TRIBHUVAN UNIVERSITY

Institute of Engineering

Thapathali Engineering Campus

Department of Civil Engineering

Kathmandu, Nepal

A FINAL YEAR PROJECT REPORT ON
"DESIGN OF RCC T-GIRDER BRIDGE"

PROJECT MEMBER

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A MAJOR PROJECT SUBMITTED IN PARTIAL FULFILLMENT OF THE FOR THE DEGREE OF BACHELOR'S IN CIVIL ENGINEERING

PROJECT SUPERVISOR	SUBMITTED TO
Er.Biswa Kumar Balla	DEPARTMENT OF CIVILENGINEERING
	Thapathali Campus IOE

TRIBHUVAN UNIVERSITY

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Department of Civil Engineering

Kathmandu, Nepal

Certificate

This is to certify that the final year project entitled **"DESIGN OF RCC T-GIRDER BRIDGE, CHEHERE, SINDUPALCHOWK"** 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.

....

Er. Biswa Kumar Balla

Project Supervisor

....

~~Er.~~ External Examiner

Dr. Rajan Suwal, Pulchowk Campus, IOE, TU

Er. Indra Narayan Yadav

Head of Department

Department of Civil Engineering

Thapathali Campus

LETTER OF APPROVAL

This is to certify that this project work entitled **"DESIGN OF RCC T-GIRDER BRIDGE, CHEHERE, SINDUPALCHOK"** was submitted by **Rojib Bhattarai (THA075BCE097), Roshan Khadka(THA075BCE100), Safal Subedi(THA075BCE103), Sagar K. Bhudathoki (THA075BCE104), Sajjan Sharma (THA075BCE107) and Sujita Shakya (THA075BCE127)** has been examined and it has been declared successful for fulfillment of the academic requirements towards the completion of the Bachelor's Degree in Civil Engineering.

....

Prof. Er. Biswa Kumar Balla

Project Supervisor

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External Examiner

Associate Prof, Dr. Rajan Suwal.

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Er. Indra Narayan Yadav

Head of Department

Department of Civil Engineering

April,2023

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DEPARTMENT OF CIVIL ENGINEERING

CERTIFICATE

This is to certify that the work contained in this report entitled **"Design of RCC T-Girder Bridge Over Sunkoshi River, Chehere, Sindupalchok"** in partial fulfillment of the requirements for the Bachelor's degree in Civil Engineering, as a record of research work has been carried out by **Rojib Bhattarai (THA075BCE097), Roshan Khadka (THA075BCE100), Safal Subedi (THA075BCE103), Sagar K. Bhudathoki (THA075BCE104), Sajjan Sharma (THA075BCE107) and Sujita Shakya (THA075BCE127)** under my supervision and guidance in the institute of Engineering, Thapathali Campus, Kathmandu, Nepal. The work embodied in this report has been submitted elsewhere for degree.

....

Prof. Er. Biswa Kumar Balla

Thapathali Campus

DEPARTMENT OF CIVIL ENGINEERING

ACKNOWLEDGEMENT

We wish to acknowledge the help received from various individuals and institution during the preparation of this report, without whom project wouldn't have been possible.

We would like to thank our project supervisor Er.Biswa Kumar Balla, resource person for this project for guiding, directing and helping to complete this project work. We would also like to present our gratitude to LOCAL ROAD AND BRIDGE SUPPORT UNIT (LRBSU) for technical and financial support to complete the bridge project of Sunkoshi River Bridge.

Our profound gratitude goes to them for getting available all the times despite their esteemed time whenever required to solve problem occurred during the project work.

We extend our gratitude to Civil Engineering Department and Administration of IOE Thapathali Campus for supporting us morally and financially respectively to prepare this report.

In a nutshell, we would like to acknowledge all individuals who have directly or indirectly helped us during our project work for the successful completion of this project.

ACKNOWLEDGEMENT

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We would also like to extend our thanks to the faculty members of the Civil Engineering Department and Administration of IOE Thapathali Campus for their intellectual input, constructive criticism, and encouragement that helped us to develop our skills and knowledge in the field.

We would like to acknowledge the contribution of our teammates who have been an integral part of the project. Each member has played significant role in the completion of the project, from the conceptualization stage to the final phase.

We express our deepest appreciation for the support extended by our families and friends who have encouraged us through the ups and downs for the project journey.

Finally, we would like to acknowledge all individuals who have directly or indirectly helped us during our project work for the successful completion of this project.

Thank you all!

Salient Features

Particulars	Required Information/Range/Values
Title of the Project	Design of RCC T-Girder Bridge
Location	
Province	Bagmati
District	Sindhupalchowk
Village	Chehere
Name of Road	Araniko Highway
Geographical Location	
Longitude	85°43'54.36" E
Latitude	27°40'23.30" N
Classification of Road	District Road
Type of the road surface	Earthen Road
Terrain/Geology	Hill
Information on the structure	
Total length of the bridge	90 m
No. of Span	3 spans
Span length	30 m
Total width of the bridge	11 m
No. of lanes	Two lanes
Width of Carriageway	7.5 m
Width of footpaths with railing	1.75 m
No. of longitudinal girder	3
No. of cross girder	4
Type of structure	Simply Supported
Type of superstructure	RCC T-Girder
Type of bearings	Elastomeric Pad Bearing
Type of Abutments	RCC (Gravity type) abutment
Type of Pier	Circular
Type of Foundation	Pile foundation
Design data	
Live load	IRC Class 70R, IRC Class A
Design discharge	2913.89 m^3/s
Linear waterway(Provided)	90 m
Catchment Area	3189 km^2

Scour depth i) Pier ii) Abutment	 7.62 m 4.84 m
Concrete grade	M30
Reinforcement	Fe500 (TMT steel)

Table of Contents

1. Introduction

1.1 Background

An important element of land transportation system is the bridge. A bridge is a structure that carries a service (which may be highway or railway traffic, a footpath, public utilities, etc.) over an obstacle (which may be another road or railway, a river, a valley, etc.), and then transfers the loads from the service to the foundations at ground level. To integrate the various aspects of the civil engineering learned throughout the undergraduate curriculum, it is mandatory to complete a real-world project work in final year of course. The six membered team of authors of this report have chosen the bridge design project for the fulfilment of degree to boost all the knowledge area.

This report is the final deliverable of the project containing site selection based on geotechnical data, planning of bridge, selection of type of bridge, hydrological and hydraulic design of bridge, structural analysis and design of each component and preparation of working drawings. The report has been organized as per standard format of department of civil engineering. This report is also prepared as a part of project work for the fulfilment of the Project-II as per the syllabus of Bachelor of Civil Engineering fourth year second part. So, for our project purpose, we have designed RCC T-girder Bridge.

This project is under jurisdiction of Department of Local Infrastructure and consulted by Local Roads Bridge Support Unit (LRBSU). This project has been completed with the financial and technical support from LRBSU. The financing agency covered all the expense incurred during completion of this project work. The geotechnical investigation report, topographical survey report, hydrological and geotechnical guidelines has been acquired through LRBSU. The topographical data were verified by limited site survey and all other data were used as provided.

In this project, we were assigned to design a bridge over Sunkoshi river which provides a link between Chehere village right of the river with Rithe Village left to the river in Sindhupalchowk district of Province-3. This bridge connects Rithe village and Sapping municipality with Araniko Highway. We are supposed to design the most economical bridge for this section based on the various data provided by LRBSU.

1.2 Title of Project Work

The key output of this project is the design of bridge and its working drawing. All of the bridge components have been designed and verified using the limit state design method, as well as the working drawing and details of the bridge across Sunkoshi River, Sindhupalchowk is included in this report. Therefore, this project is entitled as "Design of RCC T-girder Bridge ".

1.2 Location Of Project

Sunkoshi River at Chehere, Sindhupalchowk district, Nepal

1.3 Objectives

General Objectives

The general objective of the study is to cover the analysis as well as to design the technically feasible and economically justifiable RCC T-girder Bridge for the partial fulfillment of the requirements for the Bachelor degree of civil engineering program.

Specific objectives

The specific objectives of the study are:

- To select bridge site based on topographical, hydrological, and geotechnical data
- To select suitable bridge type and span arrangement
- To plan, analyze, design and detail each component of bridge
- To prepare working drawing of all components of bridge
- To prepare quantity estimate of various item of work
- To recommend any river training works or other site related works (their design is not considered)

1.4 Scope of the study

Understanding the architectural drawing and familiarity with it with respect to laws and codes

Calculation of dead load, imposed load on structure

Computation and analysis of hydrological and topographical data

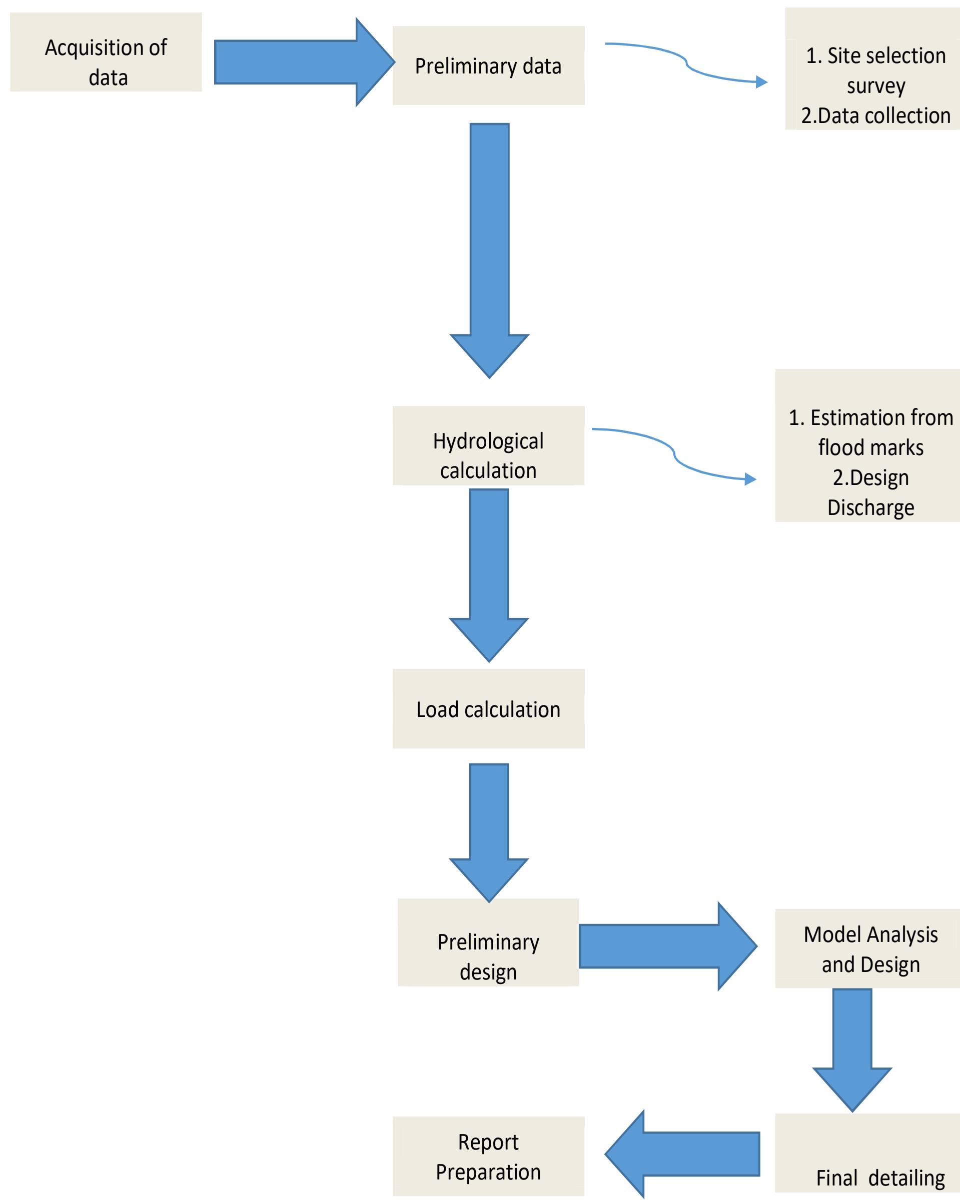
Preliminary design of individual member of structure

Computation of internal stresses and design of the members

Structural drawing and detailing of individual members

2. Methodology

In order to fulfill the above mentioned objectives, the following processes and methodology were adopted:



FLOW CHART

2.1 Acquisition of Data

2.1.1. Primary Data Collection

Primary data collection includes collecting data about local needs and bridge requirements that involves observation and interviewing local people. It also includes measurement and arrangement of span of bridge, bridge alignment and estimating the depth of the foundation.

2.1.2.Secondary Data Collection

a) Desk Study:

Desk study is the study of all scope of the topic of our project which includes literature review, past report review, study of design codes, topographic study, catchment area calculation, study of road and river characteristics and data provided by LRBSU.

b) Hydrological Analysis:

Hydrological analysis includes collecting, analyzing and calculating hydrological data provided by LRBSU or Department of Hydrology and Meteorology. It includes the calculation of following parameters:

- **i.** Maximum discharge
- **ii.** Design discharge
- **iii.** Linear waterway
- **iv.** Scour depth
- **v.** High flood level

c) Geotechnical Study:

Geotechnical investigation works includes the exploratory drilling, in-situ field testing SPT / DCPT test, borehole logging, and collection of samples and perform necessary tests on soil samples for detail information. All the investigation data was provided by LRBSU. This study also includes selection of type of foundation and analyzing depth of foundation as well as different types of soil parameters.

2.2. Data Analysis

After the selection of the proposed bridge site with alternatives and preparation of topographic maps, the Preliminary Design Report is prepared including feasibility report. After that, detail design of all structural components are done.

2.3. Span Arrangement

3 spans each of length 30 m

2.4. Loading

The Loading is considered according to clause IRC 6:2020 Table 6A : Live load Combination.

IRC CLASS 70R Tracked and Wheel Loading

This loading is to be normally adopted on all roads on which permanent bridge and culvert are constructed. Bridge design for 70R loading should also be checked for class A loading as under certain condition, heavier stresses may occur under Class A loading.

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.

2.4 Codes Uses for Design

- **a)** IRC 6: 2020 (For Load Combination and calculation for Super and Sub Structure)
- **b)** IRC 112:2020 (For Design)
- **c)** IRC 83 :2018 part II (For Design of Elastomeric Bearing)
- **d)** IRC 5 :2015 (For Bridge Specification)
- **e)** Nepal Bridge Standard (For Design Purpose)
- **f)** IRC 21 STANDARD SPECIFICATIONS (For Bridge Specification)

3 Preliminary design

For the span of 30m a RCC T-girder bridge with 3 spans shall be provided. The abutment shall be provided for depth beyond maximum scour level and adequate vertical clearance of HFL shall be made available underneath the decks.

Bridge Information

a) Carriageway: 7.5m

b) Total length of Bridge: 91.25m

c) Span length: 30m

d) Number of spans: 3

e) Width of Footpath with railing: 1.75m

f) Type of Structure: Simply Supported

g) Type of Super structure: RCC

h) Type of Bearing: Elastomeric Bearing

i) Type of Abutment: RCC (Gravity Type)

j) Type of Foundation: Pile Foundation

Preliminary Design of Super-structure

Main girder

a) Beam and Slab

For bridges having beam and slab type of super-structure the number of longitudinal shall not be less than three, except for single-lane bridges and pedestrian bridges.

If only two main girders are provided, the depth of the slab to be provided should be more, which is uneconomical. So, 3longitudinal girders are provided.

∴ Provide number of girder = 3

b) Depth of main girder= span/(12 to 15)

= 30000/(12 to 15)

=2300mm

c) c/c spacing

Distance between center of the main shall be sufficient to resist overturning or over stressing due to lateral forces and loading conditions. Otherwise, special provisions

must be made to prevent this. This distance shall not be less than L/20 of the span.

L/20 of span=1.125m

$$\frac{c}{c}\ \text{spacing of longitudinal girder} = \frac{\text{width of deck} - 2 * \text{width of cantilver}}{n-1}$$

$$= \frac{11 - 2 * 2.25}{2}$$

$$= 3.25\ \text{m}$$

∴ Provide c/c spacing 3.25 m

d) Fillet size = 150×100mm (general practice)

e) Thickness of web = shall not be less than 250mm

∴ Provide thickness of web =300mm

f)Bottom bulb width =700mm

<u>Deck slab</u>

a) Width of deck=11000mm with 7500 mm carriageway and 1750mm for footpath

b) Thickness of deck slab:

The minimum thickness of the deck slab including that at the tip of the cantilever

shall be 200 mm. However, reduction in the thickness of the slab up to a maximum

of 50 mm may be permitted at the cantilever tip subject to satisfactory detailing.

∴ Provide thickness of deck slab = 220mm to 230mm

c) Thickness of deck slab at end of cantilever =150mm

d) Wearing course:

i. Asphalt concrete for wearing coat of bridge.

ii. Thickness of wearing course = 50mm at edges and 150mm at center

Cross Girder

From IRC 2021, clause no.305.3

a) width of cross girder =400mm

b) Depth of cross girder = $\frac{3}{4}$ of main girder

$= 1725mm$

c) Depth of end cross girder =Same as intermediate girder

∴ Provide Number of cross girders = 2 end girders with 2 intermediate girders @10m and 20m each from end girder

e) c/c spacing = 10m

Railing

From IRC 5-2015, clause 109.7.2.3

a) Height of railing=shall have minimum of 1.1m height above the adjacent roadway

∴Provide Railing of Height = 1.1m

b) Height of Kerb=225mm

c) Cross section of post= 200mm×200mm concrete post

d) No. of posts = 16@ 2m c/c spacing on each side

e) 3×50mm nominal bore heavy steel pipe @6.19 kg/m, thickness of 4.5mm and sectional area of 7.88cm^2 (IS code 116:1998 Table 1)

4. Hydrological Analysis

4.1. Introduction

Bridges are very expensive structures, millions are being spent for the construction, but most of them do not last longer therefore a proper hydrological and hydraulic detail investigation is required for the proper design and construction of bridges. There is need of hydrological analysis before start of bridge construction. The result obtained from the analysis and keeping in view of suitable free board value, the construction of bridges has to be fixed. The river flood accompanied by some storms are one of the major causes for the bridge failure. Most of the people directly depend on the river basins for their livelihood, including hydropower, domestic supply, irrigation etc. so, flood profile information collected from hydrological analysis help in preventing above mentioned lives and infrastructures. The country, Nepal is located in Asia, in the lap of Himalayans. The fast flowing river with higher catchment area, increases the risk of flooding.

4.2. Study Area

The proposed bridge is to be constructed over Sunkoshi River in Chehere, Sindhupalchowk district of Bagmati province. The proposed bridge connects **Arinako** highway with **Sapping** municipality. The project area consists of different topography like rocky mountain, high hills, snow cover area etc. and different vegetation based on the data collected from GIS software.

A. Catchment Area:

fig:- Catchment Area (3189 sq.km)

The catchment Area of the proposed bridge outlet is **3189** sq.km.

B. Stream network:

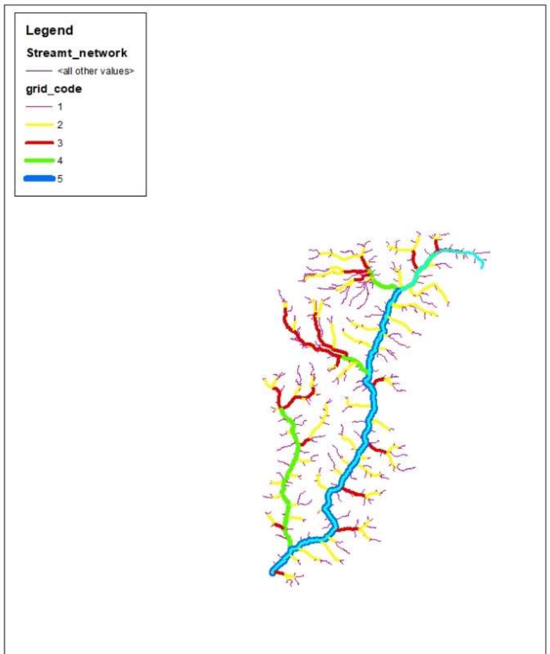
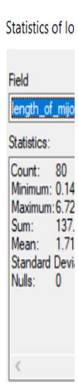


fig:- Stream Network

C. River Length:



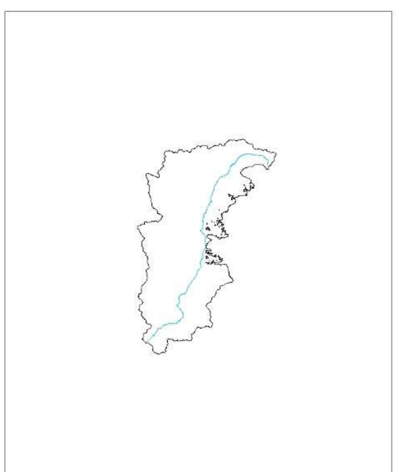


fig:- Longest River Path

The longest path measured from the GIS software is **137.54km .**

D. Land cover area:-

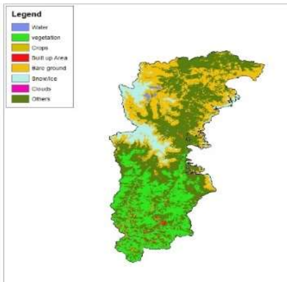


fig:-Land Cover Area

The snow cover Area is **270.71** km. sq.

E. Catchment area division: -

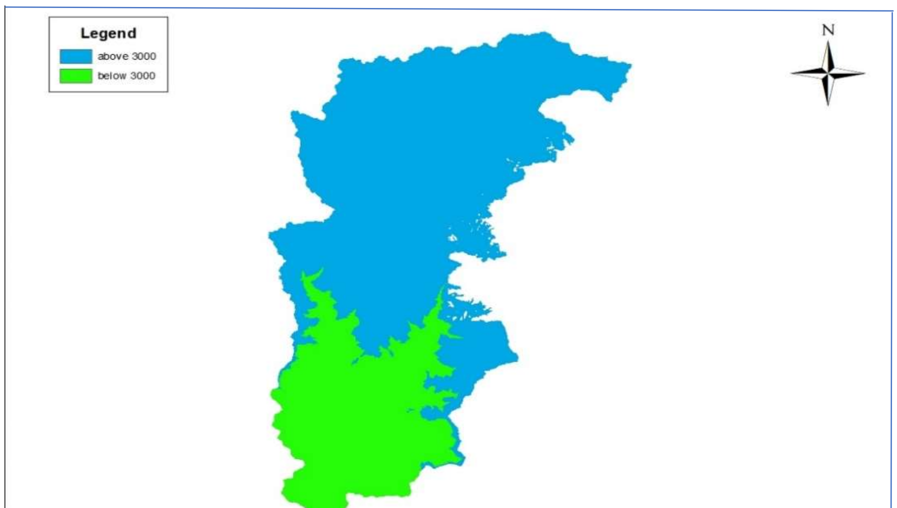


fig:-Catchment Area Division

4.3. Calculation of design discharge

1.Ryves method

According to the area of catchment and amount of rainfall, design discharge is given by:

$Q_p = C_R \times A^{2/3}$

Where,

Q_p = maximum flood discharge(m^3/s)

C_R = Ryves coefficient (10.2 for Nepal)

A = Area of catchment in sq. km

Here,

A = 3189 sq. km

$\therefore\ Q_p = 10.2 \times 3189^{2/3}$

$= \mathbf{2209.89\ m^3/s.}$

2.Fuller's method

Fuller's formula is derived from catchments in USA are a typical empirical method which is given by :

$Q=C_f \times A^{0.8}(1 + 0.8\log_{10} \mathbf{T})$

Where,

Q = maximum discharge

C_f = a constant which varies from 0.18 to 1.88

T = Return period in years

A = Catchment Area in sq. km

We have,

A=3189 sq. km

C_f=1.45 for Nepal

Then

$Q = 1.45 \times 3189^{0.8}(1+0.8\log_{10}(100))$

$= \underline{\mathbf{2394.77\ m^3/s.}}$

3. WECS method

In Nepalese context, Water and Energy Commission Secretariat (WECS) has developed empirical relationships for analyzing flood of different frequencies. According to WECS, the flood flows in any river of catchment area below 3000m of elevation is given by:

$Q = 14.63 \times (A_{3000}+1)^{0.7342}$

Where,

Q = Maximum discharge in m^3/s

A_{3000} = Catchment area below 3000m elevation = 934.42 sq.km

Then,

$Q = 14.63 \times (934.42 + 1)^{0.7342}$

$= \underline{\mathbf{2221.0409\ m^3/s}}$

4.DHM 2004

The formula for 100 years return period is given by

$Q = 20.7\ (A_{3000})^{0.72}$

Where,

Q is the design flood in m^3/s .

A_{3000} is the basin area in sq km below 3000 m elevation.

Then,

$Q = 20.7\ (934.42)^{0.72}$

$= 2849.44\ m^3/s$

5 . MODIFIED DICKENS METHOD

In this method, the T year flood discharge Q_t, in m^3/sec, is determined by:

Design discharge: $Q_t = C_D \times A^{3/4}$

Where,

C_D = Dicken's constant

$= 2.342 \times \log_{10}(0.6\ T) \times \log_{10}(1185/P) + 4$

T= return period in years = 100

$$P = 100\,\frac{(As + 6)}{(A + As)}$$

A = Catchment area in sq. km

A_s =Area of snow-covered catchment in sq. km

Here,

A = 3189 sq. km

A_S = 270.71 sq. km

$$P = 100\,\frac{(As + 6)}{(A + As)} = 100\,\frac{(270.71\ + 6)}{(3189 + 270.71\ )} = 8$$

$C_D = 2.342 \times \log_{10}(0.6T) \times \log\left(\frac{1185}{P}\right) + 4$

$= 2.342 \times \log(0.6 \times 100) \times \log\left(\frac{1185}{8}\right) + 4$

$= 13.02$

$Q_t = C_D \times A^{3/4}$

$= 13.02 \times 3189^{3/4}$

$=$ **$5505.41\ m^3/s$**

6.Gumbels & Method

Year	Thokark	Sangha chowk	Gumthung	Dolal ghat	Avg
1983	92	75	104	88	89.75
1984	92.5	54	144	60.6	87.775
1985	85	73	164.4	62.5	96.225
1986	60.7	78.4	96.3	70	76.35
1987	105	95	105.6	100.1	101.425
1988	90	75.2	98.3	57.4	80.225
1989	82.5	67.5	60.2	49.5	64.925
1990	82	62.4	126.4	74.2	86.25
1991	79	80	65	60.1	71.025
1992	79	26.2	100.4	63.5	67.275
1993	82	58.2	119.6	44.2	76
1994	82	82	157.6	72.1	98.425
1995	83	63.5	96.2	51.5	73.55
1996	64	66.8	87.3	63.1	70.3
1997	96	88.7	123	77.9	96.4
1998	82	56.2	95	57.8	72.75
1999	84	80.6	80	97.1	85.425
2000	66	115	92	67.2	85.05
2001	115	116	65	78.5	93.625
2002	72	104	50	112.6	84.65
2003	85	108.6	55	61	77.4
2004	140	77.8	65.5	90.5	93.45
2005	61	124	60.2	149.5	98.675
2006	75	64.4	65.2	59	65.9
2007	29.5	90.6	55.4	66	60.375
2008	0	100.4	55.4	75	57.7
2009	80	84	60.4	66	72.6
2010	80	99	55.2	60.3	73.625
2011	120	64.4	57.4	108	87.45
2014	100.5	66.4	35.4	56.3	64.65
2015	96.5	92	110.4	52.5	87.85
2016	72.2	66.6	350.2	47.3	134.075
2017	46.4	60	35.4	57.6	49.85
2018	52.2	141	157.4	55.5	101.525
2019	56.4	97	42.5	105.5	75.35
2020	52.2	63	48.5	54.5	54.55

$Mean(\mu\ ) = \frac{\sum X}{n} = 80.90$

$Standard\ Deviation(\sigma) = \sqrt{\frac{\sum(x-\mu)^2}{n-1}} = 16.455$

From Gumbels Table,

$Y_n = 0.5402$

$S_n = 1.128$

And we have,

$Y_t = -Ln(ln\times(T/(t-1))$

For the return period, T= 100 years

$Y_t = 4.60$

And $K_t = (Y_{t\,-}\,Y_n\,)/S_n$

Therefore $K_{100} = 3.59$

Therefore, $X_{100} = \mu + K_{100}\sigma$

$= 140.10$ mm

L=Length of the stream in Km=137.54

H=Difference in elevation of remotest point of the basin and outlet in metres.

= 5389-671

= 4718m

S=Slope of the stream = H/L

= 0.0343

$T_c = 0.019478L^{0.77}S^{-0.385}$

=645.458 min

= 10.758 hrs

$I = \frac{X_{100}}{24} * \left(\frac{24}{T_c}\right)^{\frac{2}{3}}$

= 9.96 mm/hrs

Now,

$Q = \frac{CIA}{360}$

Where I= Rainfall Intensity

C= Coefficient of runoff = 0.330

A=Catchment area

Q=Flow Discharge

$Q = \frac{0.330*9.96*3189.23}{3.6} = 2913.89$ m^3/sec.

7. Slope Area Method

The area velocity method based on 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 condition is calculated using manning's formula.

The discharge is given by Manning's formula is used here. The discharge and velocity are given by equation:

$V=\frac{1}{n}R^{2/3}S^{1/2}$

And $Q=\frac{1}{n}A.R^{2/3}S^{1/2}$

Here

V=Velocity

Q=Discharge

S=Bed Slope

R =Hydraulic Radius

=A/P

n= manning's coefficient = $((d_{50})$ ^ (1/6))/21.1

=0.30375) ^ (1/6)/21.1

=0.039

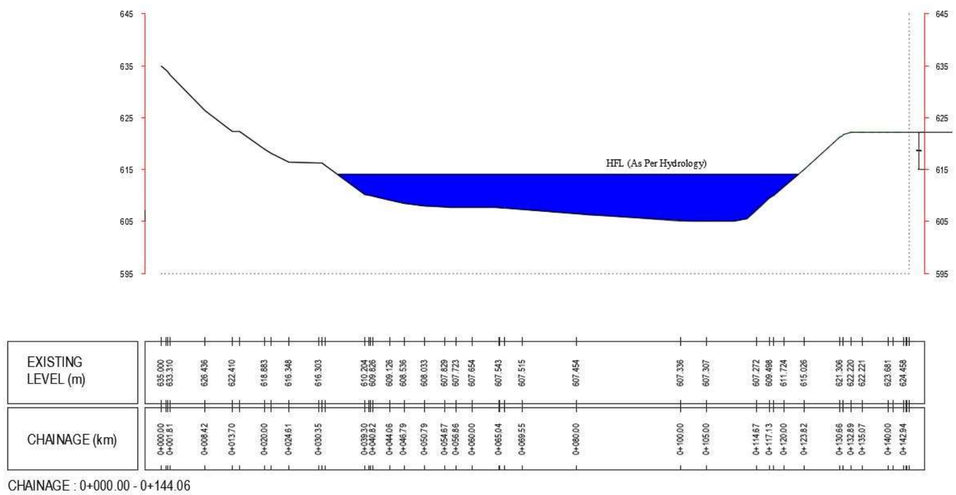
From Topo map,

Slope = 0.002500

The wetted area and wetted perimeter are obtained from the cross section of the stream at site drawn to scale with the floods levels marks therein.

From Historical Mark of High flood Level in field, From LRBSU we get the HFL as 614.14 m.

Cross section of River:



Stage (m)	A(m^2)	P (m)	R (m)	Q (m^3/s)	Remarks
614.14	588.98	93.74	6.28	2571.125	HFL

4.4. Selection of Design Flood

Considering the life period of a bridge structure, probable risk of highest flood and overall investment on the construction of a bridge, generally 50 year of return period is adopted for minor bridges whereas 100 years of return period is considered for major bridge as design flood for detailed engineering design. Hence here 100 years return period is adopted as design flood.

Design discharge

S No	Method	Discharge
1	Ryve's Formula	2209.89 m^3/sec
2	Fuller Method	2394.77 m^3/sec
3	WECS Method	2221.04 m^3/sec
4	DHM 2004 Method	2849.44 m^3/sec
5	Modified Dicken Method	5505.41 m^3/sec
6	Rational (Gumbel) Method	2913.89 m^3/sec
7	Slope Area Method	2571.125 m^3/sec

Omitting the odd discharge and selecting the maximum discharge amongst the discharges obtained from the above-mentioned methods as design discharge, we get,

Design discharge = 2913.89 m^3/sec

4.5. Rating Curve and HFL

From CAD analysis, the corresponding discharge at particular stage is calculated as follow using Manning's Formula.

Stage (m)	Area (m^2)	Perimeter (m)	Hydraulic radius(m)	Velocity m/s)	Discharge (m^3/s)
615.00	666.84	96.46	6.91	4.65	3102.57
614.90	658.03	96.17	6.84	4.62	3040.67
614.80	649.54	95.85	6.78	4.59	2982.09
614.70	640.49	95.53	6.70	4.56	2919.62
614.60	631.03	95.21	6.63	4.52	2854.52
614.50	622.50	94.89	6.56	4.49	2796.73
614.40	613.53	94.57	6.49	4.46	2736.07
614.30	604.59	94.25	6.41	4.43	2676.03
614.20	595.25	93.92	6.34	4.39	2613.46
614.14	588.99	93.74	6.28	4.37	2571.13
614.10	586.80	93.60	6.27	4.36	2557.75
614.00	577.94	93.28	6.20	4.32	2499.43
613.90	569.10	92.96	6.12	4.29	2441.61

613.80	560.28	92.64	6.05	4.26	2384.43
613.70	549.64	92.21	5.96	4.21	2316.70
613.60	542.74	92.00	5.90	4.19	2271.83
613.50	534.01	91.68	5.82	4.15	2216.39
613.40	525.30	91.35	5.75	4.11	2161.52
613.30	516.60	91.03	5.67	4.08	2107.18
613.20	507.96	90.71	5.60	4.04	2053.55
613.10	499.32	90.39	5.52	4.01	2000.41
613.00	490.71	90.07	5.45	3.97	1947.88

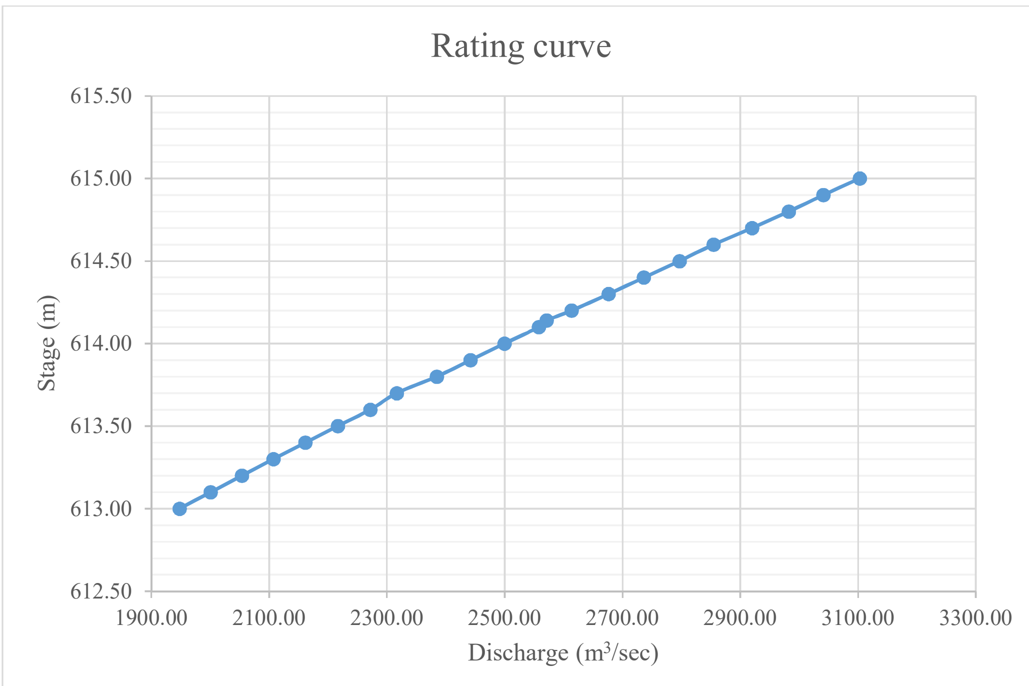


Fig: - Rating curve

HFL corresponding to design discharge **2913.89** m^3/s is 614.69 m.

4.6. Linear Waterway

According to Kellerhals, mean channel width is given by

$B=3.26Q^{0.5}$ for gravel bed channels

= 175.97 m

Where, B= mean channel length required for given discharge

Q= Design discharge

According to Lacey's formula, mean channel width is given by:

$B=4.75Q^{0.5}$ for alluvium channels

= 256.40m

where,

B= mean channel length required for given discharge

Q= Design discharge

[According to IRC 5:2015, Clause 10.6.5.1.1], the linear waterway for the natural channel bed with rigid non erodible banks shall be taken as the width of channel at HFL in which design discharge can be passed.

❖ Therefore, as per site condition, we adopt waterway = 90m.

4.7. Scour Depth

[According to IRC: 78-2014, clause 703.2]

Design discharge for foundation

For Catchment Area of 3000 km^2 increased discharge by 30 %

For Catchment Area of 1000 km^2 increased discharge by 20%

Therefore, For Catchment Area of 3189.239 km^2 increased by 29.73 %

So,

Q=3780.099 m^3/s

Mean Depth of scour below HFL (d_{sm}) = $1.34 * (\frac{D_b{}^2}{K_{sf}})^{1/3}$

Where

D_b = Design discharge in per unit width

K_{sf} = silt factor = $1.76\sqrt{d_m}$

<u>Calculation of D_b</u>

D_b= Q/w

=3780.08/90

= 42.000

Calculation of k_{sf}

$K_{sf} = 1.76\sqrt{d_m}$

Where d_m median size of bed material in mm.

For our site, from geotechnical report

$d_m = 2$ mm

Then ,

$k_{sf} = 2.49$

$\therefore D_{sm} = 1.34\ (42^2/2.49)^{1/3}$

$= 11.95$ m

Bed level from HFL = 9.6 m

Scour depth from bed(d) = 11.95 – 9.6

=2.35m

From IRC: 78-2014 clause no.703.3

Maximum scour depth

For piers

$(D_{sm})max = 2 \times d$

=2*11.95 m= 23.9

For abutment

$(D_{sm})max = 1.27 \times d$

=1.27*11.95 = 15.17 m

4.8. Afflux

Afflux is calculated according to IRC 5: 2015

It is the heading up of water over the flood level in the upstream side of a bridge caused by the constriction of the waterway at the bridge.

If the water way is restricted, it will cause afflux at the bridge and design HFL should be raised accordingly.

The afflux may be calculated by using Molesworth formula as given below: $h = [V^2 /17.88 + 0.015]\ [(A/a)^2 - 1]$ (IRC 5: 2015)

Where, h = the afflux (in meter)

V = the mean velocity of flow in the river prior to bridge construction (m/s)

= Discharge / A

= 4.947 m/s

A = Unobstructed sectional area of flow section in m^2

=588.98 m^2

a = Constricted area of the river at proposed site in m^2

= 536.43 m^2

Hence, h = $[4.947^2/17.88 + 0.015]\,[(588.98/536.43)^2 - 1]$

=0.22m

Final HFL from bed level= RL of HFL+ Afflux

=614.47 m+0.22 m

=614.69 m

4.9. FREE BOARD

In case of bridges over water bodies, the free board from the design HFL with afflux to the lowest point of bridge superstructure shall not be less than 1.0 m. The minimum freeboard shall be as shown on the following table.

Table: Free Board

Discharge m^3/s	**Minimum Free board, mm**
Less than 200	1000
201-500	1200
501-2000	1500
2001-5000	2000
5000 and above	more than 2000 (depending on the reliability of the available data for the calculation of discharge)

The design discharge is 2913.89 m^3/sec. So, the corresponding freeboard is 2000 mm.

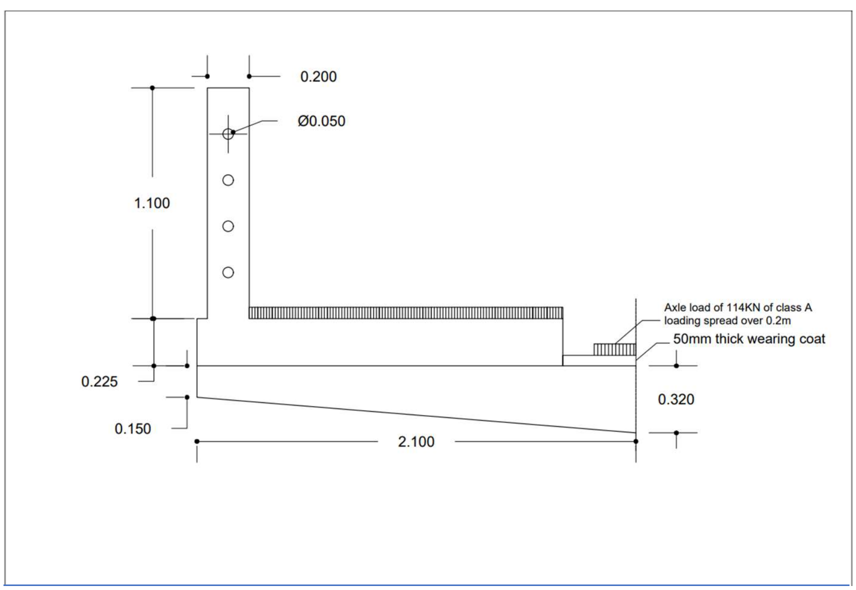
5.Detail Design of superstructure

5.1. Design of cantilever slab

5.1.1. Dead load Bending moment

Table: Calculation of dead load bending moment for cantilever slab:

S.N	Description	Formula (calculation)	FOS	Load (kN/m)	Load Arm (m)	Bending Moment (kNm/m)
1	Handrail					
	(a) Post	(25×1.1×0.2×0.2×16)/30	1.35	0.792	1.95	1.544
	(b) Rail Post	(4 × 6.19 × 9.81)/1000	1.35	0.328	1.95	0.639
2	Footpath/Kerb	25×0.225×1.75	1.35	13.289	1.225	16.279
3	Wearing Coat	22×0.05×0.35	1.75	0.674	0.175	0.118
4	Slab (Rectangular part)	25×0.17×2.1	1.35	12.049	1.05	12.651
5	Slab (Triangular part)	0.5×2.1×0.15×25	1.35	5.316	0.7	3.721
Total				**32.447**		**34.953**



5.1.2. Live load bending Moment:

Since, this bridge has 7.5m of carriageway, as per IRC-6 we have to apply 70R load or 2-lane class A load. For 70R load, clearance between end of kerb and loaded wheel, it must be minimum of 1.2m. But in our case, clearance is only 0.6m from end of cantilever, we only apply class A load whose minimum clearance is 0.15m.

Average thickness (D) = (150+320)/2 = 235 mm

Thickness of wearing coat (h) = 50 mm

Contact length of wheel load for class A = 500 mm

Dispersed length = 500+2×(50+235) = 1070 mm

Effective width of dispersion be is computed using,

$$b_{eff} = (1.2a+b1)\,) \leq l/3$$

where, b_{eff} = effective width of slab

a = distance of the CG of the concentrated load from the face of cantilever support

= 0.485/2 = 0.2425 m

b_1 = breadth of concentrated area of load

= w+2h

= 0.25+2×0.05

= 0.35

b_{eff} = 1.2×0.2425+0.35 = 0.641m

l/3 = 2.1/3 = 0.7m > 0.641. so, adopt beff = 0.641 m

Impact factor = 50% (IRC 6:208.2)

Live load per unit width including impact = (114/2×485/1070×1.5)/0.641

= 60.445 kNm

Maximum bending moment due to live load = 60.445×0.485/2×1.5

= 21.987 kNm

5.1.3. Bending moment due to pedestrian load

Loading = 400kg/m^2 = 4kN/m^2

Bending moment = 4×1.5×(0.35+1.5/2)×1.5

= 9.9 kNm

Total dead load bending moment = 34.953 kNm

Total design bending moment = 34.953 + 21.987 + 9.9

= 66.840 kNm

5.1.4. Design of section

$$\text{Effective depth required} = \sqrt{\frac{\text{M}}{0.36 * 0.48 * (1 - 0.42 * 0.48) * fck * b}}$$

$$= \sqrt{\frac{66.840}{0.36 * 0.48 * (1 - 0.42 * 0.48) * 30 * 1000}}$$

= 129.135 mm

Clear cover = 40 mm

Diameter of steel bars = 16 mm

Effective depth provided = 320 – 40 – 16/2 = 272 mm > dreq (ok)

5.1.5. Calculation of main reinforcement

$$\text{M} = 0.87 * \text{f}_\text{y} * A_{st} * \left(d - \frac{f_y\, A_{st}}{f_{ck}\, b}\right)$$

$$66.84 * 10^3 * 10^3 = 0.87 * 500 * \text{A}_\text{st} * \left(272 - \frac{500 * \text{A}_\text{st}}{30 * 1000}\right)$$

Solving, we get,

A_{st} (required) = 585.946 mm^2

A_{st} (minimum) = 0.12% of bD = 0.12/100 × 1000 × 235

= 282 mm^2 < A_{st} (required)

Provide 16 mm dia bars

Then, $A_o = \frac{\pi}{4} \times 16^2$ = 201.062 mm^2

Spacing (required) = (201.062×1000)/585.946 =343.141 mm

Provide spacing of 200 mm.

Then,

$A_{st,\ provided}$ = (201.062×1000)/200 = 1005.31 mm^2 > $A_{st,req}$ (ok)

Hence, provide 16 mm ϕ bars at 200 mm c/c spacing.

5.1.6. Calculation for Transverse Reinforcement

Effective depth provided = 320 – 40 – 16 – 10/2 = 259 mm

Bending Moment in the transverse direction(M_t) = $0.2BM_{DL} + 0.3BM_{DL}$

= 0.2×34.953+0.3×(21.987+9.9)

= 16.557 kNm

Calculation of Reinforcement

$$M = 0.87 * f_y * A_{st} * \left(d - \frac{f_y\, A_{st}}{f_{ck}\, b}\right)$$

$$16.557 * 10^3 * 10^3 = 0.87 * 500 * A_{st} * \left(259 - \frac{500 * A_{st}}{30 * 1000}\right)$$

Solving, we get,

A_{st} (required) = 148.377 mm^2

A_{st} (minimum) = 0.12% of bD = 0.12/100 × 1000 × 235

= 282 mm^2 > $A_{st,req}$ (ok) so, provide minimum reinforcement

Provide 10 mm dia bars

Then, $A_o = \frac{\pi}{4} \times 10^2 = 78.54$ mm^2

Spacing (required) = (78.54×1000)/282 =278.51 mm

Provide spacing of 200 mm.

Then,

$A_{st,\ provided}$ = (78.54×1000)/200 = 392.7 mm^2 > $A_{st,req}$ (ok)

Hence, provide 10 mm ϕ bars at 200 mm c/c spacing.

5.1.7. Check For Shear

From Irc 112 10.3.2

Design Shear Force(V_{ED})= Total DL + Live Load = 32.447 + 60.447= 92.892 kN

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{272}} = 1.85 < 2$$

$V_{R.D.C}$=[0.12×K×(80×ρ^1×fck)$^{0.33}$+0.15σ_{cp}]×A_{net}

$$\text{Therefore, } V_{RDC} = \left[0.12 * 1.85 * \left(80 * \frac{1005.31}{272*1000} * 30\right)^{0.33}\right] * 1000 * 272$$

= 124.091 kN > V_{ED}

$N_{ED} = 0$

$\sigma_{cp} = 0$

Hence $V_{rdc} > 92.892$ kN so we provide minimum shear reinforcement.

5.1.8. Check For Crack Width

Bending moment for crack width check =40.54 kNm

To calculate neutral axis depth,

$b * x^2 = kA_{st} * (d - x)$

$1000 \times x^2 = 6.45 \times (272 - x)$

Solving we get, x=53.26

We know,

$\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \dfrac{f_{ct.eff}}{\rho_{p.eff}}\left(1 + \alpha_e \rho_{p.eff}\right)}{E_s} \geq 0.6 \frac{\sigma_{sc}}{E_s}$$

$$\rho_{p,eff} = \frac{A_s}{b * h_{cef}}$$

Where h_{ceff} is least of following,

i) 2.5(h-d) = 2.5×(320-272) = 2.5×48 = 120

ii) (h-x)/3 = (320-53.26)/3 = 88.91

iii) h/2 = 320/2 = 160

So, h_{cef} taken as 88.91.

$$\rho_{p,eff} = \frac{1005.31}{1000 * 88.91} = 0.011$$

$$\sigma_{sc} = \frac{M}{\left(d - \frac{x}{3}\right) A_{st}} = \frac{40.54 * 10^6}{\left(272 - \frac{53.26}{3}\right) 1005.34} = 158.6$$

$K_t = 0.5$

Using above equation we get,

E_{sm}-E_{cm} = $1.84 \times 10^{-4} < 0.6 \times (\sigma_{sc}/Es)$

$=4.758\times10^{-}$

Calculating S_{rmax}

$$S_{(rmax)} = 3.4c + \frac{0.425k_1k_2 * \phi}{\rho_{p,eff}}$$

$=149.26$

Therefore crack width can be calculated is,

$W_k = S_{r,max} \times (E_{sm} - E_{sm})$

$=149.26\times6.60\times10^{-4}$

$=0.18mm < 0.2$ mm (ok)

5.2. Design of one-way slab panel

There are three longitudinal & four cross girders.

Wearing Coat=0.1 m

Carriageway width= 7.5m

Web thickness of girder= 0.3 m

slab thickness=0.220 m

Size of slab along longer direction = 10m

Size of slab along shorter direction =3.25m

Size of panel is 10 m × 3.25m.

5.2.1. Bending moment and shear force due to dead load

Dead load calculation for a slab

Length/breath= Size of slab along longer direction/ Size of slab along shoter direction

= 10/3.25 =3.076> 2 (one way slab)

Dead wt of slab= (0.22×1×1×25)= 5.5 kN/mm^2

Factored Dead wt of slab = 5.5×1.35 = 7.425 kN/mm^2

Dead wt. of wearing course = 0.1×1×1×25 = 2.2 kN/mm^2

Factored Dead wt. of wearing course = 2.2×1.75 =3.85 kN/mm^2

Total Dead load = 7.7 kN/mm^2

Factored Total Dead load =11.275 kN/mm^2

For 1 m span along (10 m) length of slab

Factored Dead load BM = $\frac{wl^2}{8} = \frac{11.275 * 3.25^2}{8} = 14.887$ kNm

Factored Dead load SF = $\frac{wl}{2} = \frac{11.275 * 3.25}{2} = 18.322$ kN

5.2.2. Live Load Calculation (IRC 6-2017)

5.2.2.1. IRC Class A Loading

For Bending moment due to live load

Wheel load(kN)= 57 kN

length of tire contact(W) = 0.5 m

c/c spacing of load(m)=1.2m (from code)

width of tire contact area(B) = 0.25 m

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat=0.25+2×0.1 = 0.45m

Distance of C.G. of concentrated load from the nearer support (a) = $\frac{3.25}{2} = 1.625$ m

From IRC 112-2020 Table 3.2 $\alpha = 2.6$

Effective width of slab on which load acts (beff)

$= \alpha \text{ x a x } \left(1 - \frac{\text{a}}{\text{Size of slab along shoter direction}}\right) + \text{ b1}$

$= 2.6 * 1.625 * \left(1 - \frac{1.625}{3.25}\right) + 0.45$

= 2.5625 m

Effective width (b_{eff}) = $\frac{2.5625}{2} + 1.2 + \frac{2.5625}{2} = 3.7625$ m

Effective length(l_{eff}) =0.5+2×(0.22+0.1)

= 1.14 m

Intensity of load = $\frac{114}{1.14*3.7625} = 26.578$ kN/m^2

Maximun Bending Moment

$$= \left(\frac{26.578*1.14*3.25}{4}\right) - \frac{26.578*1.14*1.14}{8}$$

=20.3 kNm/m

Continuity factor= 0.8

Impact factor= $\frac{4.5}{6+\text{Siz of slab along shorter direction}} = \frac{4.5}{6+30} = 0.125$

BM including CF and IF = Max. BM x Continuity factor x (1 + Impact factor)

=20.3 × 0.8 × (1 + 0.125) = 18.270 kNm/m

For Shear Force due to live Load

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat =0.25+2×0.1= 0.45m

Distance of C.G. of concentrated load from the nearer support (a)

$$= \frac{(\text{Effective length of BM})}{2} = 0.57\text{m}$$

Effective width of slab on which load acts (beff)

$$= \alpha \text{ x a x } \left(1 - \frac{\text{a}}{\text{Size of slab along shoter direction}}\right) + \text{ b1 } = 1.67 \text{ m}$$

Effective width (m) = beff + c/c spacing of load = 1.67 + 1.2 = 2.87 m

Effective length =1.14 m

Intensity of load for shear force =$\frac{114}{2.87*1.14} = 34.84$ kN/m^2

Maximum shear force at support = $\frac{34.84*(3.25-1.14/2)*1.14}{3.25}$

= 32.751 kN

SF including CF and IF = Max. SF x (1 + Impact factor)

=32.751 × (1 + 0.125) = 36.844 kNm/m

5.2.2.2. IRC 70R wheel Loading

For Bending moment due to live load

Wheel load(kN)= 85 kN

length of tire contact(W) = 0.86 m

c/c spacing of load(m)=1.370m (from code)

width of tire contact area(B) = 0.25 m

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat=0.25+2×0.1 = 0.45m

Distance of C.G. of concentrated load from the nearer support (a) = $\frac{3.25}{2} = 1.625$ m

From IRC 112-2020 Table 3.2 $\alpha = 2.6$

Effective width of slab on which load acts (beff)

$= \alpha \text{ x a x } \left(1 - \frac{\text{a}}{\text{Size of slab along shoter direction}}\right) + \text{ b1}$

$= 2.6 * 1.625 * \left(1 - \frac{1.625}{3.25}\right) + 0.45$

= 2.5625 m

Effective width (b_{eff}) = $\frac{2.5625}{2} + 1.37 + \frac{2.5625}{2} = 3.933$m

Effective length(l_{eff}) =0.86+2×(0.22+0.1)

= 1.5 m

Intensity of load = $\frac{170}{1.5*3.933} = 28.816$ kN/m^2

Maximun Bending Moment

$$= \left(\frac{28.816*3.25*1.5}{4}\right) - \frac{28.816*1.5*1.5}{8}$$

=27.015 kNm/m

Continuity factor= 0.8

From clause 208.3 of irc 6 the impact factor for wheel load for span less than 9m is given as 25%

BM including CF and IF = Max. BM × Continuity factor × (1 + Impact factor)

=27.015 × 0.8 × (1.25) = 27.018kNm/m

For Shear Force due to live Load

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat =0.25+2×0.1= 0.45m

Distance of C.G. of concentrated load from the nearer support (a)

$$= \frac{\text{(Effective length of BM)}}{2} = 0.75\text{m}$$

Effective width of slab on which load acts (beff)

$$= \alpha * a * \left(1 - \frac{a}{\text{Size of slab along shoter direction}}\right) + b1 = 1.95\text{m}$$

Effective width (m) = beff + c/c spacing of load = 1.95 + 1.37 = 3.32 m

Effective length =1.5 m

Intensity of load for shear force $= \frac{170}{3.32*1.5} = 34.136$ kN/m^2

Maximum shear force at support = $\frac{34.84*(3.25-1.5/2)*1.5}{3.25}$

= 39.388 kN

SF including CF and IF = Max. SF × (1 + Impact factor)

=39.388 × (1.25) = 49.235 kNm/m

5.2.2.3. IRC 70R track Loading

For Bending moment due to live load

Track load(kN)= 350 kN

length of tire contact(W) = 0.84 m

width of tire contact area(B) = 4.570 m

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat=4.570+2×0.1 = 4.77m

Distance of C.G. of concentrated load from the nearer support (a) = $\frac{3.25}{2} = 1.625$ m

From IRC 112-2020 Table 3.2 $\alpha = 2.6$

Effective width (beff)

$$= \alpha \text{ x } a \text{ x } \left(1 - \frac{a}{\text{Size of slab along shoter direction}}\right) + b1$$

$$= 2.6 * 1.625 * \left(1 - \frac{1.625}{3.25}\right) + 4.77$$

= 6.883m

Effective length(l_{eff}) =0.84+2×(0.22+0.1)

= 1.48 m

Intensity of load = $\frac{350}{1.48*6.883} = 34.361$ kN/m^2

Maximun Bending Moment

$$= \left(\frac{34.361*3.25*1.5}{4}\right) - \frac{34.361*1.5*1.5}{8}$$

=31.911 kNm/m

Continuity factor= 0.8

From clause 208.3 of irc 6 the impact factor for wheel load for span less than 9m is given as 25%

BM including CF and IF = Max. BM × Continuity factor × (1 + Impact factor)

=31.911 × 0.8 × (1.25) = 31.911kNm/m

For Shear Force due to live Load

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat =0.25+2×0.1= 0.45m

Distance of C.G. of concentrated load from the nearer support (a)

$$= \frac{\text{(Effective length of BM)}}{2} = 0.74\text{m}$$

Effective width of slab(beff)

$$= \alpha * a * \left(1 - \frac{a}{\text{Size of slab along shoter direction}}\right) + b1 = 6.255\text{m}$$

Effective length =1.48m

Intensity of load for shear force $= \frac{350}{6.255*1.48} = 37.802$ kN/m^2

Maximum shear force at support $= \frac{34.84*(3.25-1.48/2)*1.48}{3.25}$

= 43.208 kN

SF including CF and IF = Max. SF × (1 + Impact factor)

=43.208 × (1.25) = 54.010 kN

5.2.2.4. IRC 70R bogie Loading

For Bending moment due to live load

Wheel load(kN)= 100 kN

length of tire contact(W) = 0.86 m

c/c spacing of load(m)=1.2m (from code)

width of tire contact area(B) = 0.25 m

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat=0.25+2×0.1 = 0.45 m

Distance of C.G. of concentrated load from the nearer support (a) = $\frac{3.25}{2} = 1.625$ m

From IRC 112-2020 Table 3.2 $\alpha = 2.6$

Effective width (beff)

$$= \alpha \text{ x a x } \left(1 - \frac{\text{a}}{\text{Size of slab along shoter direction}}\right) + \text{ b1}$$

$$= 2.6 * 1.625 * \left(1 - \frac{1.625}{3.25}\right) + 0.45$$

= 2.563m

Effective length(l_{eff}) =0.86+2×(0.22+0.1)

= 1.5 m

Intensity of load = $\frac{200}{1.5*2.563} = 35.250$ kN/m^2

$$\text{Maximum Bending moment } = \left(\frac{35.25 * 3.25 * 1.5}{4}\right) - \frac{35.250 * 1.5 * 1.5}{8}$$

= 33.047 kNm/m

Continuity factor= 0.8

From clause 208.3 of irc 6 the impact factor for wheel load for span less than 9m is given as 25%

BM including CF and IF = Max. BM × Continuity factor × (1 + Impact factor)

=33.047 × 0.8 × (1.25) = 33.047 kNm/m

For Shear Force due to live Load

Breadth of concentration area of load (b1) = width of tire contact area+2 x Thickness of Wearing Coat =0.25+2×0.1= 0.45m

Distance of C.G. of concentrated load from the nearer support (a)

= $\frac{\text{(Effective length of BM)}}{2}$ =0.75m

Effective width of slab(beff)

$$= \alpha * \text{ a} * \left(1 - \frac{\text{a}}{\text{Size of slab along shoter direction}}\right) + \text{ b1 } = 1.95\text{m}$$

Effective width (m) = beff + c/c spacing of load = 1.95 + 1.22 = 3.17 m

Effective length =1.5m

Intensity of load for shear force =$\frac{350}{63.17*1.5} = 42.061$ kN/m^2

Maximum shear force at support = $\frac{42.061*(3.25-1.5/2)*1.5}{3.25}$

= 48.531 kN

SF including CF and IF = Max. SF × (1 + Impact factor)

=48.531× (1.25) = 60.665 kN

5.2.3. Maximum Bending Moment and Shear Force due to live load

Maximum Bending Moment = 33.047 kNm/m

Maximum Shear force =60.665 kN/m

Total Factored Maximum Bending Moment =33.047 ×1.5 +14.887 = 64.458 kNm

Total Factored Maximum Shear force = 60.665 ×1.5+18.322 = 109.320 kN

Design of Section using LSM:

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}}$ d [SP 105]

$= \frac{0.0035*172}{0.0035+0.0218}$ = 106.079 mm

Compressive force (C) = $\beta_1 \times F_{cd} \times b \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 1000 \times 106.079}{1000}$

= 1148.985 kN

CG from steel level (z) = d- $\beta_2 \times X_{lim}$ = 172 – 0.416 × 106.079

= 127.874 mm

$M_{u,\,lim}$ = C × z = 1148.985× $\frac{127.874}{1000}$ = 146.926 kNm

Since, $M_{u,\,lim}$ is less than factor bending moment assumed depth is satisfied.

5.2.4. Design of main reinforcement

Design Bending Moment, M_{Ed} = 64.458 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

To find the actual neutral axis depth corresponding to M_{Ed}

$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b\times f_{cd}}}$$

$$X_u = \frac{172}{2\times0.416} - \sqrt{\left(\frac{172}{2\times0.416}\right)^2 - \frac{64.458\times10^6}{0.810\times0.416\times1000\times13.40}}$$

X_u = 38.076 mm

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – β_2 × X_u) = (172 – 0.416 × 38.076) = 156.147 mm

$A_{st} = \frac{64.458\times10^6}{434.783\times156.147}$ = 949.450mm^2

Provide 16 mm diameter bar.

Cross sectional area of each bar (A) = 201.062 mm^2

Spacing required = $\frac{b*A}{A_{st}} = \frac{1000*201.062}{949.450}$ = 211.767 mm^2

Provide 16mm dia bars @140 mm spacing = 1436.157 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s, min} = 0.26 \frac{f_{ctm}}{f_{yk}}$ b_td but not less than 0.0013 b_td

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s, min} = 0.26\times \frac{2.5}{500} \times 1000\times172$ but not less than 0.0013×1000×172

=223.6 mm^2 but not less than 226.6mm^2

= 226.6mm^2

$A_{st, provided} > A_{s, min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s, max}$ = 0.025 A_c

$A_{s, max}$ = 0.025× [220×1000] = 5500 mm^2

$A_{st, provided} < A_{s, max}$, OK

5.2.5. Design of Transverse reinforcement.

From clause 16.6.1(3) of IRC 112 2020

The distribution bar should be 20 percent of the main reinforcement which is,

A_{st} transverse $=0.2\times A_{st}$ Provided

$= 0.2\times1436.157\ mm^2$

$= 287.234\ mm^2$

Provide 10 mm diameter bars @ 200 mm c/c

A_{st} transverse provided=(1000×78.53)/200

=392.65 > 287.234 (ok)

5.2.6. Design Of Shear Reinforcement

Design shear force, V_{Ed} =109.32 kN

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

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

$= 1+\sqrt{\frac{200}{172}}$

$= 2.078 \leq 2$

$= 2$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$= 0.031\times2^{/2}\times30^{1/2}$

$= 0.480$

$\sigma_{cp} = 0$

$\rho1 = \frac{A_{st}}{b_w.d} = 0.0083 \leq$ (0.02 - Reinforcement ratio for longitudinal reinforcement)

$\therefore\ \rho1 = 0.0083$

$\therefore V_{Rd.c} = [0.12\times2\times\ (80 \times 0.0083 \times 30)^{0.33}]\times1000\times172$

$= 111.010$ kN

And, $V_{Rd.c} = (Vmin+0.15\sigma cp)\times b_w d$

$= (0.480+0.15\times 0) \times 1000\times 172$

$=82.56$ kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\ min}$=111.010 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=109.32 kN

∴Since $V_{Ed} < V_{Rd.c}$, shear reinforcement design is not required.

5.2.7. Check for crack width

Bending moment for crack width check =44.057kNm

To calculate neutral axis depth,

$$\mathrm{b} * \mathrm{x}^2 = \mathrm{kA_{st}} * (d - x)$$

$1000\times x^2 = 6.45 \times(172- x)$

Solving we get, x=47.94

We know,

$\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \dfrac{f_{ct.eff}}{\rho_{p.eff}}\left(1+\alpha_e\rho_{p.eff}\right)}{E_s} \geq 0.6\frac{\sigma_{sc}}{E_s}$$

$$\rho_{p,eff} = \frac{A_s}{b * hcef}$$

Where h_{ceff} is least of following,

i) 2.5(h-d) = 2.5×(220-172) = 2.5×48 = 120

ii) (h-x)/3 = (220-47.94)/3 = 57.35

iii) h/2 = 220/2 = 110

So, h_{cef} taken as 57.35

$$\rho_{p,eff} = \frac{1436.157}{1000 * 57.35} = 0.025$$

$$\sigma_{sc} = \frac{\mathrm{M}}{\left(\mathrm{d} - \frac{\mathrm{x}}{3}\right)\mathrm{A_{st}}} = \frac{44.57 * 10^6}{\left(172 - \frac{47.94}{3}\right)1436.157} = 198.91$$

$K_t = 0.5$

Using above equation we get,

E_{sm}-E_{cm} = 7.04×10^{-4}

Calculating S_{rmax}

$$S_{(rmax)} = 3.4c + \frac{0.425k_1k_2 * \phi}{\rho_{p,eff}}$$

=244.8

Therefore crack width can be calculated is,

$W_k = S_{r,max} \times (E_{sm} - E_{sm})$

=244.8×7.04×10^{-4}

=0.17mm < 0.2 mm (ok)

5.3. Analysis and Design of Longitudinal Girders

5.3.1. Calculation of Loads

Deck slab

- Railing
  i. Post = (1.35×2×16×0.2×0.2×1.1×25)/30 = 1.584 kN/m
  ii. Steel pipe = (1.35×4×2×6.19×9.81)/1000 = 0.656 kN/m
- Footpath = 1.35×2×1.75×0.225×25 = 26.578kN/m
- Wearing coat = 1.75×0.1×7.5×22 = 28.875 kN/m
- Slab
  i. Cantilever portion = 1.35×2[0.15×2.1+(0.32-0.15)×0.5×2.1]×25 = 33.311kN/m
  ii. Middle portion = 1.35×0.22×6.8×25 = 50.49kN/m
  iii. Fillet = 1.35×4×0.5×0.1×0.15×25 = 1.013 kN/m

Total deck slab load on girder = 1.584+0.656+26.578+28.875+33.311+50.49+1.013

= 142.507 kN/m

Load per girder = 142.507/3 = 47.502 kN/m

T Beam Rib

Total depth of girder = 2300 mm

Slab thickness = 220 mm

Depth of girder excluding slab thickness = 2300 – 220 = 2080 mm

Width of web = 300 mm

Depth of web girder = 2080 – 250 = 1830 mm

Weight of bulb of girder = [(700×250) + (200×150)]×25 = 5.125 kN/m

Weight of web of girder = 1.83×0.3×25 = 13.725 kN/m

Total load per girder = 5.125 + 13.725 + 47.502 = 66.352 kN/m

Cross girder

Overall depth = ¾ of depth of main girder = 1725 mm

Depth excluding slab = 1725 – 220 = 1505 mm

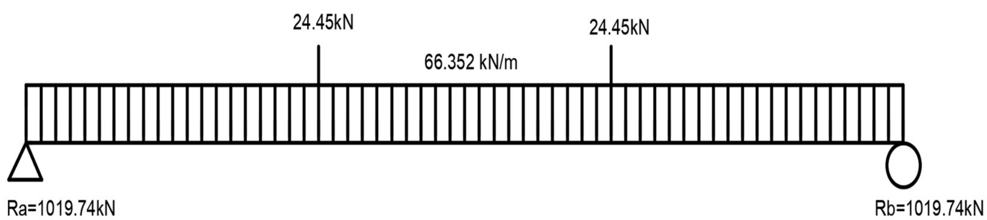
Width of cross girder = 300 mm

Weight of cross girder per m span = 1×0.3×1.505×25 = 11.288 kN/m

Weight of cross girder = 11.288×3.25×2 = 73.369 kN/m

This load is also taken equally by three girders.

Point load of cross girder on longitudinal girder = 73.369/3 = 24.456 kN/m



B.M. at L/8

$$Mu = 1036.615\times3.75-67.477\times3.75^2\times0.5$$

$$= 3357.485 \text{ kNm}$$

B.M. at quarter span (L/4) :

$$Mu = 1036.615\times7.5-67.477\times7.5^2\times0.5$$

$$= 5781.891 \text{ kNm}$$

B.M. at 3L/8 :

$$Mu = 1036.615\times11.5-67.477\times11.5^2\times0.5-24.456\times1.25$$

$$= 7234.52 \text{ kNm}$$

B.M. at mid span (L/2) :

Mu = 1036.615×15-67.477×15^2×0.5-24.456×5

= 7709.188 kNm

S.F. at support (L=0)

S.F. = 1019.74 kN

S.F. at L/8 :

SF = 1019.74 - 66.352 × 3.75

= 770.919 kN

S.F. at L/4 :

SF = 1019.74 – 66.352 × 7.5

= 522.098 kN

S.F. at 3L/8 :

SF =1019.74 – 66.352 × 11.25 - 24.456

= 232.233 kN

S.F. at mid span (L/2) :

SF = 1019.74 – 66.352 ×15 – 24.456

= 0

5.3.2. Analysis of Live Load on main girder:

a) Reaction factor

<u>For IRC Class A loading</u>

Reaction factor for outer girder (R_A):

$$R_A = \left(\frac{\sum W}{n}\right)\left[1 + \left(\frac{\sum I}{\sum dx^2 . I}\right) dx.e\right]$$

$$= \frac{2W}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (-3.25) \times (-0.7)]$$

= 0.882W

Reaction factor for inner girder (R_B):

$$R_B = \frac{2W}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times 0 \times (-0.7)]$$

= 0.67W

Reaction factor for outer girder (R_C):

$$R_C = \frac{2W}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (3.25) \times (-0.7)]$$

= 0.451W

<u>For class 70R tracked loading</u>

Reaction factor for outer girder (R_A):

$$\mathrm{R_A} = \left(\frac{\sum \mathrm{W}}{\mathrm{n}}\right)\left[1 + \left(\frac{\sum \mathrm{I}}{\sum \mathrm{dx}^2.\mathrm{I}}\right) dx.e\right]$$

$$= \frac{1\mathrm{W}}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (-3.25) \times (-1.1)]$$

= 0.503W

Reaction factor for inner girder (R_B):

$$\mathrm{R_B} = \frac{1W}{3}\left[1 + \frac{3}{2 \times (3.25)^2} \times 0 \times (-1.1)\right]$$

= 0.385W

Reaction factor for outer girder (R_C):

$$R_C = \frac{1W}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (3.25) \times (-1.1)]$$

= 0.164W

<u>For class 70R wheel loading</u>

Reaction factor for outer girder (R_A):

$$\mathrm{R_A} = \frac{1\mathrm{W}}{3}\ [1 + \frac{3}{2 * (3.25)^2} \times (-3.25) \times (-1.155)]$$

= 0.511W

Reaction factor for inner girder (R_B):

$$\mathrm{R_A} = \frac{1\mathrm{W}}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (0) \times (-1.155)]$$

= 0.388W

Reaction factor for outer girder (R_C):

$$\mathrm{R_C} = \frac{1\mathrm{W}}{3}\ [1 + \frac{3}{2 \times (3.25)^2} \times (3.25) \times (-1.155)]$$

= 0.156W

b) Impact factor

Impact factor for Class A = 1.125

Impact factor for Class 70R tracked = 1.1

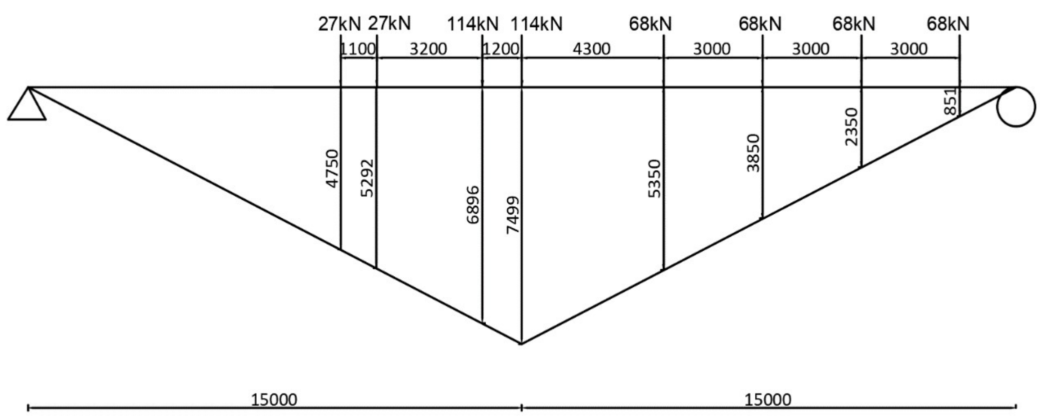
Impact factor for class 70R wheel = 1.12

c) Bending moment due to live load

<u>Class A loading</u>

Train of concentrated loads are placed in such a way that it produces maximum BM.

<u>At L/2</u>



BM = [(27×(4.75+5.3) + 114×(6.9+7.5) + 68×(5.35+3.85+2.35+0.85)] × 1.5

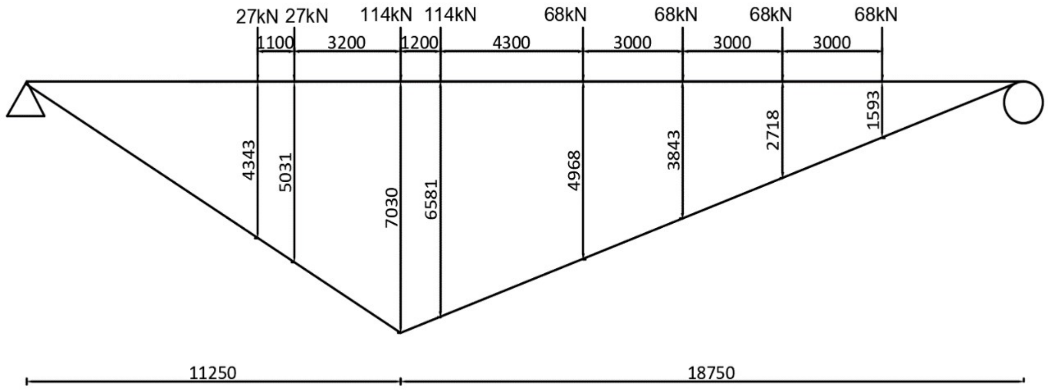
= 4134.225 kNm

Bm for outer girder A = 4134.225 × 0.882 × 1.125 × 2 = 8204.847 kNm

BM for inner girder B = 4134.225 × 0.67 × 1.125 × 2 = 6201.338 kNm

BM for outer girder C = 4134.225 × 0.451 × 1.125 × 2 = 4197.828 kNm

<u>At 3L/8</u>



BM =
[(27×(4.3435+5.031)+114×(7.031+6.581)+68×(4.9685+3.8436+2.7186+1.5936)] × 1.5

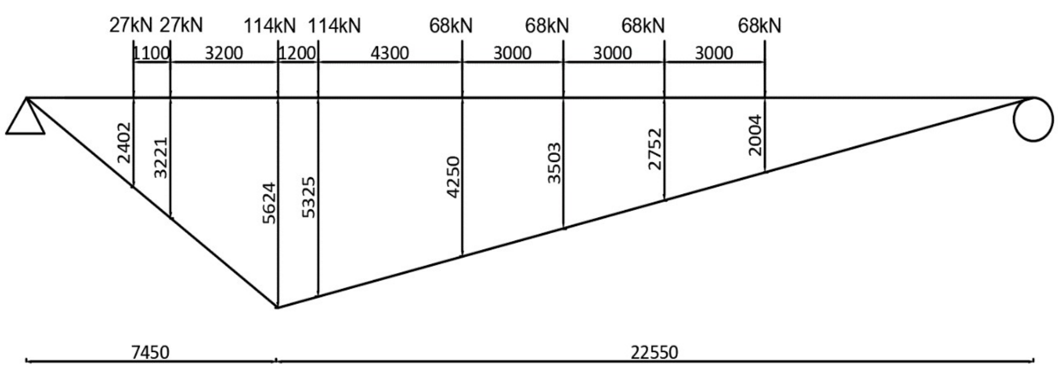
= 4045.998 kNm

BM for outer girder A = 4045.998 × 0.882 × 1.125 × 2 = 8029.75 kNm

Bm for inner girder B = 4045.998 × 067 × 1.125 × 2 = 6068.997 kNm

BM for outer girder C = 4045.998 × 0.451 × 1.125 × 2 = 4108.244 kNm

At L/4



BM = [(27×(2.4+3.225)+114×(5.625+5.325)+68×(4.25+3.5+2.75+2)] × 1.5

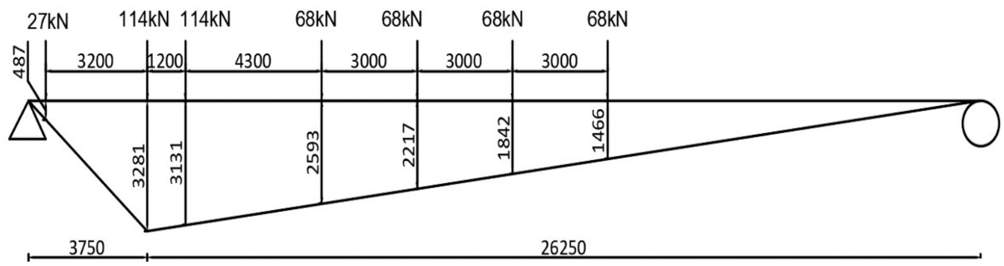
= 3375.263 kNm

BM for outer girder A = 3375.263 × 0.882 × 1.125 × 2 = 6698.598 kNm

BM for inner girder B = 3375.263 × 0.67 × 1.125 ×2 = 5062.894 kNm

BM for outer girder C = 3375.263 × 0.451 × 1.125 × 2 = 3427.19 kNm

At L/8



BM = [(27×0.4812+114×(3.2812+3.1312)+68×(2.5937+2.2187+1.8437+1.4687)] × 1.5

= 1944.739 kNm

BM for outer girder A = 1944.739 × 0.882 × 1.125 × 2 = 3859.558 kNm

BM for inner girder B = 1944.739 × 0.67 × 1.125 × 2 = 2917.108 kNm

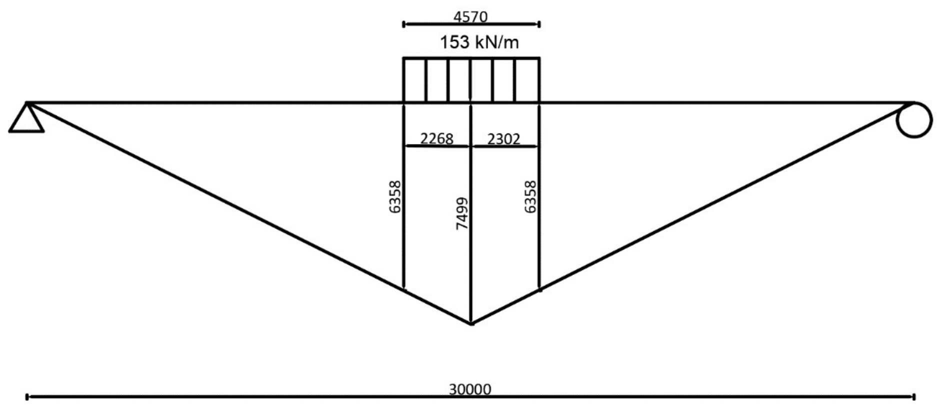
BM for outer girder C = 1944.739 × 0.451 × 1.125 × 2 = 1974.658 kNm

Class 70R Tracked vehicle :

The UDL is placed in such a way that the condition for maximum BM is satisfied, i.e

$$\frac{a'}{a} = \frac{b'}{b}$$

At L/2



BM = [1/2×2.285×(6.3575+2×7.5+6.3575)×153.173] × 1.5

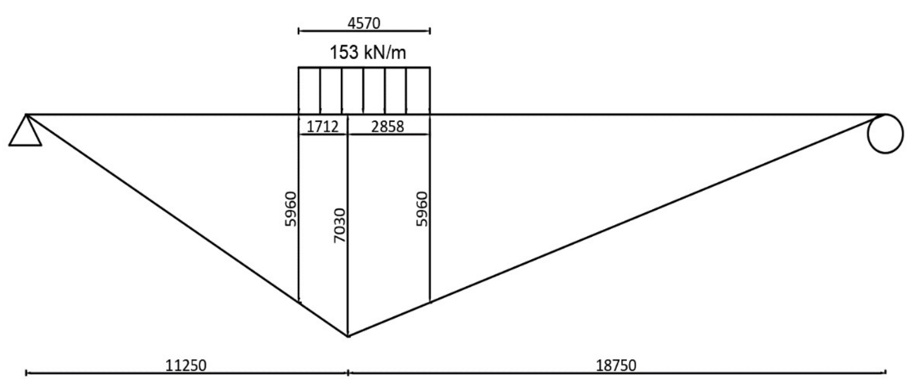
= 7275.194 kNm

BM for outer girder A = 7275.194 × 0.503 × 1.1 = 4021.876 kNm

Bm for inner girder B = 7275.194 × 0.385 × 1.1 = 3084.28 kNm

BM for outer girder C = 7275.194 × 0.164 × 1.1 = 1313.266 kNm

At 3L/8



BM = [1/2×1.741×(5.96+7.03) + 1/2×2.856×(5.96+7.03)] ×1.5×153.173

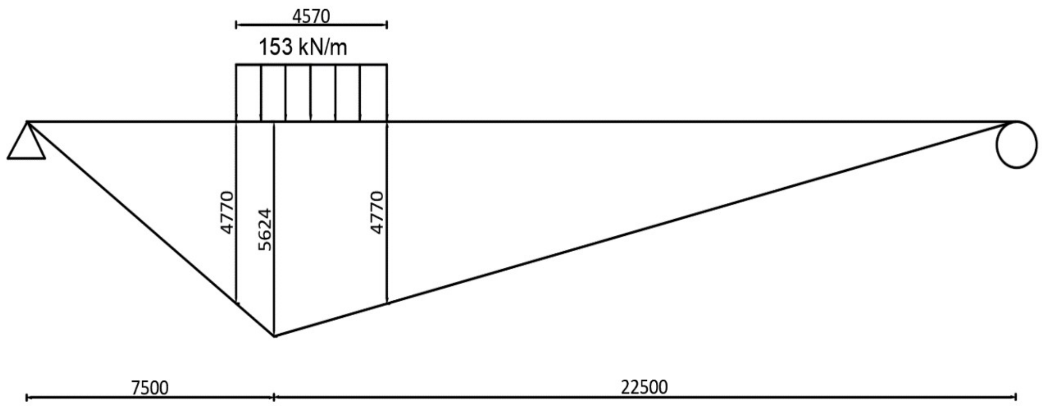
= 6860.048 kNm

BM for outer girder A = 6860.048 × 0.503 × 1.1 = 3792.375kNm

BM for inner girder B = 6860.048 × 0.385 × 1.1 = 2908.281 kNm

BM for outer girder C = 6860.048 × 0.164 ×1.1 = 1238.327kNm

At L/4



BM = [1/2×1.143×(4.77 + 5.625) + 1/2×3.427×(4.77+5.625)] ×1.5×153.173

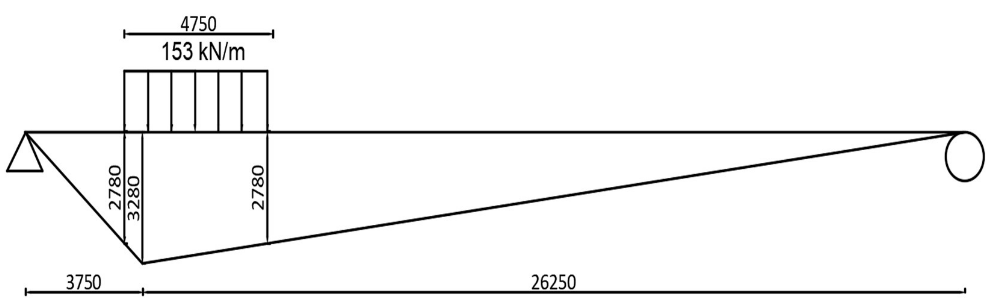
= 5457.38kNm

BM for outer girder A = 5457.38 × 0.503 × 1.1 = 3016.951 kNm

BM for inner girder B = 5457.38 × 0.385 × 1.1 = 2313.628 kNm

BM for outer girder C = 5457.38 × 0.164 × 1.1 = 985.127 kNm

At L/8



BM = [1/2×0.57×(3.28+2.78) +1 /2×4×(3.28+2.78)] ×1.5×153.137

= 3180.755 kNm

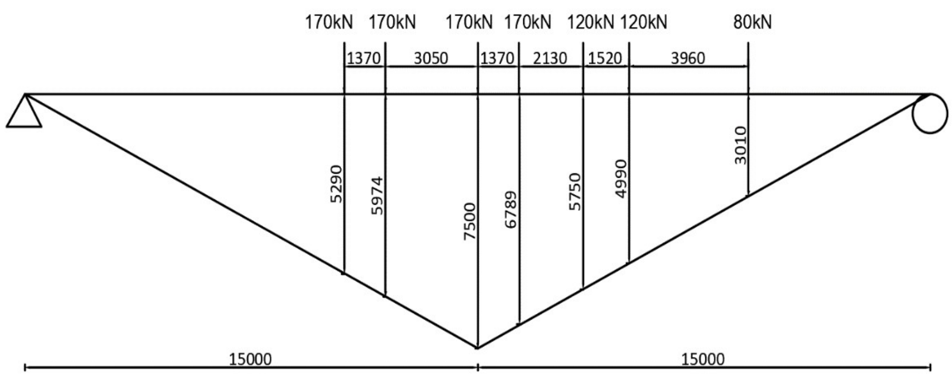
BM for outer girder A = 3180.755 × 0.503 × 1.1 = 1758.387 kNm

BM for inner girder B = 3180.755 × 0.385 × 1.1 = 1348.464 kNm

BM for outer girder C = 3180.755 × 0.164 × 1.1 = 574.167 kNm

Class 70R wheeled vehicle :

At L/2



BM = [80×3.01 + 120×(4.99+5.75) + 170×(6.815+7.5+5.975+5.29)] ×1.5

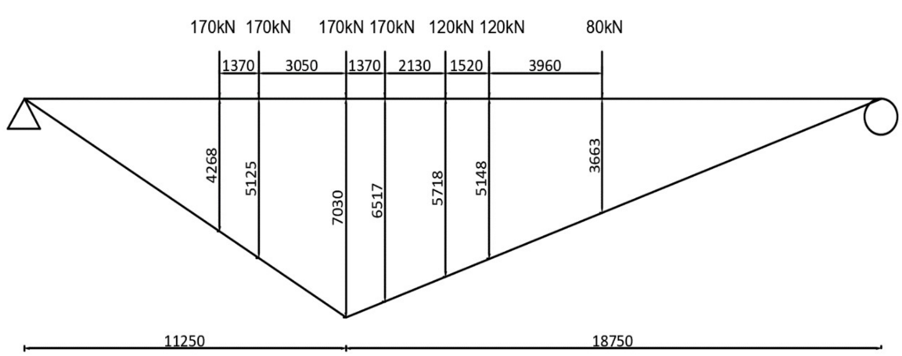
= 8817.3 kNm

BM for outer girder A = 8817.3 × 0.511 × 1.12 = 5046..570 kNm

BM for inner girder B = 8817.3 × 0.388 × 1.12 = 3831.724 kNm

BM for outer girder C = 8817.3 × 0.156 × 1.12 = 1537.014 kNm

At 3L/8



BM = [170×(4.268+5.124+7.0312+6.5174) + 120×(5.7187+5.1487) + 80×3.6637] ×1.5

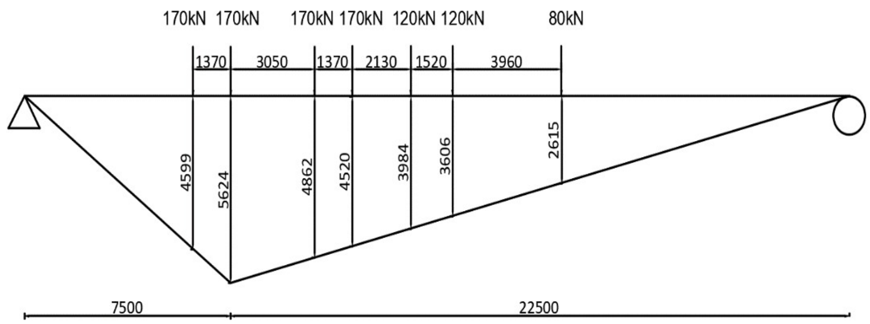
= 8245.629 kNm

BM for outer girder A = 8245.629 × 0.511 × 1.12 = 4719.375 kNm

BM for inner girder B = 8245.629 × 0.388 × 1.12 = 3583.293 kNm

BM for outer girder C = 8245.629 × 0.156 × 1.12 = 1437.361 kNm

At L/4



BM = [170×(4.5975+5.625+4.8625+4.52) + 120×(3.9875+3.6075) + 80×2.6175] ×1.5

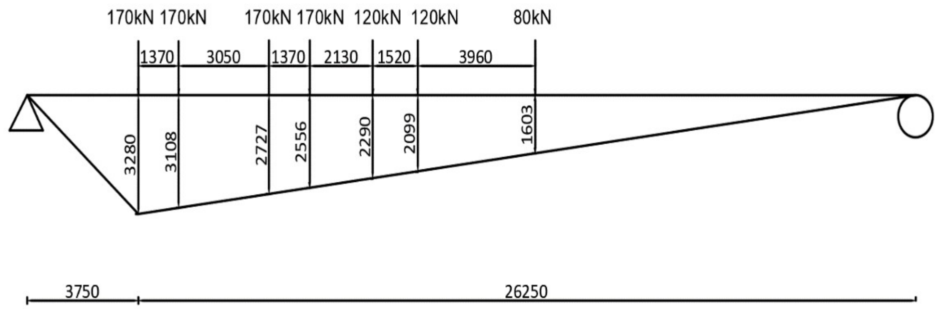
= 6680.475 kNm

BM for outer girder A = 6680.485 × 0.511 × 1.12 = 3823.561 kNm

BM for inner girder B = 6680.485 × 0.388 × 1.12 = 2903.126 kNm

BM for outer girder C = 6680.485 × 0.156 × 1.12 = 1164.527 kNm

At L/8



BM = [170×(3.28+3.1088+2.7277+2.5565) + 120×(2.2903+2.1004) + 80×1.6056] ×1.5

= 3959.613 kNm

BM for outer girder A = 3959.613 × 0.511 ×1.12 = 2266.279 kNm

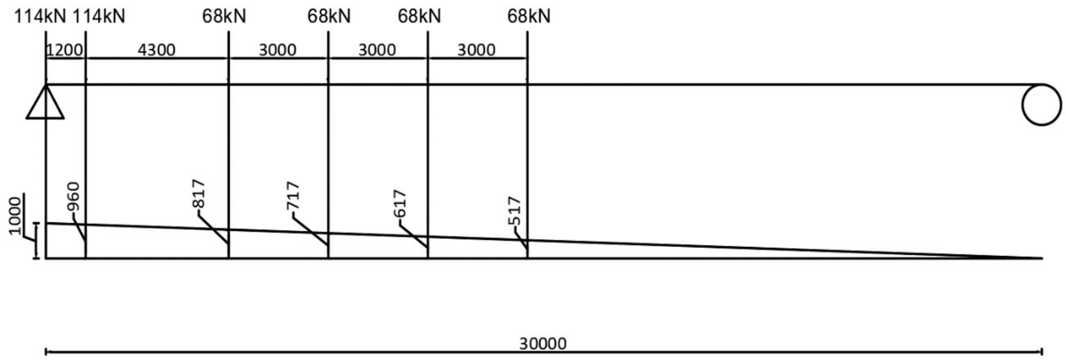
BM for inner girder B = 3959.613 × 0.388 × 1.12 = 1720.724 kNm

BM for outer girder C = 3959.613 × 0.156 × 1.12 = 690.232 kNm

d) Shear Force due to live load

<u>Class A loading</u>

At support



SF = [114×(1+0.96) + 68×(0.8166+0.7167+0.6167+0.5167)] ×1.5

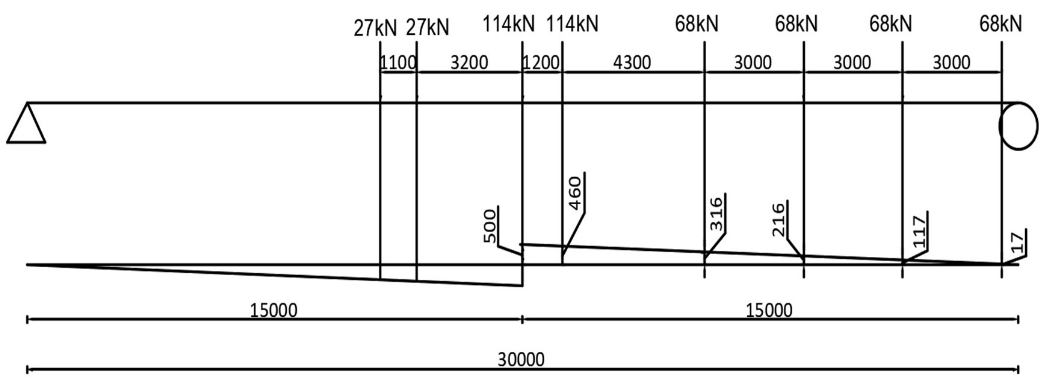
=607.163 kN

SF for outer girder A = 607.163 × 0.882 × 1.125 × 2 = 1204.986kNm

SF for inner girder B = 607.163 × 0.67 × 1.125 × 2 = 910.745 kNm

SF for outer girder C = 607.163 × 0.451 × 1.125 × 2 = 616.504 kNm

<u>At L/2</u>



SF=[27×(-0.3566-0.3933)+114×(0.5+0.46)+68×(0.3167+0.2167+0.1167+0.0167)] ×1.5

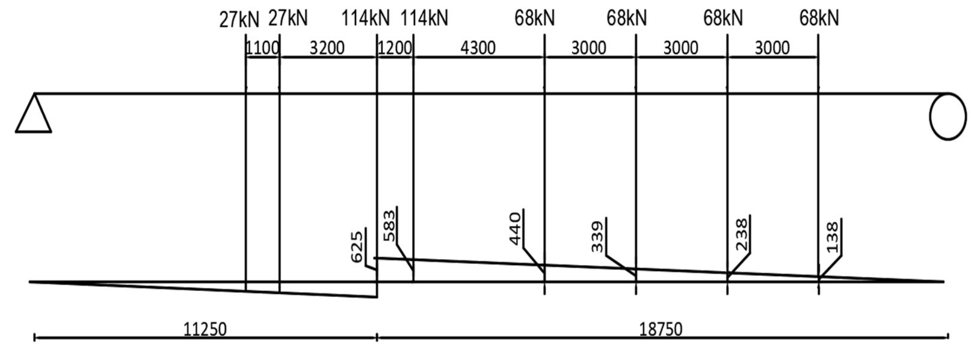
= 201.803 kN

SF for outer girder A = 201.803 × 0.882 × 1.125 × 2 = 400.501 kNm

SF for inner girder B = 201.803 × 0.67 × 1.125 × 2 = 302.704 kNm

SF for outer girder C = 201.803 × 0.451 × 1.125 = 201.907

At 3L/8



SF=[27×(0.23160.2683)+114×(0.625+0.585)+68×(0.4416+0.3416+0.2416+0.1417)] ×1.5

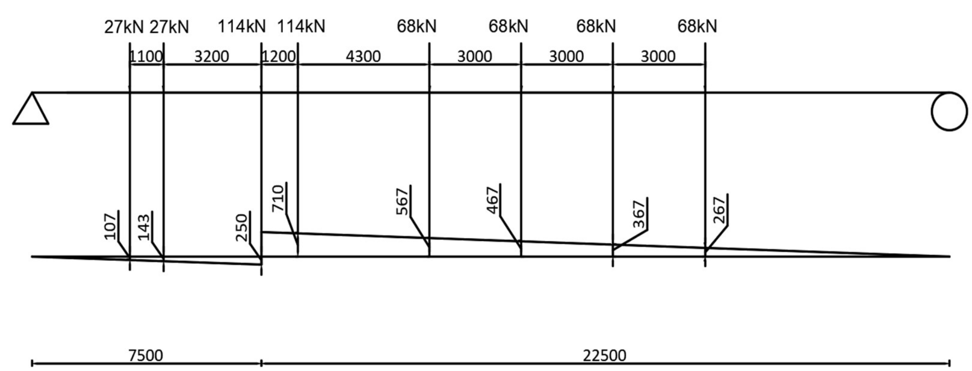
= 305.647 kN

SF for outer girder A = 305.647 × 0.882 × 1.125 × 2 = 606.592 kNm

SF for inner girder B = 305.647 × 0.67 × 1.125 × 2 = 458.471 kNm

SF for outer girder C = 305.647 × 0.451 × 1.125 × 2 = 310.349 kNm

At L/4



$$SF = [27 \times (-0.1066 - 0.1433) + 114 \times (0.75 + 0.71) + 68 \times (0.5667 + 0.4667 + 0.3667 + 0.2667)] \times 1.5$$

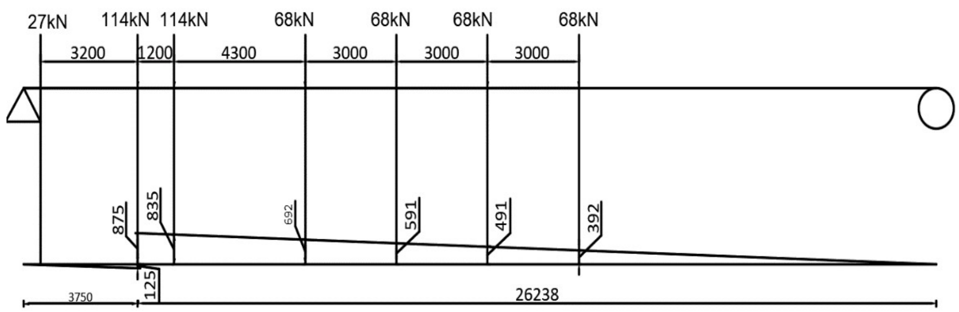
= 409.553 kN

SF for outer girder A = 409.553 × 0.882 × 1.125 × 2 = 812.804 kNm

SF for inner girder B = 409.553 × 0.67 × 1.125 × 2 = 614.329 kNm

SF for outer girder C = 409.553 × 0.451 × 1.125 × 2 = 415.853 kNm

At L/8



SF = [27×(-0.0183) + 114×(0.875+0.835) + 68×(0.6917+0.5917+0.4917+0.3917)] ×1.5

= 512.682 kN

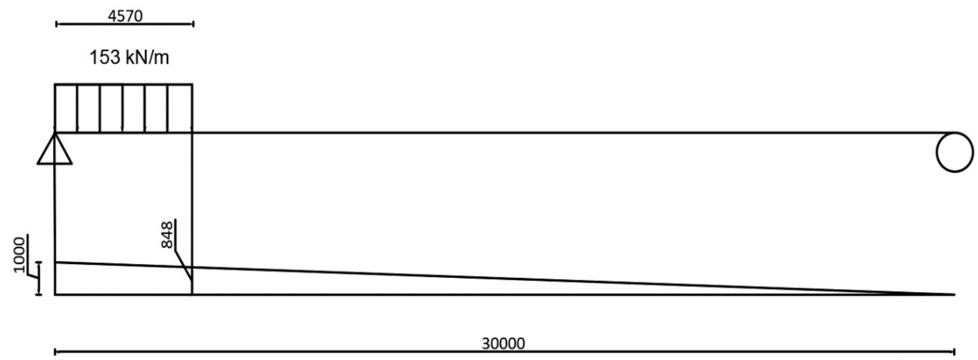
SF for outer girder A = 512.682 × 0.882 × 1.125 × 2 = 1017.477 kNm

SF for inner girder B = 512.685 × 0.67 × 1.125 × 2 = 769.024 kNm

SF for outer girder C= 512.685 × 0.451 × 1.125 × 2 = 520.570 kNm

70R Tracked vehicle

At support



SF = 1/2×4.57×(1+0.848) ×153.173×1.5

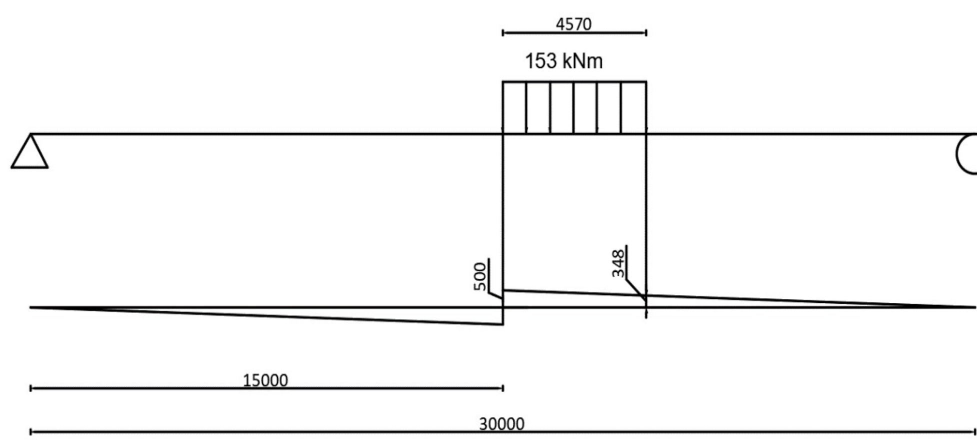
= 970.201 kN

SF for outer girder A = 970.201 × 0.503 × 1.1 = 536.347 kN

SF for inner girder B = 970.201 × 0.385 × 1.1 = 411.382 kN

SF for outer girder C = 970.201 × 0.164 × 1.1 = 175.134 kN

At L/2



SF = 0.5×4.57×(0.5+0.348)×153.173×1.5

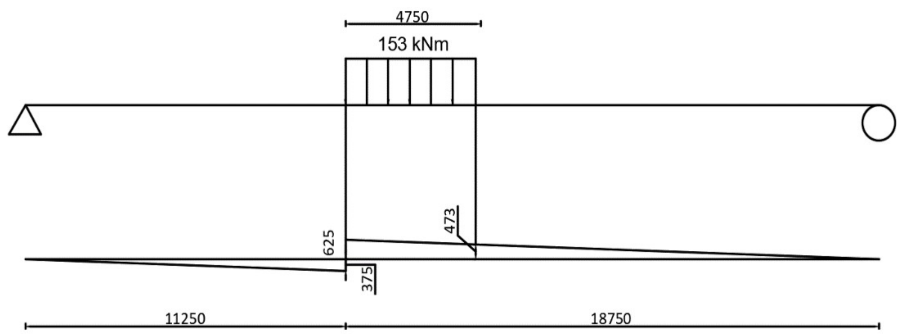
= 445.2 kN

SF for outer girder A = 445.2 × 0.503 × 1.1 = 246.116 kN

SF for inner girder B = 445.0.385 × 1.1 = 188.740 kN

SF for outer girder C =445.0385 × 0.164 × 1.1 = 80.364 kN

At 3L/8



SF = 0.5×4.57×(0.625+0.473)×153.173×1.5

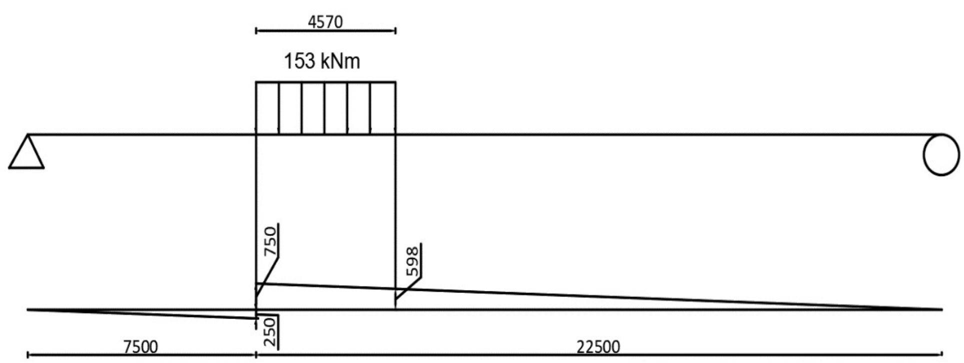
= 576.451 kN

SF for outer girder A = 576.451 × 0.503 × 1.1 = 318.674 kN

SF for inner girder B = 576.451 × 0.385 × 1.1 = 244.383 kN

SF for outer girder C = 576.451 × 0.164 × 1.1 = 104.057 kN

At L/4



SF = 0.5×4.57×(0.75+0.598)×153.173×1.5

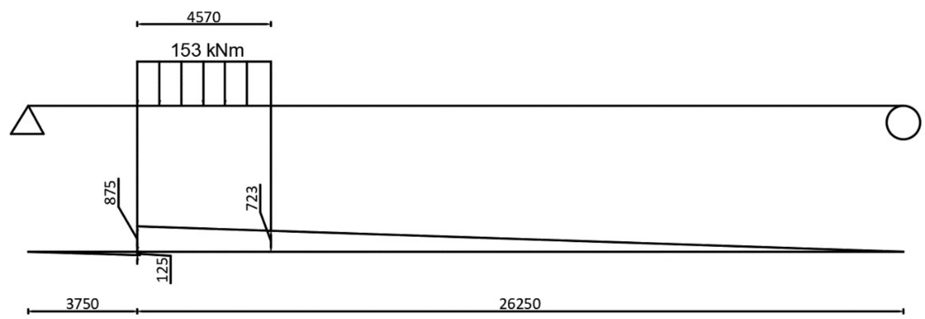
= 707.701 kN

SF for outer girder A = 707.701 × 0.503 × 1.1 = 391.231 kN

SF for inner girder B = 707.701 × 0.385 × 1.1 = 300.026 kN

SF for outer girder C = 707.701 × 0.164 × 1.1 = 127.749 kN

At L/8



SF = 0.5×4.57×(0.875+0.723)×153.173×1.5

= 838.951 kN

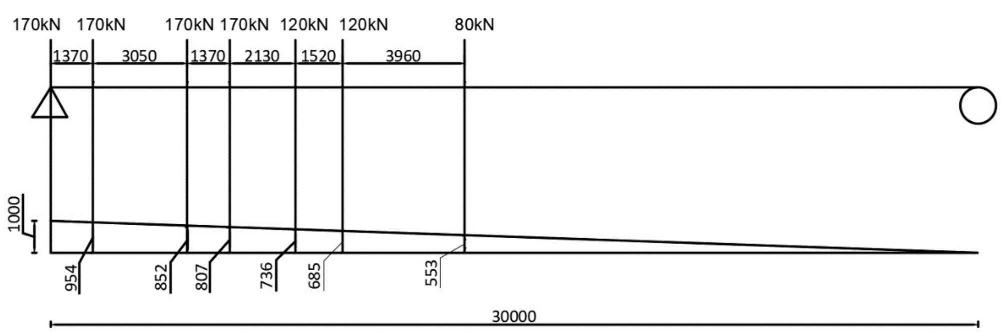
SF for outer girder A = 838.951 × 0.503 × 1.1 = 463.789 kN

SF for inner girder B = 838.951 × 0.385 × 1.1 = 355.669 kN

SF for outer girder C = 838.951 × 0.164 × 1.1 = 151.441 kN

70R Wheeled vehicle :

At support



SF = [170×(1+0.9543+0.8526+0.807) + 120×(0.736+0.6853) + 80×0.5533] ×1.5

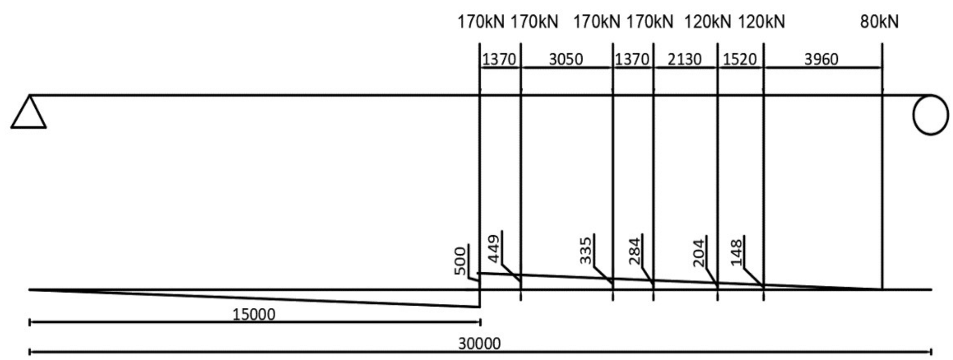
= 1243.775 kN

SF for outer girder A = 1243.775 × 0.511 ×1.12 = 711.873 kN

SF for inner girder B = 1243.775 × 0.388 × 1.12 = 540.506 kN

Sf for outer girder C =1243.775 × 0.156 × 1.12 = 216.812 kN

At L/2



SF = [170×(0.5+0.4543+0.3526+0.307) + 120×(0.236+0.1853) + 80×0.053] ×1.5

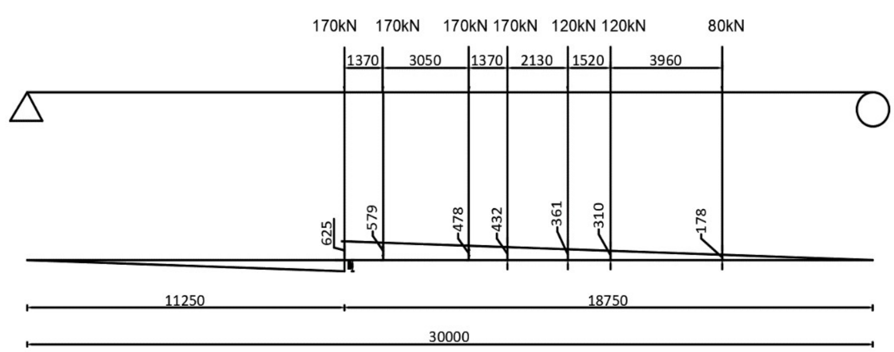
= 493.739 kN

SF for outer girder A = 493.739 × 0.511 × 1.12 = 282.591 kN

Sf for inner girder B = 493.739 × 0.388 × 1.12 = 214.563 kN

SF for outer girder C= 493.739 × 0.156 × 1.12 = 86.067 kN

At 3L/8



SF = [170×(0.625+0.8793+0.4776+0.432) + 120×(0.361+0.31)+80×0.1783] ×1.5

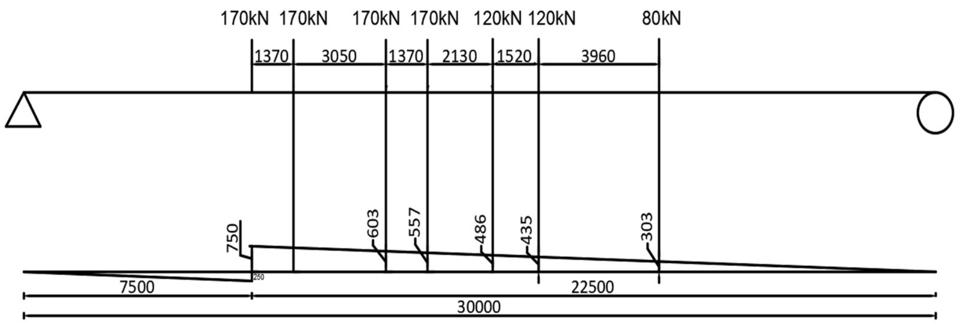
= 757.721 kN

SF for outer girder A = 757.721 × 0.511 × 1.12 = 433. 680 kN

SF for inner girder B = 757.721 × 0.388 × 1.12 = 329.282 kN

SF for outer girder C = 757.721 × 0.156 × 1.12 = 132.084 kN

At L/4



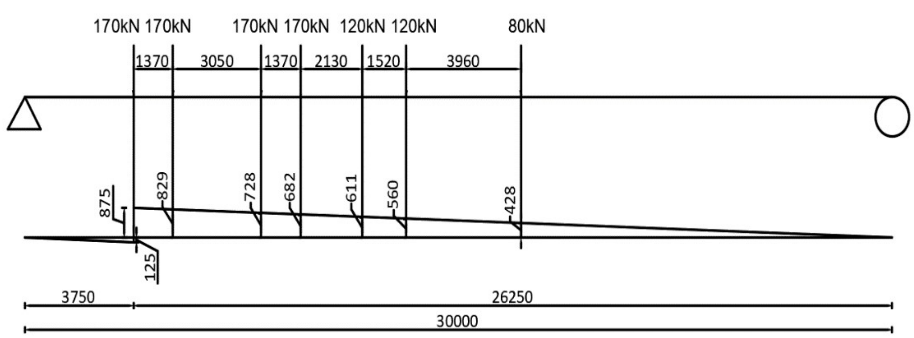
SF = [170×(0.75+0.7043+0.6026+0.557) + 120×(0.486+0.4353) + 80×0.3033] ×1.5

= 868.775 kN

SF for outer girder A = 868.775 × 0.511 × 1.12 = 497.242 kN

SF for inner girder B = 868.775 × 0.388 × 1.12 = 377.542 kN

SF for outer girder C = 868.775 × 0.156 × 1.12 = 151.443 kN



At L/8

SF = [170×(0.875+0.8293+0.7276+0.682) + 120×(0.611+0.5603) + 80×0.4283] ×1.5

= 1056.275 kN

SF for outer girder A = 1056.275 × 0.511 × 1.12 = 604.557 kN

SF for inner girder B = 1056.275 × 0.388 × 1.12 = 459.024 kN

SF for outer girder C = 1056.275× 0.156 × 1.12 = 184.128 kN

Summary:

(The Bending moment (BM) and Shear Force (SF) from class A loading is multiplied by lane distribution factor (LDF= 2) to get design BM and SF.)

Table: Total Design Bending moment and Shear Force for outer girder (A)

Section	Design Bending moment			Design shear Force		
	Class A	70R tracked	70R wheel	Class A	70R tracked	70R wheel
X=0				1204.986	536.347	711.873
X=L/2	8204.847	4021.876	5046.570	400.501	246.116	282.591
X=3L/8	8029.750	3792.375	4719.375	606.592	318.674	433.680
X=L/4	6698.598	3016.951	3823.561	812.804	391.231	497.242
X=L/8	3859.558	1758.387	2266.279	1017.477	463.789	604.557

Table: Total Design Bending moment and Shear Force for inner girder (B)

Section	Design Bending moment			Design shear Force		
	Class A	70R tracked	70R wheel	Class A	70R tracked	70R wheel
X=0				910.745	411.312	540.506
X=L/2	6201.338	3084.280	3831.724	302.704	188.740	214.563
X=3L/8	6068.997	2908.281	3583.293	458.471	244.383	329.282
X=L/4	5062.894	2313.628	2903.126	614.329	300.026	377.542
X=L/8	2917.108	1348.464	1720.724	769.024	355.669	459.024

Table: Total Design Bending moment and Shear Force for outer girder (C)

Section	Design Bending moment			Design shear Force		
	Class A	70R tracked	70R wheel	Class A	70R tracked	70R wheel
X=0				616.504	175.134	216.812
X=L/2	4197.828	1313.266	1537.014	204.907	80.364	86.067
X=3L/8	4108.244	1238.327	1437.361	310.349	104.057	132.084
X=L/4	3427.190	985.127	1164.527	415.853	127.749	151.443
X=L/8	1974.658	574.167	690.232	520.570	151.441	184.128

For the design of section, the maximum value of BM and SF is selected from the above table at various section for outer girder and intermediate main girder.

Table: Design BM and SF due to Live Load

Section	Outer Main girder		Intermediate main girder	
	BM kNm	SF kN	BM KNm	SF kN
X=0		1204.986		910.745
X=L/2	8204.847	400.501	6201.338	302.704
X=3L/8	8029.750	606.592	6068.997	458.471
X=L/4	6698.598	812.804	5062.894	614.329
X=L/8	3859.558	1017.477	2917.108	769.024

Table: BM and SF due to Dead Load

Section	Dead Load BM & SF	
	BM kNm	SF kN
X=0		1019.740
X=L/2	7709.188	0.000
X=3L/8	7234.520	232.233
X=L/4	5781.891	522.098
X=L/8	3357.485	770.919

Table: Total design BM and SF; DL BM and SF + LL BM and SF

Section	Outer Main girder		Intermediate main girder	
	BM kNm	SF kN	BM KNm	SF kN
X=0		2224.726		1930.485
X=L/2	15914.035	400.501	13910.526	302.704
X=3L/8	15264.269	838.825	13303.517	690.703
X=L/4	12480.489	1334.902	10844.785	1136.427
X=L/8	7217.043	1788.396	6274.593	1539.943

5.3.3. Design of Outer Girder

Effective width of flange:
As per clause7.6.1.2 of IRC 112, the effective flange width will be calculated.

The effective flange width b_{eff} for a T beam

$b_{eff,1} = 0.2\times b_1 + 0.1\times Lo \le 0.2\times Lo$ and $\le b_1$

$= 0.2\times 2.1 + 0.1\times 30 \le 0.2\times 30$ and ≤ 2.1

$= 3.42 \le 6$ and ≤ 2.1

$= 2.1m$

$b_{eff,2} = 0.2\times b_2 + 0.1\times Lo \le 0.2\times Lo$ and $\le b_2$

$= 0.2\times 1.475 + 0.1\times 30 <= 0.2\times 30$ and ≤ 1.475

$= 3.295 \le 6$ and ≤ 1.475

$= 1.475$ m

$b_{eff} = b_{eff,1} + b_{eff,2} + b_w \le b$

$= 2.1 + 1.475 + 0.3 \le b_1 + b_2 + b_w$

$= 3.875 \le 2.1 + 1.475 + 0.3$

$= 3.875 \le 3.875$

$= 3.875m$

Hence, effective width of flange, b_{eff} = 3.875m

Clear cover = 40mm

Let us assume 3 layers of bar of dia. 32mm

Effective depth, d = 2300 – 40 – 10 – 32 -32 - $\frac{32}{2}$ = 2170 mm = 2.170 m

Section properties:

Width of web(b_w)=300 mm

Average thickNess of left part of slab = 0.5(0.32+0.15) = 0.235 m

Average thickNess of left part of slab = 0.22 m

depth of flange (D_f)= $\frac{220+235}{2} = 223\ mm$

Overall depth of beam (D) =2300 mm

Material properties

a) Concrete used: M30 (IRC 112-2020 Table 6.4)

b) Characteristic strength, f_{ck}= 30 N/mm^2

c) Design compressive strength of concrete, f_{cd}=$\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]

d) $\alpha = 0.67$

e) $\gamma m = 1.5$

f) Design compressive strength of concrete, f_{cd}= $\frac{0.67 \times fck}{1.5} = \frac{0.67*30}{1.5}$ = 13.40 N/mm^2

g) Steel used: Fe500

h) Yield Strength of Steel, f_{yk}=500N/mm^2

i) Design yield strength of steel, f_{yd}= f_{yk}/1.15= 0.87fy = 434.783 N/mm^2

j) Young's Modulus of Elasticity, Es = 2×10^{5}N/mm^2

k) Yield strain for steel (ε_{yd}) = $\frac{\text{fyd}}{\text{Es}} = \frac{0.87*\text{fy}}{\text{Es}} = \frac{0.87*500}{200000}$ = 0.0218

l) Area factor (β_1) = 0.810

m) CG factor(β_2) = 0.416

n) Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

At L/2

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035*2170}{0.0035+ .0218} = 1338.326$ mm

CG from top = $\beta_2 \times X_{lim}$

$= 0.416 \times 1338.326$

$= 556.744$ mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

$= 4355.275$ kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim} = 2170 - 0.416 \times 1338.326$

$= 1613.256$ mm

$M_{u,\,lim1} = C \times z = 4355.275 \times \frac{1613.256}{1000} = 7026.175$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3875 - 300) \times 223}{1000}$

$= 10539.10$ kN

CG from steel level (z_2) = d- $\frac{Df}{2} = 2170 - \frac{223}{2}$

$= 2058.500$ mm

$M_{ur,\,lim2} = C \times z = 10539.10 \times \frac{2058.500}{1000} = 21990.575$ kNm

Total compressive force, C = 4355.275 + 10539.10 = 15038.090 kN

Total limiting moment, $M_{ur,\,lim} = M_{ur,\,lim1} + M_{ur,\,lim2}$

$= 7026.175 + 21990.575$

$= 29016.749$ kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{29016.749 \times 1000}{15038.090}$

$= 1929.550$ mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{15038.090}{434.783} = 34587.606 \text{ mm}^2$

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4} = 804.258 \text{ mm}^2$

Number of bars $= \frac{A_{st}}{A_0} = \frac{34587.606}{804.258} = 43.006 \approx 44$

So, this section can take up to 29016.749 kNm with 44 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 15914.035 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{15914.035\times10^6}{0.810\times0.416\times38750\times13.40}}$$

$$X_u = 180.729 \text{ mm}$$

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with $z = (d - \beta_2 \times X_u) = (2170 - 0.416 \times 180.729) = 2094.822$ mm

$A_{st} = \frac{15914.035\times10^6}{434.783\times2094.822} = 17472.737 \text{ mm}^2$

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{17472.737}{804.248} = 21.73$

Provide 22 number of bars of 32 mm diameter with area, = 17693.450 mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d$ but not less than $0.0013\, b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times 300\times2170$ but not less than $0.0013\times300\times2170$

$=846.3mm^2$ but not less than $846.3mm^2$

$= 846.3mm^2$

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max} = 0.025\times [(2300\text{-}250)\times300+700\times250+200\times150] = 20500\ mm^2$

$A_{st,\,provided} < A_{s,\,max}$, OK

Provide 2-32mm dia. bar as compressive reinforcement

A_{sc} provided = 1608.495 mm^2

SIDE REINFORCEMENT:

When the depth of beam is more than 750mm, skin(surface) reinforcement of 0.1 % of web area on each side is to be provided.

Minimum side reinforcement = 0.1% x 300 x 2170 = $651mm^2$

Providing 12mm bars at the middle section of the beam

$$\text{No of bars} = \frac{651}{\frac{\pi}{4}\times 12^2} = 5.76 \approx 6$$

Spacing of Bars =2170/6= 361.67 mm

So, provide 12 mm dia. rebar @ 250 mm c/c as side reinforcement on each side.

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =400.501 KN

<u>Maximum Allowable Shear Force (for maximum shear force take $\Theta = 45^o$)</u>

$$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta} \qquad \text{[IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]}$$

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for σ_{cp}=0 (RCC)

Lever Arm(z)=(d – $\beta_2 \times X_u$) = (2170 – 0.416 × 180.729)

= 2094.822 mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45°

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$

$= 1 \times 300 \times 2094.822 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.44 \quad ck}{\cot 45 + \tan 45}$

=2278.464 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED}=$400.501 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right) b_w d$$

K =1+$\sqrt{\frac{200}{d}}$≤2

=1+$\sqrt{\frac{200}{2170}}$

=1.304

V_{min} = 0.031$K^{3/2}$$fck^{1/2}$

=0.031×$1.304^{3/2}$×$30^{1/2}$

=0.253

σ_{cp} = 0

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0272 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ $\rho 1$=0.02

∴$V_{Rd.c}$ = [0.12×1.304× $(80 \times 0.02 \times 30)^{0.33}$] ×300×2170

=365.353 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c, min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=400.501 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

CALCULATION OF SHEAR REINFORCEMENT IRC 112:2020 Cl 10.3.3.1.-4

By equating V_{NS} and , $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 400.501 \times 1000}{1 \times 300 \times 2094.822 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

= 5.05º

∴As per the code 21.8º≤θ≤45º

Adopt θ=21.8º

∴V_{Rds}=V_{NS}=V_{ED}= $\frac{Asw}{s} \times z \times fywd \times cot\theta$

`

S=$\frac{Asw}{VE} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

$\therefore$S $= \frac{2*113.09}{400.501\times10^3} \times 2094.822 \times 434.78 \times cot21.8^0$

=1287.69 mm

$\therefore$ Provide spacing = 300mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s\times b_w} = \frac{226.195}{300\times3000}$=0.00251

<u>Minimum shear reinforcement ratio:</u>

$\therefore \rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk} = \frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing

At 3L/8

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}}$ d [SP 105]

$= \frac{0.0035}{0.0035+\ \ .0218} * 2170$ = 1338.326 mm

CG from top = $\beta_2 \times X_{lim}$

= 0.416 × 1338.326

= 556.744 mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

= 4355.275 kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim}$ = 2170 – 0.416 × 1338.326

= 1613.256 mm

$M_{u,\ lim1}$ = C × z = 4355.275 × $\frac{1613.256}{1000}$ = 7026.175 kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3875 \quad 300) \times 223}{1000}$

= 10539.10 kN

CG from steel level (z_2) = d- $\frac{Df}{2}$= 2170 – $\frac{223}{2}$

= 2058.500 mm

$$M_{ur,\,lim2} = C \times z = 10539.10 \times \frac{2058.500}{1000} = 21990.575 \text{ kNm}$$

Total compressive force, C = 4355.275 + 10539.10 = 15038.090 kN

Total limiting moment, $M_{ur,\,lim} = M_{ur,\,lim1} + M_{ur,\,lim2}$

$= 7026.175 + 21990.575$

$= 29016.749$ kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{29016.749 \times 1000}{15038.090}$

$= 1929.550$ mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{15038.090}{434.783} = 34587.606 \text{ mm}^2$

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4} = 804.258 \text{ mm}^2$

Number of bars $= \frac{A_{st}}{A_0} = \frac{34587.606}{804.258} = 43.006 \approx 44$

So, this section can take up to 29016.749 kNm with 44 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 15264.269 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2 \times \beta_2} - \sqrt{\left(\frac{d}{2 \times \beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1 \times \beta_2 \times b_{eff} \times f_{cd}}}$$

$$X_u = \frac{2170}{2 \times 0.416} - \sqrt{\left(\frac{2170}{2 \times 0.416}\right)^2 - \frac{15264.269 \times 10^6}{0.810 \times 0.416 \times 38750 \times 13.40}}$$

$X_u = 173.088$ mm

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd} \times z}$ with $z = (d - \beta_2 \times X_u) = (2170 - 0.416 \times 173.088) = 2098.001$ mm

$A_{st} = \frac{15264.269 \times 10^6}{434.783 \times 2098.001} = 16733.938 \text{ mm}^2$

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{16733.938}{804.248}$ = 20.81

Provide 22 number of bars of 32 mm diameter with area, = 17693.450 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d$ but not less than $0.0013\, b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26 \times \frac{2.5}{500} \times 300 \times 2170$ but not less than $0.0013 \times 300 \times 2170$

=846.3mm² but not less than 846.3mm²

= 846.3mm²

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\, A_c$

$A_{s,\,max} = 0.025 \times [(2300\text{-}250) \times 300 + 700 \times 250 + 200 \times 150] = 20500$ mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

Provide 2-32mm dia. bar as compressive reinforcement

A_{sc} provided = 1608.495 mm^2

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =838.825 KN

<u>Maximum Allowable Shear Force (for maximum shear force take $\Theta = 45^o$)</u>

$$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta} \qquad \text{[IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]}$$

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for σ_{cp}=0 (RCC)

Lever Arm(z)= $(d - \beta_2 \times X_u) = (2170 - 0.416 \times 173.088) = 2098.001$mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

$\theta=45^o$

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$

$$= 1 \times 300 \times 2098.001 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 4}$$

=2281.921 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED=}838.825$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [IRC 112-2020 *clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right) b_w d$$

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

$= 1 + \sqrt{\frac{200}{2170}}$

=1.304

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$=0.031\times1.304^{3/2}\times30^{1/2}$

=0.253

$\sigma_{cp\,=}\,0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0272 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1$=0.02

$\therefore V_{Rd.c} = [0.12 \times 1.304 \times (80 \times 0.02 \times 30)^{0.33}] \times 300 \times 2170$

=365.353 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=838.825 kN

∴Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Calcuation of Shear Reinforcement

By equating V_{NS} and, $\boldsymbol{V_{Rd,max}}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 838.825 \times 1000}{1 \times 300 \times 2094.822 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 10.78^{o}$

∴As per the code $21.8^{o} \leq \theta \leq 45^{o}$

Adopt $\theta = 21.8^{o}$

$\therefore V_{Rds} = V_{NS} = V_{ED} = \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S = \frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

$\therefore S = \frac{2 * 113.09}{838.825 \times 10^3} \times 2098.001 \times 434.78 \times cot 21.8^0$

=614.98 mm

∴ Provide spacing = 300mm

check

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s \times b_w} = \frac{226.195}{300 \times 300} = 0.00251$

Minimum shear reinforcement ratio:

$\therefore \rho_{min} = \frac{0.072 \times \sqrt{fck}}{fyk} = \frac{0.072 \times \sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing.

At L/8

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2} + \varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035}{0.0035 + .0218} * 2170 = 1338.326$ mm

CG from top = $\beta_2 \times X_{lim}$

= 0.416×1338.326

= 556.744 mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

= 4355.275 kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim}$ = 2170 – 0.416 × 1338.326

= 1613.256 mm

$M_{u,\, lim1} = C \times z = 4355.275 \times \frac{1613.256}{1000} = 7026.175$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3875 - 300) \times 223}{1000}$

= 10539.10 kN

CG from steel level (z_2) = d- $\frac{Df}{2} = 2170 - \frac{223}{2}$

= 2058.500 mm

$M_{ur,\, lim2} = C \times z = 10539.10 \times \frac{2058.500}{1000} = 21990.575$ kNm

Total compressive force, C = 4355.275 + 10539.10 = 15038.090 kN

Total limiting moment, $M_{ur,\,lim} = M_{ur,\,lim1} + M_{ur,\,lim2}$

$= 7026.175 + 21990.575$

$= 29016.749$ kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{29016.749 \times 1000}{15038.090}$

$= 1929.550$ mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{15038.090}{434.783} = 34587.606$ mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4} = 804.258$ mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{34587.606}{804.258} = 43.006 \approx 44$

So, this section can take up to 29016.749 kNm with 44 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, $M_{Ed} = 7217.043$kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{7217.043\times10^6}{0.810\times0.416\times3875\times13.40}}$$

$X_u = 80.359$ mm

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with $z = (d - \beta_2 \times X_u) = (2170 - 0.416 \times 80.359) = 2136.573$ mm

$A_{st} = \frac{7217.043\times10^6}{434.783 \times 2136.573} = 7769.076$ mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{7769.076}{804.248}$ = 9.66

Provide 10 number of bars of 32 mm diameter with area, = 8042.477 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}$ $b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times 300\times2170$ but not less than $0.0013\times300\times2170$

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max} = 0.025\times$ [(2300-250) ×300+700×250+200×150] = 20500 mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =1788.396 KN

<u>Maximum Allowable Shear Force (for maximum shear force take Θ = 45^o)</u>

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for σ_{cp}=0 (RCC)

Lever Arm(z)= $(d - \beta_2 \times X_u)$ = (2170 – 0.416 × 80.359) = 2136.573mm

$v_1 = 0.6\left(1-\frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45^o

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$

$= 1 \times 300 \times 2136.573 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$

=2323.875 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED}=1788.396$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d$

$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right) b_w d$

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

$= 1 + \sqrt{\frac{200}{2170}}$

=1.304

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$= 0.031 \times 1.304^{3/2} \times 30^{1/2}$

=0.253

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0124 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1$=0.0124

$\therefore V_{Rd.c} = [0.12\times1.304\times (80 \times 0.0124 \times 30)^{0.33}] \times300\times2170$

=311.652 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\ min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=1788.396 kN

∴Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Design of Shear Reinforcement

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2\times1788.396\times1000}{1\times300\times2136.573\times0.542\times0.446\times30}\right)}{2}$$

= 24.85°

∴As per the code 21.8°≤θ≤45°

Adopt θ=24.85°

$\therefore V_{Rds}=V_{NS}=V_{ED}= \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S=\frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

$\therefore S \ = \frac{2*113.09}{1788.396\times10^3} \times 2136.573 \times 434.78 \times cot24.85^0$

=253.66 mm

∴ Provide spacing = 250mm

check

Shear reinforcement ratio $\rho_w=\frac{A_{sw}}{s\times b_w} = \frac{226.195}{250\times300}$=0.00302

Minimum shear reinforcement ratio:

$\therefore \rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk} = \frac{0.072\times\sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 250 mm c/c spacing.

At L/4

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035}{0.0035+\ .0218} * 2170 = 1338.326$ mm

CG from top = $\beta_2 \times X_{lim}$

$= 0.416 \times 1338.326$

$= 556.744$ mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

$= 4355.275$ kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim} = 2170 - 0.416 \times 1338.326$

$= 1613.256$ mm

$M_{u,\,lim1} = C \times z = 4355.275 \times \frac{1613.256}{1000} = 7026.175$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3875\quad 300) \times 223}{1000}$

$= 10539.10$ kN

CG from steel level (z_2) = d- $\frac{Df}{2} = 2170 - \frac{223}{2}$

$= 2058.500$ mm

$M_{ur,\,lim2} = C \times z = 10539.10 \times \frac{2058.500}{1000} = 21990.575$ kNm

Total compressive force, C = 4355.275 + 10539.10 = 15038.090 kN

Total limiting moment, $M_{ur,\,lim} = M_{ur,\,lim1} + M_{ur,\,lim2}$

$= 7026.175 + 21990.575$

$= 29016.749$ kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{29016.749 \times 1000}{15038.090}$

=1929.550 mm

Area of reinforcement, A_{st} = $\frac{C}{F_{yd}} = \frac{15038.090}{434.783}$ = 34587.606 mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4}$ = 804.258 mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{34587.606}{804.258}$ = 43.006 ≈ 44

So, this section can take up to 29016.749 kNm with 44 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 12480.489kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{12480.489\times10^6}{0.810\times0.416\times3875\times13.40}}$$

X_u = 140.616 mm

Since X_u<D_f, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – β_2 × X_u) = (2170 – 0.416 × 140.616) = 2111.580 mm

$A_{st} = \frac{12480.489\times10^6}{434.783\times2136.573}$ = 13594.606 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{13594.606}{804.248}$ = 16.9

Provide 18 number of bars of 32 mm diameter with area = 14476.459 mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\ b_t d$ but not less than $0.0013\ b_t d$

Here, For M30

$f_{ctm} = 2.5$ [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times \frac{2.5}{500} \times 300\times2170$ but not less than $0.0013\times300\times2170$

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max} = 0.025\times$ [(2300-250) ×300+700×250+200×150] = 20500 mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =1334.902 KN

Maximum Allowable Shear Force (for maximum shear force take ϴ = 45^o)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z)= $(d - \beta_2 \times X_u) = (2170 - 0.416 \times 140.616) = 2111.580$mm

$v_1 = 0.6\left(1-\frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45^o

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$

$= 1 \times 300 \times 2111.580 \times 0.6\left(1-\frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45+\tan 4}$

=2298.236 kN

And,

V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= $V_{ED=}$1334.902 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, V_{Rds} < $V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$

$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$

K =1+$\sqrt{\frac{200}{d}}$ ≤2

=1+$\sqrt{\frac{200}{2170}}$

=1.304

V_{min} = $0.031K^{3/2}fck^{1/2}$

=$0.031\times1.304^{3/2}\times30^{1/2}$

=0.253

$\sigma_{cp\,=}$ 0

$\rho1 = \frac{A_{st}}{b_w.d} = 0.0222 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ ρ1=0.02

∴$V_{Rd.c}$ = [0.12×1.304× $(80 \times 0.02 \times 30)^{0.33}$] ×300×2170

=365.353 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×b_wd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c, min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=1334.902 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

CALCUALTION OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 1334.902 \times 1000}{1 \times 300 \times 2111.580 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 17.77^{o}$

∴As per the code $21.8^{o} \leq \theta \leq 45^{o}$

Adopt $\theta = 21.8^{o}$

∴V_{Rds}=V_{NS}=V_{ED}= $\frac{Asw}{s} \times z \times fywd \times cot\theta$

`

S=$\frac{Asw}{\mathrm{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

∴S $= \frac{2*113.09}{1788.396\times10^3} \times 2111.580 \times 434.78 \times cot21.8^0$

=388.93 mm

∴ Provide spacing = 300mm

check

Shear reinforcement ratio ρ_w=$\frac{A_{sw}}{s\times b_w} = \frac{226.195}{300\times300}$=0.00251

Minimum shear reinforcement ratio:

∴ρ_{min} = $\frac{0.072\times\sqrt{\mathrm{fck}}}{fyk}$=$\frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, ρ_w> ρ_{min}, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300 mm c/c spacing

At support

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} = 2224.726KN

Maximum Allowable Shear Force (for maximum shear force take Θ = 45°)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\ max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}$0 (RCC)

Lever Arm(z)= 2111.580mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45°

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$

$$= 1 \times 300 \times 2111.580 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$$

=2296.69 kN

And,

V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= $V_{ED=}$2224.726 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

$\therefore$Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$

$V_{Rd.c\,min} = \left(v_{mi} \;\; + 0.15\sigma_{cp}\right)b_w d$

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

$=1+\sqrt{\frac{200}{2170}}$

$=1.304$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$=0.031\times1.304^{3/2}\times30^{1/2}$

$=0.253$

$\sigma_{cp} = 0$

Since the section at L=0 has not been designed for bending, but half reinforcement is always available throughout.

A_{st} = 17472.737/2

$\rho1 = \frac{A_{st}}{b_w.d} = 0.0136 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho1$=0.0136

$\therefore V_{Rd.c} = [0.12\times1.304\times (80 \times 0.0136 \times 30)^{0.33}] \times300\times2170$

=321.610 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) $\times b_w d$

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=321.610 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=2224.726 kN

$\therefore$Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

DESIGN OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore\ \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 2224.726 \times 1000}{1 \times 300 \times 2111.580 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 36.88^o$

∴As per the code $21.8^o \le \theta \le 45^o$

Adopt $\theta = 36.88^o$

$\therefore V_{Rds} = V_{NS} = V_{ED} = \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S = \frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2-legged 12 mm stirrups

$\therefore Fywd = 500/1.15 = 434.78\ N/mm^2$

$\therefore S \ = \frac{2*113.09}{2224.726 \times 10^3} \times 2111.580 \times 434.78 \times cot 21.8^0$

$= 124.43$ mm

∴ Provide spacing = 110mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s \times b_w} = \frac{226.195}{110 \times 300} = 0.00685$

<u>Minimum shear reinforcement ratio:</u>

$\therefore \rho_{min} = \frac{0.072 \times \sqrt{fck}}{fyk} = \frac{0.072 \times \sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 110 mm c/c spacing

5.3.3. Design of Intermediate Girder

Effective width of flange:

As per clause7.6.1.2 of IRC 112, the effective flange width will be calculated.

The effective flange width b_{eff} for a T beam

$b_{eff,1} = 0.2 \times b_1 + 0.1 \times Lo \le 0.2 \times Lo$ and $\le b_1$

$= 0.2 \times 1.475 + 0.1 \times 30 \le 0.2 \times 30$ and ≤ 1.475

$= 3.295 \le 6$ and ≤ 2.1

$=1.475m$

$b_{eff,2} = 0.2 \times b_2 + 0.1 \times Lo \leq 0.2 \times Lo$ and $\leq b_2$

$=0.2 \times 1.475 + 0.1 \times 30 <= 0.2 \times 30$ and ≤ 1.475

$=3.295 \leq 6$ and ≤ 1.475

$=1.475$ m

$b_{eff} = b_{eff,1} + b_{eff,2} + b_w \leq b$

$=1.475 + 1.475 + 0.3 \leq b_1 + b_2 + b_w$

$=3.250 \leq 1.475 + 1.475 + 0.3$

$=3.250 \leq 3.250$

$=3.250m$

Hence, effective width of flange, b_{eff} = 3.250m

Clear cover = 40mm

Let us assume 3 layers of bar of dia. 32mm

Effective depth, d = $2300-40-10-32-32-\frac{32}{2}$ = 2170 mm=2.170 m

Section properties:

Width of web(b_w)=300 mm

Average thickNess of left part of slab = 0.22 m

Depth of flange (D_f)= $220\ mm$

Overall depth of beam (D) =2300 mm

Material properties

a) Concrete used: M30 (IRC 112-2020 Table 6.4)

b) Characteristic strength, f_{ck}= 30 N/mm^2

c) Design compressive strength of concrete, $f_{cd}=\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]

d) $\alpha = 0.67$

e) $\gamma m = 1.5$

f) Design compressive strength of concrete, $f_{cd}= \frac{0.67 \times fck}{1.5} = \frac{0.67*30}{1.5}$ = 13.40 N/mm^2

g) Steel used: Fe500

h) Yield Strength of Steel, f_{yk}=500N/mm^2

i) Design yield strength of steel, $f_{yd}= f_{yk}/1.15= 0.87fy = 434.783$ N/mm^2

j) Young's Modulus of Elasticity, $Es = 2\times10^5$N/mm^2

k) Yield strain for steel $(\varepsilon_{yd}) = \frac{fyd}{Es} = \frac{0.87*fy}{Es} = \frac{0.87*500}{200000} = 0.0218$

l) Area factor $(\beta_1) = 0.810$

m) CG factor$(\beta_2) = 0.416$

n) Limiting strain on extreme compressed fiber of concrete$(\varepsilon_{cu2}) = 0.0035$

At L/2

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035*2170}{0.0035+\ .0218} = 1338.326$ mm

CG from top $= \beta_2 \times X_{lim}$

$= 0.416 \times 1338.326$

$= 556.744$ mm

For Web

Compressive force $(C_1) = \beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

$= 4355.275$ kN

CG from steel level $(z_1) = d- \beta_2 \times X_{lim} = 2170 - 0.416 \times 1338.326$

$= 1613.256$ mm

$M_{u,\ lim1} = C \times z = 4355.275 \times \frac{1613.256}{1000} = 7026.175$ kNm

For Flange

Compressive force $(C_2) = F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3250 - 300) \times 220}{1000}$

$= 8696.6$ kN

CG from steel level $(z_2) = d- \frac{Df}{2} = 2170 - \frac{220}{2}$

$= 2060$ mm

$M_{ur,\ lim2} = C \times z = 8696.6 \times \frac{2060}{1000} = 17914.996$ kNm

Total compressive force, $C = 4355.275 + 8696.6 = 13051.875$ kN

Total limiting moment, $M_{ur,\ lim} = M_{ur,\ lim1} + M_{ur,\ lim2}$

$= 7026.175 + 17914.996$

$= 24941.171 \text{kNm}$

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{24941.171 \times 1000}{13051.875}$

$= 1910.926$ mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{13051.875}{434.783} = 30019.312 \text{ mm}^2$

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4} = 804.258 \text{ mm}^2$

Number of bars $= \frac{A_{st}}{A_0} = \frac{30019.312}{804.258} = 37.326 \approx 38$

So, this section can take up to 24941.171 kNm with 38 number 32 mm dia bars.

Design of main reinforcement

Design Bending Moment, M_{Ed} = 13910.526 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2 \times \beta_2} - \sqrt{\left(\frac{d}{2 \times \beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1 \times \beta_2 \times b_{eff} \times f_{cd}}}$$

$$X_u = \frac{2170}{2 \times 0.416} - \sqrt{\left(\frac{2170}{2 \times 0.416}\right)^2 - \frac{13910.526 \times 10^6}{0.810 \times 0.416 \times 3250 \times 13.40}}$$

$$X_u = 188.652 \text{ mm}$$

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd} \times z}$ with $z = (d - \beta_2 \times X_u) = (2170 - 0.416 \times 188.652) = 2091.526$ mm

$A_{st} = \frac{13910.526 \times 10^6}{434.783 \times 2091.526} = 15297.063 \text{ mm}^2$

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{15297.063}{804.248}$ = 19.02

Provide 20 number of bars of 32 mm diameter with area, = 16084.954 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\,b_t d$ but not less than $0.0013\,b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times 300\times2170$ but not less than $0.0013\times300\times2170$

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\,A_c$

$A_{s,\,max} = 0.025\times[(2300-250)\times300+700\times250+200\times150] = 20500\ mm^2$

$A_{st,\,provided} < A_{s,\,max}$, OK

Provide 2-32mm dia. bar as compressive reinforcement

A_{sc} provided = 1608.495 mm^2

Side reinforcement:

When the depth of beam is more than 750mm, skin(surface) reinforcement of 0.1 % of web area on each side is to be provided.

Minimum side reinforcement = 0.1% x 300 x 2170 = 651mm^2

Providing 12mm bars at the middle section of the beam

No of bars $= \frac{651}{\frac{\pi}{4}\times 12^2} = 5.76 \approx 6$

Spacing of Bars =2170/6= 361.67 mm

So, provide 12 mm dia. rebar @ 250 mm c/c as side reinforcement on each side.

Design Of Shear Reinforcement

Design shear force, V_{Ed} =302.704 KN

<u>Maximum Allowable Shear Force (for maximum shear force take Θ = 45°)</u>

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,max}$ =The design value of maximum shear force

a_{cw}=1 for σ_{cp}=0 (RCC)

Lever Arm(z)=(d – $\beta_2 \times X_u$) = (2170 – 0.416 × 188.652)

= 2091.526 mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45°

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$

$$= 1 \times 300 \times 2091.526 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$$

=2274.878kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED}=$302.704 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe.**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

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

$= 1+\sqrt{\frac{200}{2170}}$

$= 1.304$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$= 0.031\times1.304^{3/2}\times30^{1/2}$

$= 0.253$

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0247 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1 = 0.02$

$\therefore V_{Rd.c} = [0.12\times1.304\times(80 \times 0.02 \times 30)^{0.33}]\times300\times2170$

$= 365.353$ kN

And, $V_{Rd.c} = (Vmin+0.15\sigma cp)\times bwd$

$= (0.253+0.15\times 0) \times 300\times 2170$

$= 164.518$ kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min} = 365.353$ kN

V_{Ed} = The design shear force at a cross-section resulting from external loading

$= 400.501$ kN

$\therefore$Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Design of Shear Reinforcement

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 302.704 \times 1000}{1 \times 300 \times 2091.526 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 2.52^{o}$

∴As per the code $21.8^{o} \leq \theta \leq 45^{o}$

Adopt $\theta = 21.8^{o}$

∴$V_{Rds} = V_{NS} = V_{ED} = \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S = \frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴$Fywd = 500/1.15 = 434.78\ N/mm^2$

∴$S = \frac{2*113.09}{302.704 \times 10^3} \times 2091.526 \times 434.78 \times cot21.8^0$

$= 1698.91$ mm

∴ Provide spacing = 300mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s \times b_w} = \frac{226.195}{300 \times 300} = 0.00251$

<u>Minimum shear reinforcement ratio:</u>

∴$\rho_{min} = \frac{0.072 \times \sqrt{fck}}{fyk} = \frac{0.072 \times \sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing

At 3L/8

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu2} + \varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035*2170}{0.0035 + .0218} = 1338.326$ mm

CG from top = $\beta_2 \times X_{lim}$

$= 0.416 \times 1338.326$

$= 556.744$ mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1338.326}{1000}$

$= 4355.275$ kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim} = 2170 - 0.416 \times 1338.326$

$= 1613.256$ mm

$M_{u,\ lim1} = C \times z = 4355.275 \times \frac{1613.256}{1000} = 7026.175$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3250 - 300) \times 220}{1000}$

$= 8696.6$ kN

CG from steel level (z_2) = $d - \frac{Df}{2} = 2170 - \frac{220}{2}$

$= 2060$ mm

$M_{ur,\ lim2} = C \times z = 8696.6 \times \frac{2060}{1000} = 17914.996$ kNm

Total compressive force, C = 4355.275 + 8696.6 = 13051.875 kN

Total limiting moment, $M_{ur,\ lim} = M_{ur,\ lim1} + M_{ur,\ lim2}$

= 7026.175 + 17914.996

= 24941.171kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{24941.171 \times 1000}{13051.875}$

=1910.926 mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{13051.875}{434.783} = 30019.312$ mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4} = 804.258$ mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{30019.312}{804.258} = 37.326 \approx 38$

So, this section can take up to 24941.171 kNm with 38 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 13303.517 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{13303.517\times10^6}{0.810\times0.416\times3250\times13.40}}$$

X_u = 180.114 mm

Since X_u<D_f, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – β_2 × X_u) = (2170 – 0.416 × 180.114) = 2095..078mm

$A_{st} = \frac{13303.517\times10^6}{434.783\times2095.078}$ = 14604.751mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{14604.751}{804.248}$ = 18.16

Provide 20 number of bars of 32 mm diameter with area, = 16084.954 mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\,b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times300\times2170$ but not less than 0.0013×300×2170

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max}$ = 0.025× [(2300-250) ×300+700×250+200×150] = 20500 mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

Provide 2-32mm dia. bar as compressive reinforcement

A_{sc} provided = 1608.495 mm^2

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =690.703KN

Maximum Allowable Shear Force (for maximum shear force take Θ = 45°)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}$0 (RCC)

Lever Arm(z)= $(d - \beta_2 \times X_u) = (2170 - 0.416 \times 180.114) = 2095.078$mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45°

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$

$$= 1 \times 300 \times 2095.078 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$$

=2278.741 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED=}690.703$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = [0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}]b_w d$

$V_{Rd.c\,min} = (v_{min} + 0.15\sigma_{cp})b_w d$

K =1+$\sqrt{\frac{200}{d}}$ ≤2

=1+$\sqrt{\frac{200}{2170}}$

=1.304

V_{min} = 0.031$K^{3/2}fck^{1/2}$

=0.031×$1.304^{3/2}$×$30^{1/2}$

=0.253

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0247 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ $\rho 1$=0.02

∴$V_{Rd.c}$ = [0.12×1.304× $(80 \times 0.02 \times 30)^{0.33}$] ×300×2170

=365.353 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=838.825 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

DESIGN OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 838.825 \times 1000}{1 \times 300 \times 2095.078 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

= 8.741°

∴As per the code 21.8°≤θ≤45°

Adopt θ=21.8º

$\therefore V_{Rds}=V_{NS}=V_{ED}=\frac{Asw}{s}\times z\times fywd\times cot\theta$

`

$S=\frac{Asw}{VED}\times z\times fywd\times cot\theta$

Provide 2 legged 12 mm stirrups

$\therefore$Fywd=500/1.15=434.78 N/mm^2

$\therefore S = \frac{2*113.09}{690.703\times10^3}\times 2095.078\times 434.78\times cot21.8^0$

=745.821 mm

$\therefore$ Provide spacing = 300mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w=\frac{A_{sw}}{s\times b_w}=\frac{226.195}{300\times300}$=0.00251

<u>Minimum shear reinforcement ratio:</u>

$\therefore\rho_{min}=\frac{0.072\times\sqrt{fck}}{fyk}=\frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, $\rho_w>\rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing

At L/8

Calculation of Limiting Moment

$X_{lim}=\frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}}$ d [SP 105]

$=\frac{0.0035*2170}{0.0035+0.0218}$ = 1338.326 mm

CG from top = $\beta_2\times X_{lim}$

= 0.416 × 1338.326

= 556.744 mm

For Web

Compressive force (C_1) = $\beta_1\times F_{cd}\times b_w\times X_{lim}$

$=\frac{0.810\times13.40\times300\times1338.326}{1000}$

= 4355.275 kN

CG from steel level (z_1) = d- $\beta_2\times X_{lim}$ = 2170 – 0.416 × 1338.326

= 1613.256 mm

$M_{u,\,lim1}$ = C × z = 4355.275 × $\frac{1613.256}{1000}$ = 7026.175 kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$= \frac{13.40 \times (3250 \quad 300) \times 220}{1000}$

= 8696.6 kN

CG from steel level (z_2) = d- $\frac{Df}{2}$= 2170 – $\frac{220}{2}$

= 2060 mm

$M_{ur,\ lim2}$ = C × z = 8696.6 × $\frac{2060}{1000}$ = 17914.996 kNm

Total compressive force, C = 4355.275 + 8696.6 = 13051.875 kN

Total limiting moment, $M_{ur,\ lim}$ = $M_{ur,\ lim1}$ + $M_{ur,\ lim2}$

= 7026.175 + 17914.996

= 24941.171kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{24941.171 \times 1000}{13051.875}$

=1910.926 mm

Area of reinforcement, A_{st} = $\frac{C}{F_{yd}} = \frac{13051.875}{434.783}$ = 30019.312 mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4}$ = 804.258 mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{30019.312}{804.258}$ = 37.326 ≈ 38

So, this section can take up to 24941.171 kNm with 38 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 6274.593kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$

This, by solving becomes,

$$X_u = \frac{d}{2 \times \beta_2} - \sqrt{\left(\frac{d}{2 \times \beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1 \times \beta_2 \times b_{eff} \times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{6274.593\times10^6}{0.810\times0.416\times3250\times13.40}}$$

X_u = 83.349mm

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – β_2 × X_u) = (2170 – 0.416 × 83.349) = 2135.329 mm

$A_{st} = \frac{6274.593\times10^6}{434.783\times2135.329}$ =6758.472 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = 6758.472$ = 8.40

Provide 10 number of bars of 32 mm diameter with area, = 8042.477 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\,b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times300\times2170$ but not less than 0.0013×300×2170

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\,A_c$

$A_{s,\,max}$ = 0.025× [(2300-250) ×300+700×250+200×150] = 20500 mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =1539.943 kN

<u>Maximum Allowable Shear Force (for maximum shear force take Θ = 45º)</u>

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD, max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z)= $(d - \beta_2 \times X_u) = (2170 - 0.416 \times 83.349) = 2135.329$mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45º

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$

$$= 1 \times 300 \times 2135.329 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$$

=2322.521 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED=}1539.943$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$

$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$

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

$=1+\sqrt{\frac{200}{2170}}$

$=1.304$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$=0.031\times1.304^{3/2}\times30^{1/2}$

$=0.253$

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0124 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1=0.0124$

$\therefore V_{Rd.c} = [0.12\times1.304\times (80 \times 0.0124 \times 30)^{0.33}] \times300\times2170$

=311.652 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=1539.943 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

DESIGN OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 1539.943 \times 1000}{1 \times 300 \times 2135.329 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 20.542^{o}$

∴As per the code $21.8^{o} \leq \theta \leq 45^{o}$

Adopt θ=24.85⁰

$\therefore V_{Rds}=V_{NS}=V_{ED}=\frac{Asw}{s}\times z\times fywd\times cot\theta$

`

$S=\frac{Asw}{VE}\times z\times fywd\times cot\theta$

Provide 2 legged 12 mm stirrups

$\therefore$Fywd=500/1.15=434.78 N/mm^2

$\therefore S\ =\frac{2*113.09}{1539.943\times10^3}\times 2135.329\times 434.78\times cot24.85^0$

=340.946 mm

$\therefore$ Provide spacing = 300 mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w=\frac{A_{sw}}{s\times b_w}=\frac{226.195}{300\times300}$=0.00251

<u>Minimum shear reinforcement ratio:</u>

$\therefore\rho_{min}=\frac{0.072\times\sqrt{fck}}{fyk}=\frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, $\rho_w>\rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300 mm c/c spacing.

At L/4

Calculation of Limiting Moment

$X_{lim}=\frac{\varepsilon_{cu2}}{\varepsilon_{cu}\ +\varepsilon_{yd}}$d [SP 105]

$=\frac{0.0035*2170}{0.0035+\ .0218}$ = 1338.326 mm

CG from top = $\beta_2\times X_{lim}$

= 0.416 × 1338.326

= 556.744 mm

For Web

Compressive force (C_1) = $\beta_1\times F_{cd}\times b_w\times X_{lim}$

$=\frac{0.810\times13.40\times300\times1338.326}{1000}$

= 4355.275 kN

CG from steel level (z_1) = d- $\beta_2\times X_{lim}$ = 2170 – 0.416 × 1338.326

= 1613.256 mm

$M_{u,\ lim1}$ = C × z = 4355.275 × $\frac{1613.256}{1000}$ = 7026.175 kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times D_f$

$$= \frac{13.40 \times (3250 - 300) \times 220}{1000}$$

= 8696.6 kN

CG from steel level (z_2) = d- $\frac{Df}{2}$= 2170 – $\frac{220}{2}$

= 2060 mm

$M_{ur,\ lim2}$ = C × z = 8696.6 × $\frac{2060}{1000}$ = 17914.996 kNm

Total compressive force, C = 4355.275 + 8696.6 = 13051.875 kN

Total limiting moment, $M_{ur,\ lim} = M_{ur,\ lim1} + M_{ur,\ lim2}$

= 7026.175 + 17914.996

= 24941.171kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{24941.171 \times 1000}{13051.875}$

=1910.926 mm

Area of reinforcement, A_{st} = $\frac{C}{F_{yd}} = \frac{13051.875}{434.783}$ = 30019.312 mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4}$ = 804.258 mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{30019.312}{804.258}$ = 37.326 ≈ 38

So, this section can take up to 24941.171 kNm with 38 number 32 mm dia bars.

DESIGN OF MAIN REINFORCEMENT

Design Bending Moment, M_{Ed} = 10844.785kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{2170}{2\times0.416} - \sqrt{\left(\frac{2170}{2\times0.416}\right)^2 - \frac{10844.785\times10^6}{0.810\times0.416\times3250\times13.40}}$$

X_u = 145.833 mm

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – β_2 × X_u) = (2170 – 0.416 × 145.833) = 2109.338 mm

$A_{st} = \frac{10884.785\times10^6}{434.783\times2109.338}$ = 11825.04 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{11825.04}{804.248}$ = 14.70

Provide 16 number of bars of 32 mm diameter with area = 12867.964 mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,min} = 0.26\frac{f_{ctm}}{f_{yk}} b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,min} = 0.26\times\frac{2.5}{500}\times300\times2170$ but not less than 0.0013×300×2170

=846.3mm^2 but not less than 846.3mm^2

= 846.3mm^2

$A_{st,provided} > A_{s,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,max}$ = 0.025 A_c

$A_{s,max}$ = 0.025× [(2300-250) ×300+700×250+200×150] = 20500 mm^2

$A_{st,provided} < A_{s,max}$, OK

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =1136.427 KN

Maximum Allowable Shear Force (for maximum shear force take Θ = 45º)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for σ_{cp}=0 (RCC)

Lever Arm(z)= (d – β_2 × X_u) = (2170 – 0.416 × 143.833) = 2109.338mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45°

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan}$$

$$= 1 \times 300 \times 2109.338 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.44 \quad ck}{\cot 45 + \tan 4}$$

=2294.251 kN

And,

V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED}=1136.427kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right) b_w d$$

K =1+$\sqrt{\frac{200}{d}}$≤2

=1+$\sqrt{\frac{200}{2170}}$

=1.304

V_{min} = 0.031$K^{3/2}fck^{1/2}$

=0.031×$1.304^{3/2}$×$30^{1/2}$

=0.253

$\sigma_{cp} = 0$

$\rho1 = \frac{A_{st}}{b_w.d} = 0.0198 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ $\rho1$=0.02

∴$V_{Rd.c}$ = [0.12×1.304× $(80 \times 0.02 \times 30)^{0.33}$] ×300×2170

=365.353 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×b_wd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c, min}$=365.353 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=1334.902 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

CALCULATION OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W ZV_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 1136.427 \times 1000}{1 \times 300 \times 2109.833 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

= 14.701°

∴As per the code 21.8°≤θ≤45°

Adopt θ=21.8°

∴V_{Rds}=V_{NS}=V_{ED}= $\frac{Asw}{s} \times z \times fywd \times cot\theta$

`

S=$\frac{Asw}{\mathrm{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

$\therefore S \ = \frac{2*113.09}{1136.427\times10^3} \times 2109.833 \times 434.78 \times cot 21.8^0$

=456.384 mm

∴ Provide spacing = 300mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s \times b_w} = \frac{226.195}{300\times300}$ =0.00251

<u>Minimum shear reinforcement ratio:</u>

$\therefore \rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk} = \frac{0.072\times\sqrt{30}}{500}$ =0.00079 Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300 mm c/c spacing

At support

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} = 1930.485KN

<u>Maximum Allowable Shear Force (for maximum shear force take Θ = 45º)</u>

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z)= 2109.388mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45º

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$

$= 1 \times 300 \times 2109.388 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 4}$

=2294.306 kN

And,

$V_{Rds} = V_{NS} = V_{ED} + V_{ccd} + V_{td} = V_{ED=}$2224.726 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

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

$$= 1+\sqrt{\frac{200}{2170}}$$

$$= 1.304$$

V_{min} = 0.031$K^{3/2}$$fck^{1/2}$

$=0.031\times1.304^{3/2}\times30^{1/2}$

=0.253

$\sigma_{cp} = 0$

Since the section at L=0 has not been designed for bending, but half reinforcement is always available throughout.

A_{st} = 16084.954 /2

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0124 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ $\rho 1$=0.0124

∴$V_{Rd.c}$ = $[0.12\times1.304\times (80 \times 0.0124 \times 30)^{0.33}]\times300\times2170$

=311.652 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×b_wd

= (0.253+0.15× 0) × 300× 2170

=164.518 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\ min}$=311.652 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=1930.485 kN

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

CALCULATION OF SHEAR REINFORCEMENT

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore\ \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 1930.485 \times 1000}{1 \times 300 \times 2109.388 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 28.25^{0}$

∴As per the code $21.8^{0} \leq \theta \leq 45^{0}$

Adopt θ=28.25^{0}

∴V_{Rds}=V_{NS}=V_{ED}= $\frac{Asw}{s} \times z \times fywd \times cot\theta$

S=$\frac{Asw}{\text{VE}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

∴S $= \frac{2*113.09}{1930.485\times10^3} \times 2109.388 \times 434.78 \times cot21.8^0$

=199.97 mm

∴ Provide spacing = 150 mm

<u>check</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s\times b_w} = \frac{226.195}{150\times300}$=0.00503

<u>Minimum shear reinforcement ratio:</u>

∴$\rho_{min} = \frac{0.072\times\sqrt{\text{fck}}}{fyk} = \frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, ρ_w> ρ_{min}, (ok)

Hence provide 12mm 2- legged vertical stirrups at 150 mm c/c spacing.

5.4. Design of cross girder:

5.4.1.Intermediate Cross Girder:

(I) Calculations of Dead Load

Dead load on Cross Beam Calculation

- Intensity of slab = 1.35 × 0.22 × 25 = 7.425 kN/m^2
- Intensity of wearing coarse = 1.75 × 0.075 × 22 = 2.888kN/m^2

Total load intensity = Intensity of slab + Intensity of wearing coat

= 10.313 kN/m^2

Here the load due to self-weight of slab and wearing coat will be distributed between the cross girder and the longitudinal girder in accordance with the trapezoidal distribution of the load on panel, as shown in figure below:

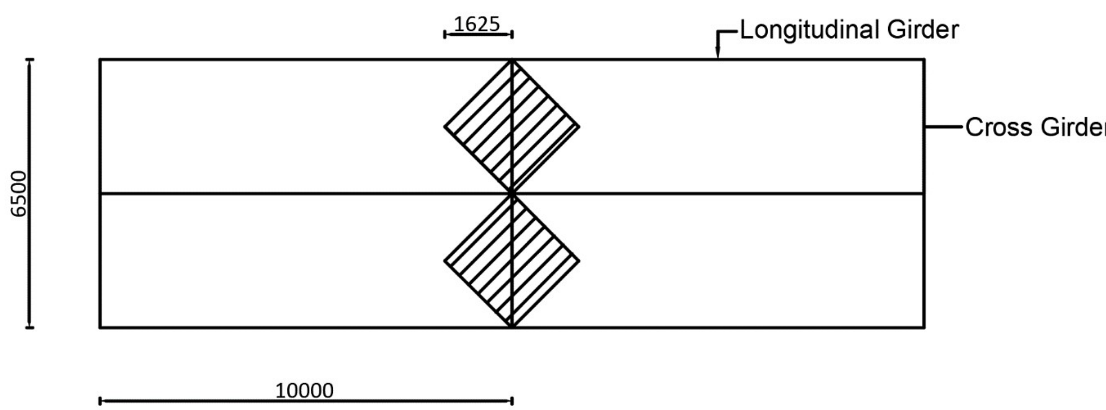


Fig: Contributory Area of Dead load on Intermediate Cross Girder

Load distribution from slab

Load on each cross girder =2 × $\frac{1}{2}$ × 3.25 × 1.625 × 10.313 = 54.463 kN

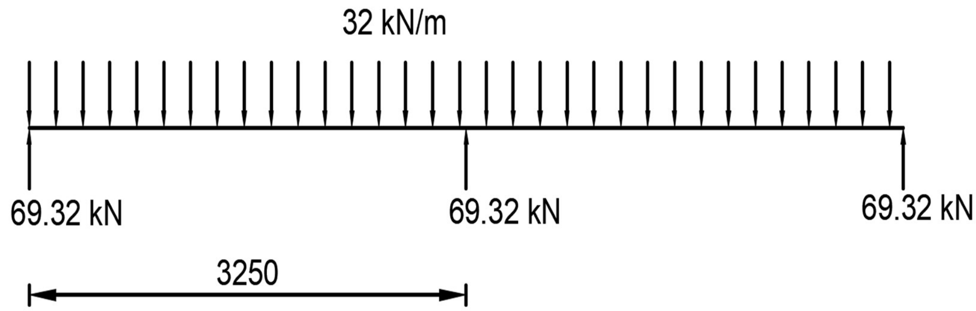
Converting to UDL we get, Dead load from slab = 54.464/3.25

= 16.758 kN/m

Self-weight of cross girder = 1.35 × 0.3 × 25 × (1.725-0.22) = 15.23 kN/m

Total DL = 15.23 + 16.758 = 31.996 kN/m

RA = RB = RC = $\frac{31.996*2*3.25}{3}$=69.324 kN/m



(II) Calculation of Live Load

Assuming intermediate cross girder as flexible beam it can be idealized as simply supported beam.

CLASS 70R TRACK LOADING

To calculate the live load on the cross girder to have maximum bending moment due to class 70R track vehicle, the load is placed symmetrically about the center line of bridge as show in figure below

let us take the span which occupy half the track of 70R load as shown in figure and a part of cross girder as the support

Track load, W=350 kN Length of track= 4.57m

Equivalent UDL = 350/4.57 = 76.58 kN/m

Reaction calculation due to live load

Taking moment about 1

$\sum M_1 = 0$

$R_2 * 10 – 76.59 *2.285/2 = 0$

$R_2 = 19.95$ kN

$\sum F_y = 0$

$R_2 + R_1 - 76.59 * 2.285 = 0$

$R_1 = 155.01$ kN

Since the reaction calculated above is from half of the track load only, hence multiplying by 2 to get the reaction for full length of the track

i.e., 155.015×2=310.03 kN.

It is actually the load acting on girder.

Calculating the reaction and finding the bending moment under the load to get the maximum value of BM. Assuming that the cross girder has rigid reaction on each longitudinal girder

Maximum LL bending moment under the load = 206.687×2.22 = 458.845 kNm

Maximum LL bending moment considering impact factor = 458.845×1.1

= 504.730kNm

Maximum shear force due to LL=reaction at support=206.687 kN

Maximum shear force due to LL considering impact factor = 206.687*1.1

= 227.356 kN

Maximum BM due to DL = 69.324*2.22-31.996*2.22*0.5*2.22

=75.055 KNm

Total design BM = BM due to DL+BM due to LL = 75.055 + 504.730*1.5

=832.150 KNm

Total design SF = SF due to DL+SF due to LL

= 69.324 +227.356*1.5

= 410.358 kN

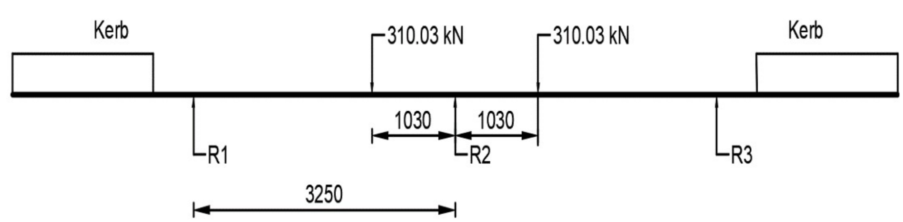
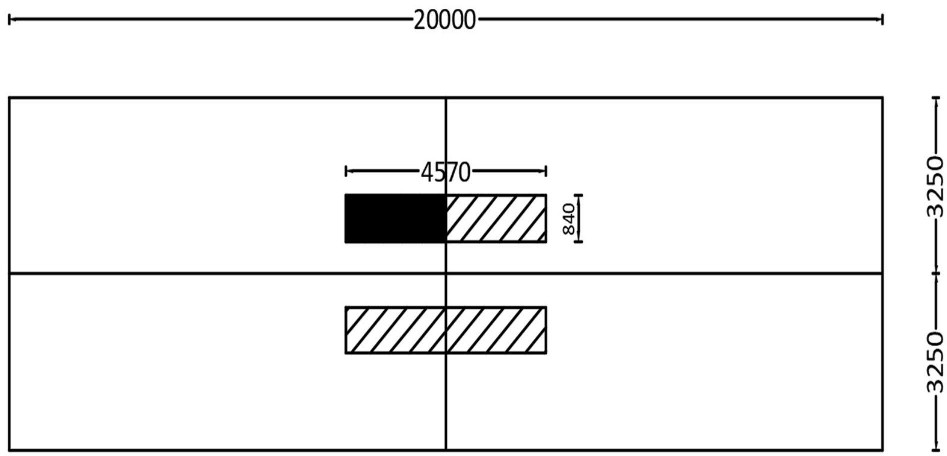


Fig: positioning of class 70R tracked load



CLASS 70R WHEELED LOADING

To calculate the live load on the cross girder to have maximum bending moment due to class 70R wheeled vehicle, the load is placed symmetrically about the center line of bridge as show in figure below:

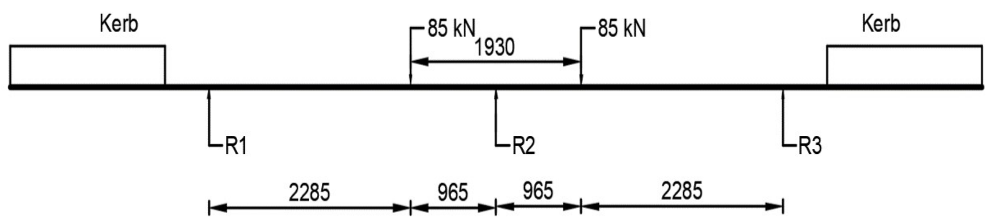


fig: Positioning of 70R wheel load

In the case of 70R wheeled load the tyre dimensions are so small in comparison with track of 70R track vehicle, so we assume that the total wheel load act as a point load on the single girder.

Reactions

Assuming that the cross girder has rigid reaction on each longitudinal girder:

$R_1 = R_2 = R_3 = \frac{2*85}{3} = 56.67$ kN

Maximum live load BM under the load = 56.67*2.285

=129.491 KNm

Maximum live load BM due to LL considering IF = 129.491*1.12

= 145.030 KNm

Shear Force = 56.67 kN

Maximum SF due to LL considering IF = 56.67*1.12 = 63.470 kN

Maximum BM due to DL = 69.324*2.285-31.996*2.285*0.5*2.285

= 74.876 KNm

Maximum SF due to DL= 69.324 kN

Total design BM = BM due to DL+BM due to LL

= 74.876+145.030*1.5

= 292.421 kN/m

Total design SF = SF due to DL+SF due to LL

= 69.324+63.470*1.5

= 164.529 kN

CLASS A LOADING

To calculate the live load on the cross girder to have maximum bending moment due to class

A vehicle, the load is placed symmetrically about the center line of bridge as shown in figure below:

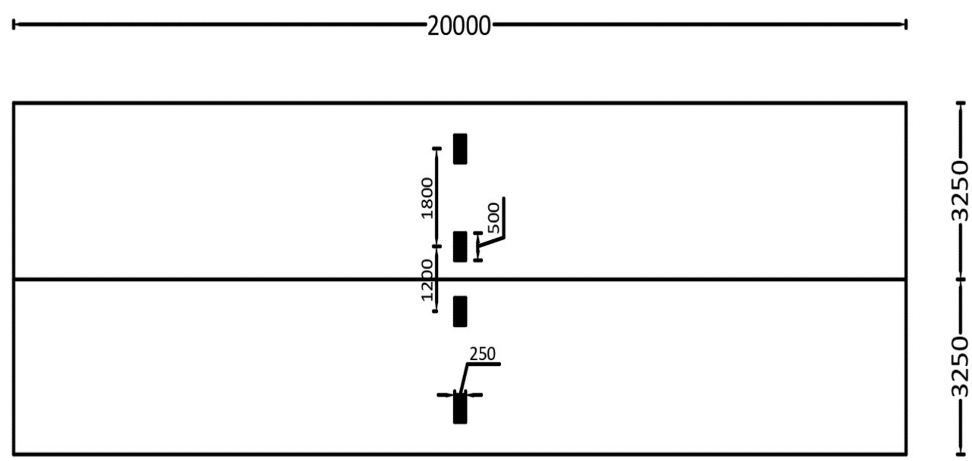


fig: Positioning of class A load

Calculation of load acting on the cross girder:

Equivalent UDL corresponding to 57 kN load =57/0.25 = 228 kN/m

Taking moment about the 1, we get,

$\sum M_1 = 0$

$R_2 * 10 - 228 * 0.125^2 = 0$

$R_2 = 0.178$ kN

$\sum F_y = 0$

$R_2 + R_1 - 228 * 0.125 = 0$

$R_1 = 28.322$ kN

Since the reaction calculated above is from half of the track load only, hence multiplying by 2 to get on the girder the reaction for full length of the track

i.e., 28.322*2=56.644 kN.

It is actually the load acting on cross girder.

Calculating the reaction and finding the bending moment under the load to get the maximum value of BM Assuming that the cross girder has rigid reaction on each longitudinal girder:

$R_1 = R_2 = R_3 = \frac{4*56.644}{3} = 75.525$ kN

Maximum LL bending moment under the load =75.525*0.6=45.315 KNm

Maximum LL bending moment considering impact factor and lane distribution factor = 45.315*1.125*2

=101.959 KNm

Maximum LL bending moment including FOS = 101.959*1.5

= 152.938kN/m

Maximum shear force due to LL = reaction at support = 75.525 kN

Maximum shear force due to LL considering impact factor and lane distribution factor = 75.525*1.125*2

=169.93 kN

Maximum Shear force including factor of safety = 169.93*1.5 = 254.897kN

Maximum BM due to DL = 69.324*0.6-31.996*0.6*0.5*0.6

= 35.835 KNm

Maximum SF due to DL= 69.324 kN

Total design BM= BM due to DL + BM due to LL

= 35.835 KNm +152.938 KNm

= 188.773 KNm

Total design SF = SF due to DL+SF due to LL

= 69.324 kN+283.218 kN

= 324.221 kN

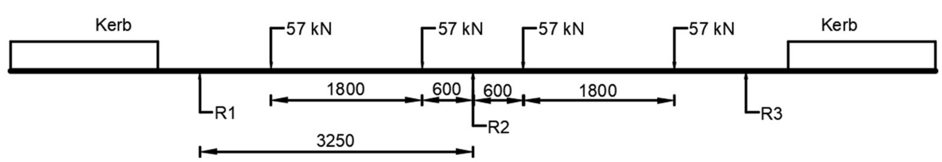


fig:Load acting on the cross girder

Table: Summary of design BM and SF:

IRC loading	BM (kNm)	SF (kN)
70R Track	832.150	410.358
70R wheel	292.421	164.529
Class A	188.773	324.221

As the bending moment and shear force for 70R tracked loading is more. The design bending moment = 832.150 KNm

The design shear force = 410.358kN

5.4.2.Design of intermediate cross girder

Bending moment = 832.150 KNm

Shear force = 410.358 kN

Section properties:

a) width of web(b_w)=300 mm

b) depth of flange (D_f)= 220 mm

c) Overall depth of beam (D) =1725 mm

Effective width of flange:

[IRC:112-2020 Clause: 7.6.1.2]

$\mathbf{b_{eff,1}}$ = $0.2\times b_1 + 0.1\times Lo \leq 0.2\times Lo$ and $\leq b_1$

= $0.2\times 4.85 + 0.1\times 3.25 \leq 0.2\times 3.25$ and ≤ 4.85

= $1.295 \leq 0.65$and ≤ 4.85

= 0.65m

$\mathbf{b_{eff,2}}$ = $0.2\times b_2 + 0.1\times Lo \leq 0.2\times Lo$ and $\leq b_2$

= $0.2\times 4.85 + 0.1\times 3.25 <= 0.2\times 3$ and≤ 4.85

= $1.295 \leq 0.65$ and≤ 4.85

= 0.65 m

$b_{eff} = b_{eff,1} + b_{eff,2} + b_w \leq b$

= $0.65 + 0.65 + 0.3 \leq b$ [$b = b1 + b2 + b_w$]

= $1.6 \leq b$

= $1.6 \leq 10$

= 1.6m

Hence, effective width of flange, $\mathbf{b_{eff}}$ = 1.6m

Clear cover=40mm

Let us provide 32 mm dia. bars

Effective depth, **d** = 1725-40--$\frac{32}{2}$ = 1669 mm=1.669m

Material properties

- Concrete used: M30 (IRC 112-2020 Table 6.4)
- Characteristic strength, f_{ck}= 30 N/mm^2
- Design compressive strength of concrete, f_{cd}=$\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]
- $\alpha = 0.67$
- $\gamma m = 1.5$
- Design compressive strength of concrete, f_{cd}=$\frac{0.67 \times fck}{1.5} = \frac{0.67*30}{1.5}$ = 13.40 N/mm^2
- Steel used: Fe500
- Yield Strength of Steel, f_{yk}=500N/mm^2
- Design yield strength of steel, f_{yd}= f_{yk}/1.15= 0.87fy = 434.783 N/mm^2
- Young's Modulus of Elasticity, Es = 2×10^{5}N/mm^2
- Yield strain for steel (ε_{yd}) = $\frac{\text{fyd}}{\text{Es}} = \frac{0.87*\text{fy}}{\text{Es}} = \frac{0.87*500}{200000}$ = 0.0218
- Area factor (β_1) = 0.810
- CG factor(β_2) = 0.416
- Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu2}+\varepsilon_{yd}}$d [SP 105]

$= \frac{0.0035*1669}{0.0035+0.0218}$ = 1029.339 mm

CG from top = $\beta_2 \times X_{lim}$

= 0.416 × 1029.339

$= 428.205$ mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1029.339}{1000}$

= 3349.748 kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim}$ = 1669 – 0.416 × 1029.339

= 1240.795 mm

$M_{u,\ lim1} = C \times z = 3349.748 \times \frac{1240.795}{1000} = 4156.350$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times X_{lim}$

$= \frac{13.40 \times (1600 - 300) \times 220}{1000}$

= 3832.400 kN

CG from steel level (z_2) = d- $\frac{Df}{2}$= 1669– $\frac{220}{2}$

= 1559 mm

$M_{ur,\ lim2} = C \times z = 3832.400 \times \frac{1559}{1000} = 5974.712$ kNm

Total compressive force, C = 3349.748 + 3832.400 = 7182.148 kN

Total limiting moment, $M_{ur,\ lim} = M_{ur,\ lim1} + M_{ur,\ lim2}$

= 4156.305 + 5974.712

= 10131.062 kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{10131.062 \times 1000}{7182.148}$

=1410.589 mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{7182.148}{434.783} = 16518.941$ mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4}$ = 804.258 mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{16518.941}{804.258}$ = 20.540 ≈ 21

So, this section can take up to 10131.062 kNm with 21 number 32 mm dia bars.

Design of main reinforcement

Design Bending Moment, M_{Ed} =832.150 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{1669}{2\times0.416} - \sqrt{\left(\frac{1669}{2\times0.416}\right)^2 - \frac{832.150\times10^6}{0.810\times0.416\times1600\times13.40}}$$

$$X_u = 28.936 \text{ mm}$$

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = $(d - \beta_2 \times X_u)$ = (1669 – 0.416 × 28.936) = 1656.964 mm

$A_{st} = \frac{832.150\times10^6}{434.783\times1656.964}$ = 1155.092 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{1155.092}{804.248}$ = 1.44

Provide 2 number of bars of 32 mm diameter with area, = 1608.495 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\,b_t d$ but not less than $0.0013\,b_t d$

Here, For M30

$f_{ctm} = 2.5$ [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times\frac{2.5}{500}\times 300\times1669$ but not less than $0.0013\times300\times1669$

=650.91 mm^2 but not less than 650.91 mm^2

= 650.91mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\,A_c$

$A_{s,\,max} = 0.025\times$ [(1725×300)] = 12937.500 mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

Design Of Shear Reinforcement

Design shear force, V_{Ed} =410.358 KN

<u>Maximum Allowable Shear Force (for maximum shear force take Θ = 45^o)</u>

$$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z) = (d – β_2 × X_u) = (1669 – 0.416 × 28.936)

= 1656.964 mm

$v_1 = 0.6\left(1-\frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45^o

Now,

$$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$$

$$= 1 \times 300 \times 1656.964 \times 0.6\left(1-\frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45+\tan 45}$$

=1802.221 kN

And,

V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= $V_{ED=}$410.358 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$

$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$

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

$=1+\sqrt{\frac{200}{1669}}$

$=1.346$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$=0.031\times1.346^{3/2}\times30^{1/2}$

$=0.265$

$\sigma_{cp} = 0$

$\rho1 = \frac{A_{st}}{b_w.d} = 0.0032 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho1=0.0032$

$\therefore V_{Rd.c} = [0.12\times1.346\times (80 \times 0.0032 \times 30)^{0.33}] \times300\times1669$

$=158.703$ kN

And, $V_{Rd.c} =(Vmin+0.15\sigma cp) \times bwd$

$= (0.265+0.15\times 0) \times 300\times 1669$

$=132.785$ kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=158.703 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=400.501 kN

∴Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Design of Shear Reinforcement

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2 \times 410.358 \times 1000}{1 \times 300 \times 1656.964 \times 0.542 \times 0.446 \times 30}\right)}{2}$$

$= 6.521^0$

∴As per the code $21.8^0 \leq \theta \leq 45^0$

Adopt $\theta = 21.8^0$

∴$V_{Rds} = V_{NS} = V_{ED} = \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S = \frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴$Fywd = 500/1.15 = 434.78$ N/mm^2

∴$S \;\; = \frac{2*113.09}{410.358 \times 10^3} \times 1656.964 \times 434.78 \times cot 21.8^0$

=992.83 mm

∴ Provide spacing = 300 mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w = \frac{A_{sw}}{s \times b_w} = \frac{226.195}{300 \times 300} = 0.00251$

<u>Minimum shear reinforcement ratio:</u>

$\therefore \rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk} = \frac{0.072\times\sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing

Side face reinforcement or skin reinforcement:

As= 0.1% of web area **(IS 456:200 cl 26.5.1.3)**

$As = \frac{0.1}{100} \times 300 \times (1725 - 220) = 451.5mm2$

Let us provide 12mm dia. Bar for skin reinforcement.

No. of bars= $\frac{451.5}{\frac{\pi\times 12^2}{4}} = 3.99$

Provide 4 bars of dia. 12 mm in the internal periphery of the beam

5.4.3 End cross girder

Load Calculation

Weight of slab = depth of slab*unit wt.*FOS

= 0.22*25*1.35

= 7.425 KN/m^2

Weight of wearing coat = Thickness*unit wt.*FOS

= 0.075*22*1.75

= 2.888 KN/m^2

Total load from slab and wearing coat =7.425+2.888

=10.313 KN/m^2

Total point load from slab and wearing coat =10.313 *area of 2 triangles

= 10.213 * 2 *1/2 *1.625^2

= 27.233 KN

Equivalent UDL from slab and wearing coat = $\frac{27.233}{3.25}$

= 8.379 KN/m

Self-weight of cross girder=width*depth*unit wt.*Fos

=0.3*(1.725-0.22) *25*1.35

=15.238 KN/m

Total UDL on cross girder=8.379+ 15.238

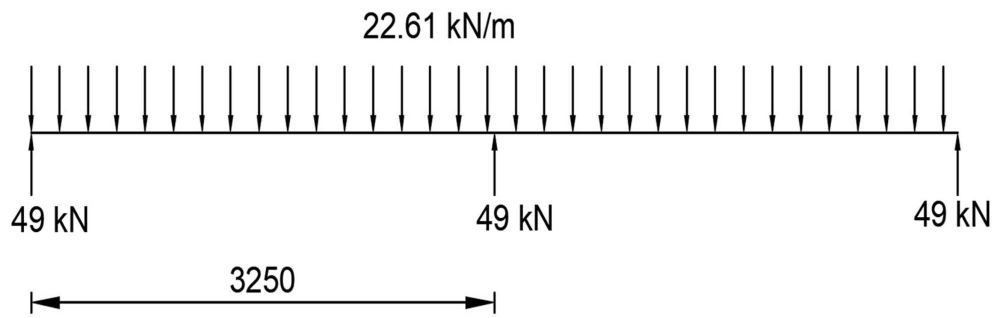
=22.617 KN/m

Let us assume, the load shared by each girder is same i.e., the girder is rigid

Hence reaction at each end point of girder=$\frac{22.617\times3.25\times2}{3}$ = 49.004KN

Maximum shear force due to DL= reaction at the support=49.004 KN

(Maximum bending moment for dead load is calculated at the same point for which the bending moment due to live load will be maximum.)



Calculation of live load on cross girder:

Class 70R track loading;

To calculate the live load on the cross girder to have maximum bending moment due to class 70R track vehicle, the load is placed symmetrically about the center line of bridge as show in figure below:

Let us take the span which occupy half the track of 70R load as shown in red color and a part of cross girder as the support.

Track load, W=350 KN

Length of track= 4.57m

Equivalent UDL=$\frac{350}{4.57}$ =76.59KN/m

<u>Reaction calculation due to live load</u>

Taking moment about the 1, we get

$\sum M_1 = 0$

$R_2\times10-76.59\times2.285\times\frac{2.285}{2}=0$

$R_2 = \frac{199.95}{10}$

$R_2 = 19.995$ KN

$\sum F_y = 0$

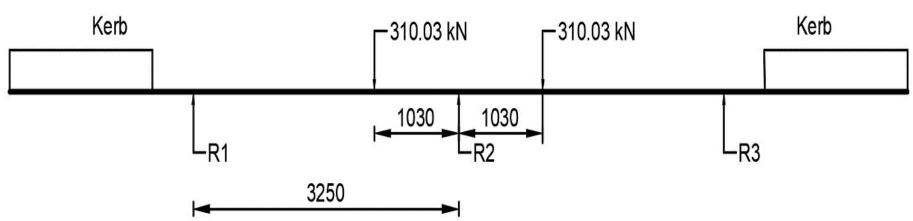
$R_2 + R_1 - 76.59 * 2.285 = 0$

$R_1 = 175.01 - 19.995$

$R_1 = 155.015$ KN

Since the reaction calculated above is from half of the track load only, hence multiplying by 2 to get the reaction for full length of the track i.e. 155.015*2=310.03 KN. It is actually the load acting on the girder.

Calculating the reaction and finding the bending moment under the load to get the maximum value of BM. Assuming that the cross girder has rigid reaction on each longitudinal girder:



$R_1 = R_2 = R_3 = \frac{2 \times 310.03}{3} = 206.687$ kN

Maximum LL bending moment under the load=206.687*2.22=458.845 KNm

Maximum LL bending moment considering impact factor=458.845*1.1

=504.730KNm

Maximum shear force due to LL=reaction at support=206.687KN

Maximum shear force due to LL considering impact factor=206.687*1.1

=227.356 KN

Maximum BM due to DL= 49.004*2.22-22.617*2.22*2.22*0.5

=53.056 KNm

Total design BM=BM due to DL + BM due to LL

=53.056 KNm+504.730*1.5KNm

=810.151 KNm

Total design SF=SF due to DL+SF due to LL

=49.004 KN+ 227.356*1.5KN

=390.038 KN

Class 70R wheeled loading:

To calculate the live load on the cross girder to have maximum bending moment due to class 70R wheeled vehicle, the load is placed symmetrically about the center line of bridge.

In the case of 70R wheeled load the tyre dimensions are so small in comparison with track of 70R track vehicle, so we assume that the total wheel load act as a point load on the single girder.

Reactions

Assuming that the cross girder has rigid reaction on each longitudinal girder:

$R_1 = R_2 = R_3 = \frac{2\times85}{3} = 56.67$ kN

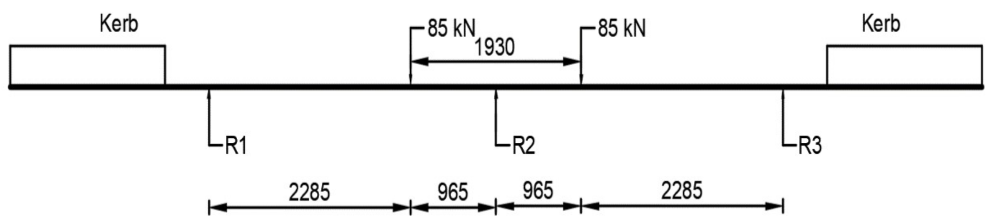


fig: Positioning of 70R wheel load

Maximum live load BM under the load=56.67*2.285

=129.491 KNm

Maximum live load BM due to LL considering IF=129.491*1.12

=145.030 KNm

Maximum SF due to LL considering IF=56.67*1.12

=63.470 KN

Maximum BM due to DL=49.004*2.285-22.617*2.285*2.285*0.5

=52.930 KNm

Maximum SF due to DL= 49.004 KN

Total design BM=BM due to DL+BM due to LL

=52.930+145.030*1.5

=270.475 KNm

Total design SF=SF due to DL+SF due to LL

=49.004+63.470*1.5

=144.209 KN

Class A loading:

To calculate the live load on the cross girder to have maximum bending moment due to class A vehicle, the load is placed symmetrically about the center line of bridge as show in figure below:

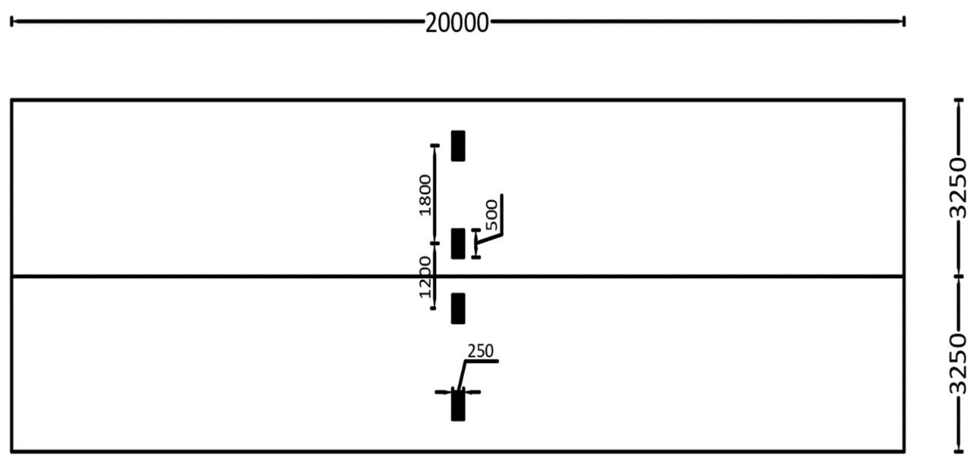


fig: Positioning of class A load

Calculation of load acting on the cross girder: Equivalent UDL corresponding to 57 KN load =57/0.25=228 KN/m

Taking moment about the 1, we get

$\sum M_1 = 0$

$R_2 * 10 - 228 * 0.125 * \frac{0.125}{2} = 0$

$R_2 = \frac{1.781}{10}$

$R_2 = 0.178$ kN

$F_y = 0$

$R_2 + R_1 - 228 * 0.125 = 0$

$R_1 = 28.5 - 0.178$

$R_1 = 28.322$ KN

Since the reaction calculated above is from half of the track load only, hence multiplying by 2 to get the reaction for full length of the track i.e., 28.322*2=56.644 KN. It is actually the load coming on the girder.

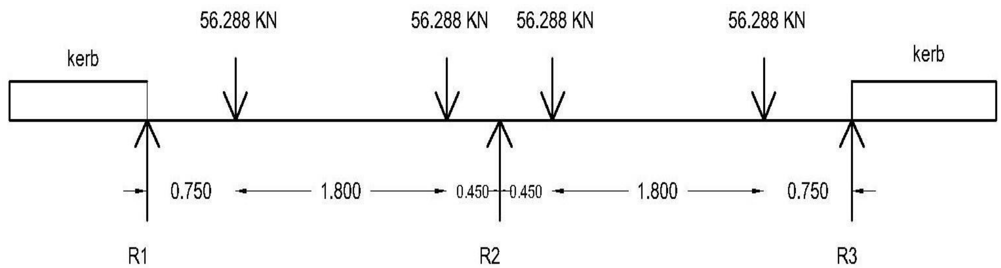


Fig:Load acting on the cross girder

Calculating the reaction and finding the bending moment under the load to get the maximum value of BM. Assuming that the cross girder has rigid reaction on each longitudinal girder:

$R_1 = R_2 = R_3 = \frac{4*56.644}{3} = 75.525$ kN

Maximum LL bending moment under the load =75.525*0.6

=45.315 KNm

Maximum LL bending moment considering impact factor, LDF and FOS

= 45.315*1.125*2*1.5

=152.938 KNm

Maximum shear force due to LL=reaction at support

=75.525 KN

Maximum shear force due to LL considering impact factor, lane distribution factor and FOS

=75.525*1.125*2*1.5

=254.896 KN

Maximum BM due to DL = 49.004*0.6-22.617 *0.6*0.6*0.5

=25.331 KNm

Maximum SF due to DL= 49.004 KN

Total design BM=BM due to DL + BM due to L

=25.331 kNm +152.938 kNm

=178.269 kNm

Total design SF =SF due to DL+SF due to LL

=49.004 KN+254.896KN

=303.900 KN

Table:Summary of design BM and SF

IRC loading	BM KNm	SF KN
70R Track	810.151	390.038
70R wheel	270.475	144.209
Class A	178.269	303.900

As the bending moment and shear force for 70R tracked loading is more.

The design bending moment = 810.151 kNm

The design shear force = 390.038kN

5.4.4.Design of end cross girder

Bending moment = 810.151KNm

Shear force = 390.038kN

Section properties:

width of web(b_w)=300 mm

depth of flange (D_f)= 220 mm

Overall depth of beam (D) =1725 mm

Effective width of flange:

[IRC:112-2020 Clause: 7.6.1.2]

$\mathbf{b_{eff,1}} = 0.2\times b_1 + 0.1\times Lo \leq 0.2\times Lo$ and $\leq b_1$

$= 0.2\times 0 + 0.1\times 3.25 \leq 0.2\times 3.25$ and ≤ 0

$= 0 \leq 0.65$ and ≤ 0

$= 0$ m

$b_{eff,2}$ $= 0.2 \times b_2 + 0.1 \times Lo \leq 0.2 \times Lo$ and $\leq b_2$

$= 0.2 \times 4.85 + 0.1 \times 3.25 <= 0.2 \times 3$ and ≤ 4.85

$= 1.295 \leq 0.65$ and ≤ 4.85

$= 0.65$ m

$b_{eff} = b_{eff,1} + b_{eff,2} + b_w \leq b$

$= 0 + 0.65 + 0.3 \leq b$ [$b = b1 + b2 + b_w$]

$= 0.95 \leq b$

$= 0.95 \leq 10$

$= 0.95$m

Hence, effective width of flange, **b_{eff}** = 0.95 m

Clear cover=40mm

Let us provide 32mm dia. Bars

Effective depth, **d** = $1725 - 40 - \frac{32}{2}$ = 1669 mm=1.669m

Material properties

- Concrete used: M30 (IRC 112-2020 Table 6.4)
- Characteristic strength, f_{ck}= 30 N/mm^2
- Design compressive strength of concrete, $f_{cd} = \frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]
- $\alpha = 0.67$
- $\gamma m = 1.5$
- Design compressive strength of concrete, $f_{cd} = \frac{0.67 \times fck}{1.5} = \frac{0.67 * 30}{1.5}$ = 13.40 N/mm^2
- Steel used: Fe500
- Yield Strength of Steel, f_{yk}=500N/mm^2
- Design yield strength of steel, $f_{yd} = f_{yk}/1.15 = 0.87fy$ = 434.783 N/mm^2
- Young's Modulus of Elasticity, Es = 2×10^5N/mm^2
- Yield strain for steel (ε_{yd}) = $\frac{fyd}{Es} = \frac{0.87 * fy}{Es} = \frac{0.87 * 500}{200000}$ = 0.0218
- Area factor (β_1) = 0.810
- CG factor(β_2) = 0.416
- Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}} d$ [SP 105]

$= \frac{0.0035*1669}{0.0035+0.0218} = 1029.339$ mm

CG from top = $\beta_2 \times X_{lim}$

= 0.416 × 1029.339

= 428.205 mm

For Web

Compressive force (C_1) = $\beta_1 \times F_{cd} \times b_w \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 300 \times 1029.339}{1000}$

= 3349.748 kN

CG from steel level (z_1) = d- $\beta_2 \times X_{lim}$ = 1669 – 0.416 × 1029.339

= 1240.795 mm

$M_{u,\ lim1} = C \times z = 3349.748 \times \frac{1240.795}{1000} = 4156.350$ kNm

For Flange

Compressive force (C_2) = $F_{cd} \times (b_{eff} - b_w) \times X_{lim}$

$= \frac{13.40 \times (950 - 300) \times 220}{1000}$

= 1916.2 kN

CG from steel level (z_2) = d- $\frac{Df}{2}$ = 1669– $\frac{220}{2}$

= 1559 mm

$M_{ur,\ lim2} = C \times z = 1916.2 \times \frac{1559}{1000} = 2987.396$ kNm

Total compressive force, C = 3349.748 + 1916.2 = 5265.948 kN

Total limiting moment, $M_{ur,\ lim} = M_{ur,\ lim1} + M_{ur,\ lim2}$

= 4156.305 + 2987.396

= 7143.706 kNm

CG of total compressive force from steel level = $\frac{M_{ur,lim}}{C} = \frac{7143.706 \times 1000}{5265.958}$

= 1356.585 mm

Area of reinforcement, $A_{st} = \frac{C}{F_{yd}} = \frac{5265.948}{434.783} = 12111.681$ mm^2

Using 32mm diameter bars

Area of each bar, $A_0 = \pi \times \frac{32^2}{4}$ = 804.258 mm^2

Number of bars = $\frac{A_{st}}{A_0} = \frac{12111.681}{804.258}$ = 15.060 ≈ 16

So, this section can take up to 12111.681 kNm with 16 number 32 mm dia bars.

Design of main reinforcement
Design Bending Moment, M_{Ed} =810.151 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

As Design bending moment is smaller than limiting bending moment, singly reinforced can be designed

Assuming NA lies in a flange.

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b_{eff}\times f_{cd}}}$$

$$X_u = \frac{1669}{2\times0.416} - \sqrt{\left(\frac{1669}{2\times0.416}\right)^2 - \frac{810.151\times10^6}{0.810\times0.416\times1600\times13.40}}$$

X_u =47.670 mm

Since X_u<D_f, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – $\beta_2 \times X_u$) = (1669 – 0.416 × 47.670) = 1649.171 mm

$A_{st} = \frac{810.151\times10^6}{434.783\times1656.964}$ = 1129.869 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

No of bar in tension = $\frac{A_{st}}{A} = \frac{1129.869}{804.248}$ = 1.40

Provide 2 number of bars of 32 mm diameter with area, = 1608.495 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26\,\frac{f_{ctm}}{f_{yk}}\,b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min}$ = 0.26× $\frac{2.5}{500}$ × 300×1669 but not less than 0.0013×300×1669

=650.91 mm^2 but not less than 650.91 mm^2

= 650.91mm^2

$A_{st,\,provided}$ > $A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max}$ = 0.025 A_c

$A_{s,\,max}$ = 0.025× [(1725×300] = 12937.500 mm^2

$A_{st,\,provided}$ < $A_{s,\,max}$, OK

Design Of Shear Reinforcement

Design shear force, V_{Ed} =390.038 KN

Maximum Allowable Shear Force (for maximum shear force take ϴ = 45º)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\,max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}$0 (RCC)

Lever Arm(z)=(d – β_2 × X_u) = (1669 – 0.416 × 47.670)

= 1649.171 mm

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45º

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$

$= 1 \times 300 \times 1649.171 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$

=1793.745 kN

And,

V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= $V_{ED=}$390.038 kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

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

$$=1+\sqrt{\frac{200}{1669}}$$

$$=1.346$$

V_{min} = $0.031K^{3/2}fck^{1/2}$

$=0.031\times1.346^{3/2}\times30^{1/2}$

$=0.265$

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0032 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1 = 0.0032$

$\therefore V_{Rd.c} = [0.12 \times 1.346 \times (80 \times 0.0032 \times 30)^{0.33}] \times 300 \times 1669$

$=158.703$ kN

And, $V_{Rd.c} = (V_{min} + 0.15\sigma cp) \times bwd$

$= (0.265 + 0.15 \times 0) \times 300 \times 1669$

$=132.785$ kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\ min}$=158.703 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=390.038 kN

$\therefore$Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Design of Shear Reinforcement IRC 112:2020 Cl 10.3.3.1.-4

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2\times390.038\times1000}{1\times300\times1649.171\times0.542\times0.446\times30}\right)}{2}$$

$= 6.233^{o}$

$\therefore$As per the code $21.8^{o} \leq \theta \leq 45^{o}$

Adopt $\theta = 21.8^{o}$

$\therefore V_{Rds} = V_{NS} = V_{ED} = \frac{Asw}{s} \times z \times fywd \times cot\theta$

`

$S=\frac{Asw}{\text{VED}} \times z \times fywd \times cot\theta$

Provide 2 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

∴S $=\frac{2*113.09}{390.038\times10^3} \times 1649.171 \times 434.78 \times cot21.8^0$

=1039.64 mm

∴ Provide spacing = 300 mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w=\frac{A_{sw}}{s\times b_w} = \frac{226.195}{300\times300}$=0.00251

<u>Minimum shear reinforcement ratio:</u>

∴$\rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk}=\frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 300mm c/c spacing

Side face reinforcement or skin reinforcement:

As= 0.1% of web area **(IS 456:200 cl 26.5.1.3)**

$As=\frac{0.1}{100} \times 300 \times (1725 - 220) = 451.5mm2$

Let us provide 12mm dia. Bar for skin reinforcement.

No. of bars$=\frac{451.5}{\frac{\pi\times12^2}{4}} = 3.99$

Provide 4 bars of dia. 12 mm in the internal periphery of the beam.

5.5. Design of Elastomeric Bearing

For the design, loads has been assessed and then for the critical load combinations of the calculated loads, bearings has been designed.

5.5.1. Calculation of Loads on bearing

Dead Load from Superstructure

Weight of Wearing Coat = $22 \times 7.5 \times 0.1 \times 30 \times 1.75$

$= 866.25\text{kN}$

Weight of Railing bar = $\dfrac{4 \times 6.19 \times 9.81 \times 30 \times 1.35 \times 2}{1000}$

$= 19.68\text{kN}$

Weight of railing post = $16 \times 0.2 \times 0.2 \times 1.1 \times 25 \times 1.35 \times 2 = 47.52\text{kN}$

Weight of Kerb = $1.35 \times 2 \times 20 \times 1.75 \times 0.225 \times 25 \times 30 = 797.34\text{kN}$

Weight of Slab = $30 \times 0.22 \times 6.8 \times 25 \times 1.35 = 1514.7\text{kN}$

Weight of Cantilever Slab = $1.35 \times 2\ [0.15 \times 2.1 + (0.32 - 0.15) \times 0.5 \times 2.1] \times 25 \times 30$

$= 999.3375\text{kN}$

Weight of fillet = $1.35 \times 4 \times 0.5 \times 0.1 \times 0.15 \times 30 \times 25 = 30.375\text{kN}$

Weight of Main girder

Web part = $1.83 \times 0.3 \times 25 \times 3 \times 30 = 1235.25\text{KN}$

Bulb part = $((0.7 \times 0.25) + (0.2 \times 0.15)) \times 25 \times 3 \times 30 = 461.25\text{kN}$

Weight of Cross girder

End Cross Girder = 2 × 1.505 × 0.3 × 5.9 × 25 = 133.1925kN

Intermediate Cross Girder = 2 × 1.505 × 0.3 × 5.9 × 25 = 133.1925kN

Total Dead Load from Superstructure (W_u) = 6238.088kN

Dead load from Superstructure on a Bearing = $\frac{6238.088}{6}$ = 1039.681kN

Dead load from superstructure without partial safety factor

$$= \frac{6238.088 - 866.25}{1.35} + \frac{866.25}{1.75} = 4474.139\text{kN}$$

Dead load from superstructure without partial safety factor on a bearing

$$= \frac{4474.139}{6} = 745.139\text{kN}$$

Minimum possible dead load from superstructure (without considering wearing course) on a bearing

$$= \frac{6238.088 - 866.25}{6} = 895.3063\text{kN}$$

Minimum possible dead load from superstructure (without considering wearing course) without partial safety factor on a bearing

$$= \frac{895.306}{1.35} = 663.19\text{kN}$$

Live Load from Superstructure

Maximum live load on a bearing = Maximum reaction of a main girder

= 1204.990/1.5

= 803.32 kN

Load due to Braking Effect

Braking Load= 0.2× 1000 = 200 KN

Horizontal braking effort on each main girder = $\frac{200}{6}$ = 33.33kN

Braking load acts at 1.2 m above wearing course (Clause 211.3 IRC 06: 2017).

Point of application of braking load = 1.2+0.075+2.3 = 3.575 m

Vertical reaction on a bearing due to braking load = $\frac{33.33 \times 3.575}{30}$ = 7.94 kN

Wind Load

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

Height of bridge < 10.0 m

From Table 5, IRC 06: 2017,

For plain terrain and basic wind 33.0 m/s,

V_Z = 27.8 m/s

P_Z = 463.7 N/m^2

From NBC 104,

Basic wind speed = 47 m/s

Then,

$$V_Z = \frac{47}{33} \times 27.8 = 39.59 \text{ m/s}$$

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

Pressure has been increased by 20% for funneling effect (Note 5 of Clause 209.2, IRC 06: 2017)

P_Z = 1.2 × 940.6 = 1128.72 N/m^2

Gust factor, G = 2 for span up to 150 m (Clause 209.3.3, IRC 06: 2017)

For single beam (Clause 209.3.3, IRC 06: 2014)

C_D = 1.3 for B/D ≥ 6

C_D = 1.5 for B/D ≤ 2

C_D = 1.361 for B/D = 4.783

209.3.3, IRC 06: For Combined effect of multiple beams $C_{D, Combined}$ = 1.5 × 1.361 = 2.04

Transverse area of bridge, A = Area of girder + Area of RC post + Area of railing

= 30 × (2.3+0.225) + 16×0.2×1.1 + (30-15×0.2) × 0.05 × 3

$= 83.32 \text{ m}^2$

$F_W^T = 1128.72 \times 83.32 \times 2 \times 2.04 = 383.95\text{kN}$

F_W^T per bearing = $\frac{383.95}{6} = 63.992\text{kN}$

Wind Load in Longitudinal direction of Bridge, $F_W^L = 0.25 \times F_W^T$ (Clause 209.3.6, IRC 06: 2017)

$= 0.25 \times 383.95$

$= 95.99\text{kN}$

F_W^L per bearing = $\frac{95.99}{6} = 15.998\text{kN}$

Wind load in Vertical direction of bridge;

Plan area = $30 \times 11 = 330 \text{ m}^2$

$F_W^V = P_Z \times A \times G \times C_L$

Where, Lift coefficient, $C_L = 0.75$ (Clause 209.3.5, IRC 06: 2017)

$F_W^V = 940.6 \times 330 \times 2 \times 0.75 = 465.597\text{kN}$

F_W^V per bearing= $\frac{465.597}{6} = 77.6\text{kN}$

Seismic Load

From Clause 218.5.1, IRC 06: 2017

Seismic load = $\frac{Z}{2} * \frac{I}{R} * \frac{S_a}{g} * W$

Where, Z = zone factor = 0.35 as per Geotechnical report

I = Importance factor = 1 for normal bridge (Clause 218.5.1.1, IRC 06: 2017)

R = Response reduction factor = 2 (Table 20, IRC 06: 2017)

$\frac{S_a}{g}$ = average response acceleration coefficient

For 5% damping of RCC structure,

$\frac{S_a}{g} = 2.5$ (Clause 218.5.1, IRC 06: 2017)

For Longitudinal direction

W_L = dead load from superstructure (without considering partial safety factor)

= 4474.139kN

Effective seismic load towards longitudinal direction,

$$F_S^L = \frac{0.36}{2} * \frac{1}{2} \times 2.5 \times 4474.139 = 2013.36 \text{ kN}$$

$$F_{S,\text{ per bearing}}^L = \frac{2013.36}{3} = 335.56 \text{ kN}$$

For Transverse direction

W_T = dead load from superstructure + 0.2 × LL (Clause 219.5.2, IRC 06: 2014)

= 4474.139 + 200 = 4674.139 kN

Effective seismic load towards transverse direction,

$$F_S^T = \frac{0.35}{2} * \frac{1}{2} \times 2.5 \times 4674.139 = 2103.36 \text{ kN}$$

$$F_{S,\text{ per bearing}}^T = \frac{2103.36}{3} = 350.56 \text{ kN}$$

Vertical reaction on support due to seismic load

Vertical reaction on bearing when seismic force is along longitudinal direction

$$F_{S,\text{ Per bearing}}^{VL} = 31.32 \text{ kN}$$

Vertical reaction on bearing when seismic force is along transverse direction

$$F_{S,\text{ Per bearing}}^{VT} = 226.52 \text{ kN}$$

Load due to Temperature variation, Creep and Shrinkage effect

For common reinforced concrete bridge deck, the longitudinal strain due to temperature variation, creep and shrinkage is $5*10^{-4}$.

Horizontal load due to creep, shrinkage and temperature has been distributed to expansion bearing only.

Horizontal deformation of bearing, $\Delta = 5*10^{-4} \times 30000 = 15$ mm.

Shear modulus of elastomeric bearing, $G = 1\ N/mm^2$ (Clause 4.2.1, IRC 83: 2018 (part II))

Approximate minimum height of bearing, $h_0 = 64.0$ mm

Approximate size of bearing = 350 mm × 500 mm

Maximum horizontal force on a bearing,

$$F_{CST} = \frac{\Delta}{2h_0} * G \times A$$

$$F_{CST} = \frac{15}{2 \times 64} * 1 * (350 - 12) \times (500 - 12) = 19.33kN$$

5.5.2. Working Stress Method of Analysis

Loads and their combination (Table B.2, IRC 06: 2014)

Only longitudinal movement of bridge has been allowed and transverse movement has been restricted by providing seismic stopper blocks. So, the bearing has been designed for loads in longitudinal direction only.

Combination I [N]:

Total vertical load = $DL_{sup} + LL + F_{br}^{V}$

= 1556.958 kN

Total horizontal load = 0

Combination II (A) [N+T]:

Total vertical load = $DL_{sup} + LL + F_{br}^{V}$

=1634.558kN

Total horizontal load = $F_{br}^{H} + F_{CST}$

= 68.662 kN

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

Total vertical load = $DL_{sup} + LL + F_{br}^{V} + F_{w}^{V}$

= 941.646 kN

Total horizontal load = $F_{br}^{H} + F_{CST} + F_{W}^{L}$

$= 371.56$ kN

Combination VI [N+T+S]:

Total vertical load = $DL_{sup} + 0.2LL + 0.5F_{br}^{V} + F_{w}^{V}$

$= 941.65$ kN

Total horizontal load = $0.5F_{br}^{H} + F_{CST} + F_{W}^{L}$

$= 371.56$ kN

Design of Bearing without Pin: - Based on IRC 83: 1987 (Part II)

Among four combinations of load, vertical load has been found maximum for combination III. So, design has been carried out considering loads from Combination III and then designed bearing is checked for other cases as per requirements.

N_{Min}= 745.69kN (Dead load without wearing course)
N_{Max}= 1634.558 kN

H_{Max} = 68.66 kN

From Table B1 (Annexure B) of IRC 83: 2018 (Part II), laminated bearing with following dimensions is chosen:

Length, b = 500.0 mm

Width, a = 350.0 mm

Thickness of steel plate, h_s = 4.0 mm

Thickness of middle elastomer layer, h_i = 12.0 mm

Thickness of elastomer layer at top and bottom, h_e = 6.0 mm

Number of steel plate = 4

Number of middle elastomer layer = 3

Total thickness of elastomer layer, h = 3 × 12 + 2 × 6 = 48.0 mm

∴ Total height of bearing, h_0 = 3 × 12 + 4 × 4 + 2 × 6 = 64.0 mm

Provide 6.0 mm gap on either side of elastomer.

So,

Effective length, b’ = (500 – 2 × 6) = 488.0 mm

Effective width, a’ = (350 – 2 × 6) = 338.0 mm

Effective area of bearing, A_1 = 488 × 338 = 164944 mm^2

Check for Geometry

i) $\frac{b}{a} = \frac{500}{350} = 1.43 < 2$ (OK)

ii) $\frac{a}{5} = \frac{350}{5} = 70 > h = 50$ mm (OK)

iii) $\frac{a}{10} = \frac{350}{10} = 35 < h = 50$ mm (OK)

iv) Shape factor, $S = \frac{A_1}{2h_i \times (a' + b')}$

$$S = \frac{164944}{2 \times 12 \times (338 + 488)} = 8.32 > 6 \ \& < 12 \text{ (OK)}$$

Check for Bearing Pressure

Bearing pressure ≤ Allowable bearing pressure

$$\text{Bearing pressure, } \sigma_m = \frac{\text{Maximum verticla load}}{\text{Bearing area}} = \frac{1634.558 \times 1000}{164944} = 9.91 \text{ N/mm}^2$$

$$\text{Allowable bearing pressure} = 0.25 \times f_{CK} \times \sqrt{\frac{A_1}{A_2}} = 0.25 \times 30 \times 2 = 15 \text{ N/mm}^2$$

(as per IS 456:2000, $\sqrt{\frac{A_1}{A_2}}$ is limited to 2)

Here, Allowable bearing pressure > Bearing pressure.

Hence, OK.

Check for translation

$$\gamma_d = \frac{\triangle_{bd}}{h} + \tau_{md} \leq 0.7$$

Where,

$\frac{\triangle_{bd}}{h}$ = Shear strain per bearing due to shrinkage, creep and temperature variation

$$\frac{\triangle_{bd}}{h} = \frac{5 \times 10^{-4} \times \frac{30000}{2}}{64} = 0.15625$$

$$\tau_{md} = \frac{H}{AG} = \frac{53.795 \times 1000}{488 \times 338 \times 1} = 0.416$$

$$\gamma_d = \frac{\triangle_{bd}}{h} + \tau_{md} = 0.15625 + 0.416 = 0.572 < 0.7 \text{ (OK)}$$

Check for rotation

Design rotation in bearing

$$\propto_{a,\,d} = \propto_d^{DL} + \propto_d^{LL} \leq \beta n \propto_{bi\ Max}$$

$$= \frac{400 \times M_{Max,\,DL} \times L \times 10^{-3}}{0.5 \times EI_{gr}} + \frac{400 \times M_{Max,\,LL} \times L \times 10^{-3}}{EI_{gr}}$$

$$= \frac{400 \times \frac{7835.751 \times 10^6}{1.35} \times 30000 \times 10^{-3}}{0.5 \times 5000 \times \sqrt{30} \times 8.873 \times 10^{11}} + \frac{400 \times \frac{9116.496 \times 10^6}{1.5} \times 30000 \times 10^{-3}}{5000 \times \sqrt{30} \times 8.873 \times 10^{11}}$$

$= 0.00834129$

$$\beta = \frac{\sigma_m}{10} = \frac{9.91}{10} = 0.99$$

$n = 4$

$$\propto_{bi\ Max} = \frac{0.5 \times \sigma_m^{Max} \times h_i}{a' \times s^2} = \frac{0.5 \times 10 \times 12}{3388 \times 8.32^2} = 0.002564$$

(σ_m^{Max} =10 MPa as per clause 916.3.5 of IRC 83: 1987 part (II))

$\beta n \propto_{bi\ Max} = 0.99 \times 4 \times 0.002564 = 0.01064$

$\propto_{a,\,d} < \beta n \propto_{bi\ Max}$ (OK)

Check for Friction

$\gamma_d \leq 0.2 + 0.1\,\sigma_m$

$0.2 + 0.1\,\sigma_m = 0.2 + 0.1 \times 10.42$

$= 1.241 > 0.482$ (OK)

Check for total shear stress

Total shear stress

$\tau_c + \tau_\gamma + \tau_\alpha \leq 5$ MPa

Where,

τ_c = shear stress due to axial compression $=1.5 \dfrac{\sigma_m}{s} = 1.5 \times \dfrac{9.91}{8.32} = 1.786$ N/mm^2

τ_γ = shear stress due to horizontal deformation = $\gamma_d = 0.572$ N/mm^2

τ_α = shear stress due to rotation $= 0.5 \times \left(\dfrac{b}{h_i}\right)^2 \times \propto_{bi\ Max}$

$$= 0.5 \times \left(\frac{488}{12}\right)^2 \times 0.00256 = 1.01 \text{ N/mm}^2$$

$\therefore 1.87 + 0.572 + 1.017 = 3.367$ N/mm$^2 \leq 5$ MPa (OK)

5.5.3. Limit State Design Method

Bearing without Pin

Loads and their combination (Table B.2, IRC 06: 2017)

(a) Basic Combination

Total Vertical load = $1.35DL + 1.75WC + 1.5LL + 1.5F_W^V + 1.15F_{br}^V$

$=1039.68 + 803.32 + 1.5 \times 77.6 + 1.15 \times 7.94$

$=2370.2$ kN

Total horizontal load along longitudinal direction=1.5 F_{CST} =1.5 × 19.32 =28.994kN

Total horizontal load along Transverse direction = 0.0 (bearing without pin has been assumed not to take transverse load)

(b) Seismic Combination

Total Vertical load due to seismicity along longitudinal direction

$$= 1.35DL + 1.75WC + 0.2LL + 1.5F_S^{VL} + 0.2F_{br}^{VL}$$

$=1039.68 + 44.63 + 1.5 \times 31.31 + 0.2 \times 7.94$

$= 1248.91$ kN

Total Vertical load due to seismicity along transverse direction

$= 1.35DL + 1.75WC + 0.2LL + 1.5F_S^{VT} + 0.2F_{br}^{VT}$

$=1039.68 + 44.63 + 1.5 \times 226.51 + 0.2 \times 7.94$

$= 1541.71$ kN

Total horizontal load along longitudinal direction = F_{CST} = 19.33kN

Total horizontal load along Transverse direction = 0.0 (bearing without pin has been assumed not to take transverse load)

Design of Bearing: - Based on IRC 83: 2018 (Part II)

N_{Min}= 1039.68kN

N_{Max}= 2370.2 kN

H_{Max}= 1.5 × 19.33 = 28.994kN

From Table B1 (Annexure B) of IRC 83: 2018 (Part II), laminated bearing with following dimensions has been chosen:

Length, b = 500.0 mm

Width, a = 350.0 mm

Thickness of steel plate, h_s = 4.0 mm

Thickness of middle elastomer layer, h_i = 12.0 mm

Thickness of elastomer layer at top and bottom, h_e = 6.0 mm

Number of steel plate = 4

Number of middle elastomer layer = 3

Total thickness of elastomer layer, h = 3 × 12 + 2 × 6 = 48.0 mm

∴ Total height of bearing, h_0 = 3 × 12 + 4 × 4 + 2 × 6 = 64.0 mm

Provide 6.0 mm gap on either side of elastomer.

So,

Effective length, b' = (500 – 2 × 6) = 488.0 mm

Effective width, a' = (350 – 2 × 6) = 338.0 mm

Effective area of bearing, A_1 = 388 × 288 = 164944 mm^2.

Check for Geometry

i) $\frac{b}{a} = \frac{400}{300} = 1.33 < 2$ (OK)

ii) $\frac{a}{5} = \frac{300}{5} = 60 > h = 50$ mm (OK)

iii) $\frac{a}{10} = \frac{300}{10} = 30 < h = 50$ mm (OK)

iv) Shape factor, $S = \frac{A_1}{l_P \times t_e}$ (Clause 5.1.3.1 IRC 83:2018 (Part II)

Where, $l_P = 2 \times (a' + b') = 2 \times (388 + 488) = 1652$ mm

$$t_e = \frac{2 \times 1.4 \times h_e + 4 \times h_i}{\text{total number of layers}} = \frac{2 \times 1.4 \times 6 + 4 \times 12}{3 + 2} = 12.96 \text{ mm}$$

$$S = \frac{164944}{1652 \times 12.96} = 7.7 > 6 \ \& < 12 \text{ (OK)}$$

Check for Bearing Pressure

Bearing pressure ≤ Allowable bearing pressure

$$\text{Bearing pressure} = \frac{\text{Maximum vertical load}}{\text{Bearing area}} = \frac{237.2 \times 1000}{164944} = 14.37 \text{ N/mm}^2$$

$$\text{Allowable bearing pressure} = 0.45 \times f_{CK} \times \sqrt{\frac{A_1}{A_2}} = 0.45 \times 30 \times 2 = 27 \text{ N/mm}^2$$

(as per IS 456:2000, $\sqrt{\frac{A_1}{A_2}}$ is limited to 2)

Here, Allowable bearing pressure > Bearing pressure. (Hence, OK.)

Check for Basic Design Requirements

a. Maximum design strain (Clause 5.1.3, IRC 83: 2018 (part II))

$$\varepsilon_{t,d} = K_L\left(\varepsilon_{c,d} + \varepsilon_{q,d} + \varepsilon_{\alpha,d}\right) \leq \varepsilon_{u,d} = \frac{\varepsilon_{u,k}}{\gamma_m}$$

Where, $K_L = 1$, is type loading factor

$\varepsilon_{c,d}$ = Strain due to compressive design load (Clause 5.1.3.2, IRC 83:2018 (Part II))

$\varepsilon_{q,d}$ = Strain due to shear (Clause 5.1.3.3, IRC 83:2018 (Part II))

$\varepsilon_{\alpha,d}$ = Strain due to angular rotation (Clause 5.1.3.4, IRC 83:2018 (Part II))

$\varepsilon_{u,k} = 7$ (Note 1 of Clause 5.1.3) and $\gamma_m = 1$

$$\therefore \varepsilon_{u,d} = \frac{\varepsilon_{u,k}}{\gamma_m} = \frac{7}{1} = 7.0$$

- Strain due to Compressive design load

$$\varepsilon_{c,d} = \frac{1.5 \times F_{Z,\,d}}{G \times A_r \times S}$$

Where, $F_{Z,}$ d = Maximum vertical load = 2370.2 kN

G = Shear modulus of elasticity of elastomer, generally taken as 1.0 N/mm^2

S = Shape factor

A_r = Reduced effective plan area due to the loading effects given by,

$$A_r = A_1 \times \left(1 - \frac{V_{x,\,d}}{a'} - \frac{V_{y,\,d}}{b'}\right)$$

$$V_{x,\,d} = \frac{\text{Maximum horizontal load in the direction of a}}{G \times A_1} \times h$$

$$= \frac{28.994 \times 1000}{1 \times 164944} \times 48$$

$$= 8.4375 \text{ mm}$$

Similarly, $V_{y,\,d} = 0$

$$A_r = 164944 \times \left(1 - \frac{8.44}{338} - 0\right) = 160826.5 \text{ mm}^2$$

$$\varepsilon_{c,d} = \frac{1.5 \times 2370.2 \times 1000}{1 \times 160826.5 \times 7.7} = 2.87 \text{ mm}$$

- Strain due to shear

$$\varepsilon_{q,d} = \frac{V_{xy,\,d}}{T_q} = \frac{\sqrt{{V_{x,\,d}}^2 + {V_{y,\,d}}^2}}{T_q} = \frac{V_{x,\,d}}{T_q} = \frac{8.44}{48} = 0.176 \text{ mm} < 1 \text{ (OK)}$$

- Strain due to angular rotation

$$\varepsilon_{\alpha,d} = \frac{a'^2 * \propto_{a,d} + \ b'^2 * \propto_{b,d}}{2 * \sum t_i^2} * \ t_i$$

Where, $\propto_{b,d} = 0$ as there is no rotation along longitudinal axis

$$\propto_{a,\,d} = \propto_d^{DL} + \propto_d^{LL}$$

$$= \frac{400 \times M_{Max,\,DL} \times 1 \times 10^{-3}}{0.5 \times EI_{gr}} + \frac{400 \times M_{Max,\,LL} \times 1 \times 10^{-3}}{EI_{gr}}$$

$$= \frac{400 \times 7835.75 \times 10^6 \times 30000 \times 10^{-3}}{0.5 \times 5000 \times \sqrt{30} \times 8.873 \times 10^{11}} + \frac{400 \times 9116.496 \times 10^6 \times 30000 \times 10^{-3}}{5000 \times \sqrt{30} \times 8.873 \times 10^{11}}$$

$$= 0.01167$$

So,

$$\varepsilon_{\alpha,d} = \frac{338^2 \times 0.01167 + 488^2 \times 0}{2 \times (3 \times 12^3 + 2 \times 6^3)} \times 12 = 1.424$$

Now,

$$\varepsilon_{t,d} = K_L \left(\varepsilon_{c,d} + \varepsilon_{q,d} + \varepsilon_{\alpha,d}\right)$$

$$= 1 \times (2.87 + 0.176 + 1.424)$$

$$= 4.469 < \varepsilon_{u,d} = 7 \text{ (OK)}$$

b. Reinforcing plate thickness (Clause 5.1.3.5, IRC 83: 2018 (part II))

$$t_s = \frac{K_p \times F_{Z,\,d} \times (t_1 + t_2) \times K_h \times \gamma_m}{A_r \times f_y}$$

Where, K_p = Stress correction factor = 1.3

t_1 and t_2 are the thickness of elastomer layer on either side of the plate

f_y = yield stress of the steel = 250.0 N/mm^2

K_h = factor for induced tensile stresses in reinforcing plate whose value is given as,

Without holes: $K_h = 1$

So, for elastomer without holes

$$t_s = \frac{1.3 \times 2370.2 \times 1000 \times (12 + 12) \times 1 \times 1}{160826.5 \times 250}$$

$$= 1.84 \text{ mm} < 3.0 \text{ mm so 4mm adopted.}$$

c. Limiting conditions (Clause 5.1.3.6, IRC 83: 2018 (part II))

i. Rotational limitation condition

For laminated rectangular bearing

$$\sum V_{Z,\,d} - \frac{a' \times \alpha_{a,\,d} + b' \times \alpha_{b,\,d}}{K_{r,\,d}} \geq 0$$

Where,

$K_{r,\,d} = 3$ (Clause 5.1.3.6, IRC 83:2018 (Part II))

$\sum V_{Z,\,d}$ is vertical deflection

From Clause 5.1.3.7, IRC 83: 2018 (Part II),

$$\sum V_{Z,\,d} = \frac{\sum F_{Z,\,d} \times t_i}{A_1} \times \left(\frac{1}{5 \times G \times S^2 + \frac{1}{E_{bearing}}} \right)$$

$E_{bearing}$ is given in Note-1of same clause as 2000 N/mm^2

$$\sum V_{Z,\,d} = \frac{2370.2 \times 1000 \times 48}{146944} \times \left(\frac{1}{5 \times 1 \times 7.7^2 + \frac{1}{2000}} \right)$$

$$= 2.24 \text{ mm}$$

Now,

$$\sum V_{Z,\,d} - \frac{a' \times \propto_{a,\,d} + b' \times \propto_{b,\,d}}{K_{r,\,d}} = 2.45 - \frac{338 \times 0.01224 + 488 \times 0}{3}$$

$$= 1.01 > 0.0 \text{ (OK)}$$

ii. Buckling stability

For laminated rectangular bearing

$$\frac{F_{Z,\,d}}{A_r} < \frac{2 \times a' \times G \times S}{3 \times T_e}$$

$$\text{Or, } \frac{2370.2 \times 1000}{160826.5} < \frac{2 \times 388 \times 1 \times 7.7}{3 \times 48}$$

i.e., 14.74 < 36.166 (OK)

iii. Non sliding condition

$$F_{xy,\,d} \leq \mu_e \times F_{z,\,d\,Min}$$

Where, $F_{xy,\,d}$ = 28.994 kN

$F_{z,\,d\,Min}$ is the minimum value of dead load from superstructure. As rubber has the unique property that it behaves differently below certain minimum load, $F_{z,\,d\,Min}$ has been taken as DL without considering wearing course, i.e.,

$F_{z,\,d\,Min}$ = 895.31kN

$$\mu_e = 0.1 + 1.5 \times \frac{K_f}{\sigma_m}$$

K_f = 0.6 for concrete

$$\sigma_m = \frac{\text{Force}}{\text{Area}} = \frac{895.31 \times 1000}{160826.5} = 5.57\text{N/mm}^2$$

Then,

$$\mu_e = 0.1 + 1.5 \times \frac{0.6}{5.57} = 0.262$$

$$\mu_e \times F_{z,\,d\,Min} = 0.3 \times 488.72 = 234.27\text{kN}$$

Here,

$F_{xy,\,d} = 28.994 \leq \mu_e \times F_{z,\,d\,Min} = 234.27$kN

Hence, OK.

6.Abutment

6.1. Calculation of Loads on Abutment

6.1.1. Unfactored Dead Load from Superstructure

Weight of Wearing Coat = 22 × 7.5 × 0.1 × 30

= 495 kN

Weight of Railing bar = $\dfrac{4 \times 6.19 \times 9.81 \times 30 \times 2}{1000}$

= 14.57 kN

Weight of railing post = 16 × 0.2 × 0.2 × 1.1 × 25 × 2 =35.2 kN

Weight of Kerb = 2× 1.75 × 0.225 × 25 × 30 =590.625 kN

Weight of Slab = 30 × 0.22 × 6.8 × 25 =1122 kN

Weight of Cantilever Slab = 2 [0.15 × 2.1 + (0.32 – 0.15) × 0.5 × 2.1] × 25×30

=740.24 kN

Weight of fillet = 4 × 0.5 × 0.1 × 0.15 × 30 × 25 = 22.5 kN

Weight of Main girder

Web part = 915 KN

Bulb part = 341.667 kN

Weight of Cross girder

End Cross Girder =98.66 kN

Intermediate Cross Girder = 99.66 kN

Total Dead Load from Superstructure (W_u) =4474.139 kN

Reaction Due to above loads: -

Slab and Cross-girder = 2082.072/2 = 1041 kN

Main Girder = 1256.66/2 = 628.33 kN

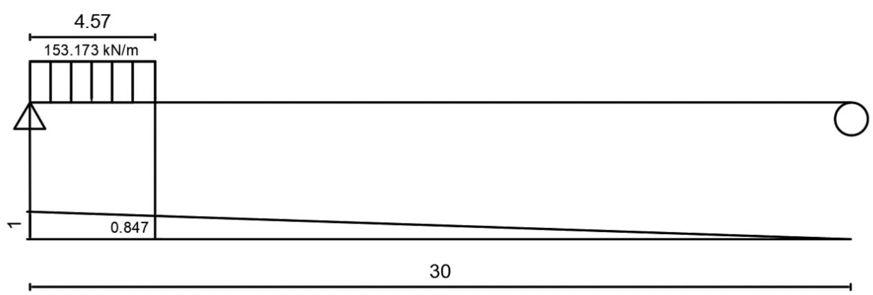
Railing Post and Railing Bar = 49.77/2 = 24.9 kN

Kerb =590.62/2 = 295.3 kN

Surfacing = 495/2 = 247 kN

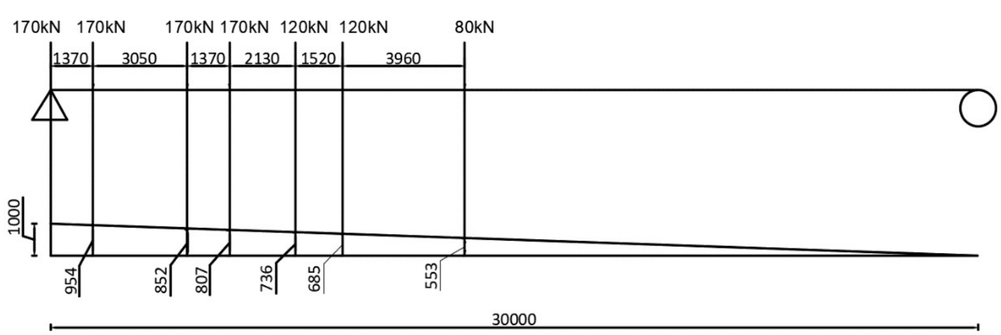
Total reaction = 2273.1 kN

6.1.2. Unfactored Live Load from Superstructure



Reaction at A due to Class70R(Track) vehicle=1/2×4.57×(1+0.848) ×153.173*1.1

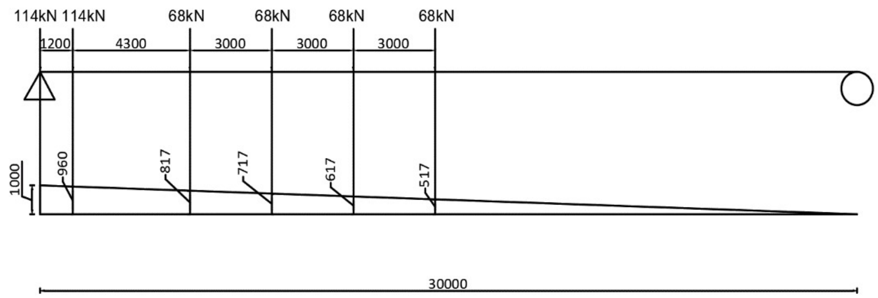
= 357.56 kN



Reaction at A due to Class70R(Wheeled) vehicle

=[170×(1+0.9543+0.8526+0.807) + 120×(0.736+0.6853) + 80×0.5533] *1.12

=474.58 kN



SF (Class A) **= [**114×(1+0.96) + 68×(0.8166+0.7167+0.6167+0.5167) *1.125 * 2

= 803.32 KN

6.1.3. Summary Live Load Reaction

Class 70R Track = 357.56 kN

Class 70R Wheeled = 474.58 kN

Class A = 803.32 kN

Maximum Reaction = 803.32 kN

6.2. Design of Abutment

6.2.1. Material and Properties:

Grade of Concrete = M30

Characteristic strength(fck) = 30N/mm^2

Reinforcement =Fe500

Yield stress of steel(fy) = 500N/mm^2

Unit weight of materials as per IRC:6-2017:

Concrete (Reinforced) = 25kN/m^3

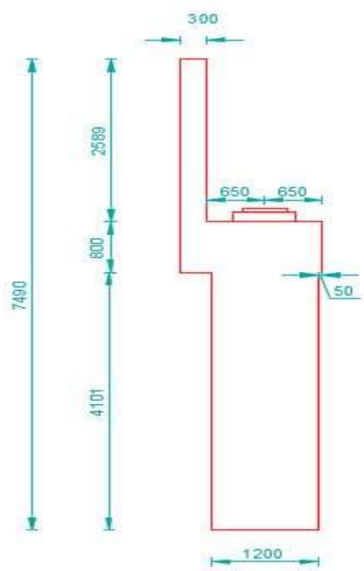
Backfill = 20kN/m^3

6.2.2. Bridge Geometry

Superstructure Geometry = 2.30m

Bearing Height = 0.150+0.064 = 0.214m (including pedestal)

6.2.3. Abutment Geometry



Total Height = 7.49m(up to pile cap)

Wearing Course Height = 0.075m

Height of Backwall = Height of superstructure + Wearing Course Height +Bearing Height

= 2.3+0.075+0.214

= 2.589m

Width of abutment =11m

Backwall Width = 0.3m

Cap Height = 0.8m

Stem Height = Total Height-Height of Backwall-Cap Height

= 7.49-2.589-0.8

= 4.101m

Stem Width = 1.2m

Approach Slab depth = 0.3m

Approach Slab Length = 3.5m

Cap Width = Stem Width+ stem hunch + Projection (0.05m)

= 1.2+0.35+0.05

=1.6m

Distance from back wall to bearing center = 0.65m

Distance from edge to bearing center = Cap Width - Backwall Width - Distance from backwall to bearing center

= 1.6-0.65-0.3

= 0.65m

6.2.4. Calculation of Loads

8.2.4.1. Load from self-weight of abutment and moment at base of stem Stem

=Stem Height*Stem Width*Width of abutment*Unit weight of concrete

= 4.101*1.2*11*25

=1353.33kN

Lever Arm = 0

DL of Cap = Cap Height*Cap Width*Width of abutment*Unit weight of concrete

= 0.8*1.6*11*25

= 352.00kN

Lever Arm = (1.6/2 - 1.2/2) = 0.2

Backwall = Backwall Height*Backwall Width*Width of abutment*Unit weight of concrete

= 2.589*0.3*11*25

= 213.59kN

Lever Arm = (-1.2/2) + 0.3/2 - 0.35 = -0.8m

Component	Wt.(kN)	Distance from center of base	Moment
Stem	1353.33	0.000	0.000
Cap	352.00	0.200	70.40

Back wall	213.59	-0.800	-170.87
Total	1918.92		-100.47

8.2.4.2. Load form superstructure

Dead Load =Weight of slab/Cross girder +Weight of Girder + Weight of railing + Weight of footpath

=1102.6 + 848.25 + 24.9 + 295.3

=2271.0kN

Lever Arm = -(0.35+1.2/2) + (0.3+0.65) = 0.00m

Surface Load = 247.5kN

Lever Arm = -(0.35+1.2/2) + (0.3+0.65) =0.00m

Live load = Maximum load among all vehicles + Pedestrian load

=803.32+0.00 = 803.32

Lever Arm = -(0.35+1.2/2) + (0.3+0.65) = 0.00m

DL	2271.0	0.00	0.00
Surface	247.5	0.00	0.00
LL	803.32	0.00	0.00

8.2.4.3.Earth Pressure

$\Phi = 35°$

$\beta = 0°$

$\alpha = 0°$

$\delta = 22.50°$ (2/3 of Φ)

γ =20 kN/m^3

Term1 = $\cos^2(\Phi - \alpha)$ = 0.671

Term2 = $\cos^2(\alpha)\cos(\delta+\alpha)$ = 0.924

Term3 = $\sin(\Phi+\delta)\sin(\Phi-\beta)$ = 0.484

Term4 = $\cos(\alpha-\beta)\cos(\delta+\alpha)$ = 0.924

$$ka = \frac{\text{Term1}}{\text{Term2}} * \left(\frac{1}{1+\sqrt{\frac{\text{Term3}}{\text{Term4}}}}\right)^2$$

$K_a = 0.244$

h = Total Height - Approach Slab depth

= 7.49 - 0.3

= 7.19m

Earth pressure = $\gamma * K_a * h$

= 20 * 0.244 * 7.19

= 35.16kN/m^2

Total Force = 0.5*Pressure*Height*Width of abutment

= 0.5*35.16*7.19*11

= 1390.29kN acts @0.42h i.e. 3.020m for dry soil from base

Horizontal Force =1390.29*cos (δ)

=1390.29*cos (22.5)

=1284.46kN acts @ 0.42h i.e. 3.020m for dry soil from base

Vertical Force =1390.29*sin (δ)

=1390.29*sin (22.5)

=532.04kN acts @ (-1.2/2= -0.6m from stem center)

	Force (KN)	LA (m)	M (KNm)
(EP h) =	1284.46	3.020	3878.82
(FP v) =	532.04	-0.6	-319.22

8.2.4.4.Braking load

Weight of 70R wheeled vehicle = 1000kN

Weight of Class A train of vehicle = 554kN

Point of application of load = 1.2m above deck

Lane	Factor	Force(kN)
1	0.2	200.0

1	0.2	110.8

Braking load = 200 kN

Lever arm = Point of application of load from deck slab + height of bridge deck + bearing height

= 1.2 + 2.3 +0.064

= 3.564 m

Moment = braking load * lever arm

= 200 * 3.564

= 995.2 kNm

Here total length of class A vehicle is 20.3m and minimum spacing between two is 18.5m. (38.8>30) m. So, factor of 10% is not taken only 20% taken.

8.4.5.Temperature

Temperature variation =30 ˚C

Coefficient Of thermal expansion(α) = 0.000012m/ ˚C / m

Length = 30m

Strain due to shrinkage = 0.0002(IRC6)

Thermal Elongation = 30*0.0000120*30

= 0.0108m

Shrinkage Elongation = 30*0.00020

=0.006m

Total strain due to temp. and shrinkage = 0.5*(0.0108+0.006)

= 0.0084m

Shear Rating of elastomer bearing = 1000 KN/m/m^2

Shear modulus of elastomeric bearing = 1.1(IRC. 83 part II)

Area of bearing = 0.175m^2

Height of bearing = 0.064m (subtracting all plates)

No. of bearing = 3

Force = 1.1*0.0084*1000*0.175/0.064

= 25.26kN

Lever arm = Bearing Height+ Cap Height + Stem Height

= 0.064+0.8+4.101

= 5.11m

Force on each abutment = 75.8 kN

Moment = 75.8 * 5.11 = 387.701kNm

8.4.6.LL Surcharge

K_a = 0.244

Height(h) = 1.2m

ɣ = 20kN/m^3

Pressure = ɣ* K_a*h

= 20*0.244*1.2

=5 .868kN/m^2

Force = Pressure*(Total Height - Approach Slab Depth) *Width of abutment

= 5.868*(7.49-0.3) *11

= 464.07kN

Lever arm = Total Height/2 = 7.49/2 = 3.74m

Moment = 464.07 * 3.74 = 1737.96 kNm

8.4.7. Seismic Loads

Zone Factor = 0.36

Imp Factor = 1

Response Reduction Factor = 3

Sa/g = 2.5

$$\frac{Z}{2} * \mathrm{I} * \frac{S_a}{g} = \frac{0.36}{2} * 1.2 * 2.5 = 0.45$$

A_h = 0.45/3 = 0.15

This term 'A_h' is multiplied with the weight and horizontal seismic force is determined. Lever arm is taken in the direction of height of abutment accordingly moment is determined. Calculation is tabulated below:

i. <u>Abutment Self Load</u>

Component	Wt.(kN)	Horizontal force(kN)	Lever arm(m)	Moment(kNm)
Stem	1353.33	203.00	2.05	416.25
Cap	352.00	52.8	4.501	237.65
Back wall	213.59	32.038	6.19	198.49
Total		287.84		852.40

ii. Superstructure Dead Load

Weight = 2518.52kN

Horizontal Force = 2518.52 * 0.15 = 377.78

Lever arm = 6.65m

Moment = 2511.60kNm

Here, live load is not taken according to Clause 219.5.2(I) of IRC:6-2017

iii. Due to backfill

$\Phi = 35°$

$\beta = 0°$

$\alpha = 0°$

$\delta = 22.50°$ (2/3 of Φ)

$\gamma = 20$ kN/m^3

$A_h = 0.15$

$Av = 2/3*0.15 = 0.1$

$\Lambda = \tan^{-1}(\frac{A_h}{1 \pm A_v}) = \tan^{-1}\frac{0.18}{1 \pm 0.12} = 0.165$ rad

$$C_a = \frac{(1 \pm A_v)\cos^2(\emptyset - \lambda - \alpha)}{\cos\lambda\cos^2(\alpha)\cos(\delta + \alpha + \lambda)} * \left(\frac{1}{1 + \left(\frac{\sin(\emptyset + \delta)\sin(\emptyset - \beta - \lambda)}{\cos(\alpha - \beta)\cos(\delta + \alpha + \lambda)}\right)^{0.5}}\right)^2$$

$C_a = 0.391$

Ca-Ka = 0.391 - 0.244 = 0.146

Height (h) = 7.19m

Width of abutment (B) = 11m

Earth Pressure Seismic = ɣ*(Ca-Ka) *h

= 20*0.146*7.19

= 21.05kN/m2

Seismic force due to backfill = 0.5*21.05*7.19*11

= 832.52kN

Lever arm = 7.19/2

= 3.59m

Moment = 832.52 * 3.59 = 2992.91 kNm

iv. Dynamic Surcharge

Force = ɣ*(Ca-Ka) *h * Height of abutment * Width of abutment

= 20*0.146*1.2*7.19*11

= 277.89kN

Lever arm = Height of abutment/2 = 7.49/2 = 3.74m

Moment = 277.79 * 3.74 = 1040.71 kNm

Unfactored

	Forces (KN)		***Moment***	**Load Factors**	
	Vertical	***Horizontal***	***(kNm)***	Basic	Seismic
Abutment Self	1918.9	0.0	-100.5	1.00	1
SS DL	2271.0		0.0	1.00	1
SS Surface	247.5		0.0	1.00	1
SS LL	803.3		0.0	0.00	0.00
EPH		1284.5	3878.8	1.50	1.00
EPV	532.0		-319.2	1.00	1.00
LL Surcharge (H)		464.1	1738.0	1.20	0.20
Braking		200.0	995.2	1.50	0.20
Temperature		75.8	387.7	0.90	0.50
	Seismic Loads				
Superstr. DL		377.8	2511.6	0.00	1.50
Abutment DL		287.8	0.0	0.00	1.50
Earth pressure		832.5	2992.9	0.00	1.50
Dynamic Surcharge		277.9	1040.7	0.00	1.50

8.2.5. Summary of Loads

	Factored (Non-Seismic/Structure)			Factored (Seismic / Structure)		
	Forces (KN)		***Moment (KNm)***	***Forces (KN)***		***Moment (KNm)***
	Vertical	***Horizontal***		***Vertical***	***Horizontal***	
Abutment Self	1918.9	0.0	-100.5	1918.9	0.0	-100.5
SS DL	2271.0	0.0	0.0	2271.0	0.0	0.0
SS Surface	247.5	0.0	0.0	247.5	0.0	0.0
SS LL	0.0	0.0	0.0	0.0	0.0	0.0
EPH	0.0	1926.7	5818.2	0.0	1284.5	3878.8
EPV	532.0	0.0	-319.2	532.0	0.0	-319.2
Surcharge (H)	0.0	556.9	2085.6	0.0	92.8	347.6
Braking	0.0	300.0	1492.8	0.0	40.0	199.0
Temperature	0.0	68.2	348.9	0.0	37.9	193.9
	SEISMIC					
Superstr. DL	0.0	0.0	0.0	0.0	566.7	3767.4
Abutment DL	0.0	0.0	0.0	0.0	431.8	0.0
Earth pressure	0.0	0.0	0.0	0.0	1248.8	4489.4
Surcharge	0.0	0.0	0.0	0.0	416.8	1561.1
	4969.5	***2851.8***	***9325.8***	***4969***	***4119***	***14017***

8.2.6. Structural Design

Width of abutment = 11m

Total Stem Base Width = 1.2m

Area (A_c) = 13.2 m^2

$F_{cd} = 0.446 * f_{ck}$

$= 13.38\ N/mm^2$

Axial Load = $0.1 * f_{cd} * A_c$

$= 17661.6\ kN$

Max Axial Load = 4969kN (Vertical load becomes axial load here)

Since, max axial load is less than $0.1 * f_{cd} * Ac$, abutment stem is designed as cantilever slab where foundation is treated as fixed support and stem is treated as slab.

6.3. Design of Abutment Stem

6.3.1. Calculation of limiting moment

E_{cu2} = 0.0035

E_{yd} = 0.0022

F_{ck} = 30 MPa

0.446*f_{ck} = 13.38 N/mm^2

Xu = 683.96 mm

0.416*X_u = 284.51 mm

Depth = Width of abutment = 1200mm

Clear cover = 75mm

Effective cover = Depth - Clear Cover - Diameter of rebar/2 = 91mm

Effective depth(d) = Depth - Effective Cover = 1109mm

Breadth of Abutment ($B_{w)}$ = 11000mm

β1 = 0.80952

β2 = 0.41597

Compressive Force(C) = β_1*0.446F_{ck}*b_w*X_u = 81491.08 kN

Cg from steel level = d-0.416X_u = 824.49mm

$M_{ulim=}$ C*CG from steel level/1000 = 67188.68 kNm

Our design moment is 14017.4kNm but we get limiting moment as 67188.68kNm which is very high. So, we have to increase tensile strain of steel in such a way that our neutral axis shifts upward and Xu value then M_{ulim} is decreased respectively.

E_{cu2} = 0.0035

E_{yd} = 0.0022

F_{ck} = 30 MPa

0.446*f_{ck} = 13.98 N/mm^2

Xu = 110.68mm

0.416*X_u = 46.04mm

Depth = Width of abutment = 1200mm

Clear cover = 75mm

Effective cover = Depth - Clear Cover - Diameter of rebar/2 = 91mm

Effective depth = Depth -Effective Cover =1109mm

B_w = 11000mm

β1 = 0.80952

β2 = 0.41597

C = β_1*0.446F_{ck}*b_w*X_u

=13187.17 kN

Cg from steel level = d-0.416X_u = 1062.96 mm

M_{ulim} = C*CG from steel level/1000 =14017.4 kNm

(Which is equal to our design moment)

6.3.2. Calculation of Reinforcement:

Fe = 500 N/mm^2

F_{yd} = 434.78 N/mm^2

Steel Required = Compressive Force*1000/ F_{yd} = 30330.49mm^2

Tensile steel as per code (IRC.112 cl.16.5.1.1)

F_{ctm} = 0.26*$f_{ck}^{2/3}$ = 2.51

F_{yk} = 500 N/mm^2

0.26*F_{ctm}/F_{yk} = 0.001144

As per code:

$$A_{s.\min} = 0.26\frac{f_{ctm}}{f_{yk}}\ b_t d, \text{ but not less than } 0.0013\ b_t d$$

So,

Minimum Area of steel = 0.0013*d*b_w = 15858.7 mm^2

Area of concrete (A_c) = D*bw = 13200000 mm^2

Maximum Area of steel = 0.025A_c = 330000 mm^2

Since, steel required is less than area of steel, we are adapting minimum area of steel.

Required A_{st} = 30330.49 mm^2

= 25mm Φ @ 178.026 mm c/c spacing

Provided A_{st} = 32mm Φ @ 150mm c/c spacing

Therefore, area of steel provided=58978.17 mm^2 (>30330.49mm^2, <330000mm^2) (ok)

This is the reinforcement provided vertically at rear end of stem.

6.3.3. Check for shear:

Horizontal load = 4119kN

Breadth of abutment stem (b_w) = 11000mm

Effective Depth of abutment stem(d) = 1109mm

Area of Concrete (A_c) = b_w*d = 12199000 mm^2

Area of Steel Provided (A_{st}) = 58978.17 mm^2

F_{ck} = 30 MPa

σ_c = 0.00

$\rho_l = A_{st}/(b_w*d) = 0.0048 \leq 0.02$

$k = 1+ (200/d)^{0.5} = 1.425 \leq 2$

$\gamma_{min} = 0.031k^{3/2} f_{ck}^{1/2} = 0.2887$

$\gamma_{Rd, c, min} = \gamma_{min} + 0.15\ \sigma_{cp} = 0.289$

$V_{Rd.c} = [0.12k(80\rho_1 * fck)^{0.33} + 0.15\sigma_{cp}$

$= (0.12*1.425*(80*0.0048*30)^{0.33}+0.15*0)$

$= 0.3839$

$\Gamma = \max(\gamma_{Rd, c, min}, V_{Rd.c}) = \max(0.289, 0.3839) = 0.3839$

$V_{Rd.c} = 0.3839*b_w*d = 4683.1kN > 4119kN$ (ok)

So, shear reinforcement is not required.

6.3.4. Crack Width:

Load factor for serviceability Limit State

Normal	Vertical	Horizontal	Moment (KNm)	Rare combination	Frequent Combination	Quasi-permanent Combination
Abutment Self	1918.9	0.0	-100.5	1	1	1
SS DL	2271.0		0.0	1	1	1
SS Surface	247.5		0.0	1.2	1.2	1.2
SS LL	0.0		0.0	1	0.75	0
EPH		1284.5	5818.2	1	1	1
EPV	532.0		-319.2	1	1	1
LL Surcharge (H)		464.1	1738.0	0.8	0	0
Braking		200.0	995.2	1	0.75	0
Temperature		75.8	387.7	0.6	0.5	0.5
Seismic Loads						
Superstructure DL		377.8	2511.6	0	0	0
Abutment DL		287.8		0	0	0
Earth pressure		832.5	2992.9	0	0	0

Factored loads(Rare combination)

	Forces(KN)		**Moment (KNm)**
	Vertical	**Horizontal**	
Abutment Self	1918.9	0.0	-100.5
SS DL	2271.0	0.0	0.0
SS Surface	297.0	0.0	0.0
SS LL	0.0	0.0	0.0
EPH	0.0	1284.5	5818.2
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	371.3	1390.4
Braking	0.0	200.0	995.2
Temperature	0.0	45.5	232.6
	Seismic Loads		
Superstr. DL		0.0	0.0
Abutment DL		0.0	0.0
Earth pressure		0.0	0.0
Total			8016.7

Factored loads(Frequent combination)

	Forces(KN)		***Moment (KNm)***
	Vertical	***Horizontal***	
Abutment Self	1918.9	0.0	-100.5
SS DL	2271.0	0.0	0.0
SS Surface	297.0	0.0	0.0
SS LL	0.0	0.0	0.0
EPH	0.0	1284.5	5818.2
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	0.0	0.0
Braking	0.0	150.0	746.4
Temperature	0.0	56.8	290.8
	Seismic Loads		
Superstr. DL		0.0	0.0
Abutment DL		0.0	0.0
Earth pressure		0.0	0.0
			6435.7

Factored loads (Quasi-permanent combination)

	Forces (KN)		**Moment (KNm)**
	Vertical	**Horizontal**	
Abutment Self	1918.9	0.0	-100.5
SS DL	2271.0	0.0	0.0
SS Surface	297.0	0.0	0.0
SS LL	0.0	0.0	0.0
EPH	0.0	1284.5	5818.2
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	0.0	0.0
Braking	0.0	0.0	0.0
Temperature	0.0	37.9	193.9
	Seismic Loads		
Superstr. DL		0.0	0.0
Abutment DL		0.0	0.0
Earth pressure		0.0	0.0
			5592.4

Bending Moment for crack width check (M_c) =8016.7 kNm (From Rare Combination)

Area of Steel provided (A_s) = 58978mm^2

Width of Abutment stem (B_w) = 11000mm

Total Depth(D) = 1200mm

Effective Depth(d) = D - Effective cover = 1200 – 91 = 1109mm

X_u = 243.91mm

Lever arm (z) = d - Xu/3 = 1028mm

$H_{c,\,eff}$ = 2.5 (h - d) or (h-Xu)/3 or h/2 (whichever is small)

= 2.5*(1200-1109)	= (1200-243.91)/3	= 1200/2
=228 mm	= 319 mm	= 600 mm

∴ $H_{c,\,eff}$ = 228 mm

$A_{c,\,eff}$ = min ($H_{c,\,eff}$) *B_w

= 228*11000

= 2502500mm^2

E_s = 200000MPa(N/mm^2)

Actual Stress(σ_{sc}) = M_c *10^6/ (A_s* z) = 132.26N/mm^2

K_t = 0.5

F_{cm} = f_{ck}+10 = 40MPa

$F_{ct,\,eff}$ = 0.259*$(F_{ck})^{2/3}$

= 0.259*$(30)^{2/3}$

=2.5

E_{cm} = 22*$(f_{cm}/12.5)^{0.3}$*1000MPa(N/mm^2)

=31187 MPa

$\rho_{1,\,eff}$ = $A_s/A_{c,\,eff}$ = 58978/2502500 = 0.0236

α_e = E_s/E_{cm} = 200000/31187 = 6.413

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t * \frac{f_{ct,eff}}{\rho_{p,eff}} * \left(1 + \alpha_e * \rho_{p,eff}\right)}{E_s} \geq 0.6\,\frac{\sigma_{sc}}{E_s}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 0.000397$$

c=75mm

$S_{r, max}$ = 3.4c+0.17*Φ/$P_{1, eff}$) = 3.4*75+0.17*32/0.0236

= 485.82mm

Crack width = c*(ε_{sm}-ε_{cm})

= 75*0.000397

= 0.192mm <0.3mm (ok for severe case)

6.3.5. Horizontal Reinforcement

Vertical main = 32mm Φ @150mm c/c spacing

A_{st} provided = 5361.65m^2/m

Stem width = 1200mm

Stem Height = 1000mm

Ac = 1200*1000 =1200000mm^2

Horizontal reinforcement = 25% A_{st} = 1340.41 mm^2

= 0.001Ac =1200 mm^2

Using 16mm Φ bar

Area of 16mm Φ bar = π*16^2/4 = 201.06mm^2

Then spacing = 1000/ (1200/201.06) = 150mm

Provide A_{st} = 20mm Φ bar @ 150mm c/c spacing

Ast provided = 2094.4 mm^2 (which is greater than 25% of Ast)

6.3.6. Vertical Reinforcement

Stem Width (W) = 1200mm

Stem L = 1000mm

A_c = 1200*1000 = 1200000 mm^2

0.0012*A_c = 0.0012*1200000 = 1440 mm^2

Using 16mm Φ bar

Area of 16mm Φ bar = π*16^2/4 = 201.06mm^2

Then spacing = 1000/ (1440/201.06) = 140mm

Provide A_{st} = 20mm Φ bar @ 150mm c/c spacing

A_{st} provided = 2094.4 mm^2 (which is greater than 0.0012Ac)

Check:

A_{st} provided at rear face = 58978.17 mm^2

A_{st} provided at front face = 2094.40 mm^2

Allowable maximum reinforcement = 0.04 * Ac = 528000mm^2 (ok)

Allowable minimum reinforcement=1440 mm^2 (ok)

6.4. Design of Pile Foundation

6.4.1. Loads from Abutment

	Unfactored		
	Forces (KN)		Moment (KNm)
	Vertical	Horizontal	
Abutment Self	1918.9	0.0	-100.5
SS DL	2253.4		0.0
SS Surface	247.5		0.0
SS LL	892.6		0.0
EPH		1284.5	6190.8
EPV	532.0		-319.2
LL Surcharge (H)		464.1	2573.3
Braking horizontal		200.0	1398.0
Braking vertical	8.0		0.0
Temperature		79.9	552.2
	Seismic Loads		
Superstr. DL		375.1	3169.3
Abutment DL		287.8	0.0
Earth pressure		832.5	4491.5
Dynamic Surcharge		277.9	1540.9

6.4.2. Load combination for design of foundation

The loads and forces may be evaluated as per IRC:6-2017 and their combinations for the purpose of stability check of the foundation should follow IRC:78-2014.

Combination I): $G + (Q + G_S) + F_{wc} + F_f + F_b + G_b + F_{cf} + F_{ep}$

CombinationII): Combination I + $W + F_{wp}$

Or

	Combinations		
	C1	C2	C3
Abutment Self	1	1	1
SS DL	1	1	1
SS Surface	1	1	1
SS LL	1	1	0
EPH	1	1	1
EPV	1	1	1
LL Surcharge (H)	1	1	0

Combination I + F_{eg} + F_{wp}

Or

Combination II + F_{im} + F_{wp}

Combination III) : $G + F_{wc} + G_b + F_{ep} + F_{ep} + F_{er} + F_f + (W \text{ or } F_{eq})$

C1 load combination

Horizontal	1	1	0
Vertical	1	1	0
Temperature	0	0	0
Superstr. DL	0	1	1
Abutment DL	0	1	1
Earth pressure	0	1	1
Dyn. Surcharge	0	1	0

Load = Respective load * Respective load factor

C1

	Forces (KN)		Moment (KNm)
	Vertical	**Horizontal**	
Abutment Self	1918.9	0.0	-100.5
SS DL	2253.4	0.0	0.0
SS Surface	247.5	0.0	0.0
SS LL	892.6	0.0	0.0
EPH	0.0	1284.5	6190.8
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	464.1	2573.3
	0.0	0.0	0.0
horizontal	0.0	200.0	1398.0
vertical	8.0	0.0	0.0
Temperature	0.0	0.0	0.0
	Seismic Loads		
Superstr. DL	0.0	0.0	0.0
Abutment DL	0.0	0.0	0.0
Earth pressure	0.0	0.0	0.0
Dynamic Surcharge	0.0	0.0	0.0
Total	**5852.5**	**1948.5**	**9742.4**

C2 load combination

Load = Respective Load * Load Factor

C2

	Forces (KN)		Moment
	Vertical	**Horizontal**	**(KNm)**
Abutment Self	1918.9	0.0	-100.5
SS DL	2253.4	0.0	0.0
SS Surface	247.5	0.0	0.0
SS LL	892.6	0.0	0.0
EPH	0.0	1284.5	6190.8
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	464.1	2573.3
	0.0	0.0	0.0
Braking horizontal	0.0	200.0	1398.0
Braking vertical	8.0	0.0	0.0
Temperature	0.0	0.0	0.0
	Seismic Loads		
Superstr. DL	0.0	375.1	3169.3
Abutment DL	0.0	287.8	0.0
Earth pressure	0.0	832.5	4491.5
Dynamic Surcharge	0.0	277.9	1540.9
Total	**5852.5**	**3721.9**	**18944.1**

C3 load Combination

Load = Respective Load * Load Factor

C3

	Forces (KN)		Moment (KNm)
	Vertical	**Horizontal**	
Abutment Self	1918.9	0.0	-100.5
SS DL	2253.4	0.0	0.0
SS Surface	247.5	0.0	0.0
SS LL	0.0	0.0	0.0
EPH	0.0	1284.5	6190.8
EPV	532.0	0.0	-319.2
LL Surcharge (H)	0.0	0.0	0.0
	0.0	0.0	0.0
Braking horizontal	0.0	0.0	0.0
Braking vertical	0.0	0.0	0.0
Temperature	0.0	0.0	0.0
	Seismic Loads		
Superstr. DL	0.0	375.1	3169.3
Abutment DL	0.0	287.8	0.0
Earth pressure	0.0	832.5	4491.5
Dynamic Surcharge	0.0	0.0	0.0
Total	**4951.9**	**2780.0**	**13431.9**

Summary of loads from each combination is given below: -

	C1	C2	C3
Vertical (kN)	5852.5	5852.5	4951.9
Horizontal (kN)	1948.5	3721.9	2780.9
Moment (kNm)	9742.4	18944.1	13431.9

6.4.3. Pile Geometry

Thickness of pile cap = 1.8 m

Length of pile cap = 11.4 m

Width of pile cap = 9.6 m

Edge projection in longitudinal axis = 0.6 m

Edge projection in transverse axis = 0.9 m

Number of piles = 9

Purposed length of pile = 23 m

Diameter of pile = 1.2 m

6.4.3.1. Configuration of piles

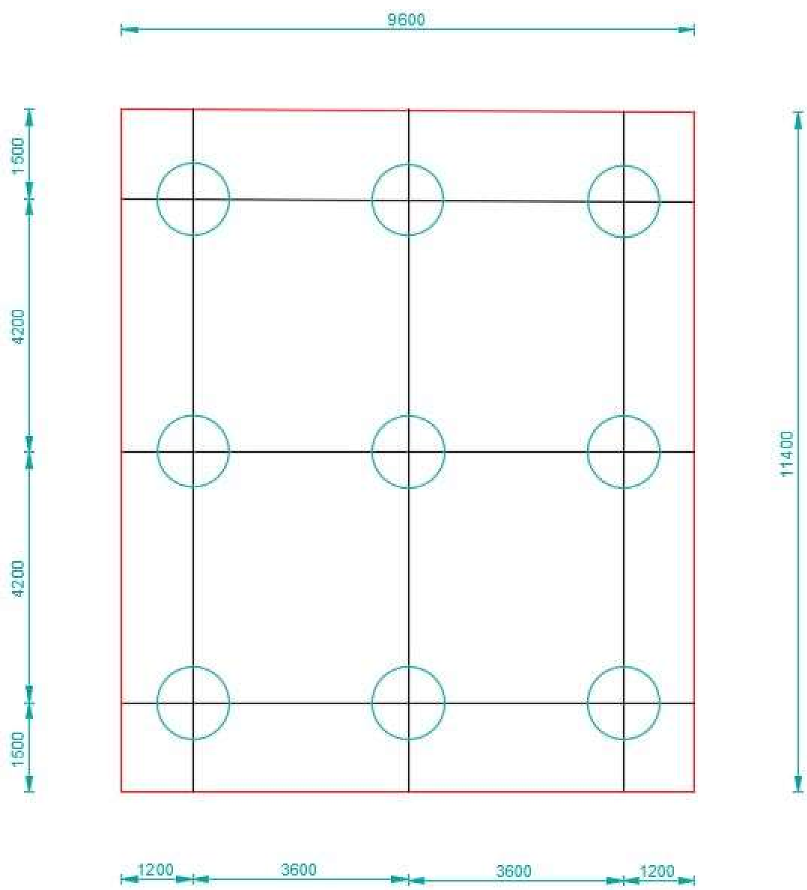


Fig : Pile configuration

piles N	Y(m)	Z(m)	y^2	z^2
1	-3.6	4.2	12.96	17.64
2	0	4.2	0	17.64
3	3.6	4.2	12.96	17.64
4	-3.6	0	12.96	0
5	0	0	0	0

6	3.6	0	12.96	0
7	-3.6	-4.2	12.96	17.64
8	0	-4.2	0	17.64
9	3.6	-4.2	12.96	17.64
			77.76	105.84

Additional load on pile cap = Length * width * thickness * unit weight of concrete

= 11.4 * 9.6 * 1.8 * 25

= 4924.8 kN

Additional load on piles = Area * length * unit weight of concrete * No of piles

$$= \pi * \frac{1.2^2}{4} * 23 * 25 * 9$$

= 5852.78 kN

6.4.3.2. Load Distribution on each pile

$$\mathbf{Total\ Axial\ Load} = \frac{\mathbf{Total\ vertical\ from\ abutment\ +\ Load\ from\ pile\ cap\ +\ Load\ from\ pile}}{\mathbf{total\ no.\ of\ piles}} + \mathbf{M_y} * \frac{\boldsymbol{z}}{\sum \boldsymbol{z^2}}$$

$$\mathbf{Horizontal\ load} = \frac{\mathbf{Horizontal\ load\ \ from\ abutment}}{\mathbf{total\ no.\ of\ piles}}$$

From combination C1

Axial	Hz	My	Total
1847.78	216.50	386.60	2234.39
1847.78	216.50	386.60	2234.39
1847.78	216.50	386.60	2234.39
1847.78	216.50	0.00	1847.78
1847.78	216.50	0.00	1847.78
1847.78	216.50	0.00	1847.78
1847.78	216.50	-386.60	1461.18
1847.78	216.50	-386.60	1461.18
1847.78	216.50	-386.60	1461.18

From combination C2

Axial	Hz	My	Total
1847.78	413.55	751.75	2599.54
1847.78	413.55	751.75	2599.54
1847.78	413.55	751.75	2599.54
1847.78	413.55	0.00	1847.78
1847.78	413.55	0.00	1847.78
1847.78	413.55	0.00	1847.78
1847.78	413.55	-751.75	1096.03
1847.78	413.55	-751.75	1096.03
1847.78	413.55	-751.75	1096.03

From Combination C3

Axial	Hz	My	Total
1747.72	308.88	533.01	2280.73
1747.72	308.88	533.01	2280.73
1747.72	308.88	533.01	2280.73
1747.72	308.88	0.00	1747.72
1747.72	308.88	0.00	1747.72
1747.72	308.88	0.00	1747.72
1747.72	308.88	-533.01	1214.71
1747.72	308.88	-533.01	1214.71
1747.72	308.88	-533.01	1214.71

Maximum Load on each pile

	Total Axial load (kN)	Total Horizontal (kN)
From combination C1	2234.39	216.50
From combination C2	2599.54	413.55
From combination C3	2280.73	308.88

Design Load on each pile

Axial load (P) = 2599.54 kN

Horizontal load (H) = 413.55 kN

6.4.4. Pile Design

Purposed pile length (L) = 23 m

Pile Diameter (D) = 1.2 m

Sub soil parameter for modeling

Modulus of subgrade reaction (η_h) for very loose sand = 5000 kN/m^3

$$\text{Moment of inertia of pile x} - \text{section} = \frac{1}{4} * \pi * \left(\frac{D}{2}\right)^4$$

$$=0.102 \text{ m}^4$$

Young's modulus of pile material (E) = 5000 * $(f_{ck})^{1/2}$*1000

$$=27386128 \text{ N/mm}^2$$

$$\text{Stiffness Factor(T)} = \left(\frac{EI}{\eta_h}\right)^{\frac{1}{5}} = 3.54\, m$$

2T = 2 * 3.54 = 7.08 m

4T = 4 * 3.54 = 14.17 m

Scour depth (e) = 12. 04 m

$$\frac{Scour\ depth\ (e)}{stiffness\ Factor(T)} = \frac{12.04}{3.54} = 3.40\, m$$

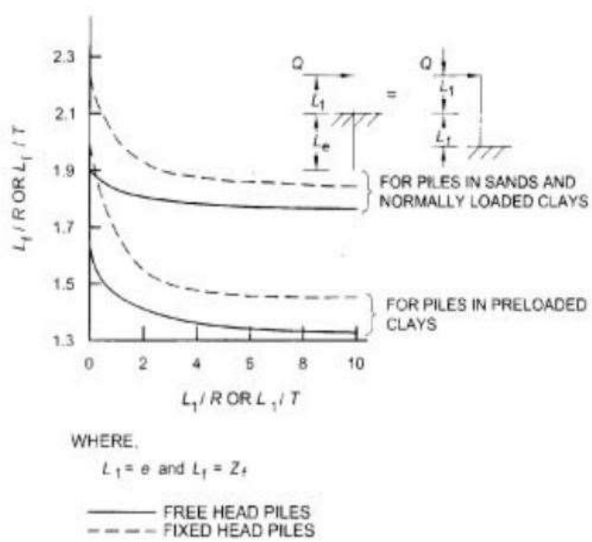


FIG. 4 DEPTH OF FIXITY

From graph,

$$\frac{\text{Depth of point of fixity } (Z_f)}{\text{stiffness factor}(T)} = 1.89$$

$Z_f = 3.54 * 1.89 = 6.71$ m

For Fixed Head moment

$$\text{Deflection}(y) = \frac{H(e + z_f)^3}{12EI} * 10^3$$

$$\text{Fixed End Moment}(M) = \frac{H(e + z_f)}{2}$$

P(kN)	H(kN)	y(mm)	M(kNm)
2234.39	216.50	0.043	2029.58
2599.54	413.55	0.081	3876.74
2280.73	308.88	0.061	2895.59

6.4.4.1. Calculation of Pile Capacity

Pile diameter (D) = 1.2 m

Pile length (L) = 23 m

unit weight of soil (γ) = 20kN/m^3

active earth pressure coefficient (ka) = 0.334

angle of friction between pile and soil (δ) = 29.94°

Frictional coefficient =tan $(\delta * \frac{\pi}{180})$ = 0.576

Cross sectional area of pile tip (A_p) = $\pi * \frac{D^2}{4}$ = 1.13 m^2

Surface area of pile shaft (A_s) = $\pi * D * L$ = 86.71 m

Assuming skin friction is applied before end bearing

Level	Overburden pressure	Force	Friction
0.0	0.00		
0.5	0.00	0.0	0.0
1.0	0.00	0.0	0.0
1.5	0.00	0.0	0.0
2.0	0.00	0.0	0.0
2.5	0.00	0.0	0.0

3.0	0.00	0.0	0.0
3.5	0.00	0.0	0.0
4.0	0.00	0.0	0.0
4.5	0.00	0.0	0.0
5.0	0.00	0.0	0.0
5.5	0.00	0.0	0.0
6.0	0.00	0.0	0.0
6.5	0.00	0.0	0.0
7.0	0.00	0.0	0.0
7.5	0.00	0.0	0.0
8.0	0.00	0.0	0.0
8.5	0.00	0.0	0.0
9.0	0.00	0.0	0.0
9.5	0.00	0.0	0.0
10.0	0.00	0.0	0.0
10.5	0.00	0.0	0.0
11.0	0.00	0.0	0.0
11.5	0.00	0.0	0.0
12.0	0.00	0.0	0.0
12.5	83.50	78.7	45.3
13.0	86.84	160.5	92.5
13.5	90.18	166.8	96.1
14.0	93.52	173.1	99.7
14.5	96.86	179.4	103.3
15.0	100.20	185.7	107.0
15.5	103.54	192.0	110.6
16.0	106.88	198.3	114.2
16.5	110.22	204.6	117.8
17.0	113.56	210.9	121.5
17.5	116.90	217.2	125.1
18.0	120.24	223.5	128.7
18.5	123.58	229.8	132.4
19.0	126.92	236.1	136.0
19.5	130.26	242.4	139.6
20.0	133.60	248.7	143.2
20.5	136.94	255.0	146.9
21.0	140.28	261.3	150.5
21.5	143.62	267.6	154.1
22.0	146.96	273.9	157.7
22.5	150.30	280.2	161.4
23.0	153.64	286.5	165.0
Total Skin Friction (kN)			**2748.6**

In the above table, following formulas are used :

Overburden pressure = Ka*γ*level

Force = Average of Overburden pressure * π * Diameter of pile * difference of levels

Skin friction = frictional coefficient * force

Total skin friction = 2748.6 kN

Note: do not take skin friction upto scour depth

6.4.4.2. End bearing

Coefficient of earth pressure (K_i)= 1

angle of friction between pile and soil (δ) = 29.94°

Frictional coefficient =tan $(\delta * \frac{\pi}{180})$ = 0.576

Effective overburden pressure at pile tip (P_d) = (L-e)*K_i* γ = 219.20 kN/m^2

Note: end bearing must be taken upto 15 times diameter of pile for greater depth

Angle of internal friction, ϕ at pile tip (N_q) = 20

Bearing capacity factor (N_γ) = 20

Cross sectional area of pile tip (A_p) = $\pi * \frac{D^2}{4}$ = 1.13 m^2

Pile diameter (D) = 1.2 m

Total end bearing = A_p * (0.5 * D * γ *N_γ + P_d * N_q)

= 5229.6 kN

FoS = 2.5

$$\text{Ultimate Load Capacity (Qu)} = \frac{Total\ end\ bearing + Total\ skin\ friction}{FoS}$$

$$= \frac{5229.6 + 2748.6}{2.5}$$

= 2902.1 kN > extreme axial force (2599.54 kN) (OK)

6.4.5. Pile Cap Design

Table: Unfactored loads from abutment

	Unfactored			Load Factors	
	Forces(KN)		Moment (KNm)		
	Vertical	Horizontal		Basic	Seismic
Abutment Self	1918.9	0.0	-100.5	1.35	1.35
SS DL	2253.4		0.0	1.35	1.35
SS Surface	247.5		0.0	1.75	1.75
SS LL	892.6		0.0	1.5	0
EPH		1284.5	6190.8	1.5	1
EPV	532.0		-319.2	1	1
LL Surcharge (H)		464.1	2573.3	1.2	0
Braking horizontal		200.0	1398.0	1.5	0
Braking vertical	8.0		0.0	1.5	0
Temperature		79.9	552.2	0.9	0.5
	Seismic Loads				
Superstr. DL		375.1	3169.3	0	1.5
Abutment DL		287.8	0.0	0	1.5
Earth pressure		832.5	4491.5	0	1.5
Dyn. Surcharge		277.9	1540.9	0	1.5

	Factored(basic)			Factored(seismic)		
	Forces(KN)		Moment (KNm)	Forces(KN)		*Moment (KNm)*
	Vertical	Horizontal		Vertical	Horizontal	
Abutment Self	2590.5	0.0	-135.6	2590.5	0.0	-135.6
SS DL	3042.1	0.0	0.0	3042.1	0.0	0.0
SS Surface	433.1	0.0	0.0	433.1	0.0	0.0
SS LL	1338.9	0.0	0.0	0.0	0.0	0.0
EPH	0.0	1926.7	9286.3	0.0	1284.5	6190.8
EPV	532.0	0.0	-319.2	532.0	0.0	-319.2
LL Surcharge (H)	0.0	556.9	3088.0	0.0	0.0	0.0
		0.0	0.0	0.0	0.0	0.0
Braking horizontal	0.0	300.0	2097.0	0.0	0.0	0.0
Braking vertical	12.0	0.0	0.0	0.0	0.0	0.0

Temperature	0.0	71.9	497.0	0.0	39.9	276.1
	Seismic Loads					
Superstr. DL	0.0	0.0	0.0		562.7	4753.9
Abutment DL	0.0	0.0	0.0		431.8	0.0
Earth pressure	0.0	0.0	0.0		1248.8	6737.2
Dyn. Surcharge	0.0	0.0	0.0		416.8	2311.4
Total	7948.7	2855.5	14513.4	6597.8	3984.5	19814.6

6.4.5.1. Summary of loads

	Basic	Seismic
Vertical(kN)	7948.7	6597.8
Horizontal (kN)	2855.5	3984.5
Moment(kNm)	14513.4	19814.6

Additional load on pile cap =1.35 * Length * width * thickness * unit weight of concrete

$$= 1.35 * 11.4 * 9.6 * 1.8 * 25$$

$$= 6648.48\text{kN}$$

Additional load on piles = 1.35 * Area * length * unit weight of concrete * No of piles

$$= 1.35 * \pi * \frac{1.2^2}{4} * 23 * 25 * 9$$

$$=7901.263 \text{ kN}$$

6.4.5.2. Load Distribution on each pile

$$\textbf{Total Axial Load} = \frac{\textbf{Total vertical from abutment} + \textbf{Load from pile cap} + \textbf{Load from pile}}{\textbf{total no. of piles}} + \mathbf{M_y} * \frac{z}{\sum z^2}$$

$$\textbf{Horizontal load} = \frac{\textbf{Horizontal load from abutment}}{\textbf{total no. of piles}}$$

Basic			
Axial	Hz	My	Total Axial load
2499.83	317.27	575.93	3075.75
2499.83	317.27	575.93	3075.75
2499.83	317.27	575.93	3075.75
2499.83	317.27	0.00	2499.83
2499.83	317.27	0.00	2499.83
2499.83	317.27	0.00	2499.83
2499.83	317.27	-575.93	1923.90
2499.83	317.27	-575.93	1923.90
2499.83	317.27	-575.93	1923.90

Maximum axial load = 3075.75 kN

Seismic			
Axial	Hz	My	Total Axial load
2349.73	442.72	786.29	3136.02
2349.73	442.72	786.29	3136.02
2349.73	442.72	786.29	3136.02
2349.73	442.72	0.00	2349.73
2349.73	442.72	0.00	2349.73
2349.73	442.72	0.00	2349.73
2349.73	442.72	-786.29	1563.44
2349.73	442.72	-786.29	1563.44
2349.73	442.72	-786.29	1563.44

Maximum axial load = 3136.02 kN

Design values

Axial load (P) = 3136.02 kN

Horizontal load (H) = 442.72 kN

Deflected length (l) = Z_f + e = 18.75 m

$$\text{Moment (M)} = \frac{\text{H} * \text{L}}{2} = 4150.212 \text{ kNm}$$

6.4.5.3. Material properties

- Concrete used: M30 (IRC 112-2020 Table 6.4)
- Characteristic strength, f_{ck}= 30 N/mm^2
- Design compressive strength of concrete, $f_{cd}=\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]
- $\alpha = 0.67$
- $\gamma m = 1.5$
- Design compressive strength of concrete, $f_{cd}=\frac{0.67 \times fck}{1.5} = \frac{0.67*30}{1.5}$ = 13.40 N/mm^2
- Steel used: Fe500
- Yield Strength of Steel, f_{yk}=500N/mm^2
- Design yield strength of steel, f_{yd}= f_{yk}/1.15= 0.87fy = 434.783 N/mm^2
- Young's Modulus of Elasticity, Es = 2×10^{5}N/mm^2
- Yield strain for steel (ε_{yd}) = $\frac{\text{fyd}}{\text{Es}} = \frac{0.87*\text{fy}}{\text{Es}} = \frac{0.87*500}{200000}$ = 0.0218
- Area factor (β_1) = 0.810
- CG factor(β_2) = 0.416
- Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

Calculation of Limiting Moment

$$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu} + \varepsilon_{yd}} d \text{ [SP 105]}$$

$$= \frac{0.0035*1700}{0.0035+ .0218} = 1048.7 \text{ mm}$$

Compressive force (C) = $\beta_1 \times F_{cd} \times b \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 3292 \times 1048.7}{1000}$

= 47776.7 kN

CG from steel level (z) = d- $\beta_2 \times X_{lim}$ = 1700– 0.416 × 1048.7

= 1263.8 mm

$M_{u,\,lim}$ = C × z = 47776.7× $\frac{1263.8}{1000}$ = 60379.7kNm

Since, $M_{u,\,lim}$ is less than factor bending moment assumed depth is satisfied.

Design of main reinforcement

Design Bending Moment, M_{Ed} = 22498.5 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b\times f_{cd}}}$$

$$X_u = \frac{1700}{2\times0.416} - \sqrt{\left(\frac{1700}{2\times0.416}\right)^2 - \frac{22498.5\times10^6}{0.810\times0.416\times3292\times13.40}}$$

X_u = 314.7 mm

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with z = (d – $\beta_2 \times X_u$) = (1700 – 0.416 × 314.7) = 1569.1 mm

$A_{st} = \frac{22498.5\times10^6}{434.783\times1569.1}$ = 32978.7 mm^2

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.25 mm^2

Spacing required = $\frac{b*A}{A_{st}} = \frac{4200*804.25}{32978.7}$ = 102.42mm

Provide 32mm dia bars @100 mm spacing = 33778.40 mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26 \frac{f_{ctm}}{f_{yk}}$ $b_t d$ but not less than 0.0013 $b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times \frac{2.5}{500} \times 4200\times1700$ but not less than $0.0013\times4200\times1700$

=9282 mm^2 but not less than 9282mm^2

= 9282 mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max} = 0.025\times [1700\times4200] = 178500$ mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

Design Of Shear Reinforcement

Design shear force, V_{Ed} =7499 kN

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

$K = 1+\sqrt{\frac{200}{d}} \le 2$

$= 1+\sqrt{\frac{200}{1700}}$

$= 1.342 \le 2$

= 1.342

$V_{min} = 0.31K^{3/2}fck^{1/2}$

$= 0.31\times1.3423^{3/2}\times30^{1/2}$

=2.63

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0014 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1$=0.0014

$\therefore V_{Rd.c}$ = [0.12×2× $(80 \times 0.014 \times 30)^{0.33}$] ×4200×1700

=5465.14 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×b_wd

= (2.63+0.15× 0) × 4200 × 1700

=18778.2 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c, min}$=18778.2kN. $V_{Ed} < V_{Rd.c}$

Horizontal reinforcement = 0.001 A_c = 0.001*1800*4200 = 7560 mm^2

Using 16mm bars with area 201 mm^2

Spacing = 223.40 mm

Adopt 16 mm bars @ 200 c/c

- **Check For Punching Shear**

  Ast provided = 25132.7 mm^2
  Therefore, ρ_l= 25132.7/(1700*1000*1000)
  = 0.0148 %

  F_{ck} = 30 Mpa
  Effective depth of pile cap(d) = 1.7 m.
  Diameter of pile(D) = 1.2 m.
  Edge projection = 0.90 m.
  Radius of control perimeter $= \frac{D}{2} + 2d = 4\ m.$

  For Internal Pile

  Perimeter (u_i) = 2*3.14*4 = 25.13 m

  Eccentricity(e) = 3136.02/4150.212= 0.756 m.

  $\beta = 1 + 0.6\pi\left(\frac{e}{D+4D}\right) = 1.178$

$V_{ED} = \beta * \frac{3136.02}{u_i} * \frac{d}{1000} = 0.250$ N/mm^2

$K = 1 + \left(\frac{200}{d}\right)^{0.5} = 1.34 < 2$

$v_{min} = 0.031 * K^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}} = 0.117\ N/mm^2$

$V_{rdc} = 0.523$ N/mm^2

Hence $V_{ED} < V_{rdc}$ Hence ok

<u>For Corner Pile</u>

Angle for corner pile = 2.34 radians.

U_1 = 9.358 m.

U_2 = 6.283 m.

$\beta = 1 + k(\frac{M_{ED}}{V_{ED}} * \frac{u_1}{W_1}) = 1.489$

$V_{ED} = 0.352$ N/mm^2

K=1.89

$V_{min} = 0.107$ N/mm^2

$V_{rdc} = 0.523$ N/mm^2 > V_{ED} (ok)

6.4.6. Design of pile stem.

Diameter of pile stem = 1.2 m.

Grade of concrete = 30 Mpa.

Height of pile Stem(L) = 23 m.

Provide Clear Cover of 75 mm

Therefore, effective cover (d') = 75 + 12.5 mm = 87.5 mm.

effective diameter = 1200 – 87.5 – 87.5 = 1025 mm.

From above analysis of loads at base of pile.

6.4.6.1. Design Forces

Design axial force (Pu) = 3136.02 kN.

Design Bending Moment (Equivalent Uniaxial) (Mu) = 4150.21 kNm.

Design Shear Force(Hr) = 442.72 kN.

Zf=6.71 m

Eccentricity = Deflected length = Zf+e =18.75m.

6.4.6.2. Design Steps

- Calculation of longitudinal reinforcement.

$$\frac{d'}{D} = \frac{87.5}{1200} = 0.072$$

$\cong 0.1$

Refer Chart 60 of Sp 16 (IS 456).

$$\frac{P_u}{f_{ck}D^2} = \frac{3136020}{30 \times 1200^2} = 0.072$$

$$\frac{M_u}{f_{ck} \times D^3} = \frac{4150210000}{30 \times 1200^3} = 0.080$$

From above value and using chart 60 of SP 16 (IS 456)

$$\frac{p}{fck} = 0.03$$

Therefore Percentage of reinforcement required,

%p = 0.07*30 = 2.1% > 0.8% (minimum)

So provide %p = 2.1%

Area of reinforcement required(Asc) $= \frac{2.1}{100} * 3.14 * \frac{1200*1200}{4}$

=23738.4 m^2

Area of 32 mm diameter bars = 804.24 m^2

No of 32 mm dia bar required $= \frac{23738.4}{804.24} = 30$

Hence provide 30 numbers of 32mm diameter bars.

Spacing of bars $= 3.14 * \frac{1025}{30} = 107.28$ mm.

Area of reinforcement provided =24127.2mm^2.

- Calculation of transverse reinforcement.

$$\tau uv = \frac{4 * 4427200}{3.14 * 1200^2} = 0.39$$

τ_{cmax} = 3.5 N/mm^2 > 0.39 N/mm^2

For p =2.1% and M30 concrete, form IS 456 table 19, τc = 0.85 N/mm>τ_{uv}

Design Shear Force V_{ED} = 442.72 kN

From IRC 112 10.3.2

$V_{R.D.C}$=[0.12*K*(80*ρ^1*fck)$^{0.33}$+0.15σ_{cp}]*A_{net} = 606.05 kN

N_{ED} = 3136.02 kN

$K = 1 + \sqrt{\frac{200}{d}} = 1.015 < 2$

$\sigma_{cp} = \frac{N_{ED}}{A_C} = \frac{3136.02 * 10^3}{1130973.3} = 2.77 > 0.2fcd$

=0.2*13.4=2.68

$V_{min} = 0.031K^{3/2}$ f ck ^{½}

V_{min} =0.031*1.015$^{3/2}$*30$^{1/2}$ = 0.173

$V_{RDC,min}$ = (v_{min} + 0.15 σ $_{cp}$) b_wd

= (0.173 + 0.15*2.68)* 813927

= 468.66

V_{rdc} = Maximum of 606.05 and 468.66 kN

= 606.05 kN

Hence V_{rdc} > 442.72 kN so we provide minimum shear reinforcement.

Diameter of tie bar

≥ 32/4 = 8 mm

≥ 8 mm

Adopt lateral ties of diameter 8mm.

Provide 4-legged 8 mm dia lateral ties (Fe 415).

$Asv = 4 * 3.14 * \frac{8^2}{4} = 200.96\ mm^2$

Spacing of ties

i) ≤ 16 × 25 = 400 mm

ii) ≤ 300mm

iii) ≤ least lateral dimension of column = 1200mm

Adopt 4-legged 8mm dia. lateral ties @ 200 mm c/c.

7. Pier

7.1. Analysis and Design of Pier

For the design of pier, following data has been obtained from hydrological and geotechnical investigation report.

Bridge span =25 m

Size of bearing = 500x350x64 mm

Lane width = 7.5 m

c/c distance between outermost girders = 6.5 m

Size of expansion joint provided = 40 mm

Depth of girder (main) = 2.3 m

Velocity of water current = 4.947 m/s

Type of foundation = Deep foundation (Pile Footing)

RL of bottom of pier = 608 m

RL of HFL = 614.69 m

Depth of pier = 614.69 - 608 + 2 = 8.69 m

7.1.1. Material

Concrete: M30

Rebar: TMT500D

7.1.2. Type of Pier

RCC single column hammer headed pier has been selected. As the length of pier is more than 5m so wall pier system will not be economical. Moreover, for 25m span, strength is also not sufficient. Also, carriage way is only 7.5 m so single column may be sufficient.

7.1.3. Pier Cap Preliminary sizing

Length of pier cap = c/c spacing of main girder + bearing length + 2 × clearance

$$= 6.5 + 0.5 + 2 \times 0.5 = 8 \text{ m}$$

(As Clearance is taken as 0.5m (0.4-0.6m)

Minimum Width of pier cap = 2 × projection beyond pier + 2 × 2 × bearing offset + 2 × bearing width

$$= 2 \times 75 + 2 \times 2 \times 150 + 2 \times 350 = 1.45 \text{ mm}$$

Width, B = diameter of pier stem + 2 × projections

Assume diameter of pier stem = 2.50 m and projections = 75mm (50-200)

$B = 2.50 + 2 \times 0.075 = 2.65$ m

Thickness of pier cap

Adopt 1500mm at the face of pier stem and 750mm at the end.

`

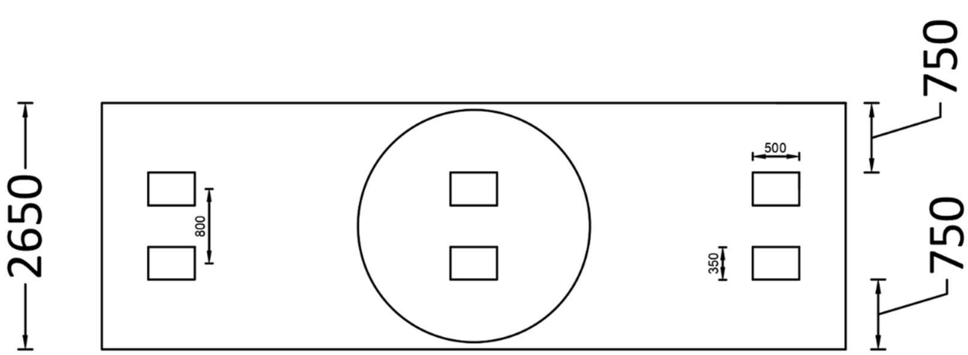


Fig: pier cap

Figure :Preliminary sizing of pier cap

7.1.4. Check for depth of pier Cap for punching shear

Depth of pier cap below bearing = 750 + (1500 – 750)/2750 × 750 = 954.55 mm

Take 40mm clear cover and 32mm dia bar, then

effective depth d= 954.55 – 40 – 32/2 = 895.55 mm

$$\text{Punching shear} = \frac{\text{Maximum vertical force on bearing}}{\text{Area}}$$

$$= \frac{1193.13 \times 1000}{2 \times (350+500+2 \times 898.55) \times 898.55} = 0.251 \text{ N/mm}^2$$

For M30 concrete, Punching shear strength $K_c\tau_c$ = 1.369 N/mm^2 > 0.68 N/mm^2

So provided thickness are sufficient.

7.1.5. Check for diameter of Pier column

Approximate axial load = (DL + LL) form superstructure + DL of pier cap

DL from superstructure = 6118.438 kN

LL from superstructure = 1338.873 kN

DL of pier cap = 1.35 × (2.5×1.5 + 0.5×2.75×2 × (1.5+0.75)) × 2.65 × 25= 888.785 kN

Therefore, design axial load P_u = 8237.119 kN

Let Ag be the sectional area required then,

$$Pu = 0.67\ fy\ Asc + 0.4\ f_{ck}\ Ac$$

Assume 1% steel reinforcement then

8237.119 × 1000 = 0.67 × 500 × 0.01 × Ag + 0.4 × 30 × 0.99 × Ag

Or, Ag = 540848.23 mm^2

For circular column, diameter D = 829.837 mm < 2500 mm

To consider eccentric loading effect, adopted 2500 mm diameter seems ok.

7.2. Analysis

Dead load from superstructure

Total load = wt. of (railing + kerb + slab+ main girder + cross girder)

= 4474.139kN (from bearing design portion)

Dead load of wearing course

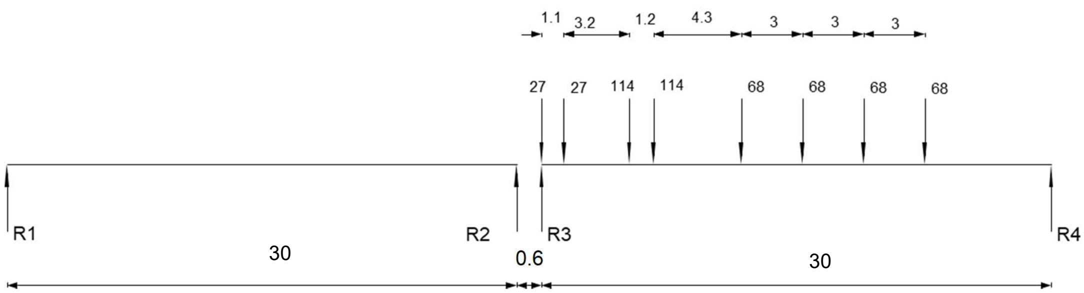
Dead load of wearing course = 495 kN

Live load from superstructure

When load is on one span only

<u>Class A vehicle</u>

When only one span is loaded

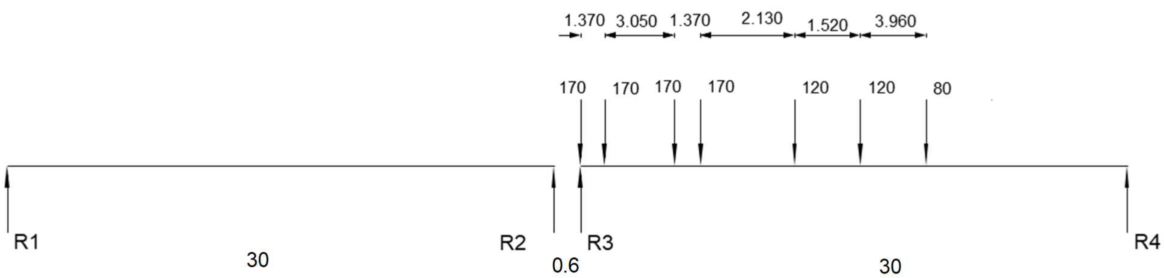


Maximum LL on pier from right side = [2 × {27 × 30 + 27 × (30-1.1) + 114 × (30 - 4.3) + 114 × (l30-5.5) + 68 × (30-9.8) + 68 × (30-12.8) + 68 × (30-15.8) + 68 × (30-18.8)}]/30 = 772.23 kN

Impact factor = 1.125

Max LL including Impact = 772.23 × 1.125 = 868.76 kN

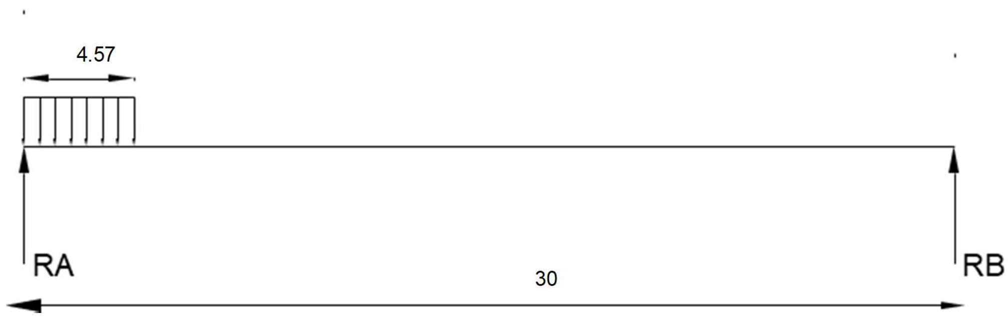
<u>Class 70R Wheel</u>



Maximum LL on pier from right side = [{170 × 30 + 170 × (30-1.37) + 170 × (30 -3.05-1.37) + 170 × (30-1.37-3.05-1.37) + 120 × (30-2.13-1.37-3.05-1.37) + 120 × (30-1.52-2.13-1.37-3.05-1.37) + 80 × (30-3.96-1.52-2.13-1.37-3.05-1.37)}]/30 = 829.21kN

Impact factor = 1.12

Max LL including Impact = 829.21 * 1.12 = 928.711 kN



Class 70R Track loading in longitudinal direction

Maximum LL on pier = 700*(30-0.5*4.57)/30 = 646.683 kN

impact factor = 1.1

Max LL including Impact = 646.683 * 1.1 = 711.352 kN

When load is both span

Total Load (due to class A) = 1038.254 kN

Total Load (due to class 70R Wheel) = 997.024kN

Load due to braking effect

(As per IRC 6 clause 211.2)

20% of first vehicle load +10% of second vehicle load in only one lane

Braking load = 0.2 × (54 + 228 + 2 × 136) + 0.1 × (54 + 228 + 2 × 136)

= 166.2 kN

This load is taken by pier only. Braking load on pier is given by:

F^{H}_{br} = 166.2 kN

$$F^{V}_{br} = \frac{166.2 \times 3.639}{30 \times 2} = 20.16 \text{ kN}$$

Wind load (from superstructure)

Calculation is as done in bearing.

F^{W}_{T} = 383.985 kN

F^{W}_{L} = 95.996 kN

F^{W}_{V} = 465.597 kN

Wind load (from substructure)

Wind load in transverse direction

$F^{T}_{W} = P_d\, A\, G\, C_D$ = 940.6 × (2.5 × 8.69 + 2.65 × 1.5) × 2 × 0.5 = 24.173 kN

Wind load in longitudinal direction (F^{L}_{W}) = 0.25 × 24.173 = 6.043 kN

Seismic load (from superstructure)

The response reduction factor for pier design (for column type as per IRC 06: 2017 table 20) is 4 instead of 1 in superstructure.

Seismic load in longitudinal direction $F^{L}{}_{S}$ = 0.15 × 4969.139 = 745.371 kN

Seismic load in Transverse direction $F^{T}{}_{S}$ = 0.15 × (4969.139+221.6) = 778.6 kN

Vertical reaction due to seismic load:

$$\text{In longitudinal direction } \mathrm{F}_{\mathrm{S}}^{\mathrm{VL}} = \frac{745.371 \times 1.4}{30} = 34.784\ kN$$

$$\text{In transverse direction } \mathrm{F}_{\mathrm{S}}^{\mathrm{VT}} = \frac{778.6 \times 1.4}{30 \times 2} = 167.7\ kN$$

Seismic load (from sub-structure)

Weight of pier cap rectangular part = (0.75+0.75) × 2.5 × 2.65 × 25 = 248.438 kN

Weight of pier cap rectangular part cantilever = 0.75 × 2.65 × 2.75 × 25 × 2 = 273.281 kN

Weight of pier cap triangular part = 0.5 × 2 × 0.75 × 2.65 × 2.75 × 25 = 136.64 kN

Self-weight of pier stem = $\pi \times \left(\frac{2.5}{2}\right)^2$ × (8.69 – 1.5) × 25 = 882.346 kN

Seismic load due to self-weight of pier cap rectangular part in longitudinal and transverse direction = 248.438 × 0.150 = 37.265 kN

Seismic load due to self-weight of pier cap rectangular part cantilever in longitudinal and transverse direction = 273.28 × 0.150 = 40.99 kN

Seismic load due to self-weight of pier cap triangular part cantilever in longitudinal and transverse direction = 136.64 × 0.150 = 20.496 kN

Seismic load due to self-weight of pier stem in longitudinal and transverse direction = 882.346 × 0.150 = 132.352 kN

Load due to temperature variation, creep and shrinkage

This load has not been considered in pier.

Self-weight of pier

Self-weight = 1540.705kN

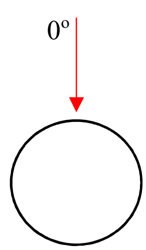
Load due to water current

$F^T{}_{WC} = P \times A$

P = $52KV^2$ where K = 0.66 for circular pier.

V = 4.947 m/s

Existing direction of flow angle with pier axis in river θ = 0° (form topo map)



Now as per clause 210.5, Design angle is $\alpha = \theta \pm 20°$

For transverse direction α_T = 20°–0° = 20°

For longitudinal direction α_L = 20°+0° = 20°

And, Area A = (614.69 - 608) × 2.5 = 16.725 m^2

$\therefore F^{T}_{wc}$ = 52 × 0.66 × (7.891 × cos20)^2 × 16.725 × 9.81/1000 = 121.695 kN

$\therefore F^{L}_{wc}$ =52 × 0.66 × (7.891 × sin20)^2 × 16.725 × 9.81/1000 = 16.12 kN

Load due to Hydrodynamic Force

$F^{T}_{hyd} = F^{L}_{hyd} = \alpha CW$

C = 0.730 (from table 9.10 Swami Saran's design of substructure)

Weight of water in enveloping cylinder W = $\pi \times \frac{2.5^2}{4}$ × 6.69 × 9.81 = 322.155 kN

For this R =4, then $\alpha = \frac{Z}{2} \times \frac{I}{R} \times \frac{S}{g}$ = 0.15

$F^{T}_{hyd} = F^{L}_{hyd}$= 0.150 × 0.730 × 322.155 = 35.276 kN

Load due to buoyancy

$F_{buoy} = V_{submerged} \times \gamma_w$

F_{buoy}= 0.25 × π × 2.5^2 × 6.69 × 9.81 = 32.84 kN

7.3. Design of Pier Cap

Pier cap has been designed as cantilever beam and detailed as per IRC-112.

Design Shear force (For basic combination)

V_{max} = (1.35DL+1.75WC+1.5LL+1.5F^{W}_{V}+1.15F^{V}_{br})$_{\text{from end girder}}$ * 1/3 +1.35DLp

V_{max} = (6238.088+1.5×1038.254+1.5×465.597+1.15×20.16) ×1/3

+1.35×2.75×2.65× (1.5+0.75)/2×25

V_{max} = 3338.46 kN

Design moment (for basic combination)

Total length from critical section = 2.892 m

Centroid of trapezoid = 1.285 m

Critical section to center of bearing = 2.142 m

M_{max} = (1.35DL+1.75WC+1.5LL+1.5F_V^W+1.15F_{br}^V)$_{\text{from end girder}}$ × 1/3 * 2.142

+1.35DL_p × (Lever arm)

M_{max} = (6238.088+1.5×1038.254+1.5×465.597+1.15×20.16) ×1/3×2.142

+1.35×2.892×2.65× (1.5+0.75)/2×25 × 1.285

M_{max} = 6933.034 kN-m

Material properties

- **o)** Concrete used: M30 (IRC 112-2020 Table 6.4)
- **p)** Characteristic strength, f_{ck}= 30 N/mm^2
- **q)** Design compressive strength of concrete, $f_{cd}=\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]
- **r)** $\alpha = 0.67$
- **s)** $\gamma m = 1.5$
- **t)** Design compressive strength of concrete, $f_{cd}= \frac{0.67 \times fck}{1.5} = \frac{0.67 * 30}{1.5} =$ 13.40 N/mm^2
- **u)** Steel used: Fe500
- **v)** Yield Strength of Steel, f_{yk}=500N/mm^2
- **w)** Design yield strength of steel, f_{yd}= f_{yk}/1.15= 0.87fy = 434.783 N/mm^2
- **x)** Young's Modulus of Elasticity, Es = 2×10^{5}N/mm^2
- **y)** Yield strain for steel (ε_{yd}) = $\frac{fyd}{Es} = \frac{0.87 * fy}{Es} = \frac{0.87 * 500}{200000}$ = 0.0218
- **z)** Area factor (β_1) = 0.810
- **aa)** CG factor(β_2) = 0.416
- **bb)** Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

Design of Section using LSM:

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu2}+\varepsilon_{yd}}$ d [SP 105]

$= \frac{0.0035 * 1444}{0.0035+0.0218}$ = 890.573 mm

Compressive force (C) = $\beta_1 \times F_{cd} \times b \times X_{lim}$

$= \frac{0.810 \times 13.40 \times 2650 \times 890.573}{1000}$

$= 25562.242$ kN

CG from steel level (z) = $d - \beta_2 \times X_{lim} = 1444 - 0.416 \times 890.573$

$= 1073.548$ mm

$M_{u,\,lim} = C \times z = 25562.242 \times \frac{1073.548}{1000} = 27442.306$ kNm

Since, $M_{u,\,lim}$ is less than factor bending moment assumed depth is satisfied.

Design of main reinforcement

Design Bending Moment, $M_{Ed} = 6933.034$ kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2\times\beta_2} - \sqrt{\left(\frac{d}{2\times\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\times\beta_2\times b\times f_{cd}}}$$

$$X_u = \frac{1444}{2\times0.416} - \sqrt{\left(\frac{1444}{2\times0.416}\right)^2 - \frac{6933.034\times10^6}{0.810\times0.416\times2650\times13.40}}$$

$$X_u = 176.218 \text{ mm}$$

Since $X_u < D_f$, NA lies in flange.

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd}\times z}$ with $z = (d - \beta_2 \times X_u) = (1444 - 0.416 \times 176.218) = 1370.698$ mm

$A_{st} = \frac{6933.034\times10^6}{434.783\times1370.698} = 11633.470 \text{ mm}^2$

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.248 mm^2

Spacing required = $\frac{b*A}{A_{st}} = \frac{2650*804.248}{11633.470} = 183.200 \text{ mm}^2$

Provide 32 mm dia bars @150 mm spacing = 14208.376mm^2

Check

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d$ but not less than $0.0013\, b_t d$

Here, For M30

$f_{ctm} = 2.5$ [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times \frac{2.5}{500} \times 2650\times1444$ but not less than $0.0013\times2650\times1444$

=4974.58 mm^2 but not less than 4974.58 mm^2

= 4974.58 mm^2

$A_{st, provided} > A_{s, min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s, max} = 0.025\ A_c$

$A_{s, max} = 0.025\times [2650\times1444] = 3975000$ mm^2

$A_{st, provided} < A_{s, max}$, OK

Design Of Shear Reinforcement

Design shear force, V_{Ed} =3338.463 KN

Maximum Allowable Shear Force (for maximum shear force take $\Theta = 45^o$)

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD, max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z)= $(d – \beta_2 \times X_u) = (1444 – 0.416 \times 176.218) = 1370.698$ mm

$v_1 = 0.6\left(1-\frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

$\theta=45^o$

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta+\tan\theta}$

$= 1\times 2650\times 1370.698\times 0.6\left(1-\frac{30}{310}\right)\times\frac{0.446 f_{ck}}{\cot 45+\tan 45}$

=13169.259 kN

And,

$V_{Rds}=V_{NS}=V_{ED}+V_{ccd}+V_{td}= V_{ED=}3338.463$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d$$

$$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right)b_w d$$

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

$= 1 + \sqrt{\frac{200}{1446}}$

$= 1.372$

$V_{min} = 0.031K^{3/2}fck^{1/2}$

$= 0.031 \times 1.372^{3/2} \times 30^{1/2}$

$= 0.273$

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.037 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

∴ $\rho 1 = 0.02$

∴$V_{Rd.c} = [0.12 \times 1.372 \times (80 \times 0.02 \times 30)^{0.33}] \times 2650 \times 1444$

$= 1296.823$ kN

And, $V_{Rd.c} = (Vmin + 0.15\sigma cp) \times b_w d$

$= (0.273 + 0.15 \times 0) \times 2650 \times 1444$

$= 1044.343$ kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\,min}$=1296.823 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=3338.463 kN

∴Since $V_{Ed} > V_{Rd.c}$, shear reinforcement design is required

Design of Shear Reinforcement **IRC 112:2020 Cl 10.3.3.1.-4**

By equating V_{NS} and, $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw} b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2\times3338.463\times1000}{1\times2650\times1370.698\times0.542\times0.446\times30}\right)}{2}$$

$= 7.276^0$

$\therefore$As per the code $21.8^0 \leq \theta \leq 45^0$

Adopt $\theta=21.8^0$

$\therefore V_{Rds}=V_{NS}=V_{ED}=\frac{Asw}{s}\times z\times fywd\times cot\theta$

`

$S=\frac{Asw}{\mathrm{VE}}\times z\times fywd\times cot\theta$

Provide 2-legged 12 mm stirrups

$\therefore$Fywd=500/1.15=434.78 N/mm^2

$\therefore S \ = \frac{2*113.09}{3338.463\times10^3}\times 1370.698\times 434.78\times cot21.8^0$

=100.95 mm

$\therefore$ Provide spacing = 90 mm

<u>*check*</u>

Shear reinforcement ratio $\rho_w=\frac{A_{sw}}{s\times b_w}=\frac{226.195}{90\times2650}$=0.0095

<u>Minimum shear reinforcement ratio:</u>

$\therefore\rho_{min}=\frac{0.072\times\sqrt{fck}}{fyk}=\frac{0.072\times\sqrt{30}}{500}$=0.00079 Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12mm 2- legged vertical stirrups at 90 mm c/c spacing

7.4.Load Analysis For Pier Stem

Different types of load combinations and their responses on the base of the pier stem are shown in the table below in following order.

1. Basic combination 1 (Live load as Leading and wind load as accompanying load).
2. Basic combination 2 (Wind load as leading load and live load as accompanying load).
3. Seismic combination 1
4. Seismic combination 2
5. Seismic combination 3

1.Basic Combination 1 (Live Load as Leading load and wind load as accompanying load)

Load(kN)		Load Factor	Eccentricity (m)		Lever Arm (m)	Design Force(kN or kN-m)				
			ex	ey		Pu	Hx	Hy	Muy	Mux
DL_{SS}	4474.1	1.35				6040.1				
DL_{WC}	495.00	1.75				866.25				
$F^{h}Br$	166.20	1.5			9.19		249.3		2291.0	
$F^{v}Br$	20.16	1.5				30.24				
$F^{L}W_{sup}$	96.00	0.9			9.19		86.40		793.99	
$F^{T}W_{sup}$	383.99	0.9			9.19			345.5		3175.9
$F^{V}W_{sup}$	465.60	0.9				419.04				
$F^{L}W_{sub}$	6.04	0.9			4.99		5.44		27.12	
$F^{T}W_{sub}$	24.17	0.9			5.06			21.75		110.00
$F^{L}WC$	16.12	1			4.46		16.12		71.90	
$F^{T}WC$	121.69	1			4.46			121.7		542.76
W_{pier}	1540.7	1.35				2079.9				
F_{buoy}	322.16	0.15				-48.32				
Total without Live Load						9387.2	357.2	489.0	3184.0	3828.7
Live Load										
LL P_u	997.02	1.5				1495.5				
LL M_L	868.76	1.5	0.4						521.26	
LL M_T	997.02	1.5		1.16						1738.8
Total With Live Load						10883	357.2	489.0	3705.33	5564

Therefore, Design Axial Force (P_u) = 10883 kN.

Design Horizontal Force (H) = $\sqrt{357.2^2 + 489^2}$ = 606 kN.

Design Bending Moment (M_u) = $\sqrt{3705.33^2 + 5564^2}$ = 6684.5 kN.

$$Eccentricity = \frac{Mu}{H} = \frac{6684.5}{606} = 11.037\ m$$

1. Basic combination 2 (Wind load as leading load and live load as accompanying load)

Load (kN)	Load factor		Eccentricity (m		Lever Arm (m)	Design Force(kN or kN-m)				
			ex	ey		Pu	Hx	Hy	Muy	Mux
DL_{SS}	4474.1	1.35				6040.1				
DL_{WC}	495	1.75				866.25				
$F^{h}Br$	166.2	1.15			9.19		191.1		1756	
$F^{v}Br$	20.16	1.15				23.184				
$F^{L}W_{sup}$	95.996	1.5			9.19		144		1323	
$F^{T}Wsup$	383.99	1.5			9.19			576		5293.2
$F^{V}W_{sup}$	465.6	1.5				698.4				
$F^{L}W_{sub}$	6.0434	1.5			4.986		9.065		45.2	
$F^{T}W_{sub}$	24.173	1.5			5.056			36.26		183.33
$F^{L}wc$	16.121	1			4.46		16.12		71.9	
$F^{T}wc$	121.69	1			4.46			121.7		542.76
W_{pier}	1540.7	1.35				2080				
F_{buoy}	322.16	0.15				-48.323				
Total Without Live Load						9659.5	360.3	733.9	3197	6019.3
Live Load										
$LL_{max}P_u$	997.02	1.15				1146.6				
LL M^{L}	868.76	1.15	0.4						399.6	
LL M^{T}	997.02	1.15		1.16						1330
Total With Live Load						10806	360.3	733.9	3597	7349.4

Therefore, Design Axial Force (P_u) = 10806 kN.

Design Horizontal Force (H) = $\sqrt{360.3^2 + 733.9^2}$ = 818 kN.

Design Bending Moment (M_u) = $\sqrt{3597^2 + 7349.4^2}$ = 8182.2 kN.

$$\text{Eccentricity} = \frac{\text{Mu}}{\text{H}} = \frac{8182.2}{818} = 10.007 \text{ m}$$

2. Seismic Combination 1 (Longitudinal Seismic force maximum).

Load (kN)		Load factor	Eccentricity		Lever Arm (m)	Design Force				
			ex	ey		Pu	Hx	Hy	Muy	Mux
DL_{SS}	4474.14	1.35				6040.09				
DL_{WC}	495.00	1.75				866.25				
F^hBr	166.20	0.2			9.19		33.24		305.5	
F^vBr	20.16	0.2				4.03201				
i.Longitudinal Seismic force on substructure.										
$F^L S_{rect1}$	37.27	1.5			7.94		55.90		443.8	
$F^L S_{rect2}$	40.99	1.5			8.315		61.49		511.3	
$F^L S_{tria}$	20.50	1.5			7.69		30.74		236.4	
$F^L S_{stem}$	132.35	1.5			3.595		198.53		713.7	
ii.Transverse seismic force on substructure.										
$F^t S_{rect1}$	11.18	1.5			7.9			16.77		133.2
$F^t S_{rect2}$	12.30	1.5			8.3			18.45		153.4
$F^t S_{tria}$	6.15	1.5			7.7			9.22		70.9
$F^t S_{stem}$	39.71	1.5			3.6			59.56		214.1
$F^L S_{sup}$	745.37	1.5			9.2		1118.1		10275	
$F^T S_{sup}$	233.58	1.5			9.2			350.4		3219.9
$F^v S_{sup}$	155.72	1.5				233.58				
$F^v S_{sup}$	46.22	1.5				69.33				
$F^L wc$	16.12	1			4.46		16.121		71.9	
$F^T wc$	121.69	1			4.46			121.7		542.76
W_{pier}	1540.71	1.35				2079.95				
F_{buoy}	322.16	1				-322.16				
Total without dead load						8971.08	1514.1	576.1	12558	4334.3
Live Load										
$LL_{max}P_u$	997.024	0.2				199.405				
LL M_L	868.76	0.2	0.4						69.5	
LL M_T	997.024	0.2		1.16						231.31
Total with Live Load						9170.49	1514.1	576.1	12627	4565.6

Therefore, Design Axial Force (P_u) = 9170.49 kN.

Design Horizontal Force (H) = $\sqrt{1514.1^2 + 576.1^2}$ = 1620 kN.

Design Bending Moment (M_u) = $\sqrt{12627^2 + 4565.6^2}$ = 13427 kN.

$$\text{Eccentricity} = \frac{Mu}{H} = \frac{13427}{1620} = 8.289 \text{ m.}$$

3. Seismic combination 2 (Transverse seismic force maximum)

Load (m)	Load factor		Eccentricity (m)		Lever Arm (m)	Design Force(kN or kN-m)				
			ex	ey		Pu	Hx	Hy	Muy	Mux
DL_{SS}	4474.1	1.35				6040.1				
DL_{WC}	495	1.75				866.25				
$F^{h}Br$	166.2	0.2			9.19		33.2		305	
$F^{v}Br$	20.16	0.2				4.032				
i.Seismic L Sub										
$F^{L}{}_{Srect1}$	11.18	1.5			7.94		16.77		133	
$F^{L}{}_{Srect2}$	12.30	1.5			8.32		18.45		153	
$F^{L}{}_{Stria}$	6.15	1.5			7.69		9.22		70.9	
$F^{L}{}_{Sstem}$	39.71	1.5			3.6		59.56		214	
ii.Seismic T sub										
$F^{t}{}_{Srect1}$	37.27	1.5			7.94			55.9		443.8
$F^{t}{}_{Srect2}$	40.99	1.5			8.32			61.5		
$F^{t}{}_{Stria}$	20.50	1.5			7.69			30.7		
$F^{t}{}_{Sstem}$	132.35	1.5			3.6			199		
$F^{L}{}_{Ssup}$	223.61	1.5			9.19		335		3082	
$F^{T}{}_{Ssup}$	778.61	1.5			9.19			1168		10733
$F^{v}{}_{Ssup}$	155.72	1.5				233.58				
$F^{v}{}_{Ssup}$	46.22	1.5				69.33				
$F^{L}wc$	16.12	1			4.46		16.1		71.9	
$F^{T}wc$	121.69	1			4.46			122		542.8
W_{pier}	1540.71	1.35				2080				
F_{buoy}	322.16	1				-322.2				
Total without dead load						8971.1	489	1636	4031	11720
Live Load										
$LL_{max}P_{u}$	997.02	0.2				199.4				
LL M_{L}	868.76	0.2	0.4						69.5	
LL M_{T}	997.02	0.2		1.16						231.3
Total with Live Load						9170.5	489	1636	4101	11951

Therefore, Design Axial Force (P_u) = 9170.5 kN.

Design Horizontal Force (H) = $\sqrt{489^2 + 1636^2}$ = 1708 kN.

Design Bending Moment (M_u) = $\sqrt{4101^2 + 11951^2}$ = 12635 kN.

$$\text{Eccentricity} = \frac{Mu}{H} = \frac{12635}{1708} = 7.398 \text{ m.}$$

4. Seismic combination 3 (Vertical Seismic force maximum)

Load (m)	Load factor		Eccentricity (m)		Lever Arm (m)	Design Force(kN or kN-m)				
			ex	ey		Pu	Hx	Hy	Muy	Mux
DL_{SS}	4474.1	1.35				6040.1				
DL_{WC}	495	1.75				866.25				
F^hBr	166.2	0.2			9.19		33.2		305	
F^vBr	20.16	0.2				4.032				
i.Seismic L Sub										
F^LS_{rect1}	11.18	1.5			7.94		16.8		133	
F^LS_{rect2}	12.298	1.5			8.32		18.4		153	
F^LS_{tria}	6.1488	1.5			7.69		9.22		70.9	
F^LS_{stem}	39.706	1.5			3.6		59.6		214	
ii.Seismic T sub										
F^tS_{rect1}	11.18	1.5			7.94			16.8		133.2
F^tS_{rect2}	12.298	1.5			8.32			18.4		153.4
F^tS_{tria}	6.1488	1.5			7.69			9.22		70.93
F^tS_{stem}	39.706	1.5			3.6			59.6		214.1
F^LS_{sup}	223.61	1.5			9.19		335		3082	
F^TS_{sup}	233.583	1.5			9.19			350		3220
F^vS_{sup}	519.07	1.5				778.61				
F^vS_{sup}	154.07	1.5				231.11				
F^Lwc	16.121	1			4.46		16.1		71.9	
F^Twc	121.69	1			4.46			122		542.8
W_{pier}	1540.7	1.35				2080				
F_{buoy}	322.16	1				-322.2				
Total without dead load						9677.9	489	576	4031	4334
Live Load										
$LL_{max}P_u$	997.02	0.2				199.4				
LL M_L	868.76	0.2	0.4						69.5	
LL M_T	997.02	0.2		1.16						231.3
Total with Live Load						9877.3	489	576	4101	4566

Therefore, Design Axial Force (P_u) = 9877.3 kN.

Design Horizontal Force (H) = $\sqrt{489^2 + 576^2}$ = 755 kN.

Design Bending Moment (M_u) = $\sqrt{4101^2 + 4566^2}$ = 6136.9 kNm.

$$\text{Eccentricity} = \frac{Mu}{H} = \frac{6136.9}{755} = 8.123 \text{ m.}$$

7.5. Design of pier stem.

Diameter of pier stem = 2.5 m.

Grade of concrete = 30 Mpa.

Height of pier Stem(L) = 8.69 m.

Provide Clear Cover of 75 mm

Therefore, effective cover (d') = 75 + 16 mm = 91 mm.

effective diameter = 2500 – 91 – 91 = 2318 mm.

From above analysis of loads at base of pier stem.

Combination	P_u (kN)	H_r (kN)	M_{ur} (kN-m)	Eccentricity (m)	$P_u/f_{ck}D^2$	$M_u/fckD^3$
1	10883	606	6684.5	11.037	0.058	0.014
2	10806	818	8182.2	10.007	0.057	0.017
3	9170.5	1620	13427	8.288	0.049	0.028
4	9170.5	1708	12635	7.398	0.049	0.027
5	9877.3	755	6136.9	8.123	0.053	0.013

Design Forces

Design axial force (Pu) = 9170.5 kN.

Design Bending Moment (Equivalent Uniaxial) (Mu) = 13427 kNm.

Design Shear Force(Hr) = 1708 kN.

Design Steps

- Slenderness Ratio

$$\text{Slenderness Ratio} = \frac{\text{KL}}{\text{D}}$$

Here bottom of pier stem is fixed and top end is free to rotate (elastometric bearing).

Therefore, K= 1.3 (IRC 112 , 2020 table 11.1)

$$\text{Slenderness Ratio } = \frac{1.3 \times 8.69}{2.5} = 4.51 < 12$$

Hence Pier stem can be designed as a short column.

- Calculation of minimum eccentricity

From Is 456 Clause 24.4.

$$e = \frac{l}{300} + \frac{D}{30} = \frac{86900}{500} + \frac{2500}{30} = 100.71 \text{ mm} > 20 \text{ mm}$$

, so eccentricity produced due to the moment needs to be consider.

Also, $0.05D = 0.05 \times 2500 = 125 \text{ mm} > \text{ e}$.

- Calculation of longitudinal reinforcement.

$$\frac{d'}{D} = \frac{91}{2500} = 0.0364$$

$\cong 0.05$

Refer Chart 59 of Sp 16 (IS 456).

$$\frac{P_u}{f_{ck}D^2} = \frac{9170500}{30 \times 2500^2} = 0.049$$

$$\frac{M_u}{f_{ck} \times D^3} = \frac{13427 \times 10^6}{30 \times 2500^3} = 0.028$$

From above value and using chart 59 of SP 16 (IS 456)

$$\frac{p}{fck} = 0.02$$

Therefore Percentage of reinforcement required,

%p = 0.0240 = 0.6% < 0.8% (minimum)

So provide %p = 0.8%

$$\text{Area of reinforcement required (Asc)} = \frac{0.8}{100} * 3.14 * \frac{2500}{4} = 39269.9$$

$$\text{No of 32 mm dia bar required} = \frac{39269.9}{804.24} = 48.82$$

Hence provide 60 numbers of bars.

$$\text{Spacing of bars} = 3.14 * \frac{2318}{60} = 121.30 \text{ mm.}$$

Area of reinforcement provided = 48254.86 mm.

- Calculation of transverse reinforcement.

$$\tau uv = \frac{4 * 1708}{3.14 * 2500^2} = 0.348$$

τ_{cmax} = 3.5 N/mm^2 > 0.348 N/mm^2

For p =0.8% and M30 concrete, form IS 456 table 19, τc = 0.59 N/mm^2 > τ_{uv}

Design Shear Force V_{ED} = 1708 kN

From IRC 112 10.3.2

$V_{R.D.C}$=[0.12*K*(80*ρ^1*fck)$^{0.33}$+0.15σ_{cp}]*A_{net} = 2850.479 kN

N_{ED} = 10882.78 kN

$$K = 1 + \sqrt{\frac{200}{d}} = 1.006 < 2$$

$$\sigma_{cp} = \frac{N_{ED}}{A_C} = \frac{10882.78}{4908739} = 2.21 < 0.2fcd$$

V_{min} = 0.031$K^{3/2}$ f ck ^{½}

V_{min} =0.031*$1.006^{3/2}$*$30^{1/2}$ = 0.171

$V_{RDC,min}$ = (v_{min} + 0.15 σ $_{cp}$) b_wd

= (0.171 + 0.15*2.21)* 4220042

= 2127.338 kN

V_{rdc} = Maximum of 2850.479 and 2127.338 kN

= 2850.479 kN

Hence V_{rdc} > 1708 kN so we provide minimum shear reinforcement.

Diameter of tie bar

≥ 32/4 = 8mm

≥ 6mm

Adopt lateral ties of diameter 10mm.

Provide 4-legged 10mm dia lateral ties (Fe 415).

$$Asv = 4 * 3.14 * \frac{10^2}{4} = 314.16 \text{ mm}^2$$

Spacing of ties

i) $\leq 16 \times 32 = 512$ mm

ii) $\leq$ 300mm

iii) $\leq$ least lateral dimension of column = 2500mm

Adopt 4-legged 10mm dia. lateral ties @ 150 mm c/c.

7.6. Design of Pile Foundation

Pile Design

Load at the base of the Pier Stem

	Vloads	Hz	My	Hy	Mz
DL	4474	0	0	0	0
Surface	495	0	0	0	0
Both span live load	997.024	0	0	0	1157
one span loaded	928.711	0	348	0	0
self load	1541	0	0	0	0
braking	20	166	1527	0	0
	0	0	0	0	0
Temp	0	0	0	0	0
Water current	0	16	72	0	0
	0	0	0	122	543
seismic	202	976	8120	0	0
	0	0	0	303	2528

Load combination for Pile Desing (IRC 78 Wokring Stress based)

Both Span Loaded

	C1	C2i (L)	C2i(T)	C3(L)	C3 (T)
DL	1	1	1	1	1
Surface	1	1	1	1	1
Both span live load	1	1	1	0	0
	0	0	0	0	0
self load	1	1	1	1	1
braking	1	1	1	0	0
	0	0	0	0	0
Temp	1	1	1	1	1
Water current	1	1	0	1	0
	1	0	1	0	1
seismic	0	1	0	1	0

0	0	1	0	1

Design Loads Case 1

	Vloads	***Hz***	***My***	***Hy***	***Mz***
DL	4474.1	0.0	0.0	0.0	0.0
Surface	495.0	0.0	0.0	0.0	0.0
Both span live load	997.0	0.0	0.0	0.0	1156.5
	0.0	0.0	0.0	0.0	0.0
self load	1540.7	0.0	0.0	0.0	0.0
braking	20.2	166.2	1527.4	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Temp	0.0	0.0	0.0	0.0	0.0
Water current	0.0	16.1	71.9	0.0	0.0
	0.0	0.0	0.0	121.7	542.8
seismic	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
	7527.0	**182.3**	**1599.3**	**121.7**	**1699.3**

Case 2	Vloads	Hz	My	Hy	Mz
DL	4474.1	0.0	0.0	0.0	0.0
Surface	495.0	0.0	0.0	0.0	0.0
Both span live load	997.0	0.0	0.0	0.0	1156.5
	0.0	0.0	0.0	0.0	0.0
self load	1540.7	0.0	0.0	0.0	0.0
braking	20.2	166.2	1527.4	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Temp	0.0	0.0	0.0	0.0	0.0
Water current	0.0	16.1	71.9	0.0	0.0
	0.0	0.0	0.0	121.7	542.8
seismic	201.9	976.5	8120.1	0.0	0.0
	0.0	0.0	0.0	302.9	2527.7
	7729.0	**1158.8**	**9719.4**	**424.6**	**4227.0**

Case 3	***Vloads***	***Hz***	***My***	***Hy***	***Mz***
DL	4474.1	0.0	0.0	0.0	0.0
Surface	495.0	0.0	0.0	0.0	0.0
Both span live load	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0

self load	1540.7	0.0	0.0	0.0	0.0
braking	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Temp	0.0	0.0	0.0	0.0	0.0
Water current	0.0	16.1	71.9	0.0	0.0
	0.0	0.0	0.0	121.7	542.8
seismic	201.9	976.5	8120.1	0.0	0.0
	0.0	0.0	0.0	302.9	2527.7
	6711.8	**992.6**	**8192.0**	**424.6**	**3070.4**

Summary	***P***	***Hz***	***My***	***Hy***	***Mz***
Case 1	7527.0	182.3	1599.3	121.7	1699.3
Case 2	7729.0	1158.8	9719.4	424.6	4227.0
Case 3	6711.8	992.6	8192.0	424.6	3070.4

7.6.1. Pile Geometry

Thickness of pile cap = 1.8 m

Length of pile cap = 8.8 m

Width of pile cap = 8.8 m

Edge projection = 0.2m

Number of piles = 9

Purposed length of pile = 23 m

Diameter of pile = 1.2 m

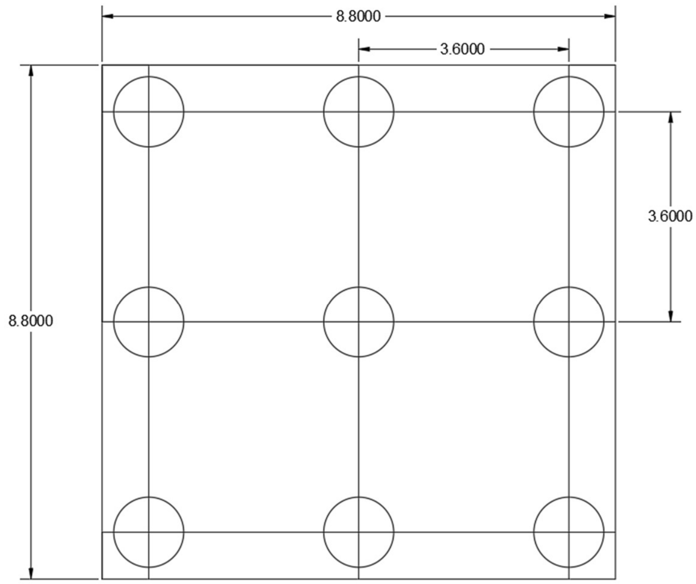


Fig: arrangement of piles

piles N	y	z	y^2	z^2
1	-3.6	3.6	12.96	12.96
2	0	3.6	0	12.96
3	3.6	3.6	12.96	12.96
4	-3.6	0	12.96	0
5	0	0	0	0
6	3.6	0	12.96	0
7	-3.6	-3.6	12.96	12.96
8	0	-3.6	0	12.96
9	3.6	-3.6	12.96	12.96
		Sum	77.76	77.76

Additional load on pile cap = Length * width * thickness * unit weight of concrete

= 8.8 * 9.6 * 1.8 * 25

= 3484.8 kN

Additional load on piles = Area * length * unit weight of concrete * No of piles

$$= \pi * \frac{1.2^2}{4} * 23 * 25 * 9$$

= 5852.78 kN

7.6.2. Load Distribution on each pile

$$\textbf{Total Axial Load} = \frac{\textbf{Total vertical load} + \textbf{Load from pile cap} + \textbf{Load from pile}}{\textbf{total no. of piles}} + \mathbf{M_y} * \frac{z}{\sum z^2}$$

$$\textbf{Horizontal load} = \frac{\textbf{Horizontal load from pile}}{\textbf{total no. of piles}}$$

Load distribution on piles

Case 1

	Axial	*Hz*	*Due My*	*Hy*	*Due MZ*	*Total*
1	1873.8	20.3	74.0	13.5	-78.7	1869.2
2	1873.8	20.3	74.0	13.5	0.0	1947.9
3	1873.8	20.3	74.0	13.5	78.7	2026.6
4	1873.8	20.3	0.0	13.5	-78.7	1795.2
5	1873.8	20.3	0.0	13.5	0.0	1873.8
6	1873.8	20.3	0.0	13.5	78.7	1952.5
7	1873.8	20.3	-74.0	13.5	-78.7	1721.1
8	1873.8	20.3	-74.0	13.5	0.0	1799.8
9	1873.8	20.3	-74.0	13.5	78.7	1878.5

Case 2	*Axial*	*Hz*	*Due My*	*Hy*	*Due MZ*	*Total*
1	1896.3	128.8	450.0	47.2	-195.7	2150.6
2	1896.3	128.8	450.0	47.2	0.0	2346.3
3	1896.3	128.8	450.0	47.2	195.7	2541.9
4	1896.3	128.8	0.0	47.2	-195.7	1700.6
5	1896.3	128.8	0.0	47.2	0.0	1896.3
6	1896.3	128.8	0.0	47.2	195.7	2092.0
7	1896.3	128.8	-450.0	47.2	-195.7	1250.6
8	1896.3	128.8	-450.0	47.2	0.0	1446.3
9	1896.3	128.8	-450.0	47.2	195.7	1642.0

Case 3	*Axial*	*Hz*	*Due My*	*Hy*	*Due MZ*	*Total*
1	1783.3	110.3	379.3	47.2	-195.7	1966.8
2	1783.3	110.3	379.3	47.2	0.0	2162.5
3	1783.3	110.3	379.3	47.2	195.7	2358.2
4	1783.3	110.3	0.0	47.2	-195.7	1587.6
5	1783.3	110.3	0.0	47.2	0.0	1783.3
6	1783.3	110.3	0.0	47.2	195.7	1979.0
7	1783.3	110.3	-379.3	47.2	-195.7	1208.3
8	1783.3	110.3	-379.3	47.2	0.0	1404.0
9	1783.3	110.3	-379.3	47.2	195.7	1599.7

Summary of the extremely loaded pile

	Axial	**Vector sum of H**
Case 1	2026.6	24.36
Case 2	2541.9	137.13
Case 3	2358.2	119.96

7.6.3. Pile Design

Purposed pile length (L) = 23 m

Pile Diameter (D) = 1.2 m

<u>Sub soil parameter for modeling</u>

Modulus of subgrade reaction (η_h) for very loose sand = 5000 kN/m^3

$$\text{Moment of inertia of pile x} - \text{section} = \frac{1}{4} * \pi * \left(\frac{D}{2}\right)^4$$

=0.102 m^4

Young's modulus of pile material (E) = 5000 * $(f_{ck})^{1/2}$*1000

=27386128 N/mm^2

$$\text{Stiffness Factor(T)} \ = \left(\frac{EI}{\eta_h}\right)^{\frac{1}{5}} = 3.478\ m$$

2T = 2 * 3.54 = 6.96 m

4T = 4 * 3.54 = 13.91 m

Scour depth (e) = 15.39 m

$$\frac{Scour\ depth\ (e)}{stiffness\ Factor(T)} = \frac{15.39}{3.478} = \ 4.42\ m$$

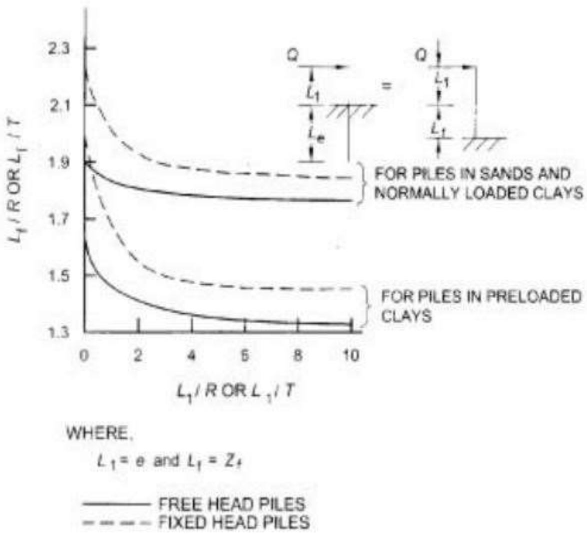


FIG. 4 DEPTH OF FIXITY

From graph,

$$\frac{\text{Depth of point of fixity }(\mathrm{Z_f})}{\text{stiffness factor(T)}} = 1.87$$

Z_f = 3.54 * 1.89 = 6.50 m

For Fixed Head moment

$$\text{Deflection(y)} = \frac{\mathrm{H(e + z_f)^3}}{\mathrm{12EI}} * 10^3$$

$$\text{Fixed End Moment(M)} = \frac{\mathrm{H(e + z_f)}}{2}$$

P(kN)	H(kN)	Deflection(m)	Moment(kNm)
2026.56	24.36	0.0084	533.2
2541.95	137.13	0.0471	3002.2
2358.22	119.96	0.0412	2626.3

Calculation of Pile Capacity

Pile diameter (D) = 1.2 m

Pile length (L) = 23 m

unit weight of soil (γ) = 20kN/m^3

active earth pressure coefficient (ka) = 0.334

angle of friction between pile and soil (δ) = 29.94°

Frictional coefficient =tan $(\delta * \frac{\pi}{180})$ = 0.576

Cross sectional area of pile tip (A_p) = $\pi * \frac{D^2}{4}$ = 1.13 m^2

Surface area of pile shaft (A_s) = $\pi * D * L$ = 86.71 m^2

Assuming skin friction is applied before end bearing

Level	Overburden pressure	Force	Friction
0.0	0.00		
0.5	0.00	0.0	0.0
1.0	0.00	0.0	0.0
1.5	0.00	0.0	0.0
2.0	0.00	0.0	0.0
2.5	0.00	0.0	0.0
3.0	0.00	0.0	0.0
3.5	0.00	0.0	0.0
4.0	0.00	0.0	0.0
4.5	0.00	0.0	0.0
5.0	0.00	0.0	0.0
5.5	0.00	0.0	0.0
6.0	0.00	0.0	0.0
6.5	0.00	0.0	0.0
7.0	0.00	0.0	0.0
7.5	0.00	0.0	0.0
8.0	0.00	0.0	0.0
8.5	0.00	0.0	0.0
9.0	0.00	0.0	0.0
9.5	0.00	0.0	0.0
10.0	0.00	0.0	0.0

10.5	0.00	0.0	0.0
11.0	0.00	0.0	0.0
11.5	0.00	0.0	0.0
12.0	0.00	0.0	0.0
12.5	0.00	0.0	0.0
13.0	0.00	0.0	0.0
13.5	0.00	0.0	0.0
14.0	0.00	0.0	0.0
14.5	0.00	0.0	0.0
15.0	0.00	0.0	0.0
15.5	310.00	292.2	168.3
16.0	320.00	593.8	342.0
16.5	330.00	612.6	352.8
17.0	340.00	631.5	363.7
17.5	350.00	650.3	374.5
18.0	360.00	669.2	385.4
~~18.5~~	~~370.00~~	~~688.0~~	396.3
19.0	380.00	706.9	407.1
19.5	390.00	725.7	418.0
20.0	400.00	744.6	428.8
20.5	410.00	763.4	439.7
21.0	420.00	782.3	450.5
21.5	430.00	801.1	461.4
22.0	440.00	820.0	472.3
22.5	450.00	838.8	483.1
23.0	460.00	857.7	494.0
		Total Skin Friction =	***4076.6***

In the above table, following formulas are used :

Overburden pressure = Ka*γ*level

Force = Average of Overburden pressure * π * Diameter of pile * difference of levels

Skin friction = frictional coefficient * force

Total skin friction = 4076.6 kN

Note: do not take skin friction upto scour depth

End bearing

Coefficient of earth pressure (K_i)= 1

angle of friction between pile and soil (δ) = 29.94°

Frictional coefficient =tan $(\delta * \frac{\pi}{180})$ = 0.576

Effective overburden pressure at pile tip (P_d) = (L-e)*K_i* γ = 152.2kN/m^2

Note: end bearing must be taken upto 15 times diameter of pile for greater depth

Angle of internal friction, ϕ at pile tip (N_q) = 20

Bearing capacity factor (N_γ) = 20

Cross sectional area of pile tip (A_p) = $\pi * \frac{D^2}{4}$ = 1.13 m^2

Pile diameter (D) = 1.2 m

Total end bearing = $A_p * (0.5 * D * \gamma * N_\gamma + P_d * N_q)$

= 3714.12 kN

FoS = 2.5

$$\text{Ultimate Load Capacity (Qu)} = \frac{Total\ end\ bearing + Total\ skin\ friction}{FoS}$$

$$= \frac{4076.6 + 3714.12}{2.5}$$

= 3116.3 kN > extreme axial force (2541.9 kN)

(OK)

Design of reinforcement

Load at the base of the Pier Stem

	Vloads	*Hz*	*My*	*Hy*	*Mz*
DL	4474	0	0	0	0
Surface	495	0	0	0	0
Live load	997	0	0	0	1157

	929	0	348	0	0
Self-Load	1541	0	0	0	0
Braking	20	166	1527	0	0
	0	0	0	0	0
Temp.	0	0	0	0	0
W Current	0	16	72	0	0
	0	0	0	122	543
Seismic	202	976	8120	0	0
	0	0	0	303	2528

Factored Loads

	1.00	***2.00***	***3.00***
Both Loaded	Basic	Seismic L	Seismic T
DL	1.35	1.35	1.35
Surface	1.75	1.75	1.75
Live load	1.50	0.00	0.00
	0.00	0.00	0.00
Self-Load	1.35	1.35	1.35
Braking	1.35	1.35	0.00
	0.00	0.00	0.00
Temp.	0.90	0.90	0.50
W Current	1.00	1.00	1.00
	1.00	1.00	1.00
Seismic	0.00	1.50	0.00
	0.00	0.00	1.50

Case 1	V loads	Hz	My	Hy	Mz
DL	6040	0	0	0	0
Surface	866	0	0	0	0
Live load	1496	0	0	0	1735

	0	0	0	0	0
Self-Load	2080	0	0	0	0
Braking	27	224	2062	0	0
	0	0	0	0	0
Temp.	0	0	0	0	0
W Current	0	16	72	0	0
	0	0	0	122	543
Seismic	0	0	0	0	0
	0	0	0	0	0
Total	***10509***	***240***	***2134***	***122***	***2278***

Case 2

DL	6040.1	0.0	0.0	0.0	0.0
Surface	866.3	0.0	0.0	0.0	0.0
Live load	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Self Load	2080.0	0.0	0.0	0.0	0.0
Braking	27.2	224.4	2062.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Temp.	0.0	0.0	0.0	0.0	0.0
W Current	0.0	16.1	71.9	0.0	0.0
	0.0	0.0	0.0	121.7	542.8
Seismic	302.9	1464.7	12180.2	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
	9316	***1705***	***14314***	***122***	***543***

Case 3

DL	6040.1	0.0	0.0	0.0	0.0
Surface	866.3	0.0	0.0	0.0	0.0

Live load	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Self Load	2080.0	0.0	0.0	0.0	0.0
Braking	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0
Temp.	0.0	0.0	0.0	0.0	0.0
W Current	0.0	16.1	71.9	0.0	0.0
	0.0	0.0	0.0	121.7	542.8
Seismic	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	454.4	3791.5
	8986	***16***	***72***	***576***	***4334***

Summary	Vloads	Hz	My	Hy	Mz
1	10509	240	2134	122	2278
2	9316	1705	14314	122	543
3	8986	16	72	576	4334

Thickness of pile cap = 1.8 m

Length of pile cap = 8.8 m

Width of pile cap = 8.8 m

Edge projection = 0.2m

Number of piles = 9

Purposed length of pile = 23 m

Diameter of pile = 1.2 m

Additional load on pile cap = Length * width * thickness * unit weight of concrete

= 8.8 * 9.6 * 1.8 * 25

= 3484.8 kN

Additional load on piles = Area * length * unit weight of concrete * No of piles

$= \pi * \frac{1.2^2}{4} * 23 * 25 * 9$

= 5852.78 kN

Factored total load = 12605.7426 kN

Effective square length of pier = $\sqrt{\frac{\pi * 2.5^2}{4}}$ =2.216 m

Width of the Slab (at edge over heavily loaded piles)= $\frac{8.8}{2} - \frac{2.216}{2} = 3.292$ m

Factored bending moment at the edge

Vertical force	Lever arm	Moment	
2773	2.492	6909.8	kNm
2674	2.492	6663.6	kNm
2575	2.492	6417.4	kNm
	Total	**19990.7**	kNm

Material properties

- Concrete used: M30 (IRC 112-2020 Table 6.4)
- Characteristic strength, f_{ck}= 30 N/mm^2
- Design compressive strength of concrete, $f_{cd}=\frac{\alpha \times fck}{\gamma m}$ [IRC:112-2020 clause 6.4.2.8]
- $\alpha = 0.67$
- $\gamma m = 1.5$
- Design compressive strength of concrete, $f_{cd}= \frac{0.67 \times fck}{1.5} = \frac{0.67*30}{1.5} = 13.40$ N/mm^2
- Steel used: Fe500
- Yield Strength of Steel, f_{yk}=500N/mm^2
- Design yield strength of steel, f_{yd}= f_{yk}/1.15= 0.87fy = 434.783 N/mm^2
- Young's Modulus of Elasticity, Es = 2×10^5N/mm^2
- Yield strain for steel (ε_{yd}) = $\frac{\text{fyd}}{\text{Es}} = \frac{0.87*\text{fy}}{\text{Es}} = \frac{0.87*500}{200000}$ = 0.0218
- Area factor (β_1) = 0.810
- CG factor(β_2) = 0.416
- Limiting strain on extreme compressed fiber of concrete(ε_{cu2}) = 0.0035

Calculation of Limiting Moment

$X_{lim} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{yd}}$ d [SP 105]

$= \frac{0.0035*1700}{0.0035+0.0218}$ = 1048.7 mm

Compressive force (C) = $\beta_1 \times F_{cd} \times b \times X_{lim}$

$$= \frac{0.810 \times 13.40 \times 8800 \times 1048.7}{1000}$$

= 100103.5 kN

CG from steel level (z) = d- $\beta_2 \times X_{lim}$ = 1700– 0.416 × 1048.7

= 1263.8 mm

$M_{u,\ lim} = C \times z = 37450.3 \times \frac{1263.8}{1000} = 105424.8$ kNm

Since, $M_{u,\ lim}$ is less than factor bending moment assumed depth is satisfied.

Design of main reinforcement

Design Bending Moment, M_{Ed} = 19990 kNm

Using IRC: SP: 105-2015 Clause 6.2 (B)

To find the actual neutral axis depth corresponding to M_{Ed}

$$M_{Ed} = \beta_1 \times f_{cd} \times b_{eff} \times X_u \times (d - \beta_2 \times X_u)$$

This, by solving becomes,

$$X_u = \frac{d}{2 \times \beta_2} - \sqrt{\left(\frac{d}{2 \times \beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1 \times \beta_2 \times b \times f_{cd}}}$$

$$X_u = \frac{1700}{2 \times 0.416} - \sqrt{\left(\frac{1700}{2 \times 0.416}\right)^2 - \frac{19990 \times 10^6}{0.810 \times 0.416 \times 8800 \times 13.40}}$$

X_u = 153.6 mm

Area of tension reinforcement

$A_{st} = \frac{M_{Ed}}{f_{yd} \times z}$ with z = (d – $\beta_2 \times X_u$) = (1700 – 0.416 × 153.6) = 1636.1024 mm

$A_{st} = \frac{19990 \times 10^6}{434.783 \times 1636.1024} = 28101.5\ mm^2$

Provide 32 mm diameter bar.

Cross sectional area of each bar (A) = 804.25 mm^2

Spacing required = $\frac{b*A}{A_{st}} = \frac{8800*804.25}{28101.5} = 251.84\ mm^2$

Provide 32mm dia bars @200 mm spacing = 35386.9 mm^2

<u>Check</u>

As per clause –16.5.1.1 (1) of IRC: 112: 2020, the minimum area of tension reinforcement shall not be less than that given by following:

$A_{s,\ min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d$ but not less than $0.0013\ b_t d$

Here, For M30

f_{ctm} = 2.5 [Table 6.5 of IRC: 112: 2020]

$A_{s,\,min} = 0.26\times \frac{2.5}{500} \times 3292\times1700$ but not less than $0.0013\times3292\times1700$

=7275 mm^2 but not less than 7275mm^2

= 7275mm^2

$A_{st,\,provided} > A_{s,\,min}$, OK

As per clause –16.5.1.1 (2) of IRC: 112: 2020, the maximum area of tension reinforcement shall not exceed:

$A_{s,\,max} = 0.025\ A_c$

$A_{s,\,max} = 0.025\times [1700\times3292] = 139910$ mm^2

$A_{st,\,provided} < A_{s,\,max}$, OK

Horizontal reinforcement = 0.001 A_c = 0.001*1800*3292 = 5925.989 mm^2

Using 16mm bars with area 201 mm^2

Spacing = 223.40 mm

Adopt 16 mm bars @ 200 c/c.

Design of top bars

Top bars (as per IRC 78 clause 707.2.8)

Minimum reinforcement in any face >250mm^2 per m

Provide 12 mm dia bar @ 200mm c/c

Area provided Ast = 1000/200 × π × 6^2 = 565.48 mm^2 > 250 mm^2 (ok)

- **Check For Punching Shear**

  Ast provided = 4021.2 mm^2

  Therefore, ρ_l= 4021.2/(1700*1000*1000)

  = 0.002365 %

  F_{ck} = 30 Mpa

  Effective depth of pile cap(d) = 1.7 m.

  Diameter of pile(D) = 1.2 m.

  Edge projection = 0.2 m.

  Radius of control perimeter $= \frac{D}{2} + 2d = 4\ m.$

  <u>For Internal Pile</u>

  Perimeter (u_i) = 2*3.14*4 = 25.13 m

Eccentricity(e) = 1357.93/3123.6 = 0.435 m.

$\beta = 1 + 0.6\pi\left(\frac{e}{D+4D}\right) = 1.102.$

$V_{ED} = \beta * \frac{3123.6}{u_i} * \frac{d}{1000} = 0.233$ N/mm^2

$K = 1 + \left(\frac{200}{d}\right)^{0.5} = 1.34 < 2$

$v_{min} = 0.031 * K^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}} = 0.117\ N/mm^2$

$V_{rdc} = 0.286$ N/mm^2

Hence $V_{ED} < V_{rdc}$ Hence ok

<u>For Corner Pile</u>

Angle for corner pile = 1.9735 radians.

U_1 = 7.894 m.

U_2 = 6.283 m.

$\beta = 1 + k(\frac{M_{ED}}{V_{ED}} * \frac{u_1}{W_1}) = 1.065$

$V_{ED} = 0.264$

K=1.34

$V_{min} = 0.117$ N/mm^2

$V_{rdc} = 0.286$ N/mm$^2 > V_{ED}$ (ok)

DESIGN OF SHEAR REINFORCEMENT

Design shear force, V_{Ed} =8021 KN

<u>Maximum Allowable Shear Force (for maximum shear force take ϴ = 45º)</u>

$V_{Rd.max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$ [IRC:112-2011, cl. no. 10.3.3.2, Eq10.8]

Here,

$V_{RD,\ max}$ =The design value of maximum shear force

a_{cw}=1 for $\sigma_{cp=}0$ (RCC)

Lever Arm(z)=0.9d

=0.9×1.7= 1.53

$v_1 = 0.6\left(1 - \frac{f_{ck}}{310}\right)$ is the strength reduction factor

$f_{cd} = 0.446 f_{ck}$

θ=45º

Now,

$\therefore V_{Rd,max} = a_{cw} b_w z v_1 \frac{f_{cd}}{\cot\theta + \tan\theta}$

$= 1 \times 8800 \times 1530 \times 0.6\left(1 - \frac{30}{310}\right) \times \frac{0.446 f_{ck}}{\cot 45 + \tan 45}$

=48814.383 kN

And,

$V_{Rds} = V_{NS} = V_{ED} + V_{ccd} + V_{td} = V_{ED} = 8021$ kN

Here,

For uniform cross section: *Vccd=Vtd=0*

V_{Rds} =The design value of the shear force

V_{NS} =Net Design Shear Force = Algebraic sum of VED, Vccd and Vtd

V_{ccd} =Design value of the shear component of the force in the compression area, in the case of an inclined compression chord

V_{td} =Design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord

∴Since, $V_{Rds} < V_{Rd,max}$, the section is **safe**

Allowable shear force without shear reinforcement: [*IRC 112-2020 clause 10.3.2*]

The design shear resistance of the member without shear reinforcement $V_{Rd.c}$ is given by:

$V_{Rd.c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d$

$V_{Rd.c\,min} = \left(v_{min} + 0.15\sigma_{cp}\right) b_w d$

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

$= 1 + \sqrt{\frac{200}{1700}}$

=1.34

$V_{min} = 0.031K^{3/2} fck^{1/2}$

$= 0.031 \times 1.34^{3/2} \times 30^{1/2}$

=0.263

$\sigma_{cp} = 0$

$\rho 1 = \frac{A_{st}}{b_w.d} = 0.0024 \leq 0.02$- Reinforcement ratio for longitudinal reinforcement

$\therefore \rho 1 = 0.0024$

$\therefore V_{Rd.c} = [0.12 \times 1.34 \times (80 \times 0.0024 \times 30)^{0.33}] \times 8800 \times 1700$

=4276 kN

And, $V_{Rd.c}$ =(Vmin+0.15σcp) ×bwd

= (0.263+0.15× 0) × 8800× 1700

=3953.362 kN

Maximum of $V_{Rd.c}$ & $V_{Rd.c,\ min}$=4276 kN

V_{Ed} =The design shear force at a cross-section resulting from external loading

=8021

∴Since V_{Ed}> $V_{Rd.c}$, shear reinforcement design is required

CALCULATION OF SHEAR REINFORCEMENT **IRC 112:2020 Cl 10.3.3.1.-4**

By equating V_{NS} and , $V_{Rd,max}$ we get

$$\therefore \theta = \frac{\sin^{-1}\left(\frac{2V_{Ed}}{a_{cw}b_W Z V_1 f_{cd}}\right)}{2}$$

$$= \frac{\sin^{-1}\left(\frac{2\times 8021\times 1000}{1\times 880\times 1530\times 0.542\times 0.446\times 30}\right)}{2}$$

= 4.686°

∴As per the code 21.8°≤θ≤45°

Adopt θ=21.8°

∴V_{Rds}=V_{NS}=V_{ED}= $\frac{Asw}{s}\times z\times fywd\times cot\theta$

`

S=$\frac{Asw}{\text{VE}}\times z\times fywd\times cot\theta$

Provide 26 legged 12 mm stirrups

∴Fywd=500/1.15=434.78 N/mm^2

∴S = $\frac{26*113.09}{8021\times 10^3}\times 1530\times 434.78\times cot21.8^0$

=609.72mm

∴ Provide spacing = 350mm

<u>*check*</u>

Shear reinforcement ratio ρ_w=$\frac{A_{sw}}{s\times b_w}=\frac{2940.531}{350\times 8800}$=0.00095

<u>Minimum shear reinforcement ratio:</u>

$\therefore \rho_{min} = \frac{0.072\times\sqrt{fck}}{fyk} = \frac{0.072\times\sqrt{30}}{500} = 0.00079$ Since, $\rho_w > \rho_{min}$, (ok)

Hence provide 12 mm 26- legged vertical stirrups at 350mm c/c spacing

Design of pile stem.

Diameter of pile stem = 1.2 m.

Grade of concrete = 30 Mpa.

Height of pile Stem(L) = 23 m.

Provide Clear Cover of 75 mm

Therefore, effective cover (d') = 75 + 12.5 mm = 87.5 mm.

effective diameter = 1200 – 87.5 – 87.5 = 1025 mm.

From above analysis of loads at base of pile.

Design Forces

Design axial force (Pu) = 3123.61 kN.

Design Bending Moment (Equivalent Uniaxial) (Mu) = 2079.37 kNm.

Design Shear Force(Hr) = 189.94 kN.

Zf=6.5 m

Eccentriity = Deflected length = Zf+e =21.89 m.

Design Steps

- Calculation of longitudinal reinforcement.

  $$\frac{d'}{D} = \frac{87.5}{1200} = 0.072$$

  $\cong 0.1$

  Refer Chart 60 of Sp 16 (IS 456).

  $$\frac{P_u}{f_{ck}D^2} = \frac{3123610}{30 \times 1200^2} = 0.072$$

  $$\frac{M_u}{f_{ck} \times D^3} = \frac{2079370 \times 10^6}{30 \times 1200^3} = 0.040$$

From above value and using chart 60 of SP 16 (IS 456)

$$\frac{p}{fck} = 0.03$$

Therefore Percentage of reinforcement required,

%p = 0.03*39 = 0.9% < 0.8% (minimum)

So provide %p = 0.9%

$$\text{Area of reinforcement required(Asc)} = \frac{0.9}{100} * 3.14 * \frac{1200}{4} = 10178.62$$

Area of 25 mm diameter bars = 490.87

$$\text{No of 25 mm dia bar required} = \frac{10178.62}{490.62} = 20.736$$

Hence provide 22 numbers of 25 diameter bars.

$$\text{Spacing of bars} = 3.14 * \frac{1025}{22} = 145.37 \text{ mm.}$$

Area of reinforcement provided =9017.14 mm.

- Calculation of transverse reinforcement.

$$\tau uv = \frac{4 * 189940}{3.14 * 1200^2} = 0.16$$

τ_{cmax} = 3.5 N/mm^2 > 0.16 N/mm^2

For p =0.9% and M30 concrete, form IS 456 table 19, τc = 0.59 N/mm>τ_{uv}

Design Shear Force V_{ED} = 189.94 kN

From IRC 112 10.3.2

$V_{R.D.C}$=[0.12*K*(80*ρ^1*fck)$^{0.33}$+0.15σ_{cp}]*A_{net} = 606.05 kN

N_{ED} = 3123.61 kN

$$K= 1 + \sqrt{\frac{200}{d}} = 1.015 < 2$$

$$\sigma_{cp} = \frac{N_{ED}}{A_C} = \frac{3123.69 * 10\text{^}3}{1130973} = 2.76 > 0.2fcd$$

=0.2*13.4=2.68

V_{min} = 0.031$K^{3/2}$ f ck ^{½}

V_{min} =0.031*1.015$^{3/2}$*30$^{1/2}$ = 0.173

$V_{RDC,min}$ = (v_{min} + 0.15 σ_{cp}) b_wd

= (0.173 + 0.15*2.68)* 813927

= 468.66

V_{rdc} = Maximum of 606.05 and 468.66 kN

= 606.05 kN

Hence V_{rdc} > 189.94 kN so we provide minimum shear reinforcement.

Diameter of tie bar

≥ 25/4 = 6.25mm

≥ 6mm

Adopt lateral ties of diameter 8mm.

Provide 4-legged 8 mm dia lateral ties (Fe 415).

$$Asv = 4 * 3.14 * \frac{8^2}{4} = 200.96\ mm^2$$

Spacing of ties

i) ≤ 16 × 25 = 400 mm

ii) ≤ 300mm

iii) ≤ least lateral dimension of column = 1200mm

Adopt 4-legged 8mm dia. lateral ties @ 200 mm c/c.

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