Kg
This acts at 1.20 m above deck level
or at a distance from base of pier 11.04m

Due to resistance in bearing due to temperature

It is possible that the frictional coefficients of two bearings on the pier may happen to be different due to unequal efficiency of the bearings. For the severest effect, it is assumed that LL to be on the right span and the frictional coefficients of bearings to be 0.25 and 0.225 on the right and left span respectively.

The total resistance offered by the right bearing= 0.25(0.5DL+max.LL)
87616.48Kg
The total resistance offered by the left bearing= 0.225(0.5DL+max.LL)
78854.83Kg
Unbalanced force at the bearing= 8761.65Kg
Unbalanced force due to temperature on DL only= 6415.63Kg
This force acts at the bearing level, i.e. at a distance from pier base=7.34m

Total longitudinal force due to Braking & temperature= 28761.65Kg
Moment due to longitudinal force= 285110.50Kg-m
Moment due to temperature on DL only= 47090.69Kg-m

Due to water current

For pier parallel to direction of water current, the intensity of pressure is given by
Ic= $52KV^2$
where,
K is a constant depending on the geometry of pier= 0.66
V is velocity of current= 2.83 m/sec
Ic= 274.56 Kg/m^2
Height of HFL from base of pier= 5.84m
Width of pier at HFL= 2.35m
a.) water current force parallel to the pier= 3768.06Kg
And corresponding moment= 11002.74Kg-m
b.) Water current varying at 30 degree=
Intensity parallel to the pier= 258.00 Kg/m^2
Intensity perpendicular to the pier= 137.28 Kg/m^2

Force parallel to the pier= 3540.82Kg
Force perpendicular to the pier= 1884.03Kg
Moment parallel to the pier= 10339.19Kg-m
Moment perpendicular to the pier= 5501.37Kg-m

Due to wind force

The area of superstructure in elevation providing 25% for railing= 78.66m²
a. height upto deck level from ground level = 9.81m
Wind pressure at that height (refer. IRC:6-2000), P = 90.16Kg/m²
Wind force on superstructure = 7092.22Kg
Wind force against moving load 20.4 m long corresponding to IRC class A @ 300.00 Kg/m
= 6120.00Kg
Total wind force= 13212.22Kg
b. Minimum force on deck at 450.00 kg/m= 11250.00Kg

c. Minimum force with wind pressure of 240.00 kg/m² in the plane of unloaded structure
= 18878.40Kg
Wind force to be considered = maximum of (a,b,c)= 18878.40Kg
Moment parallel to pier due to wind acting at 1.5m above deck = 138567.46Kg-m

Due to Buoyancy

Force due to buoyancy on the pier shaft= 15571.13Kg

4.3.1 Due to seismic forces

A. Along longitudinal direction

Description	Total Load (Kg)	Seismic Load(Kg)	Lever arm (m)	Moment (Kg-m)
Superstructure DL	509250.00	85935.94	7.34	630769.78
Pier cap	91422.94	15427.62	6.09	93954.21
Pier Shaft	55192.92	9313.81	2.55	23703.64
Total	**655865.86**	**110677.36**		**748427.63**

B. Along Transverse direction

Moment due to dead load is taken to be same as that in longitudinal direction. In addition, seismic forces and moments on LL including impact must be considered for transverse condition.

Total LL on pier= 104363.40Kg
Seismic force due to LL= 17611.32Kg
Lever arm= 7.34m
Moment due to seismic force on LL= 129267.12Kg-m

Total Seismic force= 128288.69Kg
Total Moment due to seismic force= 877694.75Kg-m

4.3.2 Summary of Loads and Moments

Description	Vertical Load (Kg)	Horizontal Force (Kg)		Moment (Kg-m)	
		Tran.	Long.	Tran.	Long.
DL (Superstr.+pier)	655865.9				
LL (Unequal loading)	104363.4			111166.4	0.0
Longitudinal force					
(Braking+Temp)			28761.6		285110.5
Water current at 30					
Degree skew		3540.8	1884.0	10339.2	5501.4
Wind forces		18878.4		138567.5	
Buoyancy	15571.1				

Seismic forces		128288.7	110677.4	877694.8	748427.6

The forces and moments due to seismicity are greater than those due to wind forces. As per standard design practice, the seismic forces and moments will be adopted neglecting the effect due to wind forces.

Case	A: DL+LL+LF+WC	B: DL+LF+WC+B	C: Case A + SF
Vertical load (Kg)	760229.27	671436.99	815567.95
Hz Load (Kg)	30645.68	30645.68	158934.37
Moment (Kg-m)	290611.87	290611.87	1029739.16

4.4 DESIGN OF PIER SHAFT SECTION

The pier section will be designed for the Case A and the section adequacy will be checked for both the cases. As the moment of inertia of the pier along Y-Y axis is greater than along X-X axis, the design needs to be done for stresses along the X-X axis only.

Design vertical load= 760229.27Kg
Design Moment= 290611.87Kg-m
Eccentricity, e= 0.38m
Diameter of the pier shaft, D= 2.35m
Effective diameter, d= 2218.00mm

Check for position of eccentricity

Length of pier, L= 7.09m
Effective length of column, l_e= 1.2 x L
= 8.51m
Lateral Dimension, D= 2.35m Ratio of effective length to lateral dimension of pier = 3.62
< 12, SHORT COLUMN
Minimum area of steel= 0.80%
Maximum area of steel= 6.00%
Assume percentage area of steel= 1.00%
Gross area of concrete, Ag= 4.337m²
Net area of concrete, Ac= 4.294m²
X-sectional area of steel, As= 43374.00mm²
Equivalent Area of Concrete = 4979588.53mm²
Equivalent Moment of Inertia = 1.91867E+12mm4
Section modulus, W= 1632911552mm³
Therefore, direct stress, σco,cal= 1.52N/mm²
Bending compressive stress, σc,cal= 1.77N/mm²

Check

i) Combined stress= (σco,cal/σco)+(σc,cal/σc)
= 0.46
0.46 < 1, OK

ii) Condition for tensile stresses to be within limit,

a) (σc,cal-σco,cal)<=0.25*(σc,cal+σco,cal)
= 0.25 <= 0.8225 OK

b) (σc,cal-σco,cal) <= 0.75* 7-day modulus of rupture of concrete
7-day modulus of rupture of concrete = 0.56*√fck
= 2.8N/mm²
0.25 <= 2.1 OK

iii) Resultant compressive stress= 3.29N/mm²
< 8.33N/mm², OK

Thus provide 55 Nos. of Φ 32.00 mm bars@spacing of 130 mm in one layer
Giving an Steel Area of= 44233.62mm²

Design for Lateral Tie

Diameter of lateral tie should be greater than $\Phi_L/4$

$\Phi_L/4$= 8mm

Pitch of lateral tie

< least lateral dimension of column= 2350mm

< 12x dia of smallest longitudinal bar= 384mm

Thus Provide, Φ 12.00 mm lateral tie @140 mm c/c at ends and @ 200 mm c/c span with cross links.

Check for stresses in concrete and steel for case B

Design vertical load= 671436.99Kg

Design Moment= 290611.87Kg-m

Eccentricity, e= 0.43m

Diameter of the pier shaft, D= 2.35m

Effective diameter, d= 2245.00mm

Check for position of eccentricity

Therefore, direct stress, σco,cal= 1.34N/mm²

Bending compressive stress, σc, cal= 1.77N/mm²

Check

i) Combined stress= (σco,cal/σco)+(σc,cal/σc)

= 0.43

0.43 < 1, OK

ii) Condition for tensile stresses to be within limit,

a) (σc,cal-σco,cal)<=0.25*(σc,cal+σco,cal)

= 0.43 <= 0.7775 OK

b) (σc,cal-σco,cal) <= 0.75* 7-day modulus of rupture of concrete

7-day modulus of rupture of concrete = 0.56*sq.root fck

= 2.8N/mm²

0.43 <= 2.1 OK

iii) Resultant compressive stress= 3.11N/mm²

< 8.33N/mm², OK

Check for stresses in concrete and steel for case C

Design vertical load= 815567.95Kg

Design Moment= 1029739.16Kg-m

Eccentricity, e= 1.26m

Diameter of the pier shaft, D= 2.35m

Effective diameter, d= 2200.00mm

Check for position of eccentricity

Therefore, direct stress, σco,cal= 1.63N/mm²

Bending compressive stress, σc, cal= 6.30N/mm²

Check

i) Combined stress= (σco,cal/σco)+(σc,cal/σc)

0.68

0.68 < 1, OK

ii) Condition for tensile stresses to be within limit,

a) (σc,cal-σco,cal)<=0.25*(σc,cal+σco,cal)

= 4.67 > 2.97375 NG

As per IS 456, permissible stress in concrete is stated as 3.2*1.5=4.8 Mpa. Thus ok for cracked section.

b) (σc,cal-σco,cal) <= 0.75* 7-day modulus of rupture of concrete

7-day modulus of rupture of concrete = $0.56*\sqrt{fck}$

= 2.8N/mm²

4.67 > 3.15 NG

As per IS 456, permissible stress in concrete is stated as 3.2*1.5=4.8 Mpa. Thus ok for cracked section.

iii) Resultant compressive stress= 7.93N/mm²

< 12.495N/mm², OK

For One span collapse condition:

Due to seismic forces

A. Along longitudinal direction

Description	Total Load (Kg)	Seismic Load(Kg)	Lever arm (m)	Moment (Kg-m)
Superstructure DL	254625.00	42967.97	7.34	315384.89
Pier cap	91422.94	15427.62	6.09	93954.21
Pier Shaft	55192.92	9313.81	2.55	23703.64
Total	**401240.86**	**67709.40**		**433042.74**

B. Along Transverse direction

Moment due to dead load is taken to be same as that in longitudinal direction. In addition, seismic forces and moments on LL including impact must be considered for transverse condition.

Total LL on pier= 0.0Kg

Seismic force due to LL= 0.00Kg

Lever arm= 11.04m

Moment due to seismic force on LL= 0.00Kg-m

Total Seismic force= 67709.40Kg

Total Moment due to seismic force= 433042.74Kg-m

4.4.1 Summary of Loads and Moments

Description	Vertical Load (Kg)	Horizontal Force (Kg)		Moment (Kg-m)	
		Tran.	Long.	Tran.	Long.
DL (Superstr.+pier)	401240.9				155321.3
LL (Unequal loading)	-			-	-
Longitudinal force					
(Braking+Temp)			4380.8		32155.2
Water current at 20					
Degree skew		3540.8	1884.0	10339.2	5501.4
Wind forces		9439.2		69283.7	
Buoyancy	15571.1				
Seismic forces		67709.4	67709.4	433042.7	433042.7

The forces and moments due to seismicity are greater than those due to wind forces. As per standard design practce, the seismic forces and moments will be adopted neglecting the effect due to wind forces.

Case	A: DL+LL+LF+WC	B: DL+LF+WC+B	C: Case A + SF
Vertical load (Kg)	401240.86	416811.99	435095.56
Hz Load (Kg)	6264.85	6264.85	73974.25
Moment (Kg-m)	192977.87	192977.87	626020.61

CHAPTER 5

3. *CONCLUSION*

Bridges connects destinations. The twentieth century finally saw two major innovations in bridge design and construction. Reinforced concrete gave the bridge engineers a most versatile construction material at hand that could be cast into literally any shape, only limited by laws if nature and the imagination of the designer. And Secondly, the new type of cable-stayed bridges appeared in the second half of the twentieth century and quickly established itself as a very economical and aesthetically satisfying member of the bridge family. In this project, the bridge is design using RCC structure implementing IRC code considering seismic analysis. Here design is done manually and the data is extracted from sap analysis. All types of load like dead load, live load, seismic forces, water pressure and etc is consider while designing this project.

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