Tribhuvan University
Institute Of Engineering
Thapathali Campus
DEPARTMENT OF CIVIL ENGINEERING

A
FINAL REPORT ON
"DESIGN OF RCC T-BEAM BRIDGE OVER ANDHERI KHOLA , NAWALPARASI"

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Tribhuvan University

Institute Of Engineering

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DEPARTMENT OF CIVIL ENGINEERING

Certificate

This is to certify that the final year project entitled **"DESIGN OF RCC T-BEAM BRIDGE OVER ANDHERI KHOLA, NAWALPARASI"** was submitted by the students to the **DEPARTMENT OF CIVIL ENGINEERING** in partial fulfillment of requirement for the degree of Bachelor of Engineering in Civil Engineering. The project was carried out under special supervision and within the time frame prescribed by the syllabus.

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ACKNOWLEDGEMENT

We are grateful to our Department of Civil Engineering, Thapathali Campus for providing us with this project to further enhance our knowledge in the field of Civil Engineering and its applications. The bridge has been a predominant feature of human progress and evaluation. It has always been a symbol of progress and development. Hence in the context of our country construction of bridge plays a vital in development which is the undoubted reason for us choosing this project. We are indebted to our advisor and supervisor Prof. Dr. Bharat Mandal for his valuable time, instruction and guidance throughout the project. We are heartily thankful to Local Road Bridge Support Unit (LRBSU) for its technical and financial support throughout the project so as to help develop a highly esteemed and efficient Bridge Engineer in the country. We are deeply obliged to Er. Chuman Babu Shrestha, capacity Building Specialist, LRBSU for arranging us the field visit, providing the design data and necessary training. We would like to express our deepest gratitude and appreciation to teachers, friends and all others who has helped the team directly or indirectly during this project work, are also the part of the team's sincere acknowledgement.

Salient Features

Particulars	Required Information/Range/Values
Name of the Project	Design of RCC Bridges over Adheri Khola
Location	
Zone	Lumbini
District	Nawalparasi
Village/Town	Left of khola:Andheri Right of khola: Karamghari
Name of the road	Arunkhola to palpa road
Geographical location	
Easting	$83^{o}57'0''$E
Northing	$27^{o}37'26''$N
Classification of road	Rural road
Type of road surface	Gravel
Terrain/Geology	Plain
Information on the structure	
Total length of the bridge	20.35m
Span arrangement	21.35m
Total width of the bridge	9m
No of lanes	two
Width of :	
Carriageway	7.5
Footpath	No footpath
Types of Superstructure	RCC T girder
Type of bearings	Elastomeric Pad bearings
Types of abutments	RCC rectangular abutments
Types of piers	No piers
Design Data	
Live loads	IRC Class 70R IRC Class A
Soil types	Gravel sand mix
Net bearing capacity of soil	300 KN/m^2
Design Discharge	48.647 m^3/s
Linear waterway	33.13 m

Notations

Φ	Diameter of Bar
τc	Shear Stress
Υt	Partial Safety Factor
Ah	Horizontal Seismic Factor
Ast	Area of Steel
bf	Width of Flange
bw	Width of Web
d	Effective Depth
D	Overall Depth
Df	Depth of Flange
E	Young's Modulus of Elasticity
fck	Characteristics Strength of Concrete
fy	Characteristics Strength of Steel
I	Moment of Inertia
Ld	Development Length
M	Bending Moment
R	Response Reduction Factor
Sv	Spacing of Stirrups
Z	Zone Factor

Abbreviations

DL	Dead Load
IRC	Indian Road Congress
LL	Live Load
LRBSU	Local Road Bridge Support Unit
RCC	Reinforced Cement Concrete

Table of Contents

1. Title of Project

The main objective of this project is to design a bridge over Andheri Khola. So, this project is entitled as **"Design of RCC Bridge over Andheri Khola, Nawalparasi".**

2. INTRODUCTION

2.1. Background

Project works are essential to prepare the students with skills required to synthesize comprehensively the knowledge gained during course work for a practical application of civil engineering discipline in real life. This project has been undertaken as a part of partial fulfillment of acquiring Bachelor's Degree in Civil Engineering as prescribed by the curriculum of Institute of Engineering (I.O.E).This project reflects the cultivation of knowledge and skills gained by the group members during the four years long studies in Bachelor of Civil Engineering. The major features include the incorporation of complete field data verification, hydraulic design analysis and preliminary design for short span double lane RCC T-beam Bridge deploying commonly practiced design principles and IRC recommendation for local road bridge. In this project, we are assigned to design a bridge over Andheri Khola, Nawalparasi. As it is a local road so a single lane bridge shall suffice. But, for our study purpose, the bridge will be designed as a two-lane bridge. We are supposed to design the most economic bridge for this section based of the various data provided by LRBSU.

2.2. Objectives

- Design of superstructure of bridge.
- Design of substructure and its foundation.

2.3. Project assignment or scope of project

Following assignments were completed during the completion of the project:

- Study of topographic, geological, hydrological, geotechnical and traffic condition of bridge site from initial survey reports provided by LRBSU.
- Visit of bridge site and preparation of site observation report including verification of data required.
- Study of different alternatives of span arrangements for the bridge and comparing them.
- Selection of an appropriate bridge type.
- Carrying out of design and detailing of selected bridge type.
- Preparation of detail drawing of bridge superstructures with its all components, abutments, pier, bearing and footing required for the construction of selected bridge type.

2.3.1. Limitations of project:

- ❖ Design on the basis of secondary data.
- ❖ Based on IRC code 5, 6, 21, 78, 83, 112.

2.4. Site Description

The proposed crossing structure over Adheri khola bridge will interlink the Karamghari and Adheri V.D.C, Nawalparasi.The salient features of the site are:

Province no	: 5
Zone	: Lumbini
District	: Nawalparasi
Muncipality	: Madhyabindu
Ward No.	: 13
Village	: Magarsingha (Karamghari/Andheri)
Access road	: Andheri to Palpa road
Co-ordinates	: E=495302.566
	: N=3056233.453
	: Z=158.270

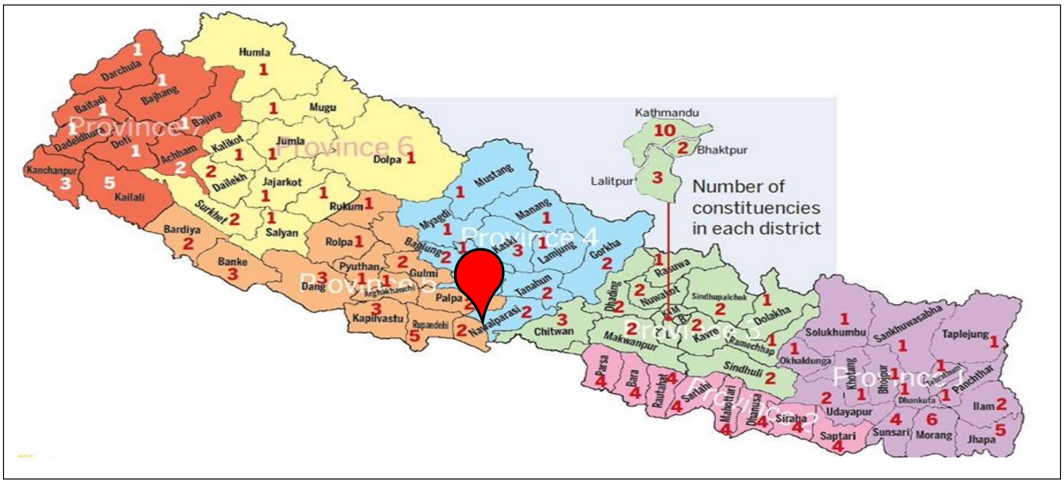


Fig2.1 Site Location

Fig2.2 Bridge axis on google map

2.5. Support from LRBSU

The bridge was designed for LRBSU as per their requirement based on the present need. LRBSU has provided us the data such as topographical survey, geotechnical data. It also helped us economically during surveying works and for learning SAP.

3. METHODOLOGY

3.1. Acquisition of Data

3.1.1. Preliminary Data

For the design of our bridge the preliminary data needed was acquired from the report prepared by consultancy but in actual practice it is done by following methods.

3.1.1.1. Site Selection Survey

Site selection survey is done by a team of bridge engineer, geotechnical engineer, surveyor and hydrologist. After consultation with local residents, technical personnel of Divisional Road Office of the site, proposed bridge alignment is fixed.

3.1.1.2. Topographical Survey

Tachometric survey is carried for detailed engineering survey of the proposed bridge site. Theodolites, level machines, staffs and measuring tape are usually used for detailed survey. After consultation with the technical personnel and the local villagers and as directed by the river morphology; an axis joining line joining left bank and right bank is fixed. Temporary Benchmark is also fixed. The bridge site detailing area covers a suitable region along the length of river both upstream and downstream. It also covers left and right banks along the existing approach roads. L -section of approach road and river measured, and the benchmarks and bridge points are shown in contour maps.

3.1.2. Geotechnical Investigation

Geotechnical investigation is one of the major parts of the project work for the design of the proposed bridge at Arun Khola in Nawalparasi district. Geotechnical investigation works includes core drilling, test pitting, visual investigation at site. Detail report and test results with bore logs are included in the actual project report. For our project the site and its contour map, hydrological data and geotechnical data were provided by LRBSU.

3.1.3. Hydrology

The maximum discharge which a bridge across a natural stream is to be designed to pass can be estimated by the following methods:

- ❖ By using one of the empirical formula applicable to the region
- ❖ By using the rational method involving the rainfall and other characteristics.
- ❖ By the area velocity method, using the hydraulic characteristics of the stream such as cross- sectional area, and the slope of the stream
- ❖ From any available records of the flood discharges observed at the or at any other site at the vicinity.

It is desirable to estimate the flood discharge by all or at least two of the above methods. These methods are briefly discussed here.

3.1.3.1. Empirical Formula

Empirical formulae for flood discharge from a catchment have been proposed of the form:

$Q=CA^n$

Where,

Q=maximum flood discharge in m3 per second.

A= catchment area in Km^2.

C= constant depending on the nature of the catchment and the location.

n= constant.

A popular empirical formula of the above type is Ryve's formula given by equation

$Q=CA^{2/3}$

The value of C is taken as 6.5 for the tracts near the coast, 8.5 for the areas between 25 and 50 km of the coast and 10 for the limited areas near hills.

3.1.3.2. Rational formulae

A rational formula for the flood discharge take into account the intensity, distribution and duration of rainfall as well as the area, shape, slope, permeability and initial wetness of catchment. Many complicated formulae are available in treatises on hydrology. A typical rational formula is:

Q=CiA/3.6

where , Q=discharge (m3/s)

C=coefficient of runoff

A=catchment area (km2)

i=intensity (mm/hr)

3.1.3.3. Area-velocity method

The area-velocity method based on the hydraulic characteristics of the stream is probably the most reliable among the methods of determining the flood discharge. The velocity obtaining in the stream under the flood conditions is calculated by Manning's or similar formula: Manning's formula is used here. The discharge is given by equation:

Q= AV

Where,

Q= discharge in m3/sec.

A= wetted area in m2.

V= velocity of flow in m/s

V=(1/n)*R2/3S0.5

n=coefficient of roughness.

S= slope of stream.

R= hydraulic mean radius in m.

R = wetted area in m2/wetted area in meters.

3.1.3.4. Estimation from flood marks

If flood marks can be observed on an existing bridge structure near the proposed site, the flood discharge passed by the structure can be estimated reasonably well, by applying appropriate formulae available in treatises on hydraulics. It is possible by inspection to ascertain the flood levels soon after a flood. Sometimes, these flood marks can be identified even years after a flood, but it is desirable to locate these as soon after the flood as possible.

3.1.3.5. Design discharge

The design discharge may be taken as the maximum values obtained from at least two methods mentioned. If the values obtained by two methods differ by 50%, then the maximum design discharge is limited to 1.5 times the lower estimate. Freak discharge of high intensity due to failure of dam or tank constructed upstream of the bridge site need not to be catered for. From consideration of economy, it is not desirable to aim to provide for the passage of the very extraordinary flood that may ever happen at a particular site. It may be adequate to design for a flood occurring once in 20 years in case of culverts and once in 100 years for bridges and to ensure that the rarer floods be passed without excessive damage to structure.

3.1.3.6. Linear waterway

When the water course to be crossed in an artificial channel for irrigation or navigation, or when the banks are well defined for artificial streams, the linear waterway should be full width of channel or stream. For large alluvial stream with undefined banks, the required linear waterway should be determined using Lacey's formula given by equation

$$L = C\sqrt{Q}$$

Where,

L= the linear waterway in m.

Q= the designed maximum discharge in m3 per second.

C= a constant, usually taken as 4.8 for regime channels, but may vary from 4.5 to 6.3 according to local conditions.

It is not desirable to reduce the linear waterway below that for regime condition. If a reduction is affected, special attention should be given to afflux and velocity of water under the bridge. With reduced waterway, velocity would increase and greater scour depth would be involved, requiring deeper foundations. Thus, any possible saving from a smaller linear waterway will be offset by extra expenditure on deeper foundations and protective works.

3.1.3.7. Afflux

Afflux is the heading up of water over the flood level caused by the constriction of Waterway bridge site.it can be computed from equation

$X = (v^2/2g) * ((L^2/c^2L^2)-1)$

Where, X= afflux

V= velocity of normal flow in stream

g= acceleration due to gravity

L= width of stream in at H.F.L

L1= linear waterway under the bridge

c= coefficient of discharge through the bridge, taken as 0.7 for sharp entry and 0.9 for bell mouth entry.

The afflux should be kept minimum and limited to 300mm. Afflux causes increase in Velocity in downstream side, leading to greater scour depth and requiring deeper Foundations. The road formation level and the top level of the guide bunds are dependent on the maximum water level on upstream side including afflux.

3.1.3.8. Economical span

Considering only the variable items for the given linear waterway, the total cost of the superstructure increases with the increase or decrease in span length. The most economical span is that for which the cost of superstructure equals the cost of substructure.

This condition may be derived as below:

Let, A= cost of approaches

B= cost of two abutments including foundations

L= total linear waterway

l= length of one span

n= number of spans

P= cost of one pier, including foundation

C= total cost of bridge.

Assuming that the cost of superstructure of one span is proportional to the square of the span length, total cost of superstructure equals n.kIs, where k is a constant. The cost of railing, flooring, etc., is proportional to the total length of bridge and can be taken as kL.

C= A + B + (n-1) P + nkl2 +K'L

For minimum cost, dC/dL should be zero,

We get: P= k.l2

Therefore, for an economic span, the cost of superstructure of one span is equal to the cost of substructure of same span.

The economical span (le) can then be computed from:

Le= $\sqrt{}$(P/k)

P and k are to be evaluated as average over a range of possible span lengths.

<u>Location of pier and abutments</u>

Pier and abutments should be as to make the use of best the foundation conditions available. As far as possible the most economical span as above should be adopted. If navigational or aesthetic requirements are to be considered, the span may be suitably modified. As a rule, the number of spans should be kept low, as pier obstructed water flow. If piers are necessary, odd number of spans should be preferred. For small bridges with open foundations and solid masonry piers and abutments, the economical span is approximately 1.5 times the height of pier or abutments, while that for masonry arch bridges it is about 2 times the height of keystone above the foundation, the question has to be examined in detail. The alignment of pier and abutments should be, as far as possible, parallel to the mean direction of flow in the stream. If any temporary variation in the direction and velocity of stream current is anticipated, suitable protective works should be provided to protect the sub-structure against the harmful effects on stability of the bridge structure.

3.1.3.9. <u>Scour depth</u>

Scour of stream bed occurs during the passage of a flood discharge, when the velocity of stream exceeds the limiting velocity that can be withstand by the particles of the bed material. The scour should be measured with reference to existing structures near the proposed bridge site, if this is possible. Due allowance should be made in the observed value for additional scour that may occur due to the designed discharge being greater than the flood discharge for which the scour was observed, and due to increased velocity due to obstruction to flow caused by the construction of bridge. Where the above practical method is not possible, the normal depth of scour may be computed by equation for natural streams in alluvial beds.

$d=0.473(Q/f)_{1/3}$

Where,

d= normal scour depth below H.F.L. for regime conditions in a channel

Q=designed discharge in m3 per second

f=Lacey’s silt factor for a representative sample of bed material

The minimum depth of foundation below H.F.L. is kept at 1.27d for abutment and 2d for piers.

3.1.4. <u>Span Arrangement of Bridge</u>

We analyzed various possible span arrangements for our site. The possibilities are:

R.C. Slab: R.C slab can withstand maximum span of about 10m. But calculated span of proposed bridge is 20.35m. So, about 2 spans are required which is uneconomical.

R.C. T- Girder: Its maximum span can be about 30m. So, 1 span of 20.35m can be used. Previous studies shows that it costs about 10-12 lakhs per meter.

Steel Truss: Its span ranges from 25-300m. Previous studies show that it costs about 20 lakhs per meter which seems to be uneconomical in our proposed bridge.

Steel Plate Girder: It costs about 15 lakhs per meter.

Pre-stressed concrete box: Its span ranges from 40-80m. Its cost is about 20 lakhs per meter. It has high structural strength compared to others. But its design and analysis procedures are complex.

So, among above possibilities pre-stressed concrete turns out to be the best option. But since we are new to the bridge design we first chose to work on R.C. T-Girder bridge.

4. LITERATURE REVIEW

4.1. Selection of Bridge Site

It may not be always possible to have a wide choice of sites for a bridge. This is particularly so in case of bridges in urban areas and flyovers. For rivers bridges, in rural areas, usually a wider choice may be available. The characteristics of an ideal site for a bridge across a river are:

a. Straight reach of river.

b. Steady river flow without whirls and across currents.

c. A narrow channel with firm banks.

d. Sustainable high banks above high flood level on each side.

e. Rock or other hard in-erodible strata close to the river bed level.

f. Proximity to a direct alignment of the road to be connected.

g. Absence of sharp curves in the approaches.

h. Absence of expensive river training works.

i. Avoidance of excessive underwater construction.

In selection a site a care should be taken to investigate a number of probable alternative sites and then decide on the site which is likely to serve the needs of the bridges at the least cost.

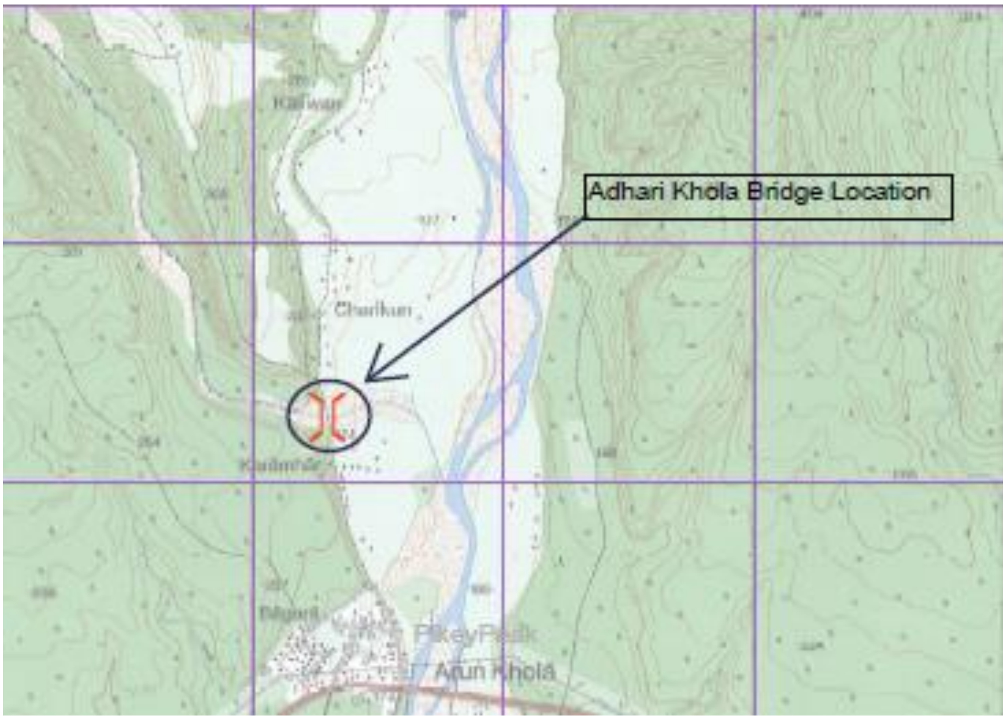


Fig4.1: Bridge site on topomap

4.2. Loading

IRC loads for the bridge design:

According to IRC: 6-2000, road bridges and culverts are classified on the basis of loadings that they are designed to carry.

IRC class 70-R loading:

This loading is to be adopted within certain limits, in certain existing or contemplated industrial areas, and along certain specified highways and areas. Bridges designed for class 70-R loading should be checked for class A loading which is considered in each lane.

IRC class A loading:

This loading is normally considered on all in which dominant bridges and culverts are constructed. One train of class A loading is considered in each lane.

4.3. Superstructure

The basic function of bridge superstructure is to permit the uninterrupted smooth passage of traffic over it and to transmit the loads and forces to the substructure safely through the bearings. Although it is difficult to stipulate the aesthetic requirements, it should, however, be ensured that the type of superstructure adopted is simple, pleasing to the eye, and blends with the environment. The superstructure of any bridge must be designed such that it satisfies geometric and load carrying requirements set forth by its owner. This geometric requirement depends upon the number and widths of traffic lanes and footpaths that have to be carried across. They also depend on overall alignment and various horizontal and vertical clearances required above and below the roadway. The superstructure designed has to meet various structural design requirements such as strength, stiffness and stability. The horizontal and vertical alignment of a bridge is governed by the geometrics of the highway, roadway or channel, it is crossing. For girder type bridges, the girders may either be curved or straight, and may be aligned on chords between supports with the deck slab built on the curve.

4.3.1. Components of superstructure

Lighting:

The lighting of the bridge is generally according to the provisions of the authority having jurisdiction on that area.

Drainage:

The transverse drainage of the roadway is usually accomplished by providing suitable crown in the roadway surface, and the longitudinal drainage is accomplished by camber or gradient.

Traffic lane:

Roads designed for traffic flow can be single lane, double lane or more. Road width in meters should be divided by 3.65 and the quotient approximated to the nearest whole number of design traffic lanes. We have designed our bridge with two traffic lane.

Road Width:

Road width is the distance between the roadside faces of the kerb which depends on the number and width of traffic lanes and the width of the bounding hard shoulders. For our project we have designed road width of 7.5 m.

Footpaths:

Footpaths or walkways are generally provided where pedestrian traffic is anticipated, but not on major arteries or in country sides. Its width is 1.5 m generally, but may be as narrow as 0.6 m and as wide as 2.5 m depending on the requirements.

Road Kerb:

The road kerb is either surmountable type or insurmountable type. In the absence of walkways, a road kerb is combined with parapet.

Parapets:

Parapets can be of many shapes and of variable sturdiness. They are designed to prevent a fast-moving vehicle of a given mass from shooting off the roadway in the event of an accidental hit. Their height varies, but it should be at least 700 mm

Handrails:

The parapets are usually mounted by metal hand rail, about 350 mm high. Their roadside face is double sloped. For our project we have designed handrail of size 50 mm50 mm.

Crash Barriers:

Sometimes walkways are protected from the erring vehicular traffic by crash barriers which act as insurmountable kerbs and deflect the hitting vehicles back into the traffic lane.

Expansion and Roadway joints:

To provide for expansion and contraction, joints should be provided at the expansion end of spans, at other points, where they may be desirable. Joints are preferably sealed to prevent erosion and filling of debris.

Medians:

On expressways and freeways, the opposing traffic flows are separated by median strips. These reduce the possibility of accidents due to head on collisions.

Super elevation:

The super-elevation of the surface of a bridge on a horizontal curve is provided in accordance with the applicable standard. This should preferably not exceed 0.06 m per meter, and never exceed 0.08 m per meter.

4.3.2. <u>Analysis of Bridge Structures</u>

4.3.2.1. <u>Slab</u>

a. <u>Cantilever slab</u>

The outer face of the main beam is taken as the support of the cantilever slab. Live load considered as class A and class AA as per IRC 6 :2014 and dead load comprises of the selfweight of the cantilever slab itself, kerb and railings whose maximum response is calculated using effective width method at the junctions of the main beam. The slab is analyzed and designed for 1 m strip. In addition, distribution reinforcement has been provided assuming the summation of 0.3 times the maximum live load bending moment and 0.2 times the dead load bending moment in accordance with IRC 21: 1987 Clause 305.18.1. The temperature and shrinkage reinforcements are provided at the top of the deck slab as per codal provision.

In case of cantilever slab, the effective width of dispersion, measured parallel to the support edge, for concentrated loads on cantilever solid slab is to be obtained from equation:

be= 1.2x + bw

Where, be= effective width x = distance of the center of gravity of the concentrated load from the face of the cantilever support

bw= the breadth of the concentration area of the load, i.e., the dimension of the tyre or track contact area over the road surface of the slab in a direction parallel to the supporting edge of the cantilever plus twice the thickness of the wearing course over the structural slab

The effective width should be limited to one-third of the length of the cantilever slab.

b. <u>Restrained Slab</u>

The interior panel of the deck slab has been designed as a two – way slab. The maximum bending moment at the critical section is determined by deploying Pigeaud’s method, considering IRC Class A and class AA loading, confirming to IRC 6 – 2014 as a live load and the self-weight of the slab and wearing course as dead load. The moment due to the load has been computed using Pigeaud’s curves. Since the curves are intended for slab simply supported on all four sides, the values of maximum bending moment have been multiplied by a continuity factor of 0.8.

The RC slab is designed as rectangular SRURS by the principle of Limit State of Design Method of slab. The temperature and shrinkage reinforcements are provided at the top of the deck slab as per codal provision.

The method developed by Pigeaud is applicable to rectangular slabs supported freely on all four sides and subjected to a symmetrical placed concentrated load as shown in the fig below:

L = long span of length

B = Short span length

u,v = Dimensions of the load spread after allowing for dispersion through the deck slab

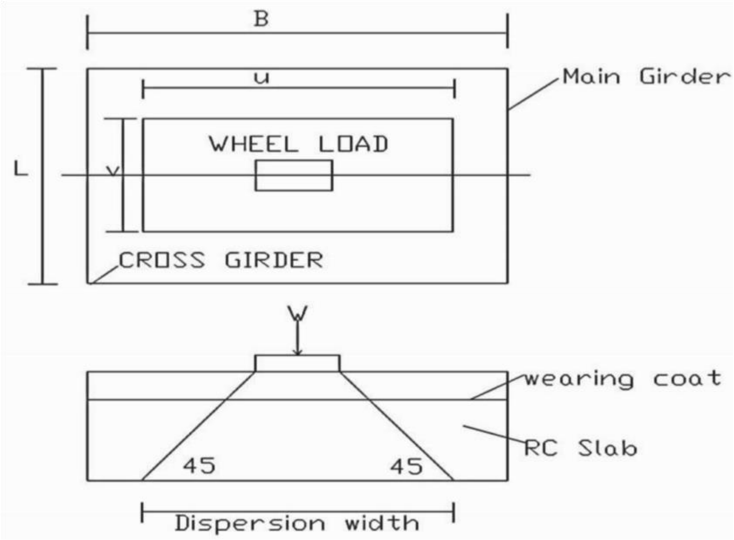


Fig4.2: Dispersion of load

K = ratio of short to long span = $\frac{B}{L}$

M_1 = Moment in the span direction

M_2 = Moment in the long span direction m_1 and m_2 = coefficient for moment along short and long direction μ = poison’s ratio for concrete generally assumed as 0.15

W = load from the wheel under construction

The dispersion of the load may be assumed to be at 45^0 through the wearing coat and deck slab according to IRC: 21-2000 code specifications. Consequently, the effect of contact wheel or track load in the direction of span shall be taken as equal to the dimension of the tyre contact area over the wearing surface of the slab inclusive of the thickness of the wearing surface. It is sometimes assumed to be at 45^0 through the wearing coat but at steeper angle through the deck slab.

The bending moment are computed as

$M_1 = (m_1 + \mu m_2)$ xW

$M_2 = (m_2 + \mu m_1)$ xW

The values of the moment coefficient m1 and m2 depend upon parameters u/L,v/L and K. Curve to compute moment coefficients of slabs completely loaded uniformly distributed load or dead load of slab for different values of K and 1/K is also given in textbook of V. Johnson.

4.3.2.2. <u>Main Girder</u>

Live load on the girder has been determined by Courbon's method assuming the supports of the deck slab as unyielding. Maximum bending moment and shear force for RCC beams due to dead and live loads have been determined at various sections of the RCC beams with the help of influence lines. The detailed calculations are presented in the design calculation.

Courbon's Method

Courbon's method is used for finding the lateral distribution of the load acting on the bridge. It helps to determine the portion of the total lead shared by each longitudinal girder. According to Courbon's method, the reaction Ri of the cross beam on any girder I of a typical bridge consisting of multiple parallel beams is computed assuming a linear variation of deflection in the transverse direction. The deflected will be maximum on the exterior girder on the side if eccentric load and minimum on the other exterior girder.

The reaction Ri is given by,

$$= \frac{PI_i}{\sum I_i} + [\frac{PI_i}{\sum I_i} \text{x} \frac{ed_i \sum I_i}{\sum I_i {d_i}^2}]$$

Where, P = Total live load

Ii = Moment of inertia of longitudinal girder

e = eccentricity of the live load (or C.G. of loads in case of multiple loads)

di = distance of girder I from the axis of the bridge

Influence Line Diagram

Usually the structures are analyses for loads which do not change their points of application on the structure. Very often structures have to be analyzed for a number of parallel moving loads which keep on changing their positions on the structures. In such cases the internal stresses in the structure at any given point, which depend on the positions of the loads, keep on varying as the loads keep up different positions on the structure. A typical instance is a bridge loaded with a number of moving vehicles, which are then said to constitute a train of wheel loads. In order to design such structures, it is not enough to analyses the structure for a given position of loads and calculates the stress resultants namely: Bending moments, radial and normal shear forces at any section in a member of the structure. The engineer must know the maximum values of stress resultants, both positive and negatives, at any section of the structure. Sometimes the designer would even like to know the maximum deflection at a given point, when a structure is subjected to moving loads. The maximum value of the resultants or the deflection at a given section could be found by taking a number of trail positions of the loads. Such a procedure apart from being time consuming is also uncertain. The task is very much simplified by using the concept of influence line. An influence line is a graph or curve showing the variation of any function such as reaction, bending moment, shearing force, deflection etc. at a given point of a structure, as a unit load parallel to a given direction, crosses the structure. An influence line is thus a relation between the required function at any given point of the structure and the position of a moving unit load on the structure. Influence line diagram for a stress is the one in which ordinate represent the value of the stress resultant for the position of unit load at the corresponding abscissa.

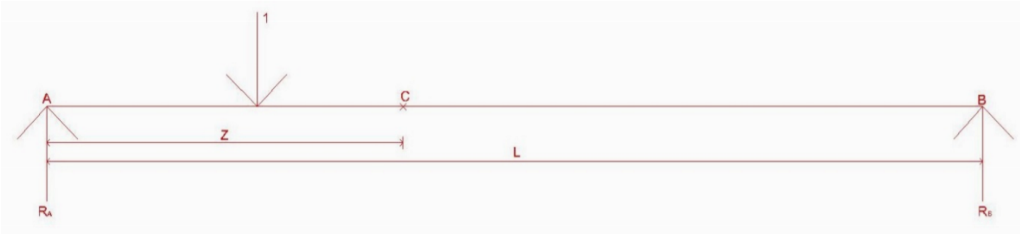


Fig4.3: Beam with unit load

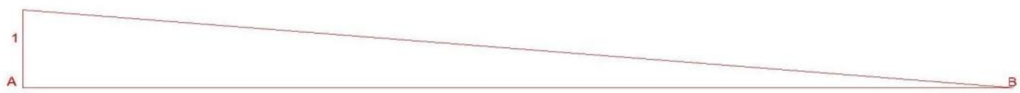


Fig4.4: ILD for R_A

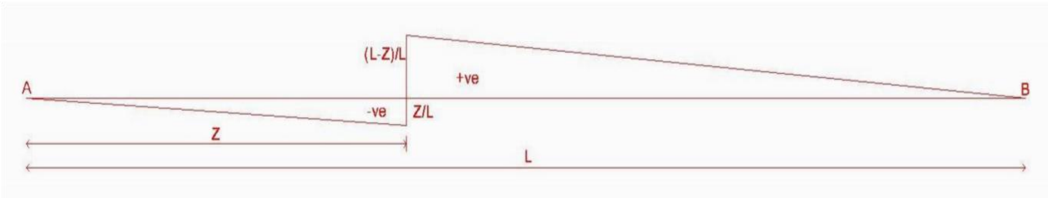


Fig4.5: ILD for V_C

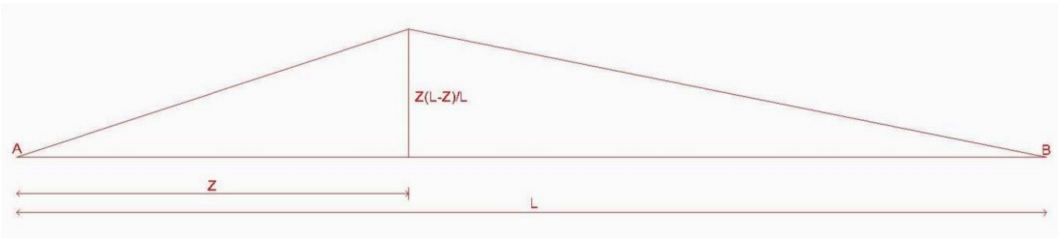


Fig4.6: ILD for M_C

4.3.2.3. <u>Cross Girder</u>

Dead load bending moment in the cross girder has been calculated considering trapezoidal distribution of weights of slab and the wearing coat. Cross girder has been considered as partially fixed beams over two supports. Live load bending moment in the cross girder has been computed considering different disposition of standard loadings over the cross beam as per IRC 6 -2014. Similar computation procedure has been used to determine the shear forces.

4.4. <u>Substructure</u>

Substructure of a bridge refers to that part of it which supports the structure that carries the roadway or the superstructure. Thus, substructure covers pier and abutment bodies together with their foundations, and also the arrangements above the piers and abutments through which the superstructure bears on the structure. The latter are called bearings

4.4.1.1. Foundation

A foundation is that part of the structure which is in direct contact with the ground and transmits loads to it. A footing is that part of the foundation that transmits the loads directly to the soil.

4.4.1.2. Deep Foundations

Deep foundations generally have depth greater than the width. They are constructed by various special means. They are of following types:

Caissons or wells

Caisson is constructed at open surface level in portions and sunk downwards mechanically by excavating soil from within the dredge hole all the way till its cutting edge reaches the desired founding level. The well is then effectively scaled at bottom and at least partly filled by sand. The surface level and the portions near it are capped. The pier or abutment is then constructed on the cap.

4.4.1.3. Shallow Foundations

A foundation is shallow if its depth is less than or equal to its width. These are generally placed after open exaction, and are called open foundations. The design of open foundations is based on complete subsoil investigations. But in case of low safe bearing capacity of soil, such foundations have to be disallowed. The selection of the appropriate type of open foundation normally depends upon the magnitude and disposition of structural loads, requirements of structures (settlement characteristics, etc.), type of soil or rock encountered, allowable bearing pressures, etc. Where rocky stratum is encountered at shallow depths, it may be preferable to adopt open foundations because of its advantage in permitting proper seating over rock and speed of construction work. They are of following types:

a. Spread Footing (Isolated footing, combined footing, strap footing)

An isolated footing is a type of shallow foundation used to transmit the load of an isolated column to the subsoil. This is the most common type of foundation. The base of the column is enlarged or spread to provide individual support for the load. A spread footing which supports two or more columns is termed as combined footing. The combined footing may be rectangular in shape if both the columns carry equal loads, or may be trapezoidal if they carry unequal loads. If the independent spread footings of two columns are connected by a beam, it is called strap footing. A strap footing may be used where the distance between the columns is so great that a combined trapezoidal footing becomes quite narrow. The strap footing consists of single continuous R.C. slab as foundation of two or three or more columns in a row. It is suitable at locations liable to earthquake activities. It also prevents differential settlement. In order to have better stability a deeper beam is constructed in between the columns. It is also known as continuous footing.

b. Mat or Raft Footing

A raft or mast is a footing that covers the entire area beneath a structure and supports all the walls and columns. When the allowable soil pressure is low or the loads are heavy the use of spread footings would cover more than one half of the area and it may prove more economical to use mat or raft foundation. The mat or raft tends to bridge over the erratic deposits and eliminates the differential settlement. It is also used to reduce settlement above highly compressive soils, by making the weight of structure and raft approximately to weight of soil excavated.

4.4.1.4. Bearing

Bearings are provided in bridges at the junction of the girders or slabs and the top of pier and abutments. Bearings transmit the load from the superstructure to substructure in such a way that the bearing stresses developed are within the safe permissible limits. The bearings also provide for small movements of the superstructure. The movements are induced due to various reasons such as:

Movement of the girders in the longitudinal direction due to variations in the temperature

a. The deflection of the girder causes rotations at the supports

b. Due to sinking of the supports the vertical movements are developed

c. Movements due to shrinkage and creep of concrete

d. In the case of pre-stressed girders, pre-stressing the girders cause movements of girders in the longitudinal direction.

Types of Bearings

a. Fixed Bearings

Fixed bearings permit rotations while preventing expansion. They are of the following types:

i. Steel Rocker bearing

ii. R.C. Hinge bearing

b. Expansion Bearings

Expansion bearings accommodate both horizontal movements and rotations, they are of following types:

i. Sliding Plate bearing

ii. Sliding cum Rocker bearing

iii. Steel Roller cum Rocker bearing

iv. R.C Rocker cum Roller bearing

v. Elastomeric bearing

Elastomeric Bearing

Elastomeric bearings are widely used in present times as they have less initial and maintenance cost. Besides occupying a smaller space, elastomeric bearings are easy to maintain and also to replace when damaged, chloroprene rubber termed as neoprene is the most commonly used type of elastomer in bridge bearings. Neoprene pad bearings are compact, weather resistant and flame resistant. Hence, nowadays elastomeric bearings have more or less completely replaced steel rocker and roller bearings.

4.4.1.5. <u>Abutment</u>

Abutments are end supports to the superstructure of a bridge. Abutments are generally built using solid stone, brick masonry or concrete. An abutment has three distinct structural components namely breast wall, wing wall and back wall. The design of abutment is done precisely in the same manner as the design of pier. The dimensions are first determined from the practical point of view and its stability is subsequently tested. The important additional force which the abutment has to withstand is the earth pressure of the earth filling behind the abutment. The minimum top width of the abutment should be 3 to 4 feet with the front batter of 1 in 24 and back batter of 1 in 6. Eddies erode the toes of the bank behind the abutment and thus the cost of maintenance of the road is increased. In order to overcome this defect and give the smooth entry and exit to the water, splayed wing walls to the abutment are constructed.

<u>Function</u>

a. To finish up the bridge and retain the earth filling

b. To transmit the reaction of the superstructure to the foundation.

<u>Design</u>

Height: Height is kept equal to that of piers.

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

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

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

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

<u>Piles</u>

Piles are essentially giant-sized nails that are driven into the subsoil or are placed in after boring holes in the subsoil. The giant-sized nails that are driven into the subsoil or are placed in after boring holes in the subsoil. The giant-sized nails are made of concrete, steel or timber and can be square, rectangular, circular or H-shaped in section. A group of piles is capped together at top, usually by a reinforced concrete cap, to support the pier of crapped together at top, usually by a reinforced concrete cap, to support the pier or abutment body above.

<u>Forces acting on abutment</u>

a. Dead load due to superstructure

b. Live load due to superstructure

c. Self-weight of the abutment

d. Longitudinal force due to tractive effort and braking

e. Forces due to temperature variation

f. Earth pressure due to backfill

Abutment should be designed in such a way that it can resist the forces mentioned above.

5. HYDROLOGY

5.1. Catchment Area Analysis

Catchment Area calculated from arc gis = 5km2

5.2. Hydro-Meteorological Data

There was no gauging station in Andheri Khola. The stream flow data for this river were not available. Hence the estimation of the flow for this river had to be determined from other approaches. To determine Design discharge, following methods are used.

5.3. Design Discharge Calculation

To calculate design discharge, we apply various methods, and among them, suitable discharge was chosen. In all methods, we choose Return Period: 100 Years .The design discharge can be calculated by using various methods as given below:

5.3.1. Modified dickens method

In this method, the T year flood discharge QT, in m3/sec, is determined by

$$Q_T = C * A^{3/4}$$

$$C_T = 2.342 \log(0.6T) \log\left(\frac{1185}{p}\right) + 4$$

$$=2.342\log(0.6x100)\log\left(\frac{1185}{120}\right) + 4$$

=8.3

Where,

p=100(A_s+6)/A =100(0+6)/5= 120

$$Q_T = 11.123x5^{3/4}$$

=27.75m^3/sec.

5.3.2. WECS

For hydrological calculation we used WECS method.

Since, this method is based on series of regression equation that are derived from analysis of all the hydrological records from Nepal.

Return period	α	β
2	1.8767	0.8783
100	14.630	0.7342

Table5.1:Coeff. Relation with time period

Qa $=\alpha(\text{area below } 3000+1)^{\beta}$m3 $/sec$

Now,

For 100 years return period ;

Q100 = $14.630(1+5)^{0.7342}$

= 54.542m3$/sec$

5.3.3. <u>Rational method:</u>

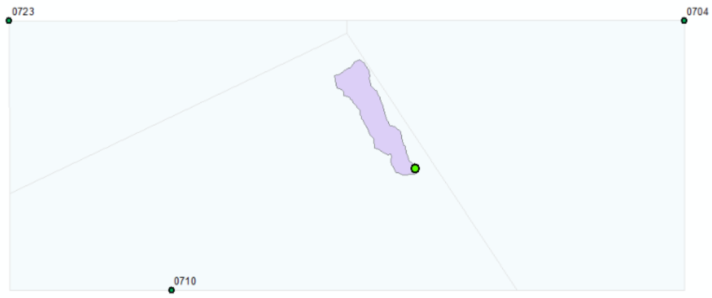


Fig5.1: Catchment area

$Q_p = \frac{CiA}{360}$ for $t \geq t_c$

$Tc = 0.01947\ L^{0.77} S^{-0.385}$

L= longest flow path

S=slope

T_c=time of concentration

L=5523m

S=slope=$\frac{\Delta H}{L} = \frac{400-200}{5523} = 0.036$

Tc=0.01947 x(5523)0.77x(0.036)-0.385

= 53.29min

=0.88hr

100 years rainfall (R_{100}) = 508.028mm

For rainfall intensity,

$I = \frac{R}{24} * (\frac{24}{Tc})$^(2/3)

$= \frac{508.028}{24} * \left(\frac{24}{0.88}\right)$^(2/3)

I=191.79 mm/hr

$Q_p = \frac{0.2*191.19*458}{360} = 48.647 m^3/\text{sec}$

Here the value of $R=R_{100}$ is calculated from Gumble method below:

GUMBLE'S EXTREME VALUE METHOD:

In this method different annual maximum rainfall patterns are used for the calculation for of 50 or 100 years return period for bridge design. Using the given extreme rainfall of meteorological station near bridge rainfall is calculated.

DUMKIBAS

Year	Extr.	Month	Day
1983	117	7	26
1984	245.3	6	17
1985	200.3	9	3
1986	275.1	8	27
1987	135.1	9	3
1988	100.3	9	6
1989	50.3	7	17
1990	329	8	27
1991	28	1	2
1993	398	9	6
1994	173	9	10
1995	188.6	6	20
1996	286	8	13
1997	114.6	7	9
1998	237	8	4
1999	310	7	11
2000	171	5	25
2001	252	7	17
2002	168	8	10
2003	393.6	8	20
2004	112.5	7	8
2005	200	8	19
2006	207.5	9	12
2007	343.7	9	30
2008	188	6	13
2009	210	8	4
2010	226.7	8	25
2011	163.2	7	31
2012	172.8	8	4
2013	266.6	6	17

Table5.2: Rainfall data Dumkibas rain gauge station

Using Cumfreq for calculation of R100 :

```
The cumulative frequency function is generalized logistic :

    Freq = 1/{1+exp(A*X^E+B)}
    The exponent E =  0.760
    A     =      -0.0805689
    B     =         4.58064

Average X:    209      Median X:    200      St.Dev. X:    91.2
```

Here below Xvalue = R100

Cum. Freq.	Ret. Per.		X-value
0.99	100	Go ->	508.028081

90% confidence limits of the X-Value

Lower	Upper
365.871354	512.723199

5.4. Linear waterway

By lacey'smethod ,

$W = c\sqrt{Q}$

$= 4.75\sqrt{48.647}$

= 33.13m (constant c varies from 4 to 6 for plain area And 2 to 3.5 for hill areas)

Discharge from field observation

From autocad,

Area of x section referring HFL a/c to local people

A=21.3158m2

Perimeter (p)=75.655m

Then,

Discharge (Q)=R2/3s1/2A/n

=(21.3158/75.655)2/3*(0.0349065)1/2*21.3158/0.05

=34.231m3/s

5.5. Design of x-section

Assume the design section is trapezoidal

Base of trapezoid =assumed linear waterway=35m

Side slope :0.5H:1V

Wetted perimeter(P)=b+2y√((1+z^2))

=20+2*y√((1+ 〖0.5〗 ^2))A=(B*y+zy2)

= (20*y+0.5*y2)

Now,

R = A/P=((20y+0.5y2))/(20+2.236y)

Q = 1/n AxR^(2/3) xs^(1/2)

n =0.05

S=0.035

Q = 48.647m3/sec

After ccalculation,

Y=0.783m

Adopt free board =0.5m

∴Total height y = 1.283m from bed level

AFFLUX

Area =15.967m2

Velocity=3.047m/s

Afflux =3.0472/(2*9.81)*(1/0.72-1)

=0.49>0.3

=0.3m

Total high flood level=0.783+afflux

=0.783+0.3

=1.083m

5.6. Scour depth and depth of foundation

Scour depth=1.34*(Db2/ksf)1/3

=1.34*(2.432/12.445)1/3

=1.045m

6. PRELIMINARY DESIGN

6.1. Design of Superstructure

1. Clear roadway = 7.5m for two lane carriage way
2. Three T-beams spaced at 3m
3. Clear span of bridge =19.65
4. Width of bearing = 0.32m
5. Effective span of bridge =20m (clear span, bearing on each side, Half of expansion joint)
6. Total span: Centre to Centre of bearing =20m
7. C/C spacing of cross beam = 5m
8. Number of cross beams per span = 5
9. Kerb height = 200mm
10. Railings:
    a. Cross section of railing post = 250mm x 250mm
    b. Height of rail post above deck = 1.2m
    c. Number of rail post = 13 @ 1.675m c/c
    d. Railing = 3nos-50mm dia. @250mm c/c
11. Slab:
    a. Thickness of deck = 250mm
    b. Thickness of wearing course = 75 mm
    c. Sidewalk width = 495 mm both side
12. Wearing coat = Take asphalt concrete for wearing coat of bridge. Thickness of wearing coat is 75 mm at crown of carriage way.

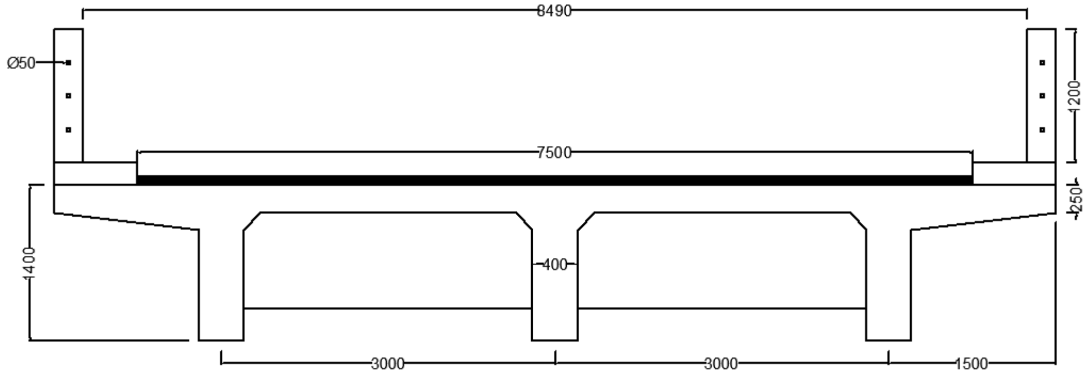


Fig6.1: Cross section of bridge deck

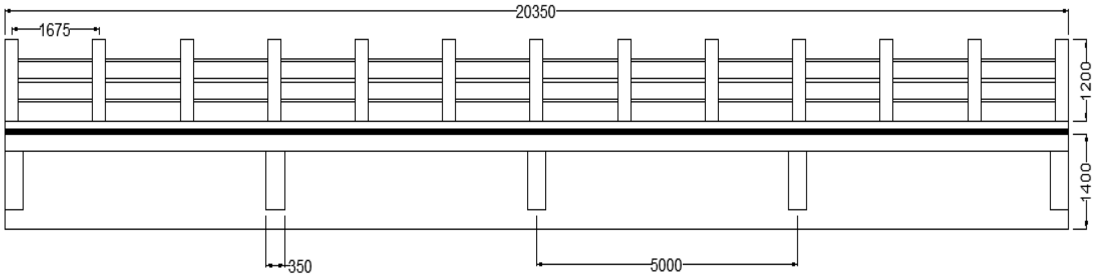


Fig6.2: Longitudinal section of bridge deck

6.2. Design of Substructure

6.2.1. Elastomeric pad Bearing

In geometrical design, approximate length, breadth and thickness of elastomeric pad and number, thickness and cover of steel laminates are found. Geometrical design is carried out using the guidelines of IRC.

[Refer standard plan dimensions of IRC 83, Part II, Cl. 916.2, Appendix I]

N_{min} = DL = 431.493 KN; Nmax = Tot. Vertical load on bearing = 893.92KN

; H = 18.47 KN

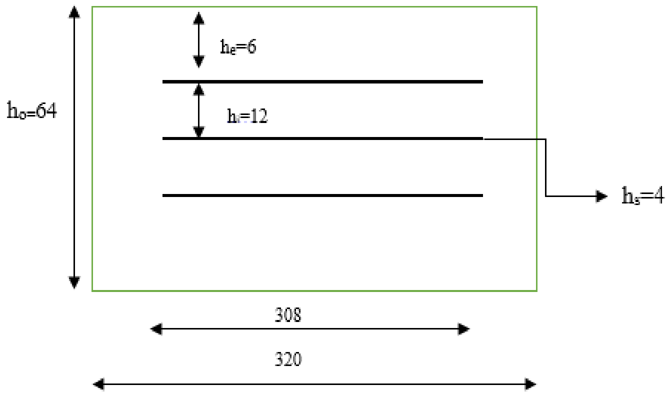


Fig6.3: Elastomeric pad bearing

b_0 = 320 mm, b = 308 mm

l_0 = 500 mm, l = 488 mm

h_i = 12 mm

$h_e = h_i/2$ = 6 mm

h_s = 4 mm

n = 3

c = 6 mm

$h_0 = (n+1)h_s + nh_i + 2h_e = 4*4+2*6+3*12=64mm$

$h=nh_i+2h_e=3*12+2*6=48mm$

Check geometry of bearing

$l_0 / b_0 \leq 2$ Ok

$h = 48 < b_0/5 = 320/5 = 64$ Ok

$> b_0/10 = 320/10 = 32$ Ok

$s = \frac{l*b}{2h_i(l+b)} = 7.867 > 6$ and < 12 <OK>

6.2.2. <u>Design of Abutment</u>

a. <u>Selection of Type of Abutment</u>

Abutment may be of masonry or reinforced cement concrete. Masonry is technically / economically feasible up to 5m height of abutment. In the particular case, abutment is greater than 5 m height. So reinforced concrete wall type abutment has been selected.

b. <u>Material Selection</u>

M20 grade of concrete for abutment stem

M25 grade of concrete for abutment cap

Fe 415 HYSD bars for all RC work

c. Seating Width

Minimum seating width = 305+2.5L+10H mm [Cl.219.9 IRC 6]

= 305+2.5x20+10x5.5 = 410 mm

Seating width $\geq$ Bearing width + 150 mm +Projection of cap + width of expansion joint Where,

Width of expansion joint $\geq 20x10^3 x0.000011/°C$ x 30 x$\frac{1}{2}$= 3.333 mm

$\geq 5x10^{-4} x20x10^3 x\frac{1}{2}$ = 5 mm

$\geq$ 20 mm

Adopting width of expansion joint = 40mm

Required seating width

= (320+150+75+40)

=585mm

Therefore seating width =585mm

d. Height of dirt wall

= depth of girder + Height of bearing – thickness of approach slab

= 1.4+0.064-0.3 = 1.164m

e. Thickness of dirt wall

$\geq$ 200 mm and $\frac{Height\ of\ dirt\ wall}{7}$= 166 mm

Adopting thickness of dirt wall = 200 mm = 0.2m

f. Width of stem of abutment

Width of stem =H/10 =550 mm

Width of stem $\geq$ thickness of dirt wall + seating width-projection

$\geq$ 0.2+0.585-0.075

$\geq$ 0.710 m

Adopting width of stem = 0.710m

g. Thickness of footing

= H/8 =687.5 mm

Therefore adopt = 800 mm

h. Width of footing

B$\approx$ 0.75 H = 0.75x5.5 = 4.125m

$$B \geq \frac{Area\ of\ footing}{Length\ of\ footing} \approx \frac{1.5xTotal\ vertical\ loads}{Allowable\ bearing\ capacity\ of\ soil\ x\ length\ of\ footing}$$

Adopting B = 4.5m

i. Thickness of Abutment cap

= 0.3 m

Minimum thickness of cap = 0.2m

j. Length of abutment

Length of abutment $\geq$ c/c distance between girders + length of Bearing + 2 x clearance

$\geq 6 + 0.5 + (2x0.4) = 7.3m$

Providing 7.3 m of abutment

6.2.3. Approach slab

Size of approach slab

= 4mx9mx0.3 m

7. STRUCTURAL ANALYSIS AND DESIGN OF BRIDGE COMPONENTS

A. Analysis and Design of Deck Slab

Here, in our case, there are two types of slab i.e. cantilever slab and restrained slab as per their support conditions. Cantilever slab lie outside the two main girders whereas the slab in between the main girders and cross-girders are restrained slabs.

7.1. Cantilever portion of slab

As the cantilever slabs on both the sides are identical in terms of their sizes, support conditions, material and loading, so, we need the design for the only one typical section. Effective width method is used for the design of cantilever slab.

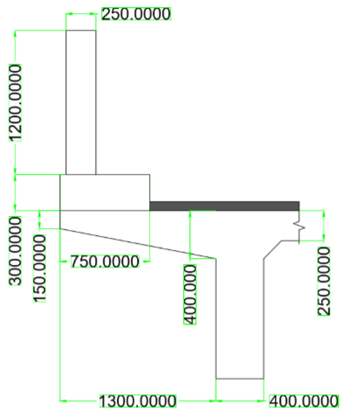


Fig7.1: Cantilever portion of slab

7.1.1. Calculation of dead load shear force and bending moment.

Description	Dead load (DL)/meter run	Lever arm (x)	B.M. =DL*x
Railing	0.25*1.2*0.25*13/20=1.219	1.3-0.05-0.25/2=1.125	1.371
Kerb	0.75*0.3*25=5.625	0.75/2+0.55=0.925	5.203
Slab (rectangular)	0.15*1.3*25=4.875	1.3/2=0.65	3.169
Slab (triangular)	½*0.1*1.3*25=1.625	1/3*1.3=0.433	0.704
Wearing coat	0.075*0.55*22=0.9075	0.55/2=0.275	0.25
∑	=16.689 KN		=10.697 KNm

Table7.1: Calculation for dead load,Sf,BM

7.1.2. <u>Calculation of live load bending moment</u>

For class 70R load, clearance between end of kerb and loaded wheel, it must be minimum of 1.2m. But in our case, clearance is only 0.550m from end of cantilever, we only apply class A load whose minimum clearance is 0.15m. SO placing a wheel 114KN axle at 0.15m from face of the kerb as shown in fig below.

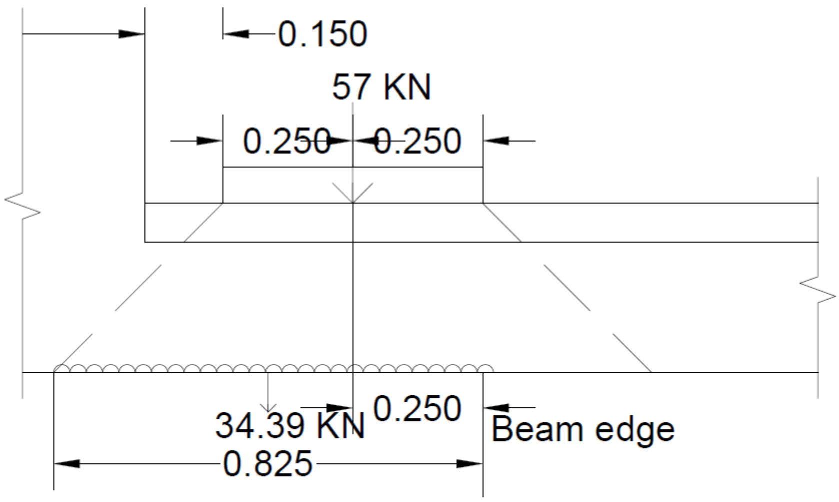


Fig7.2: Live Load on Cantilever Portion

Distance of c.g. of wheel from edge of the cantilever = 0.55 – (0.15 + 0.25)

= 0.15 m

Dispersed length of the load along span = 0.5+2*(0.4 + 0.075)

= 1.45 m

Out of this length, only 0.875 m (1.45/2 + 0.15) will be covered by load effect. Hence, the actual load effective on the cantilever portion is

=114/2 *0.875/1.45

=34.29 KN

Effective width of slab be is computed using,

$b_e = (1.2x + b_w) \leq l/3$ [Refer Cl. IRC 21 305.16.2 (2)(i)]

Where, be= effective width

x = distance of the center of gravity of the concentrated load from the face of the cantilever support

= 0.875/2

= 0.4375 m

b_w = 0.25 + 2*0.075

= 0.4 m

b_e = (1.2*0.4375 + 0.4)

= 0.925 m and l/3 =1.3/2 = 0.433 m

But according to code $b_e \leq l/3$

So b_e = 0.433 m

Impact factor = 4.5/(6+l)

= 0.616

But, the maximum value of impact factor is 0.5

Bending moment owing to live load = 1.5 * 35.93/0.433 *0.875/2

= 54.46 KNm

Design longitudinal bending moment at face of the main girder

= 54.46*1.5 + 11.752*1.35

= 97.555 KNm

Design shear = 16.689*1.5 + 1.5*35.93*1.35

= 97.79 KN

Design transverse bending moment in the direction of traffic

= 0.3*54.46*1.5 +0.2*11.752*1.35

= 27.68 KNm

Check of Slab Depth

Taking 25 mm of clear cover and 10 mm Ø primary reinforcement

Effective depth of primary reinforcement = 250 + 150 -25 -10/2 = 370 mm

$d_{bal} = \sqrt{\frac{Mu}{Q * B}} = \sqrt{\frac{97.555*10^2}{4.017*1000}}$ = 155.84 mm

$Q=0.36*f_{ck}*0.46(1-0.416*0.46) = 4.01$

Hence, $d_{bal} < d_{provided}$

Primary Reinforcement

$A_{st} = \sqrt{\frac{Mu}{0.87fy(d-0.416xu)}}$

$A_{st} = \frac{97.555*10^6}{0.87*500(370-0.416*\frac{0.87*500*Ast}{0.36*30*1000})}$

= 623.738 mm^2

A_{st}^{min} = 0.12% of bD

=0.12*1000*400/100

=480 mm^2

Hence, A_{st} provided > A_{st}

Spacing $=\frac{\pi d^2}{4Ast}*1000$ = 125.92 mm

Use 10 mm Ø bar @ 125 mm c/c

Transverse Reinforcement

$A_{st} = \sqrt{\frac{Mu}{0.87fy(d-0.416xu)}}$

$A_{st} = \frac{27.68*10^6}{0.87*500(370-0.416*\frac{0.87*500*Ast}{0.36*30*1000})}$

$=173.339 \text{ mm}^2 < A_{st}^{min}$

Provide $A_{st} = A_{st}^{min} = 480 \text{ mm}^2$

Spacing $=\frac{\pi d^2}{4Ast}*1000 = 163.624$ mm

Use 10 mm Ø bar @ 160 mm c/c

Temperature Reinforcement

According to provision stated in IRC 21-2000 cl. 305.17.4, for cantilever slab, minimum reinforcement of 4 no.s of 16 mm Ø HYSD bars shall be provided parallel to the free edge @ 150 mm spacing at the tip divided equally between the top and bottom surfaces.

Astmin = $\frac{4*\pi/4*16^2}{2}$ = 402.2 mm2

Hence, 10mm Ø bar @ 300 mm c/c spacing is provided per m width as temperature reinforcement in both the direction of slab at the bottom.

Spacing = $\frac{1000*\pi/4*10^2}{402.2}$ = 195.27 mm

Ast provided = $\frac{\pi*10^2}{4*190}$*1000 =413.367 mm2

Hence, Ast provided > Ast reqd

Check for Shear

No need to check shear for cantilever slab.

Reinforcement	Design Moment kN-m	Area of steel reqd mm^2	Area of steel provided		spacing reqd mm	spacing provided mm
			Dia (mm)	Area (m2)		
primary	97.555	623.738	10	628.319	125.92	125
transverse	27.68	173.339	10	173.339	163.624	160
temperature		402.2	10	413.367		300

7.2. Restrained slab

Restrained slab in the slab which is supported on all the four sides and its ends are not free to rotate. In our case, the beams supporting the slab undergo large deformation due to very long span and heavy moving load. So, it behaves like a flexible beam. Hence, we have used Pigeaud's method for the analysis of restrained slab.

7.2.1. Analysis of Restrained Slab

Maximum bending moment due to dead load

Effective span=clear span (cl-305-4-3 IRC 21)

Effective span in traverse direction =3-0.4 =2.6m

In longitudinal direction=5-0.35 =4.65

7.2.1.1. Calculation of dead load

Weight of deck slab = 0.25*25=6.25 KN/m^2

Weight of wearing course =0.075*22=1.65 KN/m^2

Total weight =7.9 KN/m^2

Total dead load on panel = total weight x area of panel= 7.9 x 2.9x4.65 +(2.616(fillet)=98.126 kN

Since the slab is supported on all four sides and is continuous, Pigeaud's curve will be used to get influence coefficients to compute moments. Ratio K =short span / Long span k= 2.6/4.65 =0.559

,$\frac{1}{k} = 1.78$

From Pigeaud's curve, for K = 0.559, m_1 = 0.047

for $\frac{1}{K}$= 1.78 , m_2 = 0.018

Maximum bending moment along short span (M_1) = (m_1+μ m_2)xP

= (0.047+0.15*0.018) x 98.126

= 4.86 KNm

Maximum bending moment along longer span (M2) = (m2+μ m1)xPx0.8

= (0.018+0.047*0.15) x 98.126

= 2.455 KNm

7.2.1.2. Live load B.M. due to IRC Class A wheel vehicle

U = (0.5+2*0.075)=0.65

V = (0.25+2*0.075) = 0.4m

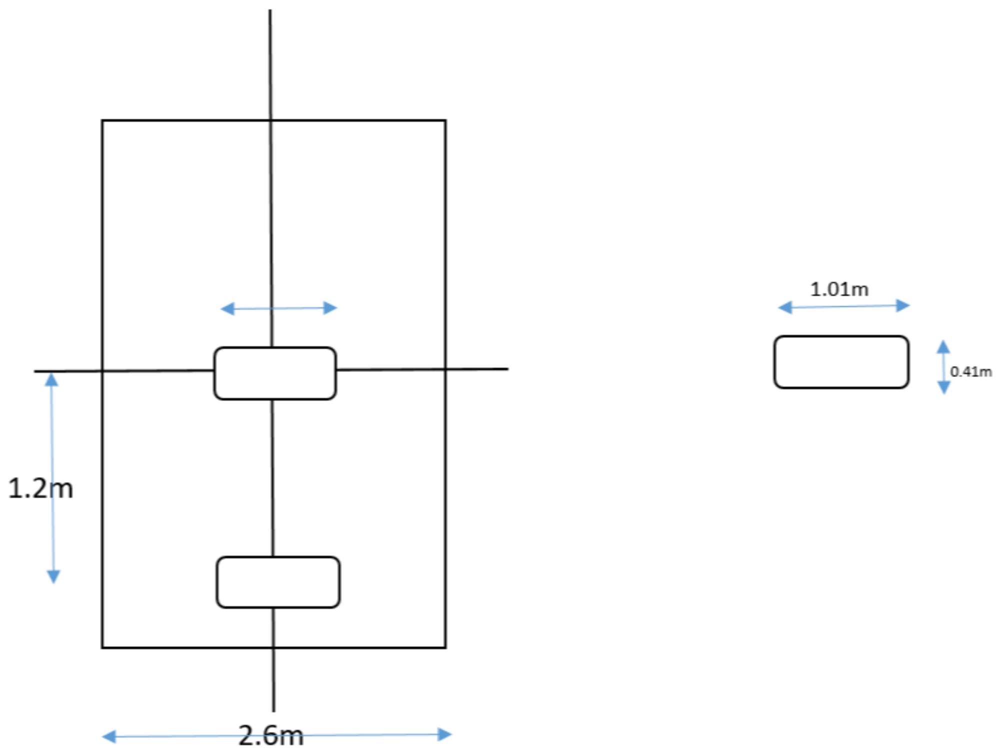


Fig7.3:Arrangement of wheel load for class A loading

For wheel I

U/B = 0.65/2.6 =0.25

V/L = 0.4/4.65 =0.086

From pigeaud's curve for K=0.559 ~ 0.6

M_1 = 0.1959

M_2 = 0.1641

μ = 0.15

Load=57*I.F = 57*1.173

=66.69 KN

BM in short span = (m1+μ m2) xW*0.8

=(0.195+0.15*0.164)*66.69*0.8

=11.71 KN

BM in long span =(m2+μ m1)*W*0.8

=(0.164+0.15*0.195)*66.69*0.8

=10.31 KN

For Wheel II

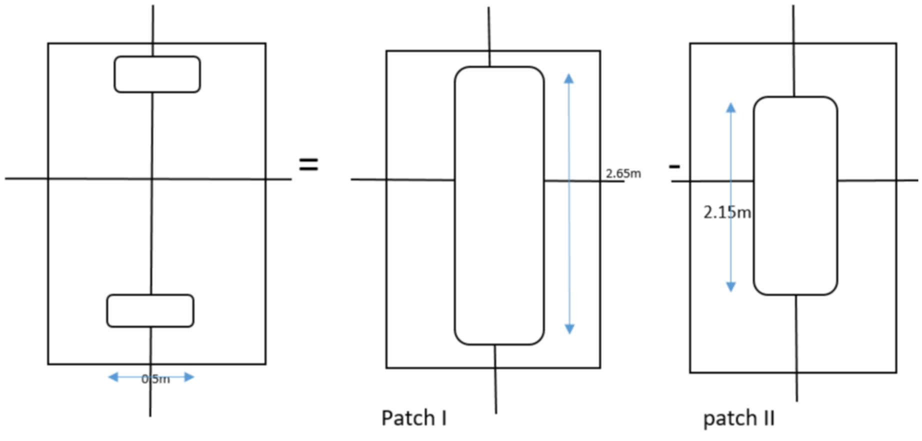


Fig 7.4:Arrangemt of wheel load for wheel ii for class A loading

Patch I

U/B =(0.5+2*0.075)/2.6 = 0.25

V/L =(2.65+2*0.075)/4.65 =0.602

Patch II

U/B= (0.5 +2*0.075)/2.6 =0.25

V/L=(2.15+2*0.075)/4.65 = 0.494

K= 0.559 ≈ 0.6

For Patch I

M_1=0.142

M_2 = 0.042

BM at short span =(0.142+0.15*0.042)*(2.65/0.25)*57*0.8*1.17

= 83.868 KNm

BM long span=(0.042+0.15*0.142)*(2.65/0.25)*57*0.8*1.17

=35.798 KNm

For Patch II

M1=0.152

M2 =0.049

BM at short span = (0.152+0.15*0.049)*(2.15/0.25)*57*0.8*1.17

=73.11 KNm

BM at long span = (0.049+0.15*0.152)*(2.15/0.25)*57*0.8*1.17

=32.943 KNm

Total for wheel II:

For short span =(83.868-73.11)/2 =5.379 KNm

For long span =(35.798-32.943)/2=1.653KNm

Total BM short span =(11.71+5.379)=17.089 KNm

Total BM long span =(10.31+1.653)=11.963 KNm

Overall BM due to live and dead load

Short span = $(BM)_{LL}+(BM)_{DL}$

=17.089 +4.801

=21.95KNm

Long span =$(BM)_{LL}+(BM)_{DL}$

= 2.455 + 11.963

= 14.418 KNm

7.2.1.3. <u>Calculation of BM due to LL by IRC 70R wheel loading</u>

Impact factor (IF)= (4.5/(6+20)) = 0.173

For wheel I

U=(0.86+2*0.075) = 1.01

V=(0.26 +2*0.075)=0.41

(U/B) = 1.01/3 = 0.336

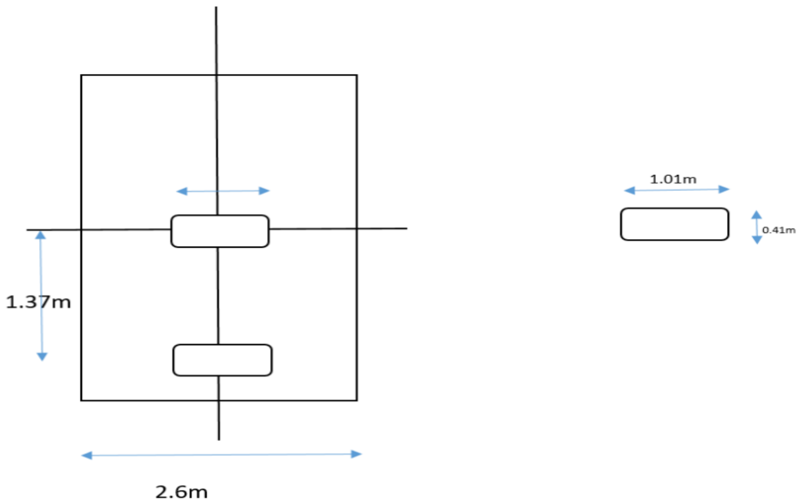


Fig 7.5:arrangement of wheel load for 70R loading for BM

(V/L) = 0.41/5 =0.082

K= B/L =3/5 =0.6

From pigeaud's curve

K= 0.6, m1=0.198 , m2=0.139

W= (170/2)*IF

BM in short span(MB) =(m1+μ m2) * W

Due to continuous slab design = (MB)*0.8

M_B= (0.198+0.15*0.139)*85*0.173*0.8

=17.4594 KNm

BM at long span

ML = (0.139+0.15*0.198)*85*1.173*0.8

=13.4562 KNm

For wheel II

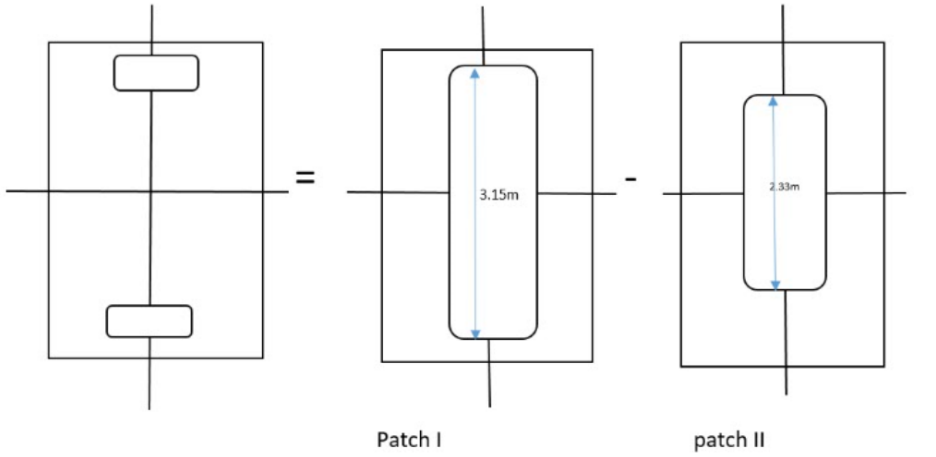


Fig 7.6: Arrangement of wheel ii.for 70R loading for BM

Patch I	patch II
U/B = (1.01/3) =0.3367	U/B =1.01/3 = 0.3367
V/L = (3.15/5) = 0.63	V/L =2.33/5 =0.466
K=0.6	K = 0.6
M_1=0.198	M_1=0.198
M_2=0.042	M_2=0.055

For patch I

BM at short span

M_B= (0.198 +0.15*0.042)85*3.15*1.173*0.8/0.41

=125.1993KNm

BM at long span

M_L=(0.042 + 0.15*0.198)*85*3.15*1.173*0.8

=43.9393KNm

For patch II

M_B =(0.198 ++0.15*0.055)*85*2.33*1.173*0.8/0.41

=93.4917 KNm

M_L=(0.055+0.15*0.198)*85*2.33*1.173*0.8/0.41

=38.394KNm

Actual

M_B=(125.1993-93.4917)/2

=15.8538 KNm

M_L=(43.9393-38.394)/2

=2.7727KNm

Total BM due to both wheel

M_B=17.4594+15.8538

=33.312 KNm

M_L=13.4562+2.7727

=16.2289KNm

Total BM due DD and LL

M_B=33.3132+4.801 =38.1142 KNm

M_L=16.2289 + 2.455 =18.6839 KNm

7.2.1.4. <u>Calculation of shear force</u>

a. <u>Due to dead load</u>

DL of slab and wc =(0.25*25+0.075*22)=7.9 KN/m^2

From code ,

25% of weight of fillet is taken for shear calculation

Wt. of fillet=0.5*2*0.15*0.15*4.65*25

=2.615 KN

Max SF at support due to DL = (0.25*2.165/2)+(7.9*1*2.6/2)

=10.596 KN

b. <u>SF due to live load of class A loading</u>

Location of load for maximum SF

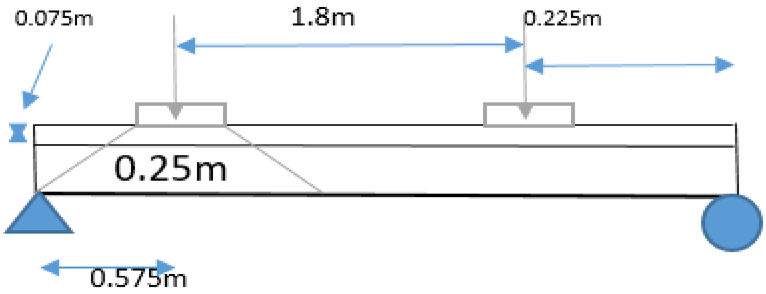


Fig7.7: Load arrangement for SF due to class A

Depression length for wheel= 0.5 + 2*(0.075+0.25)

=1.15m

For wheel I

Effective width of the slab,

$b_{ef} = a.\alpha.\left(1 - \frac{a}{l}\right) + b_1$

We have, $\frac{longer\ span}{shorted\ span} = \frac{4.65}{2.6} = 1.788$

From table of IRC 112:2011, B3.2, α = 2.592($for\ continious\ slab$)

a= the distance of center of gravity of the concentrated load from the nearer support

= 0.575m

b_1= 0.25+2x0.075= 0.4m

$$\therefore\ b_{ef}1 = 2.592x0.575x\left(1 - \frac{0.575}{2.6}\right) + 0.4$$

=1.56m

Since the b_{ef} of individual wheel overlaps, find modified effective width.

$b_{ef}{}^{IMod}$= 1.2+0.78+0.78 = 2.76 m

(Width of beam is taken into account for effective width calculation)

Load due to wheel I (F_I) =$\frac{2x57xI_f x\gamma_f}{b_{ef(mod)}} = 2x57x\frac{1.4}{2.275} = 48.32KN/m$

For wheel II

a= the distance of center of gravity of the concentrated load from the nearer support = 0.225

l= the effective span = 2.6m,

b_1= 0.25+2x0.075 = 0.4m

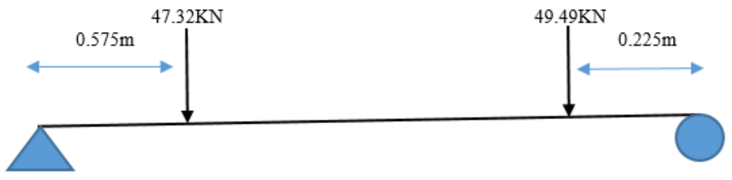
$b_{ef} = 2.592x0.225x\left(1 - \frac{0.225}{2.6}\right) + 0.4 = 0.932$m

Since b_{ef} doesn't overlaps

Beff. =0.932*2

=1.864 m

Load due to wheel I (F_{II}) =$\frac{2x57xI_f x\gamma_f x110/500}{b_{ef(mod)}} = 2x57x\frac{1.17}{1.864}$ =71.155 KN/m



Reaction at A and B

R_A = 55.89 KN

R_B = 41.916 KN

Max SF due to LL of class A wheel load = 55.89*1.5

=83.835KN

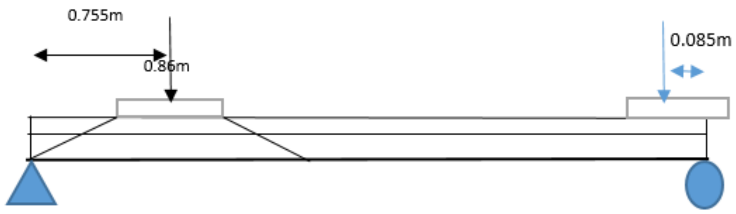
Total SF =$(SF)_{LL}$ +$(SF)_{DL}$

=75.68*1.5 +10.596

=98.1396KN

c. <u>Shear force due to 70 R wheel loading</u>

Location of wheel load for maximum loading



Dispersion length = 0.86 + 2*(0.075 + 0.25)

= 1.51

a=0.755m

For wheel I

$b_{eff}= kx * \left(1 - \frac{x}{L}\right) + bw$

L=2.6

bw=0.4

K=2.96

Hence ,

b_{eff}=1.793m> 1.37 (overlaps)

modified b_{eff} = 1.793 + 2.13 + 1.793

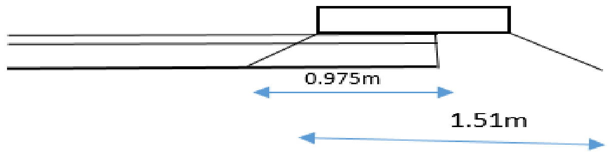
= 5.293 m

Impact factor (IF) = 1.117

Total load due to wheel I = (230/5.293)*1.17

=50.84 KN

Load due to wheel II

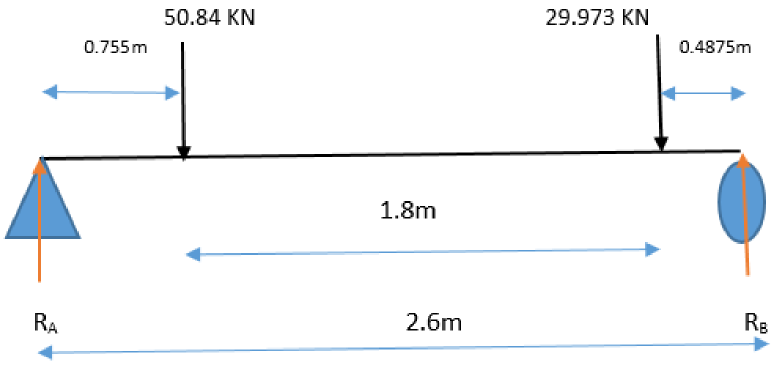


Load acting = (0.975/1.51)*210

=135.596 KN

So load due to wheel II = (135.596/5.293)*1.17

= 29.973 KN



R_A=36.077 KN = 54.116 KN (factored)

R_B=44.736 KN = 67.104 KN (factored)

Total SF maximum = 67.104 + 14.305 (dead SF)

=81.409KN

7.2.2. <u>Design of restrained slab</u>

(BM)short span (M_B)= 38.1142 KNm

(BM)long span (M_L)=18.6839 KNm

Check depth of slab

Let us provide clear cover of 40mm and dia of 10mm.

Taking M30 Fe500

d = D – CC – ø/2 = 250 – 40 – 10/2 = 205 mm

$d_{bal} = \sqrt{\frac{M_u}{Q \times b}} = \sqrt{\frac{38.1142\times10^6}{4.183\times1000}} =$ 95.45mm <205 ok

Where, Q = 0.36 fck × 0.48 × (1 – 0.42 × 0.46) = 4.183

$d_{prov} > d_{bal}$ hence, assumed depth is sufficient.

Since $d_{prov} > d_{bal}$, section is designed as Singly Reinforced Under-Reinforced Section (SRURS). Section design has been carried out by using IS 456:2000.

Area of reinforcement

$$= \frac{M}{0.87\times fy\times(d-0.42xu)}$$

$$= 38.1142 * \frac{10^6}{0.87 * 500 * (205 - 0.42 * 0.48 * 205)}$$

Ast = 535.33mm2

% of steel =0.26%

Spacing of reinforcement $= \frac{\pi x\frac{10^2}{4}\times1000}{535.33}$ =146.713mm

Providing 10mm diameter bar @140mm c/c spacing.

Area provided $= \frac{\pi x10^2}{140*4} x1000$=560.99 mm2

Similarly for long span

Area of reinforcement

$$= \frac{M}{0.87\times fy\times(d-0.42xu)}$$

$$= 18.68 * \frac{10^6}{0.87 * 500 * (205 - 0.42 * 0.48 * 205)}$$

Ast = 262.37mm2

% of steel =0.127% <0.12

Spacing of reinforcement $= \frac{\pi x\frac{10^2}{4}\times1000}{262.37}$ =299.34mm

Providing 10mm diameter bar @295mm c/c spacing.

<u>Check for Shear</u>

Maximum shear force = 98.139 KN

At mid span,

$V_{rd.c}$ = [0.12*k* 80*ρ*f_{ck})$^{0.33}$] *b_w*d

K = 1 + $\sqrt{200/d} \leq 2$

= 1.75 < 2

$\rho = \frac{Ast}{b*d}$

$\rho = \frac{560.99}{1000*355}$

=0.00158

$V_{rd.c}$ = [0.12*1.75*(80*0.00158*30)$^{0.33}$]*(1000*355)

= 115.73KN < 98.139 KN

Hence no shear reinforcement is required.

Design Summary:

SN	Description	BM(KNm)		SF(KN)
		Short span	Long span	
1	Dead load	4.86	2.455	10.596
2	Class A(W) live load	21.95	14.418	**98.1396**
3	70R(W)	**38.1142**	18.6839	81.409

SN	Description	Size of ϕ	Spacing
1	Short span	**10mm**	**140mm C/C**
2	long span	**10mm**	**295mm C/C**

7.3. Design of Longitudinal girder:

Bridge deck consist of three main girder and five cross girder with rigidly connected deck slab. Here one main girder and two cross girder have been analyzed and design.

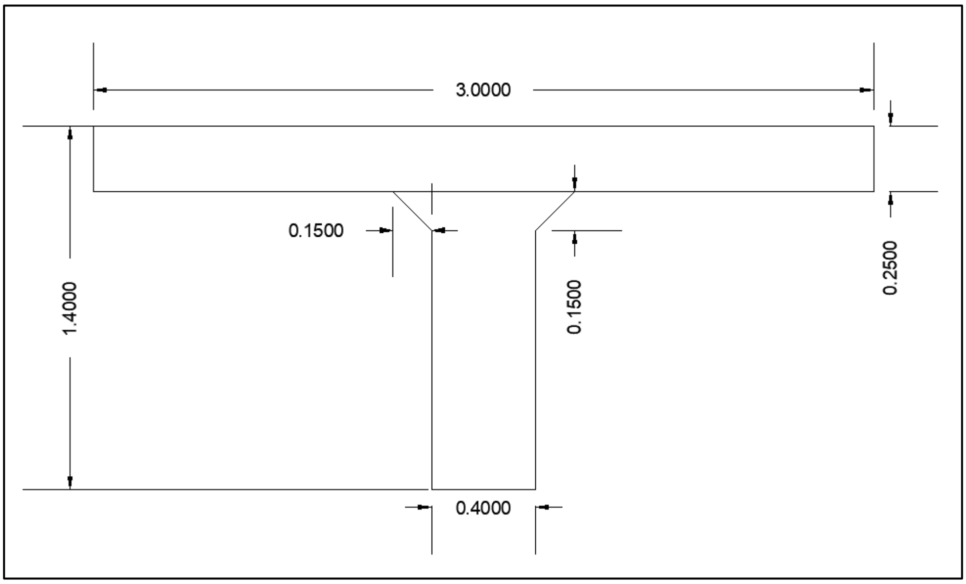


Fig7.8: Cross section of T beam (All dimension are in m)

7.3.1. Calculation of dead load on a main girder per running meter of span

Weight of wearing coat = 0.075*7.5*22 = 12.375 KN/m

Weight of railing = 2*1.219 = 2.438 KN/m

Weight of kerb =2*5.625 = 11.25 KN/m

Weight of slab = (4.0625 + 4.875)*2 + 0.25*6.4*25 = 57.875 KN/m

Weight of Fillet= 4*1/2*0.15*0.15*25 =1.125 KN/m

Weight of girder = 3*1.15*0.4*25 = 34.5 KN/m

Total weight of bridge = 119.563 KN/m

Dead load per meter per girder= 119.563/3 = 39.854 KN/m

Self-wt. of cross girder on a girder = ½*0.865*0.35*(3-0.4)*25 =9.839 KN/m

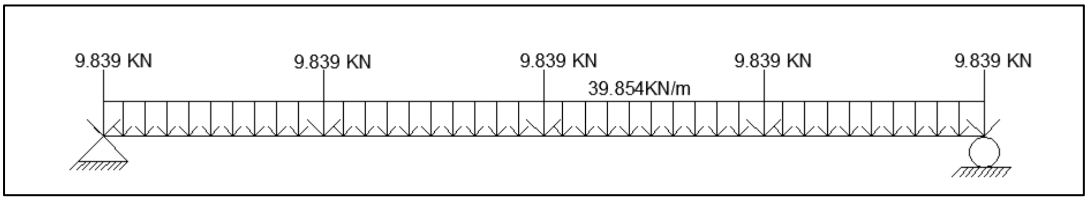


Fig7.9: Dead load on main girder.

7.3.2. Calculation of max BM and SF at critical section

Reaction at support (Ra) = (9.839*5 + 39.854*20)/2 = 423.14 KN/m

Shear force due to dead load

At support= 423.14 - 9.839= 413.301 KN

At L/2 = 0 KN

BM due to dead load

At mid span (L/2) = Ra*10 -9.839*10 - 9.839*5 - 39.854*10*10/2 = 2001.115 KNm

At quarter span (L/4) = 423.14*5 - 9.839*5 - 39.854*5*5/2 =1568.33 KNm

At L/8 = 423.14*2.5 - 9.839*2.5 - 39.854*2.5*2.5/2 = 908.61 KNm

At 3L/8 = 423*7.5 -9.839*7.5 - 9.839*2.5 - 39.854*7.5*7.5/2 = 1954.27 KNm

Distribution of live load on longitudinal girder for bending moment

IRC class 70R - tracked loading

Reaction on the girder will be maximum when eccentricity is maximum. Eccentricity will be maximum when the loads are very near to the kerb. Position of the loads for maximum eccentricity is shown in each case below.

(All girder are assumed to have same MOI)

Distribution coefficient (k)

$$K = \frac{Load\ carried\ by\ any\ beam}{average\ load\ per\ beam}$$

$$= \frac{\frac{w}{n}\left(1+\frac{n*e*x}{\in x\hat{}2}\right)}{\frac{w}{n}}$$

$$= \left(1 + \frac{n*e*x}{\in x\hat{}2}\right)\text{*avg load per beam}$$

Where,

e = max eccentricity for any load from center line

x = distance from center line to the required beam

n = number of girder

∑x2 = sum of square of distance of girder from center line

For Class A loading

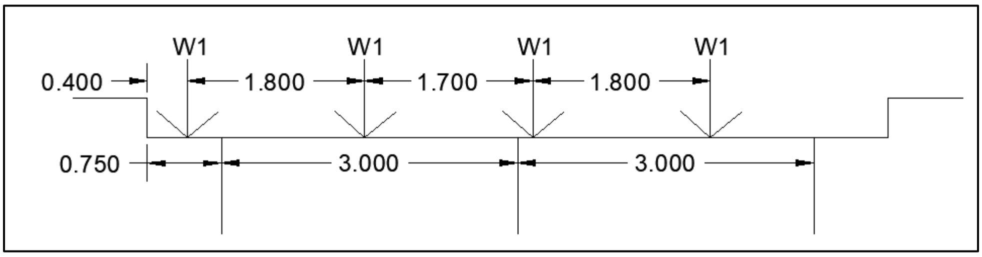


Fig7.10: Class A loading for maximum eccentricity

Reaction for girder A = $\left(1+\frac{n*e*x}{\in x\text{^}2}\right)*\frac{Wt}{3}$

= 1.8 W1

where,

n = 3

x = 3m

∑x2 = 32 + 32 + 0 = 18m2

e = 3.15 - (0.4 + 1.8 + 1.7/2) = 0.7m

Reaction for inner girder = 4/3 *W1 (1+0)

= 4/3 W1

<u>For class 70R (W)</u>

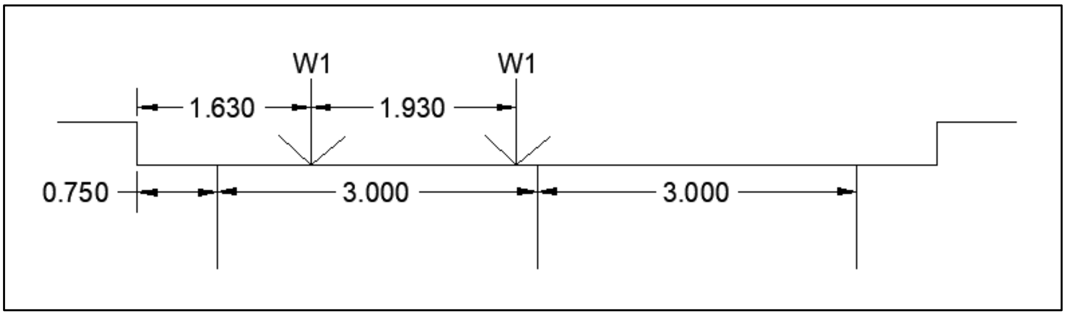


Fig7.11: Class 70R wheel loading for maximum eccentricity

Reaction factor for A = 2/3 *W1*(1 + $\frac{3*1.155*3}{3^2+3^2}$) = 1.052 W1

Reaction factor for B = 2/3 *W1

<u>For class 70R (T)</u>

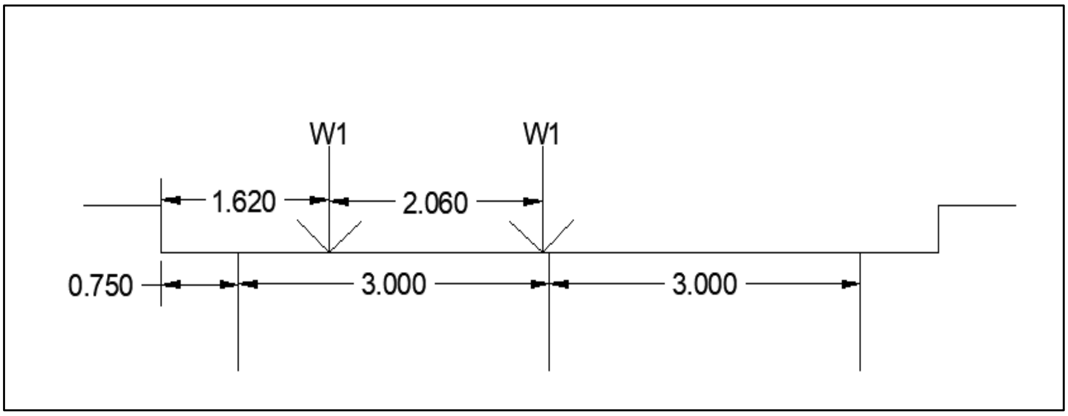


Fig7.12: Class 70R tacked loading for maximum eccentricity

Reaction factor for A = 2/3 *W1*(1 +$\frac{3*1.1*3}{3^2+3^2}$) = 1.033 W1

Reaction factor for B = 2/3 *W1

7.3.3. Calculation of Live Load BM and SF:

Impact factor [Refer IRC: 6-2014 Cl. 208]:

Vehicle Impact factor

IRC Class	Impact factor
A	1.17
70R (W)	1.17
70R (T)	1.1

Bending moment and shear force due to Live load and distribution to inner and outer girder

The influence line diagram for bending moment is shown in figure

Effective span of girder, le = 20m

Ordinate of bending moment at considered section, Mx = (1-x/L)*x

7.3.3.1. For IRC Class A Vehicle loading

Calculation of maximum bending moment at L/2

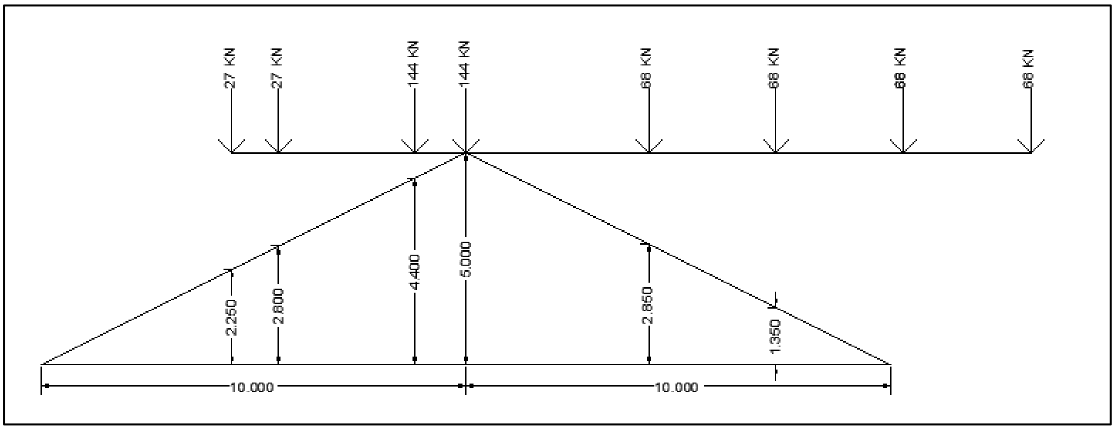


Fig7.13: ILD of BM at mid span for class A loading

Maximum BM at mid span = 27*2.25 +27*2.8 + 144*4.4 + 144*5 + 68*2.85 + 68*1350

= 1493.55 KN-m

Total bending moment for outer girder = 2362.221 KN-m

Total bending moment for inner girder = 1750.669 KN-m

Calculation of maximum bending moment at L/4

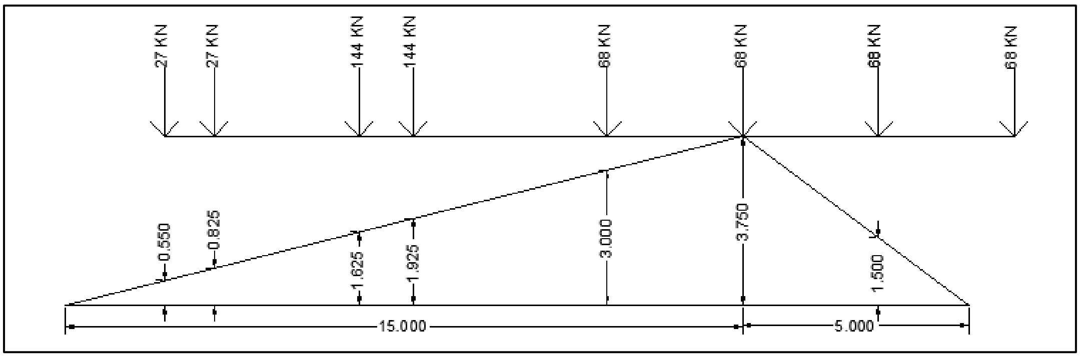


Fig7.14: ILD of BM at L/4 for class A loading

Maximum BM at quarter span = 1002.83 KN-m

Total bending moment for outer girder= 1584.065 KN-m`

Total bending moment for inner girder=1173.97 KN-m

Calculation of maximum bending moment at L/8

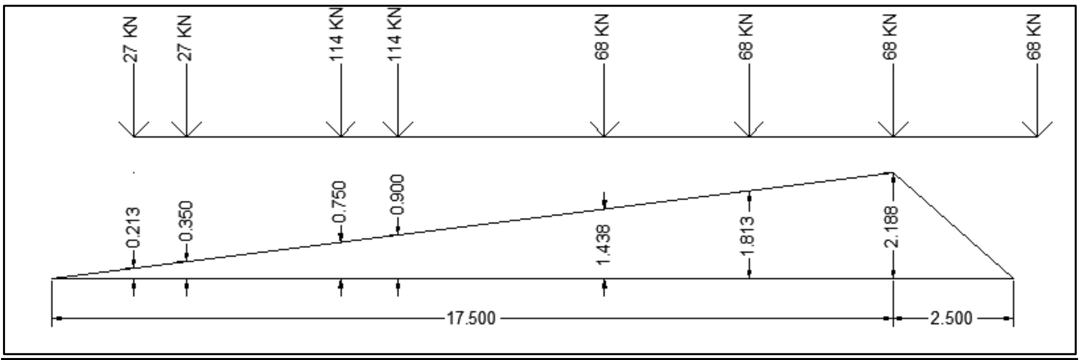


Fig7.15: ILD of BM at L/8 for class A loading

Maximum BM at L/8 span= 572.97 KN-m

Total bending moment for outer girder= 905 KN-m

Total bending moment for inner girder= 670.71 KN-m

Calculation of maximum bending moment at 3L/8

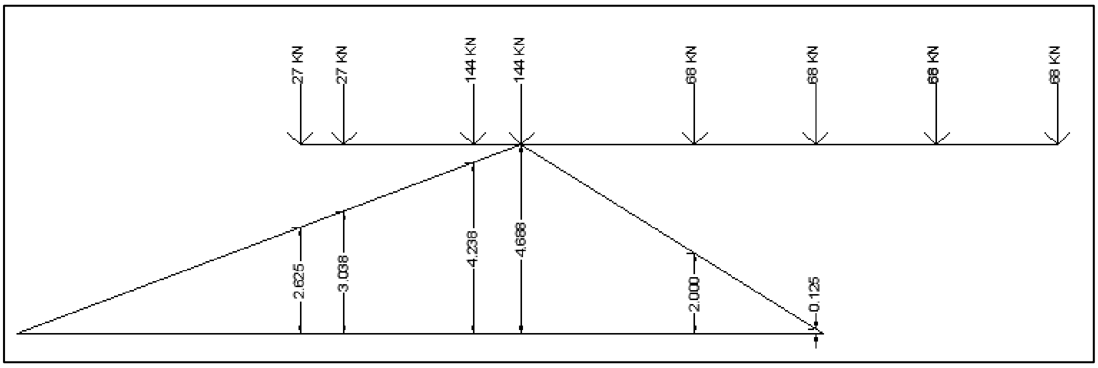


Fig7.16: ILD of BM at 3L/8 for class A loading

Maximum BM at 3L/8 span = 1289.478 KN-m

Total bending moment for outer girder= 2036.731 KN-m

Total bending moment for inner girder= 1509.444 KN-m

<u>Calculation of maximum SF at left support:</u>

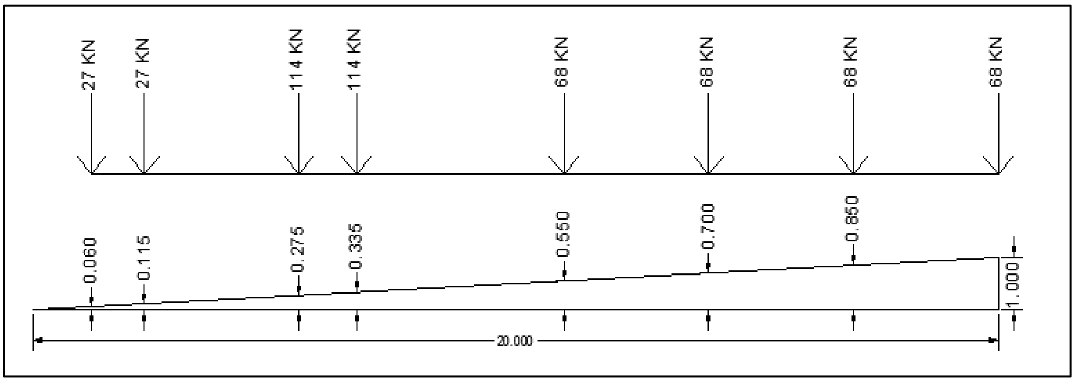


Fig7.17: ILD of SF at support for class A loading

Maximum SF at support = 27*0.06 + 27*0.115 + 144*0.275 + 144*0.335 + 68*0.55 + 68*0.7 + 68*0.85 + 68*1

= 357.16 KN

Total Shear force including impact and reaction factor for outer girder = 564.134 KN

Total Shear force including impact and reaction factor for inner girder = 418.086 KN

<u>Calculation of maximum Shear force at x=L/8 of span</u>

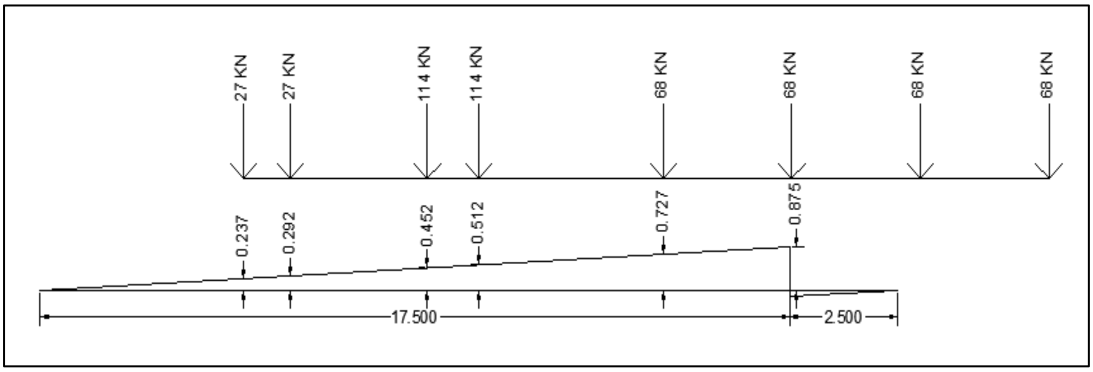


Fig7.18: ILD of SF at L/8 for class A loading

Maximum SF at L/8 = 232.415 KN

Total Shear force including impact and reaction factor for outer girder = 367.099 KN

Total Shear force including impact and reaction factor for inner girder = 272.062 KN

Calculation of maximum shear force at x= L/4 of span

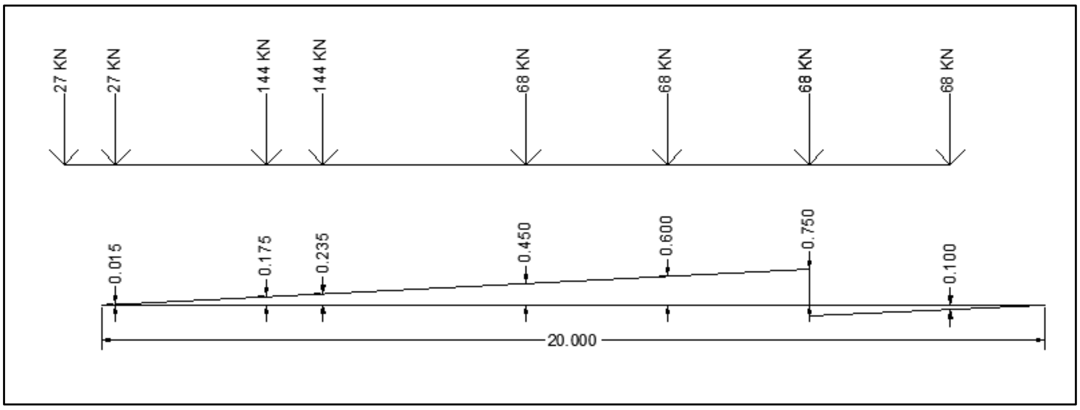


Fig7.19: ILD of SF at L/4 for class A loading

Maximum SF at L/4 = 173.36 KN

Total Shear force including impact and reaction factor for outer girder = 273.82 KN

Total Shear force including impact and reaction factor for inner girder = 202.933 KN

Calculation of maximum shear force at x=3L/8 of span

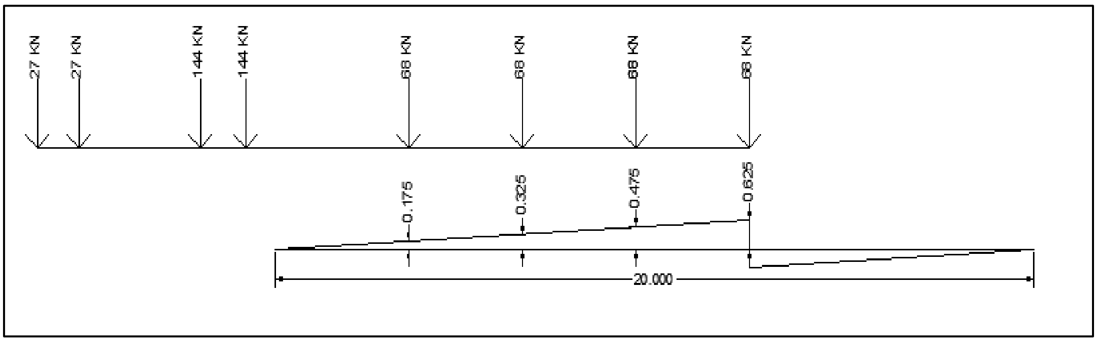


Fig7.20: ILD of SF at 3L/8 for class A loading

Maximum SF at 3L/8 = 108.8 KN

Total Shear force including impact and reaction factor for outer girder = 171.85 KN

Total Shear force including impact and reaction factor for inner girder = 127.36 KN

Calculation of maximum shear force at x= L/2 of span

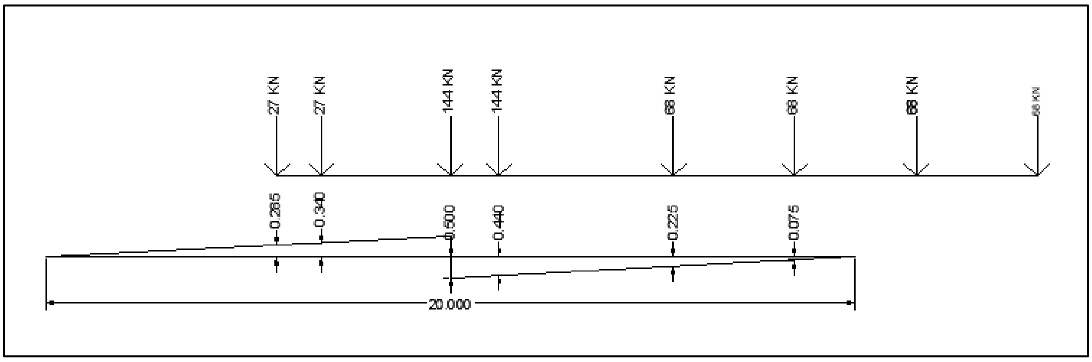


Fig7.21: ILD of SF at mid span for class A loading

Maximum SF at mid span = 110.683 KN

Total Shear force including impact and reaction factor for outer girder = 174.823 KN

Total Shear force including impact and reaction factor for inner girder = 129.564 KN

7.3.3.2. <u>For IRC Class 70R wheel vehicle loading</u>

<u>Calculation of maximum bending moment at L/2</u>

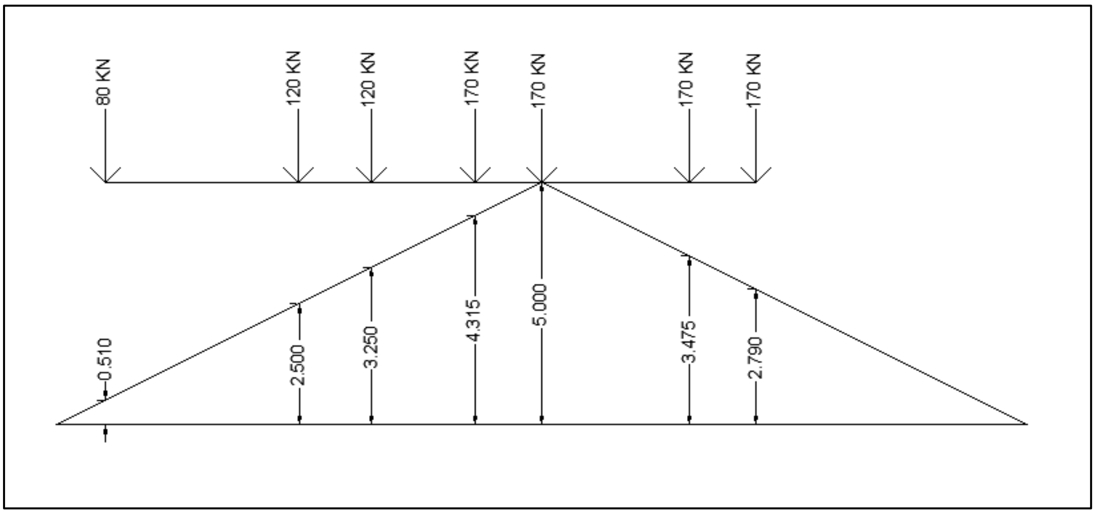


Fig7.22: ILD of BM at mid span for class 70R wheel loading

Maximum Bending Moment = 3377.35 KNm

Total Bending moment including impact and reaction factor for outer girder

= 3117.733 KNm

Total Bending moment including impact and reaction factor for inner girder

= 1973.774 KNm

<u>Calculation of maximum bending moment at L/4</u>

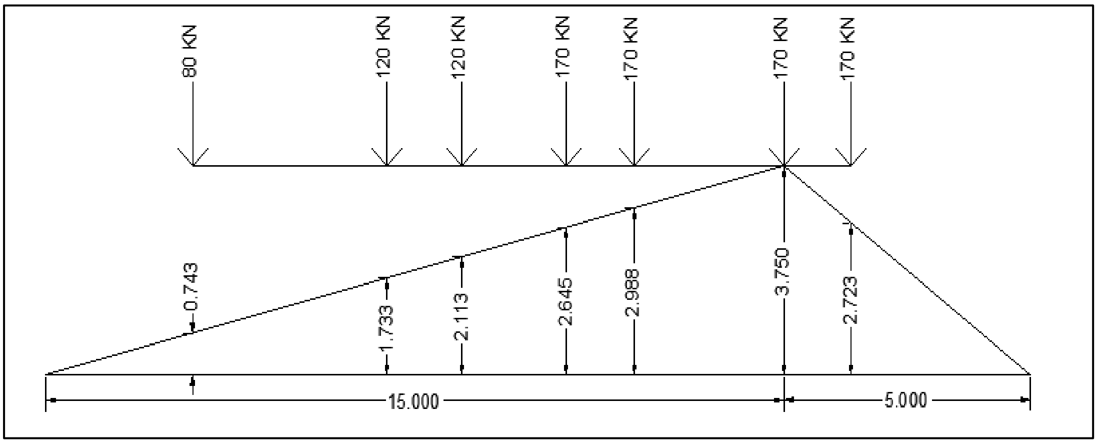


Fig7.23: ILD of BM at l/4 for class 70R wheel loading

Maximum Bending Moment= 2578.81 KNm

Total Bending moment including impact and reaction factor for outer girder

= 2380.577 KNm

Total Bending moment including impact and reaction factor for inner girder

= 1507.095 KNm

Calculation of maximum bending moment at L/8

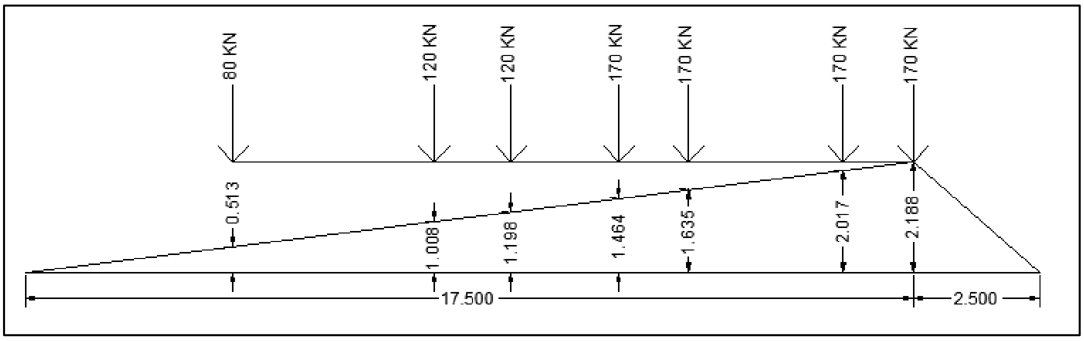


Fig7.24: ILD of BM at l/8 for class 70R wheel loading

Maximum Bending Moment = 1547.61 KNm

Total Bending moment including impact and reaction factor for outer girder

= 1428.645 KNm

Total Bending moment including impact and reaction factor for inner girder

= 904.446 KNm

Calculation of maximum bending moment at 3L/8

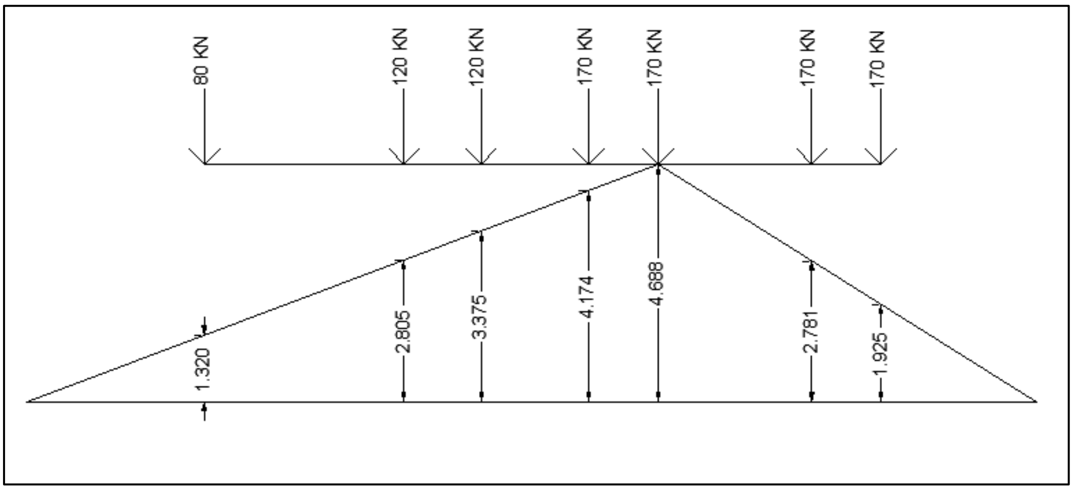


Fig7.25: ILD of BM at 3L/8 for class 70R wheel loading

Maximum Bending Moment= 3153.93 KNm

Total Bending moment including impact and reaction factor for outer girder

= 2911.487 KNm

Total Bending moment including impact and reaction factor for inner girder

= 1843.204 KNm

Calculation of maximum SF at left support:

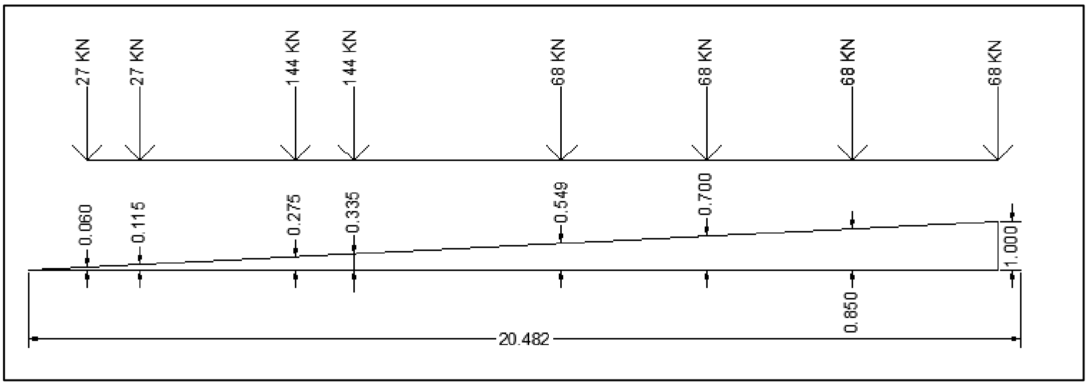


Fig7.26: ILD of SF at support for class 70R wheel loading

Maximum SF at left support = 743.38 KN

Total Shear force including impact and reaction factor for outer girder = 686.236 KN

Total Shear force including impact and reaction factor for inner girder = 434.44 KN

Calculation of maximum SF at L/8 span

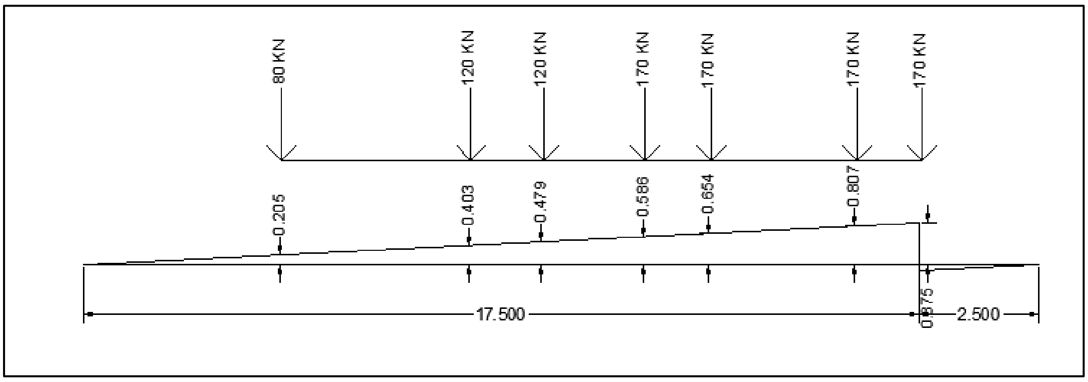


Fig7.27: ILD of SF at L/8 for class 70R wheel loading

Maximum SF at L/8 from left support = 618.98 KN

Total Shear force including impact and reaction factor for outer girder = 571.399 KN

Total Shear force including impact and reaction factor for inner girder = 361.741KN

Calculation of maximum SF at L/4 span

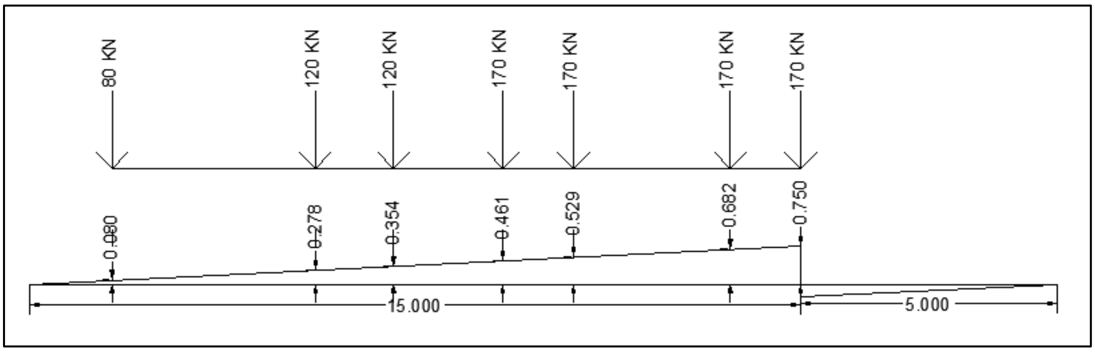


Fig7.28: ILD of SF at L/4 for class 70R wheel loading

Maximum SF at L/4 from left support= 493.895 KN

Total Shear force including impact and reaction factor for outer girder = 455.929 KN

Total Shear force including impact and reaction factor for inner girder = 288.639 KN

Calculation of maximum SF at 3L/8 span

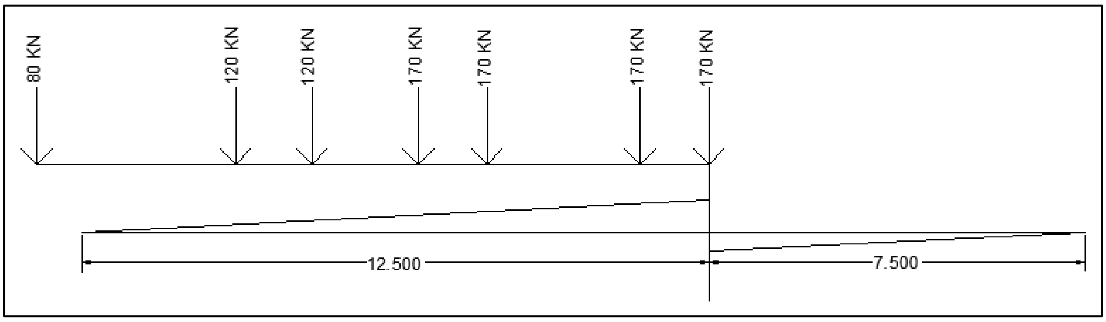


Fig7.29: ILD of SF at 3L/8 for class 70R wheel loading

Maximum SF at 3L/8 from left support= 357.7 KN

Total Shear force including impact and reaction factor for outer girder = 330.203 KN

Total Shear force including impact and reaction factor for inner girder = 209.045 KN

Calculation of maximum SF at L/2 span

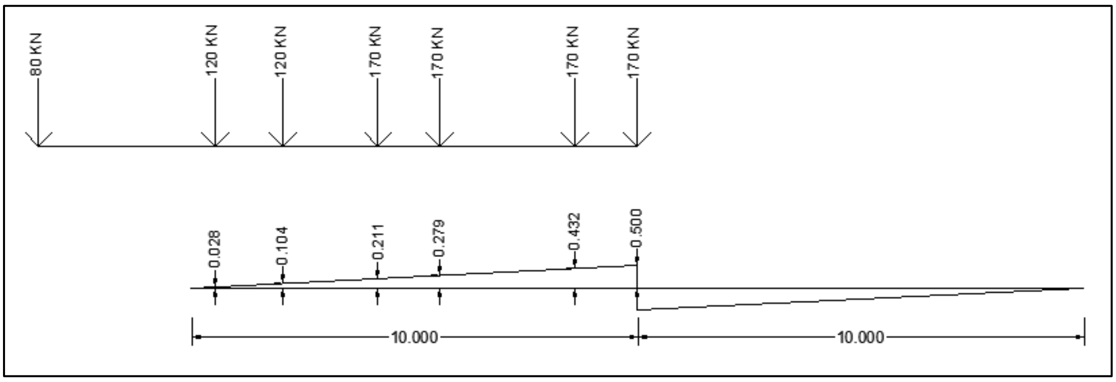


Fig7.30: ILD of SF at mid span for class 70R wheel loading

Maximum SF at L/2 from left support= 257.41 KN

Total Shear force including impact and reaction factor for outer girder = 237.622 KN

Total Shear force including impact and reaction factor for inner girder = 150.434 KN

7.3.3.3. For Class 70R tracked vehicle loading

Calculation of maximum bending moment at L/2:

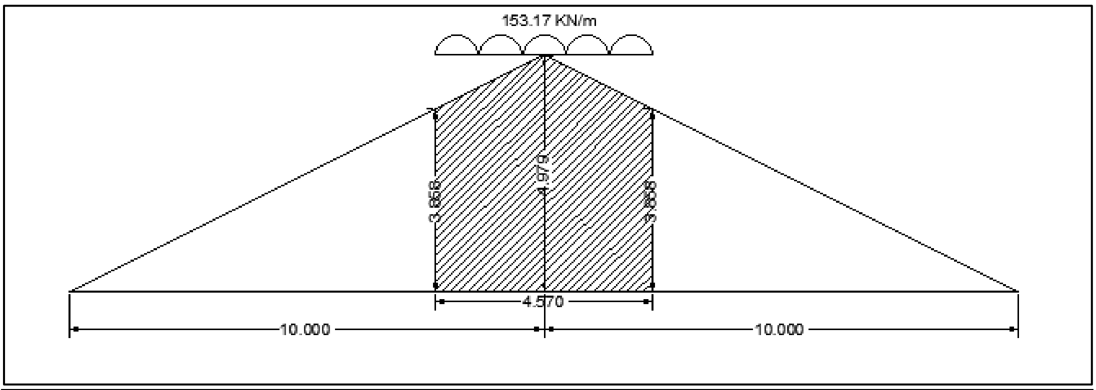


Fig7.31: ILD of BM at mid span for class 70R tracked loading

Maximum Bending Moment

= [3.868 + ½*(4.979 - 3.868)]*4.57*153.17

= 3099.89 KN-m

Total Bending moment including impact and reaction factor for outer girder

= 2809.918KNm

Total Bending moment including impact and reaction factor for inner girder

= 1811.622 KNm

Calculation of maximum bending moment at 3L/8

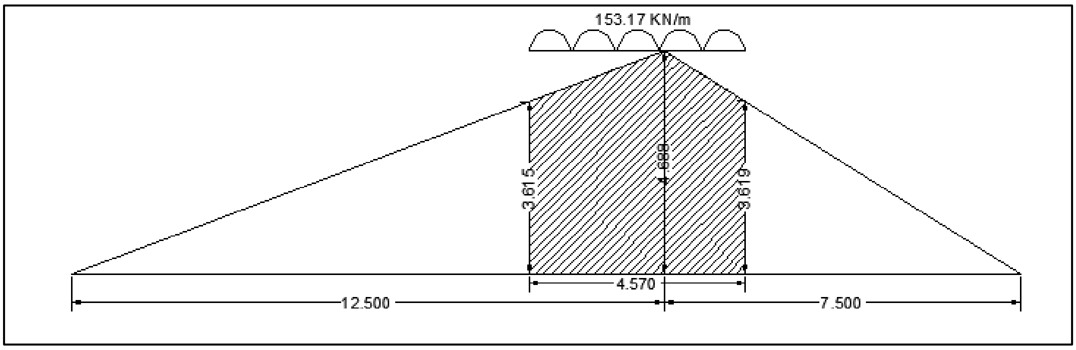


Fig7.32: ILD of BM at 3L/8 for class 70R tracked loading

Maximum Bending Moment= 2906.75 KN-m

Total Bending moment including impact and reaction factor for outer girder

= 2634.845 KNm

Total Bending moment including impact and reaction factor for inner girder

= 1698.748 KNm

Calculation of maximum bending moment at L/4

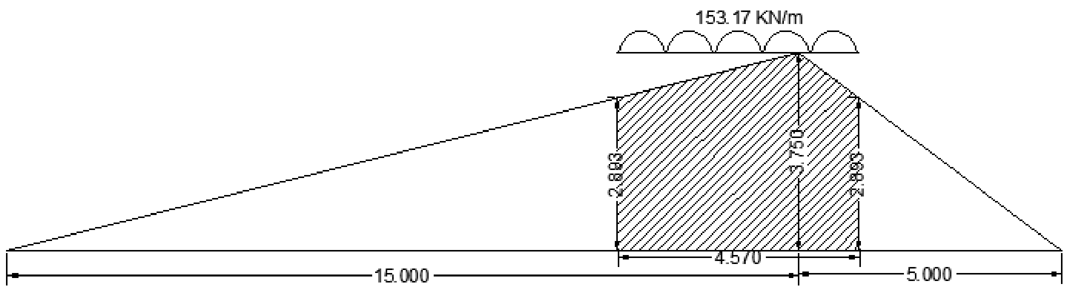


Fig7.33: ILD of BM at L/4 for class 70R tracked loading

Maximum Bending Moment= 2324.789 KN-m

Total Bending moment including impact and reaction factor for outer girder

= 2107.286 KNm

Total Bending moment including impact and reaction factor for inner girder

= 1358.618 KNm

Calculation of maximum bending moment at L/8

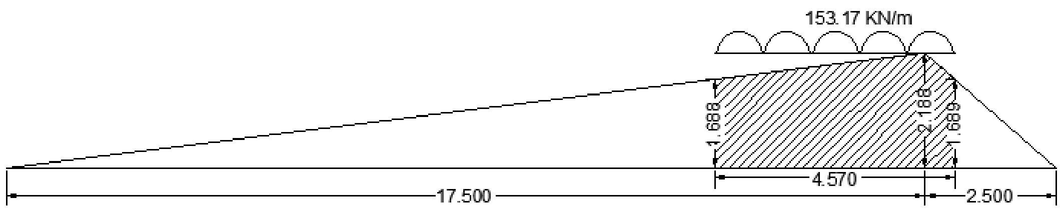


Fig7.34: ILD of BM at L/8 for class 70R tracked loading

Maximum Bending Moment= 1356.22 KN-m

Total Bending moment including impact and reaction factor for outer girder

= 1229.355 KNm

Total Bending moment including impact and reaction factor for inner girder

= 792.595 KNm

Calculation of maximum SF at left support

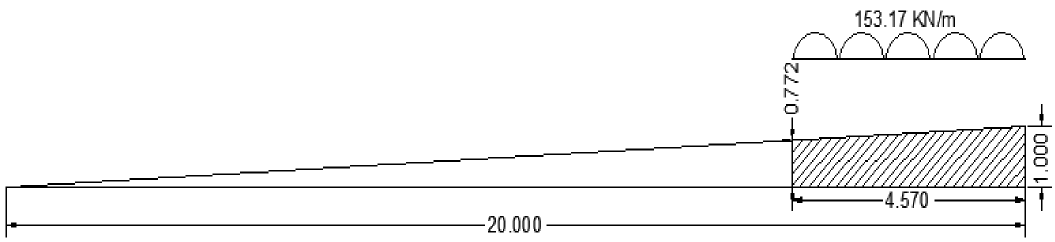


Fig7.35: ILD of SF at support for class 70R tracked loading

Maximum SF at left support= 620.01 KN

Total Shear force including impact and reaction factor for outer girder = 562.012 KN

Total Shear force including impact and reaction factor for inner girder = 362.343 KN

Calculation maximum of SF at L/8 span

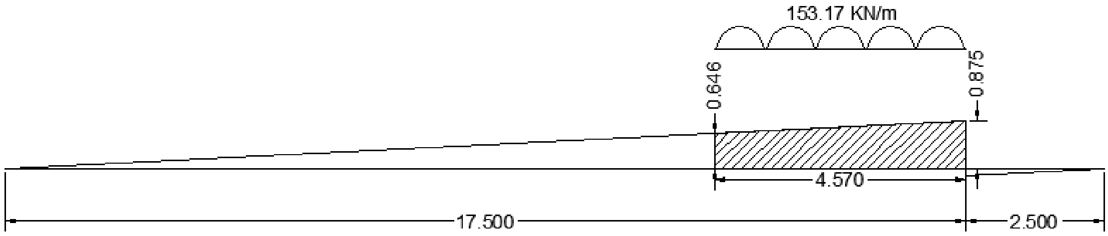


Fig7.36: ILD of SF at L/8 for class 70R tracked loading

Maximum SF at L/8 from left support= 532.515 KN

Total Shear force including impact and reaction factor for outer girder = 482.702

Total Shear force including impact and reaction factor for inner girder = 311.209KN

Calculation of maximum SF at L/4 span

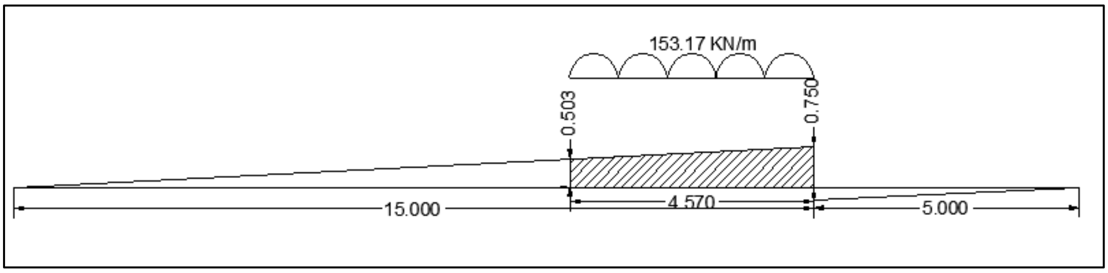


Fig7.37: ILD of SF at L/4 for class 70R tracked loading

Maximum SF at L/4 from left support= 445.55 KN

Total Shear force including impact and reaction factor for outer girder = 403.872 KN

Total Shear force including impact and reaction factor for inner girder = 260.386KN

Calculation of maximum SF at 3L/8 span

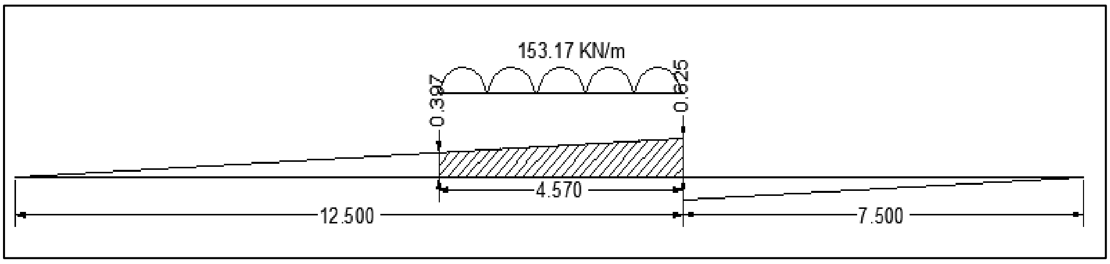


Fig7.38: ILD of SF at 3L/8 for class 70R tracked loading

Maximum SF at L/8 from left support= 372.58 KN

Total Shear force including impact and reaction factor for outer girder = 337.73 KN

Total Shear force including impact and reaction factor for inner girder = 217.741 KN

Calculation of maximum SF at mid span

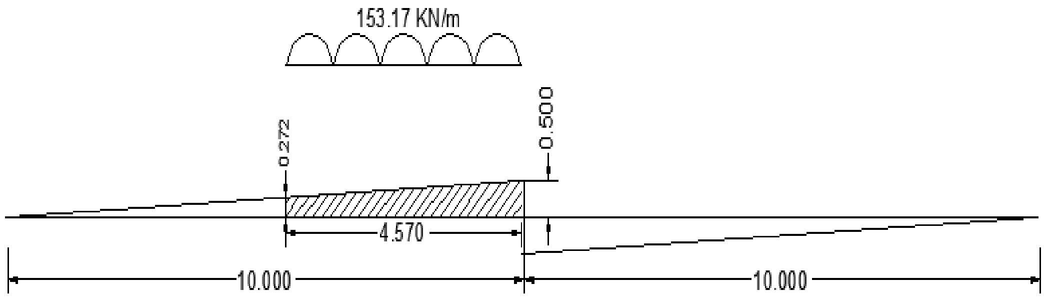


Fig7.39: ILD of SF at mid span for class 70R tracked loading

Maximum SF at L/8 from left support= 270.01 KN

Total Shear force including impact and reaction factor for outer girder = 244.752 KN

Total Shear force including impact and reaction factor for inner girder = 157.798 KN

Logitudinal outer girder

Actual section

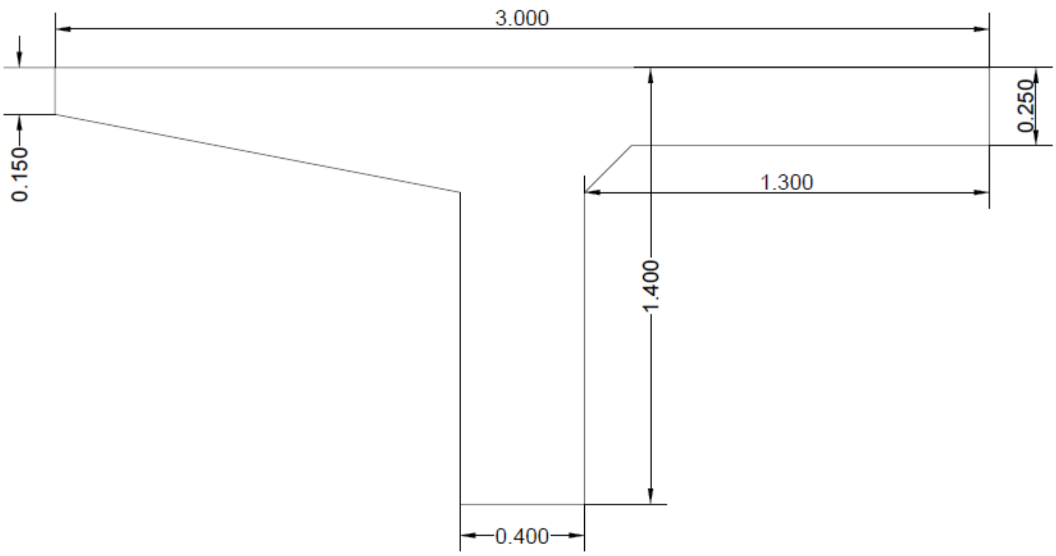


Fig7.40: Actual section of outer girder

At L/2 mid span

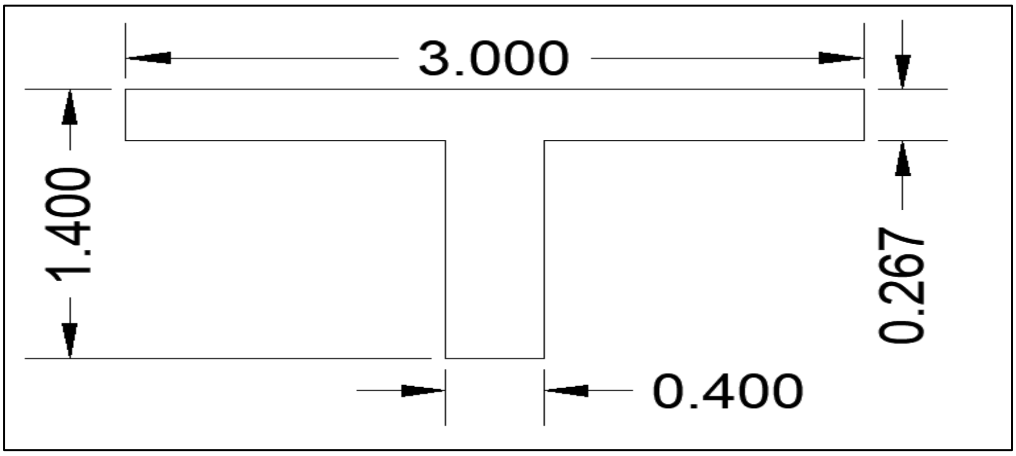


Fig7.41: Section of outer girder at mid span

$M_{ultimate}$ = 5940.718 KNm

$5940.718*10^6 = 0.36*30*3000*x_u*[1300-0.42x_u]$

$\frac{5940.718*106}{0.36*30*3000} = 1300 - 0.42{x_u}^2$

$0.42{x_u}^2-1300x_u + 183355.4938$

X_u=198.13mm < D_f so lies in the flange

From neutral axis method,

$M=0.87f_y*A_{st}*[d-0.42x_u]$

$5940.718*10^6 = 0.87*500*A_{st}*[1300-0.42*148.13]$

A_{st} = 11033.27mm^2

Providing, 32mm diameter bar

No. of bar required = $\frac{11033.27*4}{\pi*32^2}$

= 13.718 = 14 no.s

Check for minimum/maximum

Ast,min= 0.2% of bD [from clause 305.1 IRC 21]

$=\frac{0.2}{100}*400*14002$

=1120 mm2 [OK]

Ast provided $=\frac{14*\pi*32}{4}$

=11259.47mm2

% of steel = $\frac{11259.47}{400*1300}$

=2% < 2.5% [OK]

At quarter span

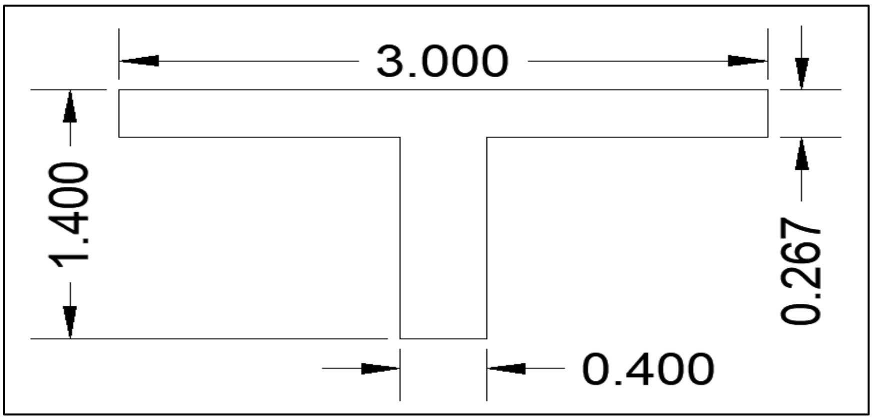


Fig7.42: Section of outer girder at quarter span

M_{live} = 2380.577KNm

Due to dead load

M_{dead} = 1568.33*1.35

Mu= 2380.577+1568.33*1.35

=4497.82KNm

Assuming that x_u lies in the flange

0.87fy A_{st} =0.36fck*b*x_u

0.87*500*A_{st} =0.36*30*3000*x_u

x_u = 0.0139A_{st}

M_u = 0.36fck*b*x_u*[d-0.92x_u]

M_u=0.87fyA_{st}*[d-0.42x_u]

4497.82*10^6 =0.87*500*A_{st}*[1300-0.42*0.0134A_{st}]

$\frac{4497.82*106}{0.87*500}$ = 1300A_{st} - 5.628*$10^{-3}$$A_{st}$

10339816.09 = 1300A_{st} - 5.628*$10^{-3}$$A_{st}$

A_{st1} = 222739.68mm^2

A_{st2} = 8248.237mm^2

We have to use small area

X_u = 0.0134*8248.237

= 110.526<Df [which shows that lies in the flange]

So, required area = 8248.237mm^2

Let, the diameter of 32mm is used

X = $\frac{8248.237*4}{\pi*32^2}$

=10.256

=11 numbers

Check for min/max area required

$A_{st,min}$= 0.2% of bD [from clause 305.1 IRC 21]

$=\frac{0.2}{100}$*400*1300 =1120 mm^2 [OK]

A_{st} provided $=\frac{11*\pi*32^2}{4}$

=8876.725mm^2

% of steel = $\frac{8876.725}{400*1400}$

=1.58% < 2.5% [OK]

<u>Design of section for shear</u>

At support,

$V_{rd.c}$ = [0.12*k* 80*ρ*f_{ck})$^{0.33}$] *b_w*d

K = 1 + $\sqrt{200/d} \leq 2$

= 1.392 < 2

$\rho = \frac{Ast}{bw*d}$

$\rho = \frac{11259.47}{250*1300}$

=0.0346

$V_{rd.c}$ = [0.12*1.392*(80*0.0346*30)$^{0.33}$]*(250*1300)

= 233.382KN<1222.234KN

$V_{rd.s}$ = 1222.234-233.382

=988.852KN

Use 10mm Ø4 legged stimps, the spacing is,

$S_v = \frac{0.87*fy*Asv*z}{\text{Vrd.s}}$

= 162.69 mm

Take 160 mm

At quarter span,

$V_{rd.c}$ = [0.12*k* 80*ρ*f_{ck})$^{0.33}$] *b_w*d

K = 1 + $\sqrt{200/d} \leq 2$

= 1.392 < 2

$\rho = \frac{Ast}{bw*d}$

$\rho = \frac{8846.725}{250*1300}$

=0.027

$V_{rd.c}$ = [0.12*1.392*(80*0.027*30)$^{0.33}$]*(250*1300)

= 215.04 KN < 733.88 KN

$V_{rd.s}$ =733.88 - 215.04

= 518.48 KN

Use 10mm Ø 2 −legged stimps, the spacing is,

$S_v = \frac{0.87*fy*Asv*z}{\text{Vrd.s}}$

= 155.095 mm

Take 155 mm

Inner longitudinal girder

At mid span,

Bending moment from live load (maximum occur from 70R wheel load)

B_{live} = 1973.774KNm

B_{dead} = 2091.115*1.35

=2823.00KNm

Total= 4796.774KNm

Assuming that x_u lies in the flange,

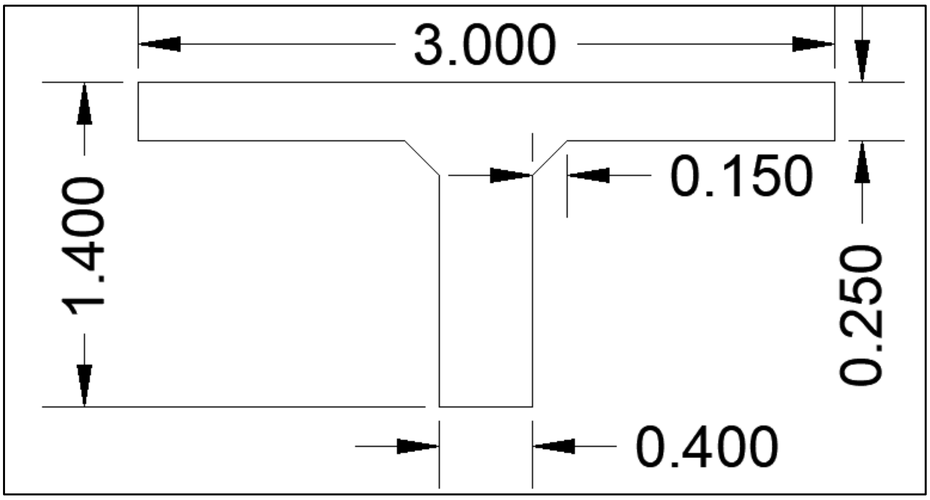


Fig7.43: Section of inner girder at mid span

By using equivalent thickness method we get,

Average thickness on left part of slab = $\frac{0.25*1.15+0.325*0.15}{1.3}$

= 0.258

Average thickness on right part of slab = $\frac{0.25*1.15+0.325*0.15}{1.3}$

= 0.258mm

Thickness of flange = $\frac{0.258*2}{2}$ = 258mm

New section,

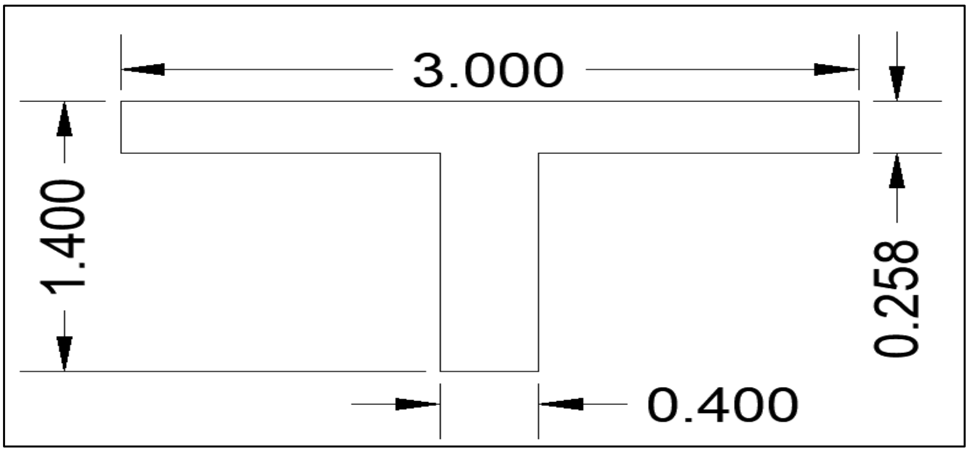


Fig7.44: New section of inner girder at mid span

$M_{u,lim}$ = 0.36fck *b*x_u*(d-0.42x_u)

$X_{u,lim}$ = 0.46*d

= 0.36fck*b*0.46d *(d-0.42*0.46d)

= 0.133*fck*b*d^2

4796.774*10^6 = 0.133*30*3000*d^2

$D = \sqrt{\frac{4796.774*106}{0.133*30*3000}}$

=633.034mm < 1400mm (OK)

M_u = 0.36fck*b*x_u*(d-0.42x_u)

4796.774*10^6 =0.36*30*3000*x_u*(1300 - 0.42*x_u)

4796.77*10^6 = 0.36*30*3000*x_u*(1300 - 0.42*x_u)

$\frac{4796.77*106}{0.36*30*3000}$ = 1300 - 0.42*x_u^2

0.42*x_u^2 - 1300x_u + 148048.456 = 0

X_u = 118.41mm < 258mm (OK)

which shows that in flange

Calculation of area of steel,

M_u = 0.87fy A_{st}*(1300-0.42*x_u)

4796.77*10^6 = 0.87*500* A_{st}*(1300-0.42*x_u)

A_{st} = 8819.756mm^2

Providing 32mm^2

No. of bar required = $\frac{8819.756*4}{\pi*32^2}$

= 10.966 = 11 no.s

Area of steel provided = $11*\frac{\pi*32^2}{4}$

= $8846.72mm^2$

Check for min/max area of steel

$A_{min} = \frac{0.2}{100}*400*1400$

= $1120mm^2$ (OK)

$A_{max} = 2.5\% > \frac{8846.72*100}{400*1300}$

= 1.58% (OK)

At quarter section

BM_{live} = 1507.095KNm

BM_{dead} = 1568.33*1.35

=2117.245KNm

Total BM = 1507.095 + 2117.245

= 3624.34KNm

$D = \sqrt{\frac{3624.34*10^6}{0.133*30*3000}}$

= 550.259mm < 1300mm (OK)

Let x_u lies in the flange,

$M_u = 0.36fck*b*x_u*(d-0.42x_u)$

$3624.34*10^6=0.36*30*3000*x_u*(1300 - 0.42*x_u)$

$\frac{3624.34*10^6}{0.36*30*3000} = 1300\, x_u - 0.42*x_u^2$

$0.42*x_u^2 - 1300x_u + 111862.35 = 0$

$X_u =$ 88.583mm (OK) [lies in the flange]

$M_u = 0.87*500*A_{st}*(1300-0.42*x_u)$

$3624.34*10^6 = 0.87*500*A_{st}*(1300-0.42*88.583)$

$A_{st} = 6597.91mm^2$

Providing n = $\frac{6597.977*4}{\pi*32^2}$

Area of steel provided = $\frac{9*\pi*32^2}{4}$

= $7238.23mm^2$

Check for min/max area

$A_{min} = 120mm^2$(OK)

Area of steel = $\frac{7238.23*100}{400*1300}$

= 1.293% < 2.5% (max)[OK]

At support,

$V_{rd.c}$ = [0.12*k* 80*ρ*f_{ck})$^{0.33}$] *b_w*d

K = 1 + $\sqrt{200/d} \leq 2$

= 1.392 < 2

$\rho = \frac{Ast}{bw*d}$

$\rho \quad = \frac{8846.725}{250*1300}$

= 0.027

$V_{rd.c}$ = [0.12*1.392*(80*0.027*30)$^{0.33}$]*(250*1300)

= 215.04 KN < 970.44 KN

$V_{rd.s}$ = 970.44 - 215.04

= 755.37 KN

Use 10mm Ø 4-legged stirrups

The spacing is,

$S_v = \frac{0.87*fy*Asv*z}{\mathrm{Vrd.s}}$

= 212.9 mm

Take 210 mm

At quarter span

$V_{rd.c}$ = [0.12*k* 80*ρ*f_{ck})$^{0.33}$] *b_w*d

K = 1 + $\sqrt{200/d} \leq 2$

= 1.392 < 2

$\rho = \frac{Ast}{bw*d}$

$\rho = \frac{7238.23}{250*1300}$

=0.0223

$V_{rd.c}$ = [0.12*1.392*(80*0.0223*30)$^{0.33}$]*(250*1300)

= 201.889 KN < 639.69 KN

$V_{rd.s}$ = 639.69 - 201.889

= 437.801 KN

Use 10mm Ø 2 −legged stimps, the spacing is,

$S_v = \frac{0.87*fy*Asv*z}{\mathrm{Vrd.s}}$

= 183.67 mm

Take 180 mm

Summary

Materials (Fe 500, M30)

Outer girder

Section	Design Moment kN-m	Area of steel reqd mm^2	Area of steel provided		
			Number	Dia (mm)	Area (mm^2)
L/2	5940.718	11033.27	14	32	11259.47
3L/8	5549.752	11033.27	14	32	11259.47
L/4	4497.8225	8298.237	11	32	8876.725
L/8	2655.2685	8298.237	11	32	8876.725
support	0	0	11	32	8876.725

Inner girder

Section	Design Moment kN-m	Area of steel reqd mm^2	Area of steel provided		
			Number	Dia (mm)	Area (mm^2)
L/2	4796.77925	8819.756	11	32	8846.72
3L/8	4481.4685	8819.756	11	32	8846.72
L/4	3624.354	6597.91	9	32	7238.23
L/8	2131.0695	6597.91	9	32	7238.23
support	0	0	9	32	7238.23

Outer girder

Section	Design SF kN	Spacing required mm	vertical stirrup provided		
			No. of legged	Dia (mm)	spacing (mm)
L/2	0	0	2	10	155
3L/8	478.88195	155.095	2	10	155
L/4	733.88	155.095	2	10	155
L/8	994.8481	162.69	4	10	160
support	1222.234	162.69	4	10	160

Inner girder

Section	Design SF kN	Spacing required mm	vertical stirrup provided		
			No. of legged	Dia (mm)	spacing (mm)
L/2	0	0	2	10	180
3L/8	358.893	183.67	2	10	180
L/4	639.69	183.67	2	10	180
L/8	785.19	212.9	4	10	210
support	970.44	212.9	4	10	210

7.4. Design of cross girder

7.4.1. Dead Load of intermediate cross girder

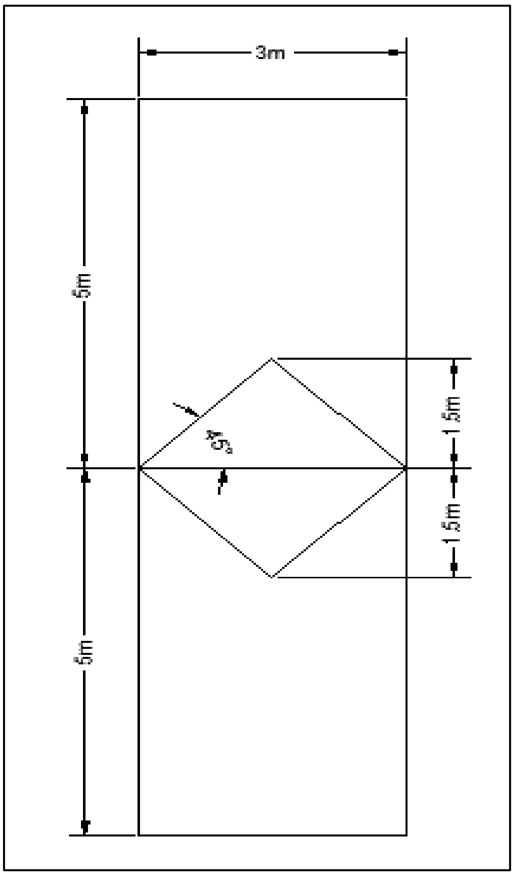


Fig7.45: Intermediate cross-girder

Self weight of wearing course = 0.075*1/2*2*1.5*3*22

= 7.425 KN

Self weight of slab = 0.25*1/2*2*1.5*3*25

= 28.125 KN

Total uniformly varying load = (7.425 + 28.125) KN

= 35.55 KN

= 23.7 KN/m

Self weight of cross girder = 25*3*0.865*0.35

= 22.706 KN

Uniformly distributed load = 7.569 KN/m

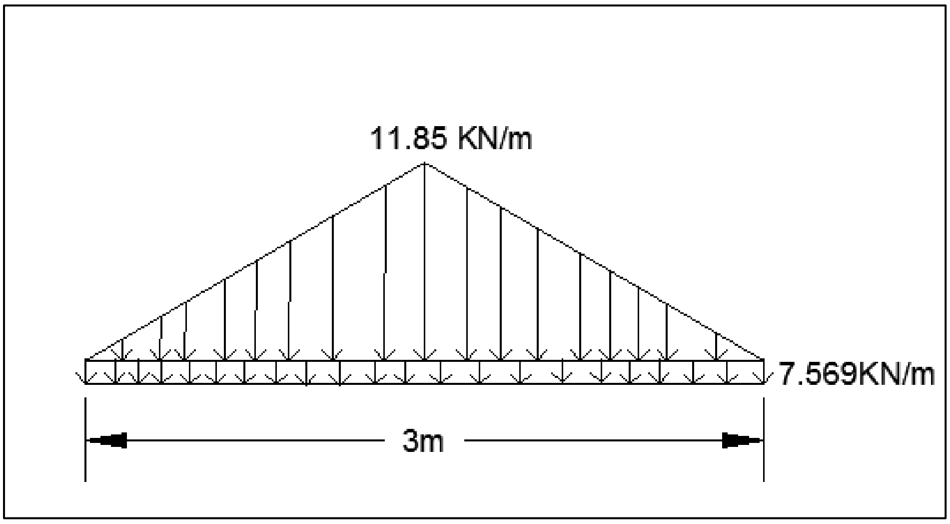


Fig7.46: Dead load on cross girder

Shear force = (1/2*23.7*3+ 3*7.569)/2

= 29.129 KN

Factored shear force = 29.129*1.35

= 39.32 KN

Bending moment = 29.12*1.5 – 7.569*1.5^2/2 – ½*1.5*23.7*1/3*1.5

= 26.277 KNm

Factored bending moment = 26.277*1.35

= 35.465 KNm

7.4.2. <u>Calculation of live load on intermediate cross girder:</u>

<u>Due to Class A loading</u>

Maximum bending moment occurs when one wheel of a vehicle lies near center of span.
Interior cross girder

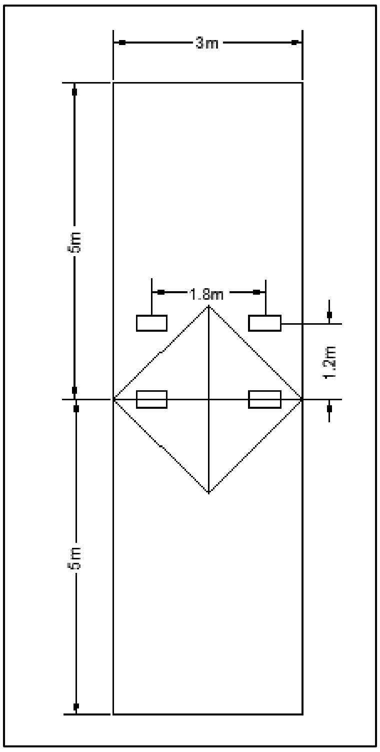


Fig7.47: Class A load on cross girder

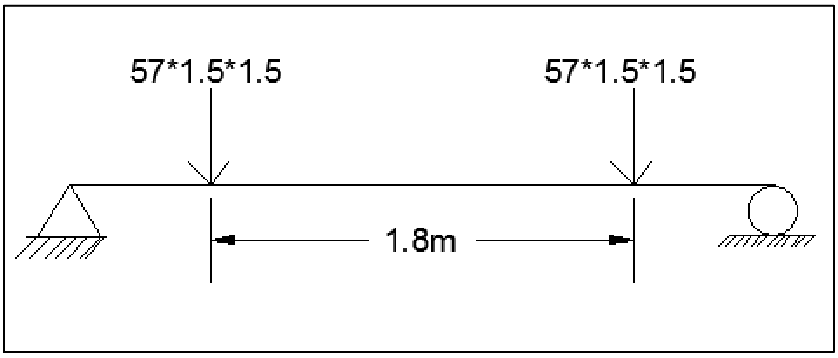


Fig7.48: location of loads

Maximum bending moment = 96.188 KNm

7.4.3. <u>Design of intermediate cross girder</u>

Total bending moment = 35.465+96.188 = 131.645 KNm

Total shear force = 39.32 + 196.65 = 235.97 KN

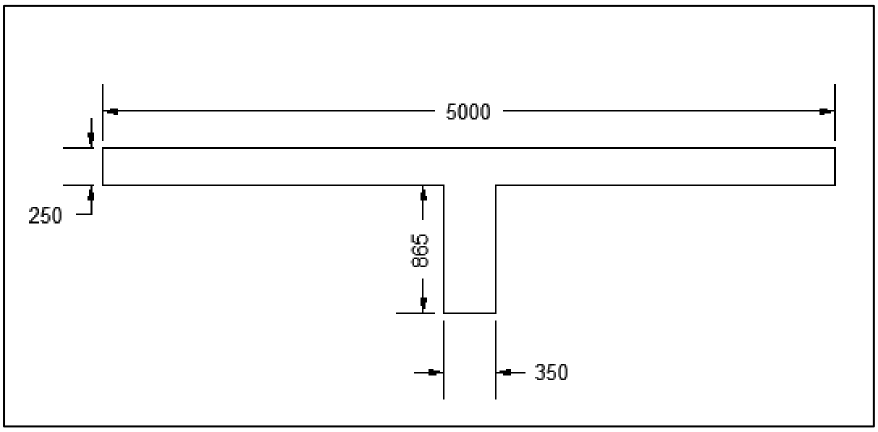


Fig7.49: Intermediate cross girder

Check for moment

Effective depth (deff) =1115 – 100 = 1015mm

First, Assuming that Xu lies in Df

0.36fck*bf*Xu = 0.87fy*Ast

Where, bf = (L0/6 + bw + 6Df) = (3000/6 + 350 + 6*250) = 2350mm

Now,

0.36*30*Xu*2350 = 0.87*500*Ast

Xu = 0.017*Ast

Now,

Mu = 0.36fck*bf*Xu(d – 0.42Xu)

131.645*106 = 0.36*30*2350* Xu (1015 – 0.42 Xu)

Solving for Ast , we get Ast = 301.24 mm2

Therefore, Xu = 0.017*301.24 = 5.121mm (lies in Df)

Mu,lim = 0.36fck*b*Xu(d – 0.42Xu)

= 0.36*30*2350*0.46d*(d – 0.42*0.46d)

= 0.36*30*2350*0.46*1015*(1015 – 0.42*0.46*1015)

= 9704 * 106 KNm

Check,

Ast,min = 0.2% of bdeff = (0.2/100 *350*1015) =710 mm2

So providing 12mm Φ bar

Number of bars (n) = (710*4 / πd2) = 7 nos.

Area provided = 7*πd2/4 = 791.679 mm2

Check for maximum,

Percentage = 791.697 / 1015*350 = 0.215% < 2.5% (Hence ok)

Check for shear

Nominal shear stress (Tv) = Vu/bd = 235.97*1000 / 350*1115 = 0.6 N/mm2

Percentage of steel = 0.215%

From IS 456:2000

By interpolating,

Ʈc = 0.342 N/mm2

Ʈc,max = 3.1N/mm2

Ʈc <Tv < Ʈc,max

So,

Vus = 0.87fyAsvd / Sv

Providing 12mm Φ , 2- legged vertical stirrup,

Sv = $\frac{0.87*500*2*\pi*12*12/4*1015}{235.97*1000-0.342*350*1015}$ = 872.42 mm > 300mm

So, provide 12mm Φ , 2- legged vertical stirrup @ 300mm c/c spacing.

7.4.4. <u>Calculation of dead loads on end cross girder</u>

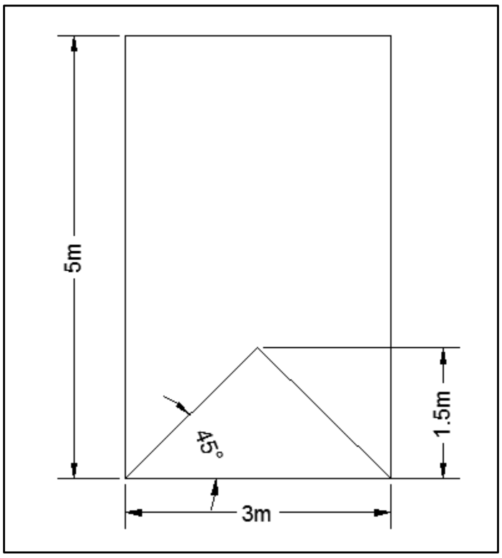


Fig7.50: End cross-girder

Self-weight of wearing course = 7.425/2

= 3.713 KN

Self-weight of slab =28.125/2 = 14.063 KN

Self-weight of cross girder = 22.706 KN

Therefore,UDL=7.569KN

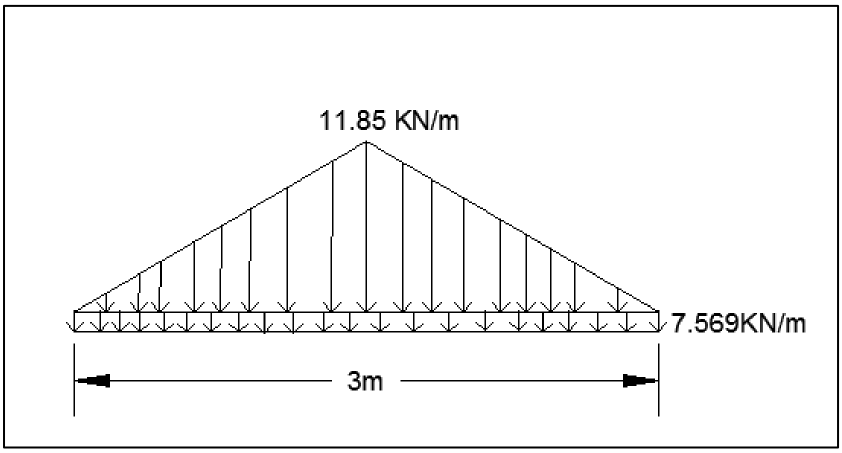


Fig7.51: total loads

Shear force = (11.85*3*0.5 + 7.569*3)/2 = 20.241 KN

Factored shear force = 27.325 KN

Bending moment = 20.241*1.5 – 7.569*1.52/2 – 11.85*0.5*1.5*1.5/3 = 17.40 KNm

Factored bending moment = 17.40*1.35 = 23.49 KNm

7.4.5. Calculation for live load on end cross gider:

Due to Class A loading

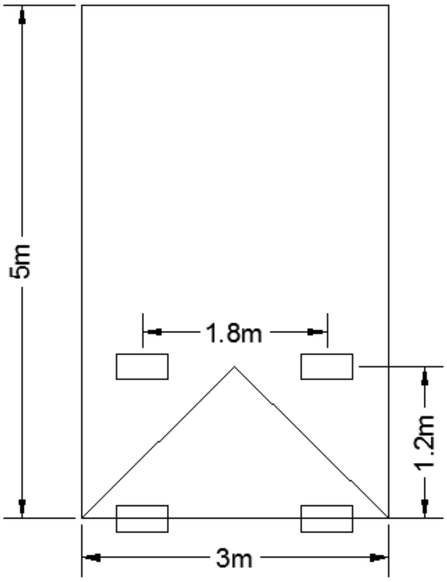


Fig7.52: Class A loading on end cross girder

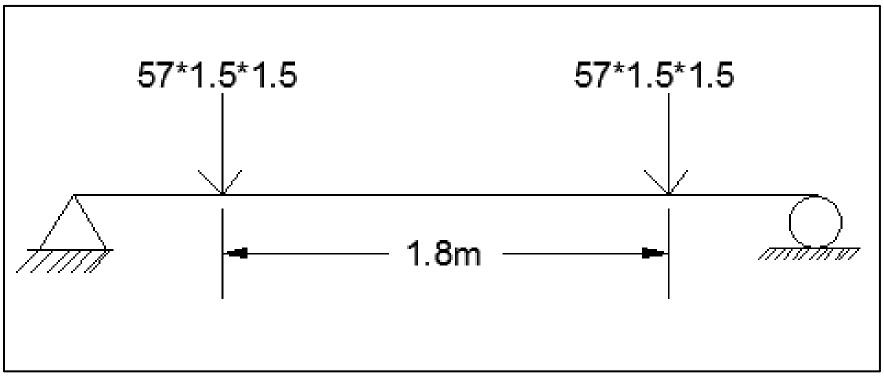


Fig7.53: Load configuration

Maximum bending moment = 96.188 KNm

7.4.6. <u>Design of end cross girder</u>

Total bending moment = 23.49 + 96.188 = 119.682 KNm

Total shear force = 27.325 + 196.65 = 223.98 KN

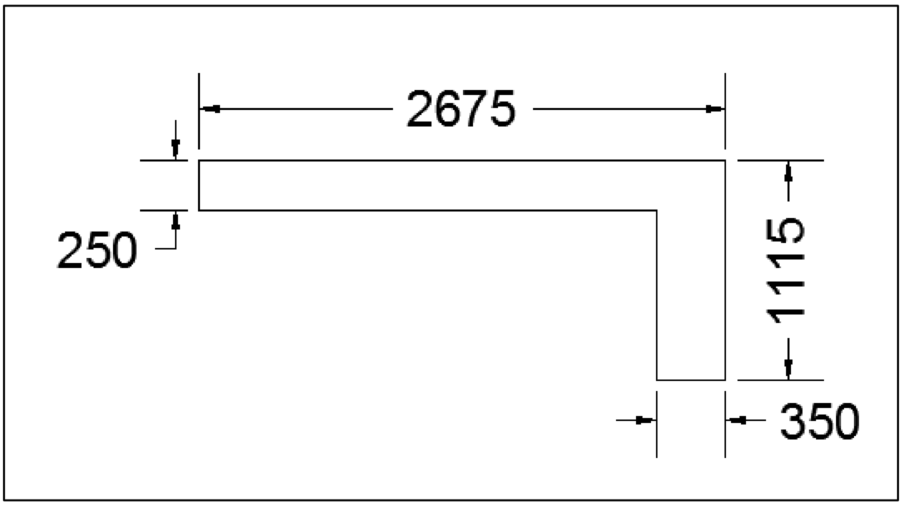


Fig7.54: End croos-girder

From IS 456:2000

$b_{eff} = L_0 /12 + b_w + 3D_f$ = 3000/12 + 350 + 3*250 = 1350 mm < 2675 mm

So, effective width (b) = 1350mm

Now,

Assuming that X_u lies in D_f

$0.36f_{ck}bX_u = 0.87f_yA_{st}$

$0.36*30*1350*X_u = 0.87*500*A_{st}$

$X_u = 0.0298\ A_{st}$

Now,

Mu = 0.36fck*bf*Xu(d – 0.42Xu)

119.682*106 = 0.36*30*1350* 0.0298Ast(1015 – 0.42 * 0.0298Ast)

Ast = 272.3 mm2

Therefore, Xu = 0.0298*272.3 = 8.11mm (lies in Df)

Check,

Ast,min = 0.2% of bdeff = (0.2/100 *350*1015) =710 mm2

So providing 12mm Φ bar

Number of bars (n) = (710*4 / πd2) = 7 nos.

Area provided = 7*πd2/4 = 791.679 mm2

Check for maximum,

Percentage = 791.697 / 1015*350 = 0.215% < 2.5% (Hence ok)

<u>Check for shear</u>

Nominal shear stress (T_v) = V_u/bd = 223.98 *1000 / 350*1115 = 0.57 N/mm^2

Percentage of steel = 0.215%

From IS 456:2000

By interpolating,

τ_c = 0.342 N/mm^2

$\tau_{c,max}$ = 3.5N/mm^2

$\tau_c < T_v < \tau_{c,max}$

So,

$V_{us} = 0.87 f_y A_{sv} d / S_v$

Providing 12mm Φ , 2- legged vertical stirrup,

$S_v = \frac{0.87*500*2*\pi*12*12/4*1015}{235.97*1000-0.342*350*1015}$ = 872.42 mm > 300mm

So, provide 12mm Φ , 2- legged vertical stirrup @ 300mm c/c spacing.

<u>Side face reinforcement</u>

As the depth of cross girder exceeds 750mm, side face reinforcement shall be provided along the two face. The total area of such reinforcement shall not be less than .1% of web area and shall be distributed equally.

As required =0.001*350*865=302.75 mm2

Provide 8 mm dia. bar, Number of bars. = 302.75 / $2\pi\times8\times8/4$ = 3.01

Provided 4-8mm bar on each face.

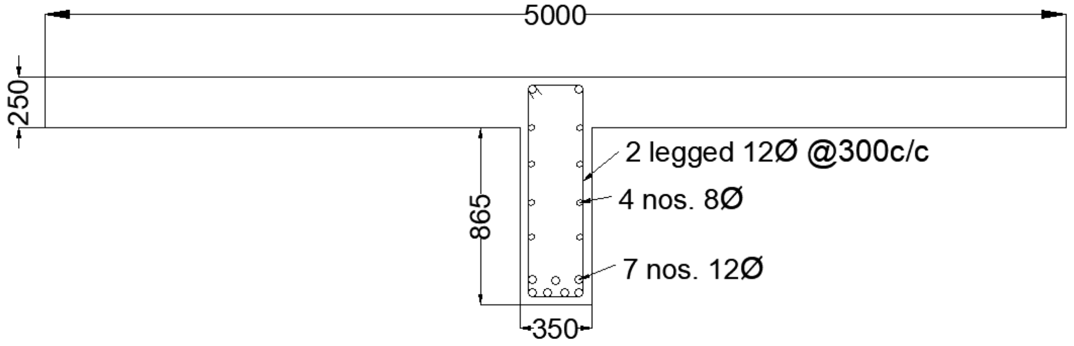


Fig7.55: Reinforcement detailing of interior cross girder

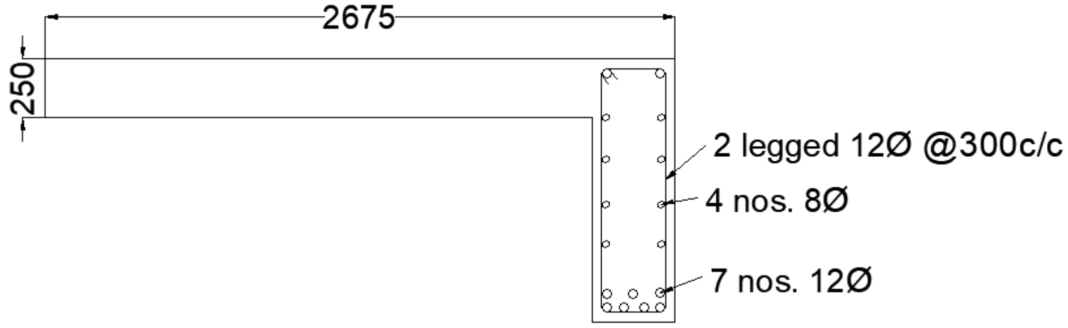


Fig7.56: Reinforcement detailing of end cross girder

Design summary

S.N	Description	BM (KNm)		SF(KN)		Reinforcement	
		DL	LL	DD	LL	Longitudinal	Shear
1	Intermediate girder	35.46	96.188	39.32	196.65	7nos-Φ12mm	12mmΦ, 2-legged vertical stirrups@300 mm c/c
2	End girder	23.49	96.188	27.32	196.65	7nos-Φ12mm	12mmΦ, 2-legged vertical stirrups@300 mm c/c

B. Analysis and design of substructure

7.5. Analysis and design of bearing

7.5.1. Calculation of Loads on Bearing

a. DL from Superstructure

Weight of wearing coat = 7.5*0.08*22*20 = 247.5KN

Weight of railing =2.438KN/m = 2.438*20 =48.76 KN

Weight of Kerb = 11.25KN/m = 11.25*20 = 225 KN

Weight of slab = 57.875 KN/m = 57.875*20 = 1157.5 KN

Weight of fillet=1.125KN/m=1.125*20=22.5KN

Where weight of slab consists

- Middle portion
- Fillet
- Cantilever part

Weight of main girder = 0.4*1.15*20*25*3 =690KN

Weight of cross girder = 0.35*0.865*5.2*5*25=196.78KN

Total DL from super structure (Wu)) = 247.5KN+48.76KN+225KN+1157.5KN +22.5KN+690KN+196.78KN=2588.96KN

DL from superstructure on a bearing (DL_{sup}) = $\frac{2588.96}{6}$ = 431.493KN

b. LL from Superstructure

Maximum LL on a bearing (LL) = Maximum shear force due to LL on main girder = $\frac{686.236}{1.5}$ =457.490KN

c. Load due to braking effort of Class A load

Braking load = 0.2 × (2 × 27 +2 x 114+ 4 × 68) = 110.8 KN

Horizontal Braking load on a bearing (F_{br}^{H}) = 110.8/6 = 18.47 KN

Braking loads acts at 1.2m above wearing coat. Point of application of braking load is (1.2+0.075+0.25+1.2) =2.675m from bearing. It induces vertical reaction on bearing.

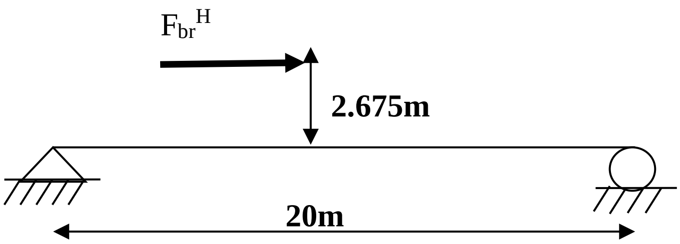


Fig7.57: Line of action of horizontal load

Vertical reaction on a bearing due to Braking load (F_{br}^V) $=\frac{110.8*2.675}{20*3}$ =4.939KN

d. Wind load

Wind load in transverse direction of bridge (F_w^T) = $P_Z \times A \times G \times C_D$ =

Take, Ht. of bridge = 10 m,

Basic wind speed = 47 m/s and Terrain with no obstruction

Where,

V_a = 27.80 × 47/33 = 39.6 m/s [Refer Cl. 209, IRC 6]

$P_Z = 463.7 \times \frac{47^2}{33^2} = 940.6\ N/m^2$

G = 2 up to 150 m span

$C_D = 1.56785\ [\frac{B}{D} = \frac{7.5}{1.4} = 5.357]$ (By Interpolation)

A = 1.4 ×20+0.2×20+1.2×0.25×13= 35.9m^2

$F_w^T = P_z \times A \times G \times C_D$= 940.6*35.9*2*1.1 = 74.288 KN

Wind load in transverse direction on a bearing (F_W^T) = 74.288 /6 = 12.38 KN

Wind load in longitudinal direction of bridge (F_W^L) = 0.25 × F_W^T = 0.25 x 74.288 = 18.572KN

Wind load in longitudinal direction on a bearing (F_W^L) = 18.572/6 = 3.095KN

Wind load in vert. dir. of bridge (F_W^V) = $P_Z \times A \times G \times C_L$

= 940.6 × (20 x 9) × 2 × 0.75

= 253.962 KN

Wind load in vertical direction on a bearing (F_W^V) =253.962/6 = 42.327 KN

ertical reaction per bearing$=\frac{14.819}{3}$ =4.939KN

e. Seismic Load

Seismic load (F_S^h) $= \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} \times W$ [Refer Cl. 219, IRC 6]

Take, Seismic Zone - V, Soil Strata - Medium, Damping - 5 %, Bridge Class - Normal

Where,

$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} = 0.225$, Z = 0.36, I = 1, R = 2,

$\frac{S_a}{g}$=2.5(for medium soil type)

Annex (IRC 6) For calculation of $\frac{S_a}{g}$

$= \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} = \frac{0.36}{2} \times \frac{1}{2} \times 2.5 = 0.225$

Fundamental Time Period (T) = $2\sqrt{\frac{D}{100F}}$

D= Appropriate DL of Superstructure and LL in KN

F= Horizontal force to be applied at C.M of Superstructures for earthquake in transverse direction and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

Longitudinal = W_L =DL from superstructure = 2588.96KN

Transverse = W_T = W_L +0.2x(2x27+2x114+4x68) = 2588.96KN +110.8KN

=2699.76KN

Seismic Load (Longitudinal) = $\frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} x W_L$

= 0.225x2588.96

= 582.516KN

Seismic Load (Transverse) = 2699.76x0.225 = 607.446KN

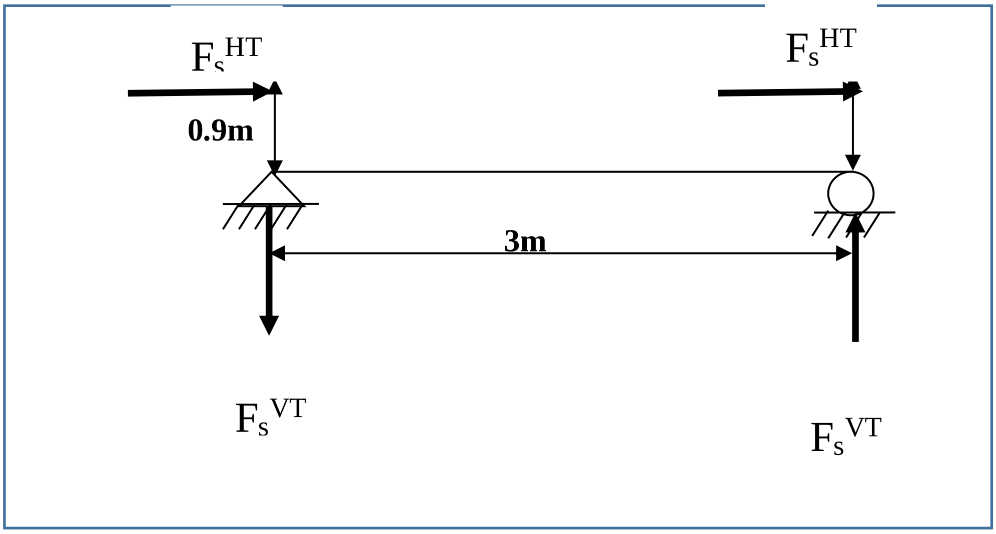
Seismic load in transverse direction on a bearing (F_{ShT}) = 607.446/6 = 101.24KN

Seismic load in longitudinal direction on a bearing (F_{ShL}) = 582.516/6 = 97.086KN

Vertical reaction due to seismic load on support of bridge (F_S^v)

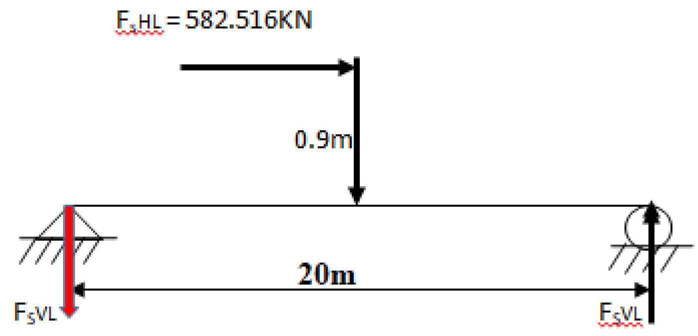
Seismic loads acts on c. g. of seismic weight. It creates additional vertical load on bearing. Consider c. g. of seismic weight = 0.9 m from bearing.

V. reaction on a bearing when s. load acts in tr. dir. (F_S^{VT}) $\frac{101.24*0.9*2}{3}$ = 60.774KN



Vertical react. on a bearing when seismic. load acts in long. dir.

(F_S^{VL}) $\frac{582.516*0.9}{20*3}$ = 8.737KN



f. Load due to temperature variation, creep and shrinkage effect

Maximum horizontal force on a bearing (Fcst) = $\frac{\Delta}{h_0} \times G \times \frac{A}{2} = \frac{5}{64} \times 1 \times \frac{150304}{2}$ =11.75 KN

Where,

- Strain due to temp., creep and shrinkage = 5×10^{-4} [Refer IRC 83 Part II Cl. 916.3.4]
- Horizontal deformation of bearing (Δ) = $5 \times 10^{-4} \times 20 \times 10^3 \times ½$ = 5 mm
- Shear modulus of elastomer (G) = 1 N/mm^2 [Refer IRC 83 Part II, Cl. 915.2.1]
- Preliminary height of bearing (h_0) = 64 mm
- Preliminary effective sectional area of bearing (A) = b × l =308 × 488 = 150304 mm^2

Load Combinations [IRC 6 Table 1]

Calculation of Loads on Bearing According to Combination of Loads

Vertical and horizontal loads subjected to bearing in the direction of traffic are only taken for design.

Combination I [N]

Total Vertical load = $DL^{Sup} + LL + F_{br}^{V}$ = 431.493 + 457.490 + 4.939 = 893.92KN

Total Horizontal load = F_{br}^{H} = 18.47 KN

Combination II (A) [N+T]

Total Vertical load = DL + LL + F_{br}^{V} = 893.92KN

Total Horizontal load = $F_{br}^{H} + F_{cst}$ = 18.47 + 11.75 = 26.27 KN

Combination III (A) [N+T+W]

Total Vertical load = DL + LL+ $F_{br}^{V} + F_{W}^{V}$ = 893.92+42.327= 936.247 KN

Total Horizontal load

= $F_{br}^{H} + F_{cst} + F_{W}^{L}$ = 3.095+26.27 = 29.365 KN

Combination VI[N+T+S]

Total Vertical load = D L+ 0.2 × LL + 0.5 × F_{br}^{V} + F_{s}^{vL}

=431.493+0.2*457.490+0.5*4.939+8.73

=534.190 KN

Total Horizontal load = 0.2 × F_{br}^{H} + F_{cst} + F_{s}^{hL} = 0.2 × 18.47 + 11.75+ 97.086

= 112.53KN

7.5.2. Design of Elastomeric Pad Bearing for Combination I [N]

a. Geometrical design

In geometrical design, approximate length, breadth and thicKNess of elastomeric pad and number, thicKNess and cover of steel laminates are found. Geometrical design is carried out using the guidelines of IRC.

[Refer standard plan dimensions of IRC 83, Part II, Cl. 916.2, Appendix I]

N_{min} = DL = 431.493 KN; Nmax = Tot. Vertical load on bearing = 893.92KN

H = 18.47 KN

Take,

b_0 = 320 mm, b = 308 mm

l_0 = 500 mm, l = 488 mm

h_i = 12 mm

$h_e = h_i/2$ = 6 mm

h_s = 4 mm

n = 3

c = 6 mm

$h_0 = (n+1)h_s + nh_i + 2h_e$ = 4*4+2*6+3*12=64mm

$h = nh_i + 2h_e$ = 3*12+2*6=48mm

Check geometry of bearing

$l_0 / b_0 \leq 2$ Ok

h = 48 < b_0/5 =320/5= 64 Ok

> b_0/10 =320/10= 32 Ok

$s = \frac{l*b}{2h_i(l+b)}$ = 7.867 > 6 and < 12 <OK>

Bearing stress in concrete ≤ Allowable Bearing stress <OK>

BS in concrete (σ_m) $= \frac{N_{max}}{l*b} = \frac{893.92*10000}{488*308} = 5.95 N/mm^2$

Allowable BS $= 0.25 \times f_{ck} \times \sqrt{\frac{A_1}{A_2}} = 0.25 \times 30 \times \sqrt{2} = 10.60\ N/mm^2$

<OK>

b. <u>Structural design</u>

Bearing is further checked for translation, rotation, friction and shear

Check for translation (IRC 83 PART II Cl. 916.3.4)

Design strain in bearing (γ_d) $<$ 0.7

$\gamma d = \gamma bd = \frac{\Delta_{bd}}{h} + \tau_{md} = 0.078 + 0.122 = 0.2 < 0.7$ **<OK>**

Where,

$$\frac{\Delta_{bd}}{h} = 5 * 10^{-4} * 20 * \frac{10^3}{2*64} = 0.078$$

$$\tau_{md} = \frac{H}{A*G} = \frac{18.47*10^3}{308*488*1} = 0..122;$$ G=shear mod of Elasticity=1 N/mm^2

Check for rotation (IRC 83 PART II Cl. 916.3.5)

Design rotation in bearing (α_d) $\leq \beta n \alpha_{bimax}$

$$\alpha_d = \alpha_d{}^{DL} + \alpha_d{}^{LL} = \frac{400 \times M_{DL} \times L}{\frac{E_c}{2} \times I_{gr}} \times 10^{-3} + \frac{400 \times M_{LL} \times L}{E_c \times I_{gr}} \times 10^{-3}$$

$$=\left(\frac{500*5000.39*20}{12500*0.3586} + \frac{500*1326.01*20}{25000*0.3586}\right)*10^{-3}$$

$= 0.00985 < \beta n \alpha_{bimax}$

$\beta = 0.1\sigma_m = 0.1 \times 5.95 = 0.595$

$n = 3$

$\sigma_m{}^{max} = 10$

$$\alpha_{bimax} = \frac{0.5\ \sigma_m{}^{max} \times h_i}{b \times s^2}$$

$$= \frac{0.5*10*12}{308*7.867^2} = 0.00315$$

$\beta n \alpha_{bimax} = 0.5950*3*0.00315 = 0.00562$

(ok)

c. Check for friction (IRC 83 PART II Cl. 916.3.6)

Design strain in bearing (γ_d) $\leq 0.2 + 0.1\sigma_m$

Normal stress in bearing (σ_m) > 2 N/mm^2 and $\leq$ 10 N/mm^2

Where, $\gamma_d = 0.2$

$< 0.2 + 0.1\sigma_m = 0.2 + 0.1 \times 5.95 = 0.795\ N/mm^2$ <OK>

$\sigma_m = 5.95 N/mm^2 > 2\ N/mm^2$ and $< 10\ N/mm^2$ <OK>

$\gamma d \leq 0.2+0.1\sigma m$

d. Check for shear stress (IRC 83 PART II Cl. 916.3.7)

Total shear stress $\leq$ 5 N/mm^2

$\tau_c + \tau_r + \tau_\alpha = 2.8236$ N/mm^2 < 5 N/mm^2

Where, Shear stress due to axial compression (τ_c)=$1.5*\frac{\sigma_m}{s} = 1.5 * \frac{5.95}{7.867}$

= 1.134 N/mm^2

Check of Elastomeric Pad Bearing for Combination VI [N+T+S]

Loads are not varied significantly in first three combinations of loads. But in seismic combination i.e. in N + T + S case, horizontal load is about two times greater than other combinations. So designed bearing has been checked for seismic combination (along the traffic) of loads only.

N_{min} = 431.493 KN

N_{max} = =534.190 KN

H =112.53KN

Check bearing stress in concrete

Bearing Stress in concrete $\leq$ Allowable bearing stress in concrete

Where,

Bearing stress in concrete (σ_m) ==$\frac{N_{max}}{l*b}$ =$\frac{534.190*10000}{308*488}$=3.55 N/mm^2

Allowable bearing stress in concrete = $0.25 \times f_{ck} \times \sqrt{\frac{A_1}{A_2}} = 0.25 \times 30 \times \sqrt{2}$= 10.60 N/mm^2

Check bearing for translation

Total strain in bearing (γ_d) < 0.7

Where,

$$\gamma d = \gamma bd = \frac{\Delta_{bd}}{h} + \tau_{md}$$

$$\frac{\Delta_{bd}}{h} = 0.078$$

$$\tau_{md} = \frac{H}{A*G} = \frac{112.53*10^3}{308*488*1} = 0.748$$

$\gamma_{d =}$ 0.078+0.748= 0.826 > 0.7 <NOT OK>

Size of bearing provided for loads of combination I (N) is not sufficient for loads of combination VI (N+T+S). Check for translation shows that provided size of bearing could not accommodate the horizontal force of combination VI. In the situation it is required to provide elastomeric pad bearing with pin on one side of support to resist horizontal load

Diameter of Pin

Shear stress in pin due to horizontal loads $\leq$ Allowable shear stress in pin ($\tau \leq \tau_a$)

Take pin of Fe250 grade

$$\frac{\text{Horizontal Loads}}{\text{Cross Section Area of Pin}} = \frac{112.5*10^3}{\pi r^2} = 0.4f_y\text{; Where } f_y = 250$$

$$r = 18.92\ mm$$

Provide 40 mm dia. stainless steel pin. Pin should be extended up to the depth of cap of support.

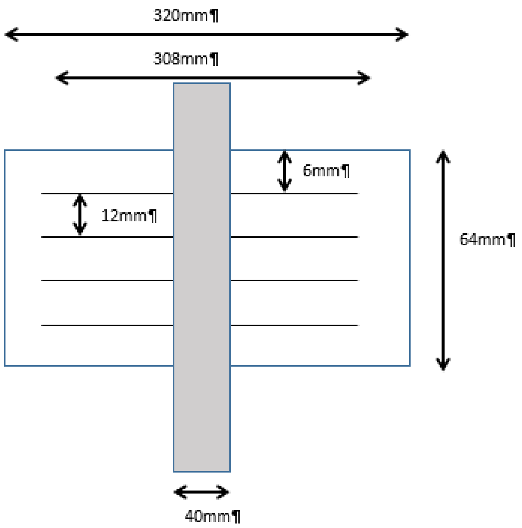
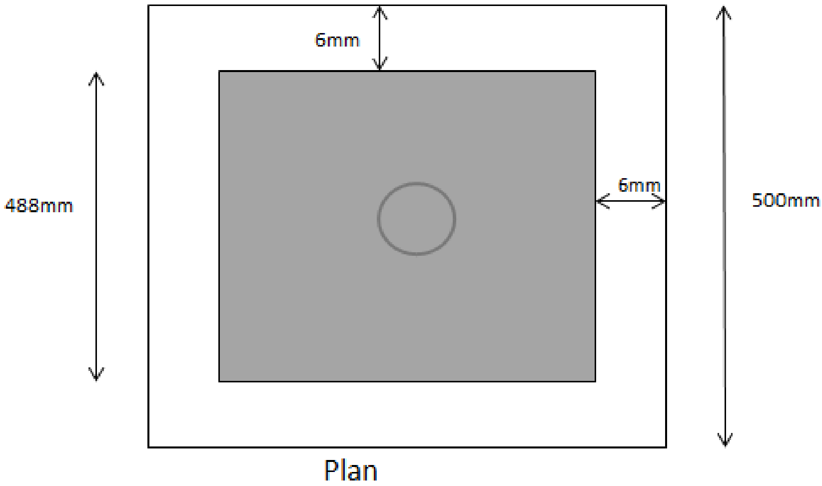


Fig7.58: cross section of bearing



Provide 64*320 mm bearing which will satisfy the Transitional failure at one side and 500*320 mm with pin at the other side.

Design Summary

Type of bearing	elastomeric pad bearing
Size of bearing	500*320*40
Diameter of pin	40mm
Material of pin	Fe 250

7.6. Design of RC Abutment with Spread Footing

RC Abutment for a 20 m span T-Beam Bridge to meet the following requirements.

Type of Bridge – T-Beam bridge of 20 m span

Carriage way – Two lane without footpath

Height of Abutment

Scour depth $(D_{sd})=1.34*(D_b^2/k_{sf})^{0.33}$

Where

D_b = (discharge/span)=48.647/2.34=2.43m^3/ms

K_{sf} = silt factor=1.25

Therefore

D_{sm} = 2.24m

Again max depth of scour below HFL is taken as

D=1.5*2.24=3.37m

Let assume free board =1.5+affluxe

=1.5+0.5=2m

Therefore total height of abutment =2+3.37=5.37

=5.5m

Back fill characteristics =Angle of internal friction of soil ($\emptyset$) = 40°

Unit weight of soil (γ) = 18 KN/m3

Backfill slope (i) = 0°

Bearing size = 320 mm × 500 mm × 64 mm

7.6.1. Analysis and Design of Abutment of Right Bank

7.6.1.1. Planning and Preliminary Design

a. Selection of Type of Abutment

Abutment may be of masonry or reinforced cement concrete. Masonry is technically / economically feasible up to 5m height of abutment. In the particular case, abutment is greater than 5 m height. So reinforced concrete wall type abutment has been selected.

b. Material Selection

M20 grade of concrete for abutment stem

M25 grade of concrete for abutment cap

Fe 415 HYSD bars for all RC work

c. Seating Width

Minimum seating width = 305+2.5L+10H mm [Cl.219.9 IRC 6]

= 305+2.5x20+10x5.5 = 410 mm

Seating width ≥ Bearing width + 150 mm +Projection of cap + width of expansion joint
Where,

Width of expansion joint $\geq$ 20x10^{3}x0.000011/$°C$ x 30 x$\frac{1}{2}$= 3.333 mm

$\geq$ 5x10^{-4}x20x10^{3}x$\frac{1}{2}$ = 5 mm

$\geq$ 20 mm

Adopting width of expansion joint = 40mm

Required seating width

= (320+150+75+40)

=585mm

Therefore seating width =585mm

d. Height of dirt wall

= depth of girder + Height of bearing – thickness of approach slab

= 1.4+0.064-0.3 = 1.164m

e. Thickness of dirt wall

≥ 200 mm and $\frac{Height\ of\ dirt\ wall}{7}$= 166 mm

Adopting thickness of dirt wall = 200 mm = 0.2m

f. Width of stem of abutment

Width of stem =H/10 =550 mm

Width of stem ≥ thickness of dirt wall + seating width-projection

≥ 0.2+0.585-0.075

≥ 0.710 m

Adopting width of stem = 0.710m

g. Thickness of footing

= H/8 =687.5 mm

Therefore adopt = 800 mm

h. Width of footing

B≈ 0.75 H = 0.75x5.5 = 4.125m

$$B \geq \frac{Area\ of\ footing}{Length\ of\ footing} \approx \frac{1.5xTotal\ vertical\ loads}{Allowable\ bearing\ capacity\ of\ soil\ x\ length\ of\ footing}$$

Adopting B = 4.5m

i. Thickness of Abutment cap

= 0.3 m

Minimum thickness of cap = 0.2m

j. Length of abutment

Length of abutment ≥ c/c distance between girders + length of Bearing + 2 x clearance

≥ 6 + 0.5 + (2x0.4) =7.3m

Providing 7.3 m of abutment

k. Size of approach slab

= 4mx9mx0.3 m

l. Diagram

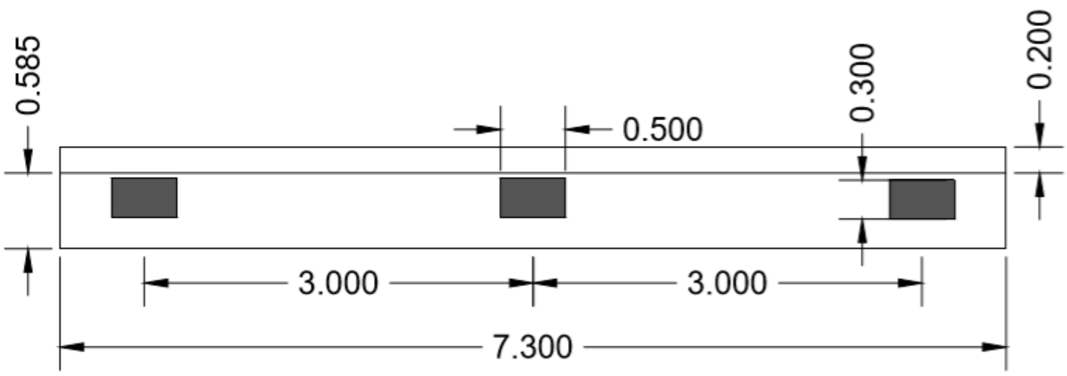


fig: Plan of bearing and cap of abutment

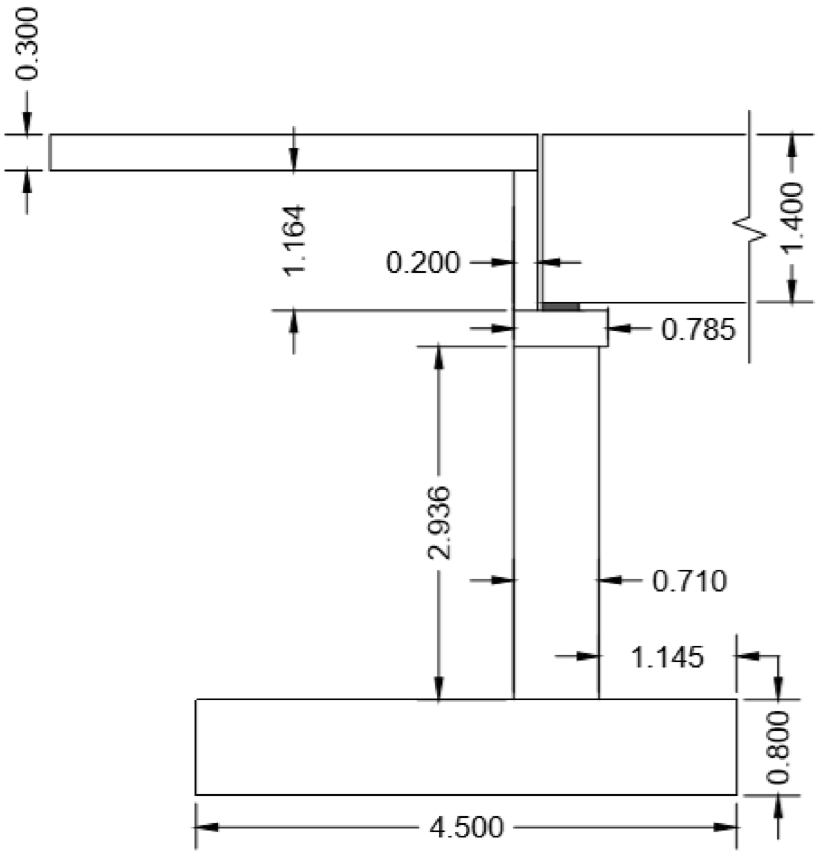


Fig7.59: Abutment X-section

7.6.1.2. <u>Analysis and design of Abutment Stem</u>

<u>Load Calculation(calculation from design of bearing)</u>

a. <u>Dead load from superstructure</u>

Total dead load = 431.49*6

= 2588.94 KN

Load on abutment per unit length (Dlss) = $\frac{2588.94}{7.3x2}$ = 177.32 KN/m

b. <u>Weight of approach slab</u>

Taking half of total weight of approach slab = 0.3*9*4*25x$\frac{1}{2}$= 135 KN

Load on abutment per unit length (Dl_{Aps})=$\frac{135}{7.3}$ =18.5 KN/m

c. <u>LL from superstructure</u>

Class A is applied:

Two train of class A

Impact factor = 1.173

LL = 114x (1+0.94) + 68x2x (0.725+0.575+0.425+0.275) = 357.16 KN

Load on abutment per unit length (LL)= $\frac{357.16*1.17}{7.73}$ = 57.24 KN/m

Adopting, Load on abutment per unit length (LL) = 57.24 KN/m

d. Load due to braking effort

Horizontal breaking load per unit length (F_{Br}^{H})= $\frac{112.44}{7.3x2}$= 7.7 KN/m

Vertical reaction due to braking load per unit length (F_{Br}^{V})= $\frac{4.939*3}{7.3}$ =2.029 KN/m

e. Wind Load

Transverse wind load per unit length (F_W^T) = $\frac{74.288}{7.3x2}$ = 5.08 KN/m

Longitudinal wind load per unit length (F_W^L) = $\frac{18.572}{7.3x2}$ = 1.27 KN/m

Vertical wind load per unit length (F_W^V) = $\frac{253.962}{7.3x2}$ = 17.39 KN/m

f. Seismic load due to the DL and LL from superstructure

Seismic load in longitudinal direction ($FshL$) = 582.516 KN

Seismic load in transverse direction (Fsht) = 604.44 KN

Seismic load in transverse direction per unit length (Fsht) = $\frac{607.44}{7.3x2}$ =41.605 KN/m

Seismic load in longitudinal direction per unit length (Fshl) = $\frac{582.516}{7.3x2}$ = 39.898 KN/m

Vertical reaction on abutment per unit length when seismic load acts in transverse direction (Fsvl) = $\frac{8.737*6}{2*7.3}$ = 3.59 KN/m

Vertical reaction on abutment per unit length when seismic load acts in longitudinal direction (Fsvt) = $\frac{61.062*6}{2*7.3}$ =25.09KN/m

g. Load due to temperature variation, creep and shrinkage effect

Load on three bearing due to FCST = 11.75*3 = 35.25 KN

Load per unit length (Fcst) = $\frac{35.25}{7.3}$ = 4.828 KN/m

h. Self-weight of Abutment

Self-weight = (1.4*0.2+0.3*0.785+0.71*0.2936)*7.3*25 = 474.51 KN

Load per unit length (DlAb) = $\frac{474.51}{7.3}$ = 65 KN/m

i. Seismic load due to the self-weight of abutment

$$(F_{sAbt}^{ht}) = \frac{Z}{2} x \frac{I}{R} x \frac{S_a}{g} x\ w$$

$$= \frac{0.36}{2} x \frac{1}{3} x 2.5 x 47451$$

$$= 7.1176 \text{ KN}$$

Load per unit length $(F_{sAbt}{}^{bt}) = \frac{7.1176}{7.3}$ =0.975 KN/m

j. Load due to static earth pressure

Load due to active earth pressure has been found Coulomb's Theory.

Where,

$P_A = 0.5 \times \gamma_{soil} \times H^2 \times K_A$

Where

$$K_A = \frac{sin^2(\beta+\emptyset)}{Sin^2\beta \times Sin(\beta-\delta)} \times \frac{1}{\left(1+\sqrt{\frac{Sin(\emptyset+\delta).Sin(\emptyset-i)}{Sin(\beta-\delta)\times SIn((\beta)+i)}}\right)^2}$$

$$= 0.199$$

$P_A = 0.5 \times 18 \times 5.5^2 \times 0.199 = 54.18$ KN/m

Where,

$\emptyset = 40^0$, $i = 0^0$, $\beta = 90^0$, $\delta = (2/3)*40 = 26.67^0$, $\gamma_{soil} = 18$ KN/m^3, H= 5.5 m

Horizontal component of load per unit length	$P_{EP}{}^{H(s)} = P_a x cos(26.67^\circ) = 48.62$ KN/m
Vertical component of load per unit length	$P_{EP}{}^{V(s)} = P_a x sin(26.67^\circ) = 23.89$ KN/m

This load acts at 0.42H from base of foundation i.e. 0.42x5.5 = 2.31m

k. Load due to dynamic earth pressure

Load due to active earth pressure has been found by Mononobe Okabe Theory.

$P_A = 0.5 \times \gamma_{soil} \times H^2 \times K_A{}^{dyn}$

Where,

$$K_A^{dyn} = (1 \pm \alpha_v)\frac{Cos^2(\varphi-\alpha-\psi)}{Cos\psi \times Cos^2\alpha \times Cos(\delta+\alpha+\psi)} \times \frac{1}{\left\{1+\left(\frac{Sin(\varphi+\delta)x\,Sin(\varphi-i-\psi)}{Cos(\alpha-i)x\,Cos(\alpha+\delta+\psi)}\right)^{\frac{1}{2}}\right\}^2}$$

$\varphi = 40^0$, $i = 0^0$, $\delta = 2/3 \times 40^0 = 26.67^0$, $\alpha = 0^0$

$\alpha_h = \frac{Z}{2}\times\frac{I}{R}\times\frac{S_a}{g} = \frac{0.36}{2}\times\frac{1}{2} \times 2.5 = 0.15$

$\alpha_v = \alpha_h \text{ x } \frac{2}{3} = 0.15 \text{ x } \frac{2}{3} = 0.1$

$\psi = \tan^{-1}\left(\frac{\alpha_h}{1+\alpha_v}\right) = 7.765°$

$$\therefore K_A^{dyn} = 0.372$$

γ_{soil} = 18 KN/m^3, H= 5.5 m

P_A= 0.5 × 18 × 5.5^2 ×0.372 = 101.277 KN/m

Horizontal component of load per unit length (P_{sdh}) = PA cos(26.6°)

=90.555KN/m

Vertical component of load per unit length (P_{sdv}) = PA sin(26.26°)

= 45.34 KN/m

This load acts at 0.6H from base of foundation i.e. 0.6x5.5 = 3.3m

l. **Surcharge load** [cl.214.1 IRC6]

1.2m earth fill from road surface is taken as surcharge load.

P_{sur}= $K_a x \gamma_s x h x W$= 0.199x18x1.2x5.5 = 23.6412 KN/m

Horizontal component of load per unit length $P_{Sur}{}^{H}$= P_{Sur}cos(26.67°) = 21.12 KN/m

Vertical component of load per unit length $P_{Sur}{}^{V}$= P_{Sur}sin(26.67°) = 10.61 KN/m

m. Backfill weight on heel slab of footing

W_{BF}= (5.5-0.8-0.3) x 2.645x18 = 209.48 KN/m

Load per unit length W_{Bf} =209.48 KN/m

n. Weight of Footing

$W_{Footing}$=0.8x4.5 x 25 = 90 KN/m

7.6.1.3. Analysis of abutment stem

In this stage, loads on the abutment stem and dirt wall are assessed for different combination of loads, maximum responses to loads at critical sections are found, stability of abutment is checked and critical section of abutment are designed and detailed for maximum responses.

(Refer IRC 6, 21, 78 and IRC 112, IS456, SP16 and SP34 for RC design and detailings)

In the example, responses of abutment at bottom and 1.5m from the bottom for basic combination and seismic combination of loads have been calculated. Loads taken are vertical and longitudinal loads. Although seismic and wind loads in traverse direction are greater than in longitudinal direction, reduction in earth pressure and other loads in transverse direction in earth pressure and other loads in traverse direction makes the longitudinal direction load critical .

(Refer annex B, IRC 6-2010)

Responses of abutment at its bottom in basic combination of loads

Load(KN)		Υ_f	Eccentricity		P_u(KN)	M_{uy}(KNm)	M_{ux}(KNm)	Hy
			x(m)	y(m)				
DL_{ss}	177.32	1.35		0.045	234.7785	10.565	0	0
Dl_{aps}	18.5	1.35		-1.155	24.975	-28.84	0	0
LL	57.24	1.5		0.045	85.86	3.86	0	0
F_{br}^{H}	7.7	1.15	3.236		8.72735	0	28.24	8.73
F_{br}^{V}	2.029	1.5		0.045	3.3285	0.149	0	
F_W^{L}	1.27	1.5	3.236		1.905	0	6.16	1.91
F_{cst}	4.828	1.35	3.236		6.5178	0	21.09	6.51
Dl_{ab}	65	1.5		0	97.5	0	0	
$P_{EP}{}^{H(s)}$	23.89	1.5		-0.355	35.835	-12.721	0	
$P_{EP}{}^{V(s)}$	48.62	1.5	1.51		72.93	0	110.12	72.93
P_{sur}^{H}	21.126	1.2	1.95		25.3512	0	49.43	25.35
P_{sur}^{V}	10.611	1.2		-0.355	12.7332	-4.52	0	
				Total	610.44	-31.51	215.05	115.43
					Total B.M	184.155 KNm		

Response of abutment at its bottom in seismic combination of loads

Load(KN)		Υ_f	Eccentricity		P_u(KN)	M_{uy}(KNm)	M_{ux}(KNm)	Hy
			x(m)	y(m)				
DL_{ss}	177.32	1		0.045	177.32	7.82595	0	0
Dl_{aps}	18.5	1		1.155	18.5	-21.3675	0	0
LL	57.24	0.2		0.045	11.448	0.51516	0	0
F_{br}^{H}	7.7	0.2	3.236		1.5178	0	4.9116	1.5178
F_{br}^{V}	2.029	0.2		0.045	0.4438	0.019971	0	
F_{cst}	4.828	1	3.236		4.828	0	15.623	4.828
F_S^{HL}	39.898	1	3.236		39.898	0	126.61	39.126
F_S^{VL}	3.59	1		0.045	3.59	0.1584	0	
Dl_{ab}	65	1		0	65	0	0	
F_{sAbt}^{bt}	0.975	1		0	0.975	0	0	
P_{sdv}	45.34	1		0.355	45.34	-16.0957	0	
P_{sdh}	90.55	1	2.5		90.55	0	226.38	90.55
P_{sur}^{H}	21.126	0.2	1.95		4.2252	0	8.2391	4.2252
P_{sur}^{V}	10.611	0.2		0.355	2.1222	-0.75338	0	
				Total	461.506	-29.6971	381.76	140.25
				Total B.M	354.788	KNm		

Responses of abutment at 1.5m of stem from its bottom in basic

Load(KN)		Υ_f	Eccentricity		P_u(KN)	M_{uy}(KNm)	M_{ux}(KNm)	Hy
			x(m)	y(m)				
DL_{ss}	177.32	1.35		0.045	234.7785	10.56503	0	0
Dl_{aps}	18.5	1.35		-1.155	24.975	-28.8461	0	0
LL	57.24	1.5		0.045	85.86	3.8637	0	0
F_{br}^{H}	7.7	1.15	1.736		8.72735	0	15.15	8.7
F_{br}^{V}	2.029	1.5		0.045	3.3285	0.149783	0	
F_W^{L}	1.27	1.5	1.736		1.905	0	3.307	1.9
F_{cst}	4.828	1.35	1.736		6.5178	0	11.314	6.5
Dl_{ab}	65	1.5		0	97.5	0	0	
$P_{EP}{}^{H(s)}$	23.89	1.5		-0.355	35.835	-12.7214	0	
$P_{EP}{}^{V(s)}$	48.62	1.5	0.01		72.93	0	0.729	73
P_{sur}^{H}	21.126	1.2	0.45		25.3512	0	11.408	25
P_{sur}^{V}	10.611	1.2		-0.355	12.7332	-4.52029	0	
				Total	610.4416	-31.5093	41.91	115
				Total		10.4	KNm	

Responses of abutment at 1.5m from its bottom of stem in Seismic Combination of loads

Load(KN)		Υ_f	Eccentricity		P_u(KN)	M_{uy}(KNm)	M_{ux}(KNm)	Hy
		x(m)	y(m)					
DL_{ss}	177.32	1		0.045	173.91	7.826	0	0
Dl_{aps}	18.5	1		-1.155	18.5	-21.4	0	0
LL	57.24	0.2		0.045	11.448	0.515	0	0
F_{br}^{H}	7.7	0.2	1.736		1.5178	0	2.635	1.52
F_{br}^{V}	2.029	0.2		0.045	0.4438	0.02	0	
F_{cst}	4.828	1	1.736		4.828	0	8.381	4.83
F_S^{HL}	39.898	1	1.736		39.126	0	67.92	39.1
F_S^{VL}	3.59	1		0.045	3.52	0.158	0	
Dl_{ab}	65	1		0	65	0	0	
F_{sAbt}^{bt}	0.975	1		0	0.975	0	0	
P_{sdv}	45.34	1		-0.355	45.34	-16.1	0	
P_{sdh}	90.55	1	1		90.55	0	90.55	90.6
P_{sur}^{H}	21.126	0.2	0.45		4.2252	0	1.901	4.23
P_{sur}^{V}	10.611	0.2		-0.355	2.1222	-0.75	0	
				Total	461.51	-29.7	171.4	140
				Total B.M=141.6933				

7.6.1.4. Design of abutment stem

Max axial load = (610.4461-115.434)

= 495.0076KN

Also, $0.1*f_{ck}*A_c$=(0.1*20*710*1000)/1000

=1420>495.0076KN

For these case, compression member is treated as flexural member so abutment is design as a cantilever slab.

Since design BM is higher in the seismic combination of load, design of abutment stem is carried for seismic combination of loads.

A. Design of bottom section

Moment at bottom section =354.788 KNm *(from table above)*

Check depth of slab

Let us provide clear cover of 50mm and dia of 25mm.

Taking M20 Fe415

d = D – CC – ø/2 = 710 – 50 – 25/2 = 652.5 mm

$d_{bal} = \sqrt{\frac{M_u}{Q \times b}} = \sqrt{\frac{354.788\times10^6}{3.457\times1000}}$ = 357.202 mm

Where, Q = 0.36 fck × 0.48 × (1 – 0.416 × 0.48) = 3.457

$d_{prov}>d_{bal}$ hence, assumed depth is sufficient.

Find reinforcing bars

Since $d_{prov}>d_{bal}$, section is designed as Singly Reinforced Under-Reinforced Section (SRURS). Section design has been carried out by using IS 456:2000.

a. Main vertical bars (vertical bars in the side of backfill)

Area of reinforcement

$$= \frac{M}{0.87\times fy\times(d-0.42xu)}$$

$$= 354.788 * \frac{10^6}{0.87 * 415 * ((710 - 62.50) - 0.42 * 0.48 * 647.5)}$$

A_{st} = 1886.227mm^2

% of steel =0.29%

Spacing of reinforcement $=\frac{\pi x\frac{25^2}{4}\times1000}{1886.227}$ =260mm

Providing 25mm diameter bar @250mm c/c spacing.

Area provided $=\frac{\pi x25^2}{250}x1000$=1943.75mm2

b. <u>Outer vertical reinforcement (vertical bars in the side of river)</u>

[Refer detailing criteria of IRC 112-2011 and IS 4]

Take 0.12 % of gross sectional area of abutment as outer vertical reinforcement

As $=\frac{0.12}{100} \times 1000 \times 710$ = 852 mm2

Provide 16mm diameter bar,

Spacing required = $\frac{\pi x 16^2}{4 \times 852} \times 1000$

=235.86mm

Provide 16 mm Øbar @ 230 mm c/c spacing .

c. <u>Horizontal Reinforcement</u>

Take, As = 0.1% of stem area of abutment or 25% of main vertical bars

= 0.001 × 710×2936 = 2084.56 mm2

Provide 12mm Ø bar,

Spacing of the reinforcement $=\frac{\pi * \frac{12^2}{4}}{2084.56} * 1000$ = 54.223mm

Provide 12 mm Øbar @ 50 mm on each face of abutment

<u>Check bottom section for shear</u>

Check $\tau_{uv} \leq$ K τ_{uc}

Where,

$\tau_{uv} = \frac{H_y}{\text{bd}} = \frac{140.427 \times 1000}{1000 \times 647.5}$ = 0.216 N/mm^2

For M20 and pt =0.3% (table 19 of IS 456)

τ_{uc} = 0.24 N/mm^2

$\tau_{uc,\,\max}$ = 2.8 N/mm^2

K =1 [Refer table 19 & 20, IS 456]

Sinceτ_{uv}< K τ_{uc}, shear reinforcement is not required.

Design of abutment section at mid span of stem (2.65m) from its bottom

Moment at 1.5m from bottom of stem =141.693KNm (from above table)

We have,

$M_{ulim} = 0.138 f_{ck} bd^2$

$=0.138 \times 20 \times 1000 \times 647.5^2$

=1157.14KNm>141.693KNm

Which show that singly reinforced section is design.

B. Design of abutment at 1.5m from bottom

a. Main vertical bars (vertical bars in the side of backfill)

Area of reinforcement

$$= \frac{M}{0.87 \times fy \times (d - 0.42xu)}$$

$$= 141.693 * \frac{10^6}{0.87 * 415 * ((710 - 62.50) - 0.42 * 0.48 * 647.5)}$$

$=759.13mm^2$

Spacing of reinforcement $= \frac{\pi x \frac{25^2}{4} \times 1000}{759.13}$ =646.29mm

Providing 25mm diameter bar @300mm c/c spacing

Horizontal reinforcement is same as in bottom on both face of reinforcement.

7.6.1.5. Analysis and design of dirt wall

Design of Dirt Wall

Material: M20 & Fe 415

Preliminary Size: 0.2m x 1.164m x 9 m

The dirt wall is designed as a cantilever slab of span 1.164 m. To determine the responses of the dirt wall, basic combination and seismic combination of loads are considered.

Surcharge load = 1.2 x 18 x k_a x 1.164 x1 = 1.2 x18 x1 x 0.199×1.164 = 5 kN/m

Load due to earth pressure

$$= \frac{1}{2} \times K_A^{dyn} \times \gamma_{soil} \times h^2 \times 1 = \frac{1}{2} \times 0.372 \times 18 \times 1.164^2 = \mathbf{4.536 kN/m}$$

Seismic load due to weight of the dirt wall (A_hxW) = $\frac{0.36}{2}$ x $\frac{1}{3}$x2.5x1.164x0.2x1x25

= 0.875 KN/m

Mu at bottom = 5 x $\frac{\mathbf{1.164}}{\mathbf{2}}$ + 4.536 x 0.6 x 1.164 + 0.873 x $\frac{\mathbf{1.164}}{\mathbf{2}}$ = 6.586 kN/m

Check depth of slab

Let us provide clear cover of 40mm and dia of 12mm.

Taking M20 Fe415

d = D – CC – ø/2 = 200 – 40 – 12/2 = 154 mm

$$d_{bal} = \sqrt{\frac{M_u}{Q \times b}} = \sqrt{\frac{6.586 \times 10^6}{3.457 \times 1000}} = 43.64 \text{ mm} < 200\text{mm ok}$$

Where, Q = 0.36 fck × 0.48 × (1 – 0.416 × 0.48) = 3.457

$d_{prov} > d_{bal}$ hence, assumed depth is sufficient.

Since $d_{prov} > d_{bal}$, section is designed as Singly Reinforced Under-Reinforced Section (SRURS). Section design has been carried out by using IS 456:2000

Area of reinforcement required

$$= \frac{M}{0.87 \times fy \times (d - 0.42xu)}$$

$$= 65.456 * \frac{10^6}{0.87 * 415 * ((200 - 46) - 0.42 * 0.48 * 154)}$$

A_{st} = 148.35mm^2

% of steel =0.096%<0.12% Hence, provide mimimum reinforcement.

A_{st} required =240mm^2

Spacing of reinforcement $= \frac{\pi x \frac{12^2}{4} \times 1000}{240}$ =471mm

Providing 12mm diameter bar @300mm c/c spacing.

7.6.1.6. <u>Stability check</u>

Stability of abutment is checked for overturning and sliding. In the following table overturning moment and restoring moment about the toe of footing of abutment and shear at the base of footing have been calculated.

(In the table below the symbol have following notation

OT= overturning

RT=restoring

OTM=overturning moment

RTM=restoring moment

ϒf= partial safety factor

SF=shear force

VF=vertical force)

Stability check for basic combination of load

Load(KN)		Ɣf OT	RT	Lever arm (m)	OTM	RTM	S.F	VF
DL_{ss}	177.3		0.95	1.455	0	240.39	0	165.21
Dl_{aps}	18.5		0.95	1.856	0	32.619	0	17.575
LL	57.24		0	1.455	0	0	0	0
F_{br}^{H}	7.7	1.15	0	4.036	35.22	0	8.7274	0
F_{br}^{V}	2.029		0	1.455	0	0	0	0
F_{W}^{L}	1.27	1.5	0	4.036	7.689	0	1.905	0
F_{cst}	4.828	1.5	0	4.036	29.23	0	7.242	0
Dl_{ab}	65		0.95	1.5	0	92.625	0	0
$P_{EP}{}^{H(s)}$	23.89			1.85	0	0	0	0
$P_{EP}{}^{V(s)}$	48.62	1.5		2.2	160.4	0	72.93	0
P_{sur}^{H}	21.13	1.2		2.75	69.72	0	25.351	0
P_{sur}^{V}	10.61			1.85	0	0	0	0
W_{BF}	209.5		0.95	3.177	0	632.25	0	199.01
W_F	90		0.95	2.25	0		0	85.5
				TOTAL	302.3	997.89	116.16	529.05

Total overturning moment = 302.309KN-m

Total restoring moment = 997.854KN-m

Total shear at base of footing = 116.16KN

Total vertical load at the base of footing = 529.05KN

CHECK

$$\frac{\text{Total restoring moment}}{\textit{Overturning moment}} = \frac{997.854}{302.33} = 3.3 > 2 \text{ OK}$$

$$\frac{\text{Total shear strength}}{\text{Total shear at base}} = \frac{529.049 \tan(40)}{302.33} = 3.82 > 1.5 \text{ OK}$$

Hence, abutment is safe in both overturning and slidding for basic combination of loads

Stability check for seismic combination of loads

Load(KN)		ϒf OT	RT	Lever arm (m)	OTM	RTM	S.F	VF
DL_{ss}	177.3		1	1.455	0	253	0	173.91
Dl_{aps}	18.5		1	1.856	0	34.34	0	18.5
LL	57.24		0	1.455	0	0	0	0
F_{br}^{H}	7.7	0.2		4.036	6.13	0	1.518	0
F_{br}^{V}	2.029		0	1.455	0	0	0	0
F_{cst}	4.828	0.5		4.036	9.74	0	2.414	0
F_S^{HL}	39.89	1		4.036	158	0	39.13	0
F_S^{VL}	3.59	0		1.455	0	0	0	0
Dl_{ab}	65		1	1.5	0	97.5	0	65
F_{sAbt}^{bt}	0.975	1		2.268	2.21	0	0.975	0
P_{sdv}	45.34			1.85	0	0	0	0
P_{sdh}	90.55	1		3.3	299	0	90.55	0
P_{sur}^{H}	21.13	1		2.75	58.1	0	21.13	0
P_{sur}^{V}	10.61			1.85	0	0	0	0
W_{BF}	209.5		1	3.177	0	665.5	0	209.48
W_F	90		1	2.25	0	202.5	0	90
				Total	532.91	1253.9	155.7	556.89

Total overturning moment = 532.91KN-m

Total restoring moment = 1252.9KN-m

Total shear at base of footing = 155.7KN

Total vertical load at the base of footing = 556.89KN

Check

$$\frac{\text{Total restoring moment}}{\textit{Overturning moment}} = \frac{1252.9}{532.91} = 2.35 > 2 \text{ OK}$$

$$\frac{\text{Total shear strength}}{\text{Total shear at base}} = \frac{556.894 * \tan(40)}{155.708} = 3.00 > 1.5 \text{ OK}$$

Hence, abutment is safe in both overturning and slidding for basic combination of loads

7.6.2. <u>Analysis and design of Spread Footing</u>

<u>Responses of footing at its base in basic combination of loads</u>

(Here DFC=distance from centroid

e=eccentricity along width of footing from centroid

P_u=vertical force +horizontal force

VF=vertical force

M_{TC}=moment through centroid axis of footing)

Load(KN)		ϒf	DFB	e	Pu	M_{TC}	Horizontal
DL_{ss}	177.3	1.35		0.795	234.779	186.65	
Dl_{aps}	18.5	1.35		-1.405	24.975	-35.09	
LL	57.24	1.5		0.795	85.86	68.259	
F_{br}^{H}	7.7	1.15	3.636		8.72735	31.733	8.72735
F_{br}^{V}	2.02	1.5		0.795	3.3285	2.6462	
F_W^{L}	1.27	1.5	3.636		1.905	6.9266	1.905
F_{cst}	4.828	1.35	3.636		6.5178	23.699	6.5178
Dl_{ab}	65	1.5		0.75	97.5	73.125	
$P_{EP}{}^{H(s)}$	23.89	1.5		0.395	35.835	14.155	
$P_{EP}{}^{V(s)}$	48.62	1.5	1.8		72.93	131.27	72.93
P_{sur}^{H}	21.126	1.2	2.35		25.3512	59.575	25.3512
P_{sur}^{V}	10.611	1.2		0.395	12.7332	5.0296	
W_{BF}	209.484	1.35		-0.9275	282.803	-262.3	
W_F	90	1.35		0	121.5	0	
			Total		1014.74	305.68	115.431
			VF		899.309		

From table,

Vertical force (P_u) =899.309KN

Total moment (M_u) =305.68KNm

Responses of footing at its base in seismic combination of loads

Load(KN)		Yf	DFB	e	Pu	Moment	Horizontal
DL_{ss}	177.3	1		0.795	173.91	138.26	
Dl_{aps}	18.5	1		-1.405	18.5	-25.99	
LL	57.24	0.2		0.795	11.448	9.1012	
F_{br}^{H}	7.7	0.2	3.636		1.5178	5.5187	1.5178
F_{br}^{V}	2.02	0.2		0.795	0.4438	0.3528	
F_{cst}	4.828	1	3.636		4.828	17.555	4.828
F_S^{HL}	39.898	1	3.636		39.126	142.26	39.126
F_S^{VL}	3.59	1		0.795	3.52	2.7984	
Dl_{ab}	65	1		0.75	65	48.75	
F_{sAbt}^{bt}	0.975	1	1.868		0.975	1.8213	0.975
P_{sdv}	45.34	1		0.395	45.34	17.909	
P_{sdh}	90.55	1	2.9		90.55	262.6	90.55
P_{sur}^{H}	21.126	0.2	2.35		4.2252	9.9292	4.2252
P_{sur}^{V}	10.611	0.2		0.395	2.1222	0.8383	
W_{BF}	209.484	1		-0.9275	209.484	-194.3	
W_F	90	1		0	90	0	
				Total	760.99	437.4	141.222
				VF	619.768		

From table above,

Vertical force (P_u) =619.768KN

Total moment (M_u) =437.4KNm

Calculation of soil pressure

For basic combination

Soil pressure(p_u)= $\frac{P_u}{A} \pm \frac{M'_{ux} \text{ x } y}{I_{x'x'}} = \frac{899.31\times1000}{1000\times4500} \pm \frac{305.68\times10^6}{3.375\times10^9}$ = 290 KN/m^2 and 190 KN/m^2

Similarly

Seismic combination

Soil pressure(p_u) = $\frac{P_u}{A} \pm \frac{M'_{ux} \text{ x } y}{I_{x'x'}} = \frac{619.765\times1000}{1000\times4500} \pm \frac{437.4\times10^6}{3.375\times10^9}$ = 267 KN/m^2 and 138 KN/m^2

Bearing strength of soil =300KN/m^2>290KN/m^2 .Hence safe

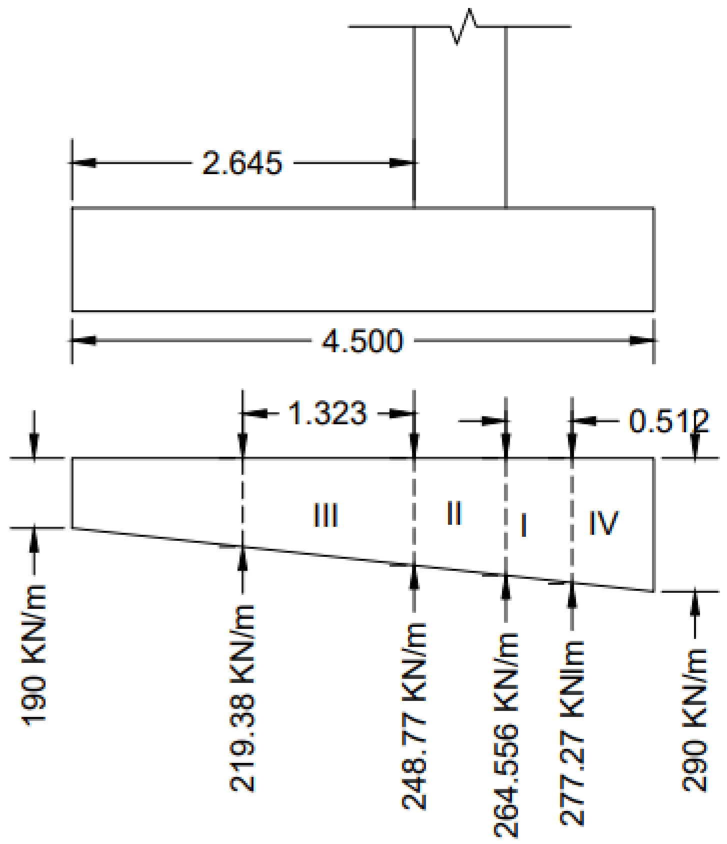


Fig7.60: Stress distribution in footing(dimension in meter)

7.6.2.1. <u>Analysis of footing</u>

Find maximum BM at the face of abutment and one way shear at the section lying at d distance from the face of abutment.

Maximum BM at ‘I_M’ = $264.556\times 1.145^2\times\frac{1}{2}+\frac{1}{2}\times 25.44\times 1.145\times 1.145\times\frac{2}{3}$ = 184.537 KN-m

Maximum BM at ‘II_M’ = $\frac{190\times2.645\times2.645}{2}+0.5\times58.77\times2.645\times\frac{2.645}{3}$ = 733.148 KN-m

Maximum BM at ‘III_M’ = $\frac{190\times1.322\times1.322}{2}+0.5\times29.38\times1.322\times\frac{1.322}{3}$ = 174.58 KN-m

Maximum BM at ‘IV_M’ = $\frac{277.27\times0.572\times0.572}{2}+0.5\times12.73\times0.572\times\frac{2\times 0.572}{3}$ = 46.74 KN-m

SF is critical at ‘d’ distance from the face of stem (IS 456;2000)

Where d =0.716

Maximum SF at ‘d’ie 0.716m from face of stem = $\frac{290+280.44}{2}\times(1.145-0.715)$

= 122.466KN

7.6.2.2. **Design of footing for basic combination of loads**

Check depth of footing

Let us provide clear cover of 75mm and dia of 20mm.

Taking M20 Fe415

d = D – CC – ø/2 = 800 – 75 – 20/2 = 715mm

$d_{bal} = \sqrt{\frac{M_u}{Q \times b}} = \sqrt{\frac{733.148 \times 10^6}{0.138 \times 20 \times 1000}}$ = 515 mm <800 mm (provided)

$d_{prov} > d_{bal}$ hence, assumed depth is sufficient.

a. Main reinforcing bars at bottom (along the width of footing)

Since $d_{prov} > d_{bal}$, section is designed as Singly Reinforced Under-Reinforced Section (SRURS). Section design has been carried out by using IS 456:2000.

Area of reinforcement

$$= \frac{M}{0.87 \times fy \times (d - 0.42xu)}$$

$$= 733.148 * \frac{10^6}{0.87 * 415 * (715 - 0.42 * 0.48 * 715)}$$

A_{st} = 3557.113mm^2

% of steel =0.497%

Spacing of reinforcement $= \frac{\pi x \frac{20^2}{4} \times 1000}{3557.113}$ =88.31mm

Providing 20mm diameter bar @85mm c/c spacing for bottom main reinforcement bar.

Area provided $= \frac{\pi x 20^2}{85} x 1000$=3695.99mm^2

At section I

Area of reinforcement

$$= \frac{M}{0.87 \times fy \times (d - 0.42xu)}$$

$$= 184.537 * \frac{10^6}{0.87 * 415 * (715 - 0.42 * 0.48 * 715)}$$

Ast = 895.3mm2

Spacing of reinforcement $= \frac{\pi x \frac{20^2}{4} \times 1000}{895.3}$ =350mm

Providing 20mm diameter bar @300mm c/c spacing for bottom at section I.

b. Reinforcing bars at top and distribution bars at bottom

Taking 0.12% of bD for top and distribution bars

$$A_s = \frac{0.12}{100} \times 1000 \times 800 = 960mm^2$$

Spacing of reinforcement =$\frac{\pi x \frac{16^2}{4} \times 1000}{960}$ =209.33mm

Provide 16mmØ bar@205mm c/c as distribution bars at bottom.

Provide 16mmØ bar@205mm c/c at top of footing in both directions.

c. Check depth of slab for one-way shear at 'd'=0.715m**.**

Nominal shear stress τ_{uv}=$\frac{V_u}{bd} = \frac{122.466 \times 1000}{1000 \times 715} = 0.157\ N/mm^2$

For M20 and $P_t = 0.46\%$

From table (IS 456:2000)

$$\tau_{uc} = 0.48\ N/mm^2$$

Depth factor k = 1

Since , $\tau_{uv} < k\tau_{uc}$

No need of shear reinforcement

d. Check for development length

Ld =α1 α2l0

α1=1

α2=base required/base provided

=1886.227/1943.75

=0.97

L0=nΦ

=29*25=725mm

Ld =α1 α2l0

=703.25mm

Minimum footing width=1110mm

Since provide length=1110mm>703.23mm

Additional anchoring bar not required

Design Summary

For bottom section

SN	Description	Size of ϕ	Spacing
1	Vertical bars in the side of backfill	**25mm**	**250mm**
2	Vertical bars in the side of river	**16mm**	**230mm**
3	Horizontal Reinforcement	**12mm**	**50mm**

For 1.5mabove from bottom

SN	Description	Size of ϕ	Spacing
1	Vertical bars in the side of backfill	**25mm**	**300mm**
2	Vertical bars in the side of river	**16mm**	**230mm**
3	Horizontal Reinforcement	**12mm**	**50mm**

For Dirt wall

SN	Description	Size of ϕ	Spacing
1	Vertical bars in the side of backfill	**12mm**	**300mm**

For footing

SN	Description	Size of ϕ	Spacing
1	Longitudinal bottom bars	**20mm**	**300mm**
2	Bars at top and bottom distribution bar at bottom	**16mm**	**205mm**

8. CONCLUSION AND RECOMMENDATIONS

With our relentless effort for about a year, we finally have concluded our final year project work entitled 'Design of Bridge over Andheri Khola'. This would not have been possible without the valuable guidance from our supervisor to understand some difficult concepts of Bridge Analysis and Design. It was our teamwork that helped us overcome the lengthy design procedures and surpass our own expectations towards the final year project. The field visit helped us collect a real time experience to be a Bridge Engineer while the incorporation of superstructure and substructure type helped us explore the possibilities in bridge design in terms of safety, economy and aesthetics. The report encompasses all our work and has been prepared with the best of our knowledge and skills. The project work is apparently complete but we have not been able to include analysis results from SAP2000 in this report. As the use of computers helps make structural analysis easier and quicker, use of computer packages along with manual calculations cannot be averted and hence, their use highly recommended. But it should be forethought that the results from such software are only as reliable as the knowledge and acquaintance that the user has of the software thus needing a very meticulous application.

This hereby concludes our final year project work.

9. QUANTITY ESTIMATE

SN	Description	No	Length m	Breadth m	Height m	Quantity	Unit
1	**superstructure**						
	Railing(M30)	26	0.25	0.25	1.2	1.95	cu.m
	Railing bar	6	20.35	0.00196m2		0.239	cu.m
	kerb(M30)	2	20	0.75	0.2	6	cu.m
	wearing course		20	7.5	0.075	11.25	cu.m
a	**Deck slab (M30)**						
	cantilever slab	2	20	0.275	1.3	14.3	cu.m
	Restarined slab		20	6.4	0.25	32	cu.m
	Approach slab	2	4	9	0.3	10.8	cu.m
b	**Main girder(M30)**						
	web	3	20	0.4	1.15	27.6	cu.m
	Fillet	4	20	0.01125m2		0.9	cu.m
	Cross girder(M30)						
	web	5	6.4	0.35	0.865	9.688	cu.m
2	**substructure**						
a	**Abutment**						
	Dirt wall(M30)	2	7.3	0.2	1.1	3.21	cu.m
	Abutment cap(M30)	2	7.3	0.785	0.3	3.438	cu.m
	Abutment stem	2	7.3	0.71	2.936	30.372	cu.m
	Footing(M30)	2	7.3	4.5	0.8	52.56	cu.m
3	Bearing	6	0.5	0.32	0.064	0.0617	cu.m
4	Expansion joint	2	7.3	0.04	1.4	0.817	cu.m
	Reinforcement						
	10mmΦbar					9875.36	kg
	12mmΦbar					4566.52	kg
	32mmΦbar					5645.8	kg

10. REFERENCES

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11. ANNEX

Bridge site photographs

Fig1.1:Karamghari side to Adheri side

Fig11.2: Data collection at the site

Fig11.3: D/S of Andheri Khola from u/s right side with marked bridge axis

Fig11.4: Measuring cross section at site

Fig11.5: Interaction with local people

Fig11.6: Group photo near site