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PROJECT REPORT

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1.0 EXECUTIVE SUMMARY

1.1 Introduction

National Highways and Infrastructure Development Corporation (NHIDCL) is a fully owned company of the **Ministry of Road Transport & Highways (MoRT&H)**, Government of India. The company promotes surveys, establishes, designs, builds, operates, maintains and upgrades National Highways and Strategic Roads including interconnecting roads in parts of the country which share international boundaries with neighboring countries. The regional connectivity so enhanced would promote cross border trade and commerce and help safeguard India's international borders. This would lead to the formation of a more integrated and economically consolidated South and South East Asia. In addition, there would be overall economic benefits for the local population and helps to integrate the peripheral areas with the mainstream in a more robust manner.

National Highways & Infrastructure Development Corporation Ltd. is the employer and executing agency for the consultancy services and the standards of output required from the appointed consultants are of international level both in terms of quality and adherence to the agreed time schedule.

Sl. No.	Location of Bridge
1	At km 95.50 over the Irang river on Imphal to Jiribam road (NH-37, Old NH-53)

Pursuant to Clause 10.2 of the Terms of Reference (TOR), this Draft Detailed Project Report is being submitted for Imphal-Jiribam road section (length- 220 Km) on NH-37(old NH-53) in the State of Manipur.

1.2 Project Background

In context of the above mentioned points, the existing Bailey bridge at km 95.50 on at NH-37 in the district of Noney in the state of Manipur shall be replaced with a new PSC Bridge just on the upstream side of the existing bridge. The approaches of Bridge will be 4-Lane with paved shoulder configuration as per specification. However, a short approach road which connects the nearest existing road to maintain the connectivity of Imphal to Jirbam has been proposed for the time period of construction for proposed 4-lane road by next 2-3 years.

The consultancy services for the same is to include design of best possible alignment near the existing bailey bridge and design of Bridge, Approaches and other structures in addition to Financial Analysis of costs, prioritization of Bridge depending on project viability and anticipation of hazards during construction, preparation of Land Acquisition Plan, if required and obtaining of all requisite clearances.

1.3 Salient Features of the Proposed Road:

Descriptions		At present	Proposed
Terrain	:	Hilly	Hilly
Length	:	Proposed Length =43.7m	Proposed Length =123.00m (3 x41m)
Type of Bridge	:	Bailey Bridge	PSC Girder Bridge
Alignment	:	The horizontal alignment of the existing road is curvilinear. There are some sub-standard stiff curve and also deficiency in transition length as per MoRT&H standards.	Minimum Design Speed Considered - 25 kmph
Cross-Section	:	Carriageway: 4m Total width: 5.6m	**Proposed Bridge & Approach Road Cross Sections :** **For Bridge Portion (Each part of Twin Bridge)** a)Carriageway width - 9.5m b)Crash barrier – 0.5m on either side c)Footpath – 1.5m on single side d)Railing – 0.5m on single side e)Total width – 12.5m
CBR Considered	:	-	8%
Pavement Design Life (Approach Road)	:	-	Flexible Pavement-15 Years
Protection Work	:	Nil	Bed & Bank Protection on Approach
Total Civil Cost in Rs.	:	-	Rs 30.75 Cr.

1.4 Abstract of Cost Estimates:

Rate Analysis for different items of works has been carried out considering **Manipur Schedule of Rates for National Highways-Works 2016**.

Sr No	Description of Bill Items	Amount (INR Crore)
A	**Road Portion (Approach road)**	
I	Cutting , Earth filling & Disposal	1.65
II	Sub base	0.24
III	Non-Bituminous Base Course	0.47
IV	Bituminous Base Course	0.25
V	Wearing Coat	0.14
	Sub Total A	**2.75**
B	**Culvert (Sub Total B)**	**0.38**
C	**Bridge**	
I	Foundation	9.10
II	Substructure	5.52
III	Superstructure	10.28
IV	Protection work	0.17
V	Miscellaneous	0.03
	Sub Total C	**25.1**
D	**Grand Total (A+B+C) (As per SOR 2016)**	**28.23**
E	Inflation @ 2.93%	0.83
F	Add GST @ 6%	1.69
G	Civil work without Maintenance **(D+E+F)**	**30.75**

2.0 PROJECT BACKGROUND

2.1 State Profile

Manipur is a state located in the north-eastern part of India. It is surrounded by the Indian states of Nagaland to the north, Assam to the west and Mizoram to the southwest and by Myanmar to the south and east. The state was established on 21 January 1972. Manipur is the 24th largest state by area in India, and the 24th largest by population. Imphal is the capital of the state. The state is located between 23.49'N and 25.68'N latitude and between 93.03'E and 94.78'E longitude. The state is divided into 16 districts. The total area of the State is 22,327 sq. kms.

A current map of the state of Manipur is appended below as Plate – 2.1:

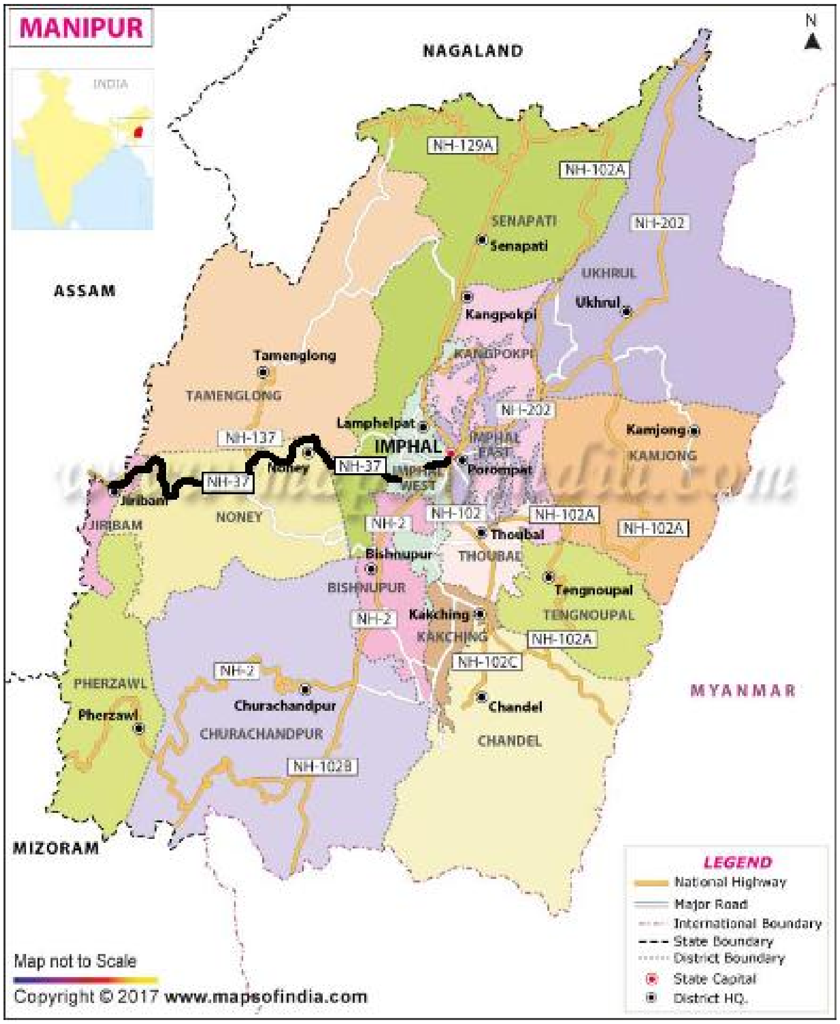


Plate – 2.1: Manipur State Map

2.2 District Profile

Noney district is situated in the Western part of Manipur. It is the third least populous district of Manipur with its headquarters located at Longmai. The district is bounded by Tamenglong district in the north, Imphal district in the east, Churachandpur and Pherzawl districts in the South, Jiribam and Pherzawl districts in the West. The district is divided into four Sub-Divisions, i.e. Longmai, Nungba, khoupum, Haochong Sub-Divisions.

According to the 2011 census Noney district has a population of 36671. This gives it a ranking of the third least populous district in Manipur (out of a total of 16). Noney has a sex ratio of 939 females for every 1000 males. Noney has a higher literacy rate as a whole. In 2011, the literacy rate of Noney was 86.90%. Male literacy was 93.40%, while female literacy rate was 78.71%.

2.3 Manipur – Roadway Network

Manipur serves as a key logistical centre for northeastern states. Main National Highways which connects Imphal are,

a) National Highway NH-39 links Manipur with the rest of the country through the railway stations at Dimapur in Nagaland at a distance of 215 km (134 mi) from Imphal.

b) National Highway 37 (old NH – 53) connects Manipur with another railway station at Silchar in Assam, which is 269 km (167 mi) away from Imphal.

The road network of Manipur, with a length of 7,170 km (4,460 mi) connects all the important towns and distant villages. However, the road condition throughout the state is often deplorable.In 2010, Indian government announced that it is considering an Asian infrastructure network via Manipur to Vietnam.The proposed Trans-Asian Railway (TAR), if constructed, will pass through Manipur, connecting India & Burma, Thailand, Malaysia and Singapore.

Road map Network is appended below as Plate – 2.2:

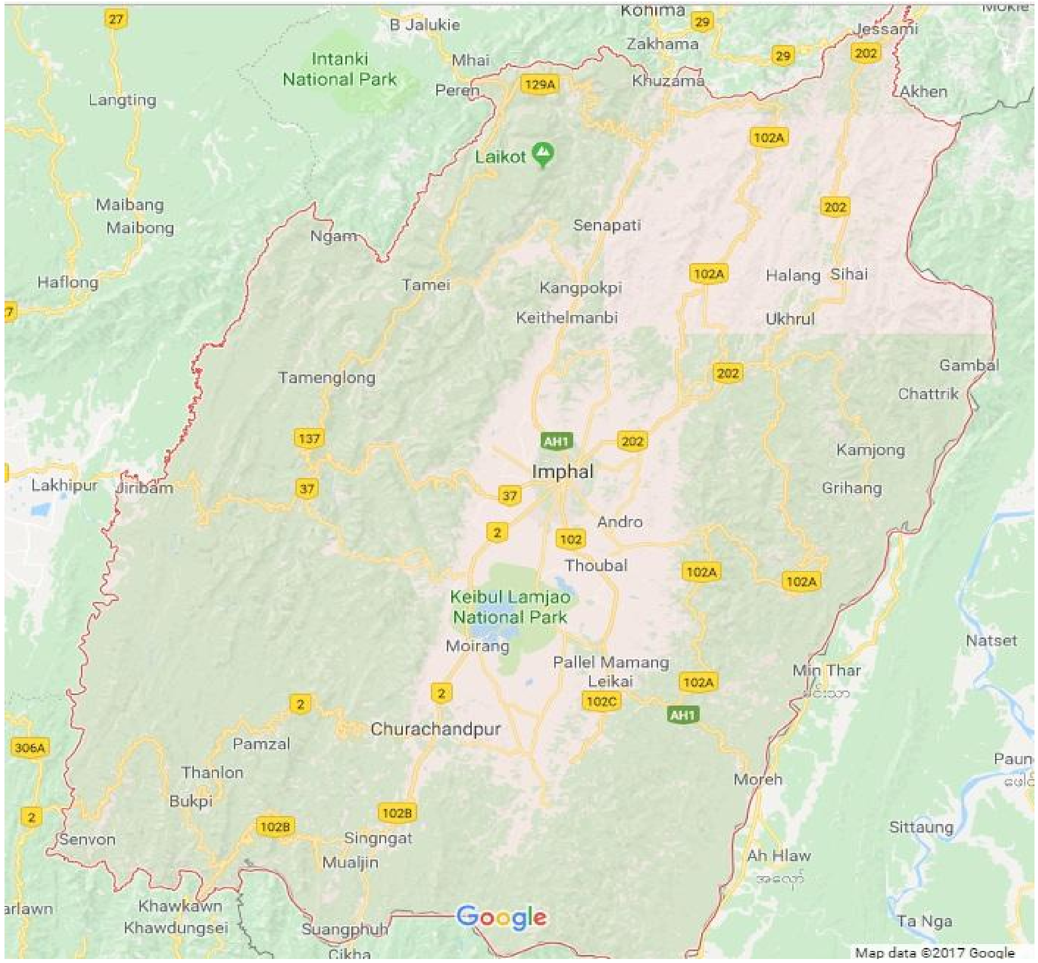


Plate – 2.2: Roadway Network in Manipur

3.0 PROJECT DESCRIPTION

3.1 General

The proposed Bridge lies over National Highway NH-37(old NH-53) in Noney district of Manipur. NH-37 in India links Imphal, Tupul, Noney, Khungsang, Nungba, Jiribam etc. The existing Bailey Bridge is unable to carry the current traffic load of the NH also this narrow bridge causes congestion in that location. To avoid the congestion of traffic and considering the present poor condition of existing bridge NHIDCL has decided to provide a new 4-Lane bridge as per IRC standard.

3.2 Existing Right of Way (ROW)

The existing ROW width along the Approach road has been observed to be around 6 m to 14 m. However, the existing ROW does not cater to the requirement of land i.e. 42m ROW of Hill Road hence land is required to be acquired.

3.3 Settlements

During reconnaissance survey it has been observed that there are 8 nos. pakka and semi pakka hutment along the bridge approach.

3.4 Terrain Classification

The project road passes through mountainous / hilly terrain. The topography is mostly rural in nature.

3.5 Geology and Soil Types

Mainly rocky strata and soil type is clayey silt with decomposed rock. A considerable depth of Boulder layer is also found in top layer.

3.6 Existing Bridge

The existing Bridge is Single Lane Bailey Bridge. It is a single span bridge with span length of 43.7m. The carriageway width of the existing bridge is 4 meter. The existing bridge condition is poor. Some photographs of existing bridge are enclosed in the following page as Plate – 3.1

Plate – 3.1: View of existing Bridge condition near proposed Bridge location

4.0 DRAFT DESIGN STANDARD

4.1 Design Standards: PSC Girder Structures

(a) GEOMETRIC DESIGN

i) The overall width of the deck slab will be kept twin of 12.5m with 9.5m width carriageway.

ii) The span type and arrangement will be proposed considering site constraint, optimize the pier number and aesthetics.

(b) LOADING STANDARD

i) All structures will be designed according to load specified in IRC-6:2017.

ii) LL on footpath will be taken as 5 KN/m2

iii) Environmental loadings such as earth pressure, water current, seismic forces and temperature effect will be taken as per IRC/BIS Codes. IS-1893 will be followed in evaluating dynamic increment of earth pressure.

(c) GUIDING STANDARDS FOR STRUCTURES

The Structural planning of new bridges or culverts will be guided by the layout of existing structures.

The preliminary designs of proposed structures will be carried out in accordance with the provisions of the following IRC Codes/guidelines.

- IRC:5-2015 - Section I, General Features of Design
- IRC:6-2017 - Section II, loads and Stresses
- IRC:112-2011 - Code for Concrete Road Bridges
- IRC:22-2015 - Section VI, Composite Construction
- IRC:40-2002 - Section IV, Brick, stone & Block Masonry
- IRC:45-1972 - Recommendations for estimating the Resistance of soil Below Maximum Scour level in the Design of Well
- IRC:SP:84-2014 -- Manual for Four Lane Highway with Paved Shoulder
- IRC:SP:13-2004 -- Guidelines for design of small bridges and culverts
- IRC:78-2014 - Section VII, Foundations and Structure
- IRC:83-2015 - Section IX,(Part I), Metallic Bearings
- IRC:83-2015 - Section IX,(Part II), Elastomeric Bearings
- IRC:83-2002 - Section IX,(Part III), POT Bearings

- IRC:87-2011 - Guidelines for the Design & Erection of False work for Road Bridges
- IRC:SP-33-1989 - Guidelines on Supplemental Measures for Design, Detailing and Durability of Important Bridge Structures
- IRC:89-1997 - Guidelines for design and construction of river training and control works for road bridges (1st Revision)

Where IRC Codes are silent relevant BIS Codes will be followed. And where even BIS codes are silent, international codes / MOST, MORTH guidelines will be adopted.

(d) SEISMIC DESIGN

The project road falls in Seismic Zone V, as per the classification specified in IRC:6-2017. All bridges will be designed for Seismic forces as per clause 219.1 of the said code.

(e) SOIL PARAMETERS

The Soil parameters used in the preliminary design of foundations for Bridges will be taken from the report of soil investigation and information obtained from local authorities / existing bridge design data.

The following soil parameters will be used for material for back fill behind abutment of bridges and culverts and the abutment structure will be designed accordingly.

ϕ = 30°

δ = 20°

γ_d = 18 KN/m^3

γ_{sub} = 10 KN/m^3

A 600 mm thick granular material filter behind abutment and fin wall and adequate weep holes in abutment walls and fin wall will be provided for proper drainage.

(f) FOUNDATIONS:

For this Open foundation / Pile Foundation has been adopted based on the geotechnical investigation data for the bridge.

(g) SUBSTRUCTURE:

RC wall type piers and wall type / spill through type abutment will be provided in the bridges, matching the requirements and site conditions. Their design will be carried out in conformity with IRC-78-2014. The shape, size and alignment will be considered from aesthetic and hydraulic aspects.

(h) SUPERSTRUCTURE:

I) Type & Span arrangement of superstructure has been chosen considering alignment, obstruction due to at grade intersection, space constraint and other site constraints. Generally, Precast or Pre fabricated type superstructure such as: PSC Box Girder, PSC I-Girder, Steel-Concrete composite T-girder, Steel-Concrete composite Box-girder, steel truss girder, steel bow string girder, and Precast RC T-girder has been considered in selecting the superstructure. In this case, PSC I-Girder has been used.

II) **BEARINGS:**

Pot-PTFE bearings will be used in this bridge as required for specific span & Elastomeric bearings for Arrester Block.

III) **RAILINGS:**

Reinforced concrete railings in M-30 grade concrete following MoRT&H standard will be provided.

IV) **CRASH BARRIER:**

Reinforced concrete crash barrier in M-40 grade concrete following MoRT&H standard will be provided.

V) **EXPANSION JOINTS**:

Buried type expansion/strip seal joints as per MoRT&H standard will be used.

VI) **WEARING COURSE**:

65mm thick bituminous concrete wearing course will be adopted.

VII) **APPROACH SLAB:**

R.C. approach slabs, 3.50m long and 300mm thick in M-30 concrete will be used at either end of the bridges and culverts to ensure riding comfort and to reduce vehicular surcharge on the abutment walls. One end of the approach slab is supported on R.C. bracket projecting out, from dirt wall while the rest of the slab is placed on compacted soil as per the guidelines issued by MoRT&H. A leveling course, 150mm thick in M-15 grade concrete will be used under the approach slab.

VIII) **DRAINAGE SPOUTS:**

4 nos. of 100mm dia drainage spout has been used for deck drainage in one side of carriageway per span of the Twin Structure.

IX) **TMT REINFORCEMENT AND PRESTRESSING CABLES:**

Fe-500 high yield strength deformed bars conforming to IS-1789 will be used as reinforcement in all R.C. works. Uncoated stress relieved low relaxation strands conforming to IS-14268 will be used in PSC works.

Prestressing Stages: As far as possible, single stage prestressing will be proposed.

5.0 COST ESTIMATE

5.1 General

Bill of Quantities (BOQ) and the project cost estimates has been prepared considering the various items of works associated with identified improvement proposals so as to assess the total cost of the project. The cost estimates have been based on the available data/documents supplemented by the consultants' surveys, site visits and experience in similar type of works. Rate analyses of major item of works have been worked out to verify the adopted rates.

5.2 Methodology

Cost estimate methodology involves the following:

- Computation of quantities for improvement proposal
- Unit rate analysis
- Bill pricing and finalization of cost estimates

5.3 Estimation of Quantities

The major items of the work considered for the purpose of cost estimation are:

A. Bridge over Irang River

B. Approach Road

A. Bridge

1. Structure Cost
2. Drainage Work
3. Traffic Sign, Markings and Other Road Appurtenances
4. Miscellaneous

B. Approach Road

1. Site Clearance & Dismantling
2. Earthwork
3. Granular Sub-base Course and Base Courses
4. Bituminous Courses
5. Culverts

6. Drainage and Protection Work
7. Traffic Sign, Markings and Other Road Appurtenances
8. Miscellaneous
9. Detailed BOQ has been given in Volume-III

5.4 Material Sources

The sources of materials are given in tabulated form in Table no. 5.1

Coarse Aggregates

Hard stone aggregate, fulfilling the requirements of concrete works, base, sub base and asphaltic works are considered from Stone Quarry located at 60Km from Project road .Its available from Noney.

Sand

Coarse Sand is available from Noney an average lead of 60 km from project road.

Fine aggregates are available Noney with an average lead of 60 km from project Road.

Bitumen

Bitumen of viscosity grade VG-40 is available from Numaligarh Refinery, Assam in bulk or packed condition. Distance from Numaligarh Refinery, Assam to bridge location lead is 417 km.

Cement

Cement to be used in the construction work shall be any of the following types with the prior approval of the Engineer:

Ordinary Portland cement, 33 Grade, conforming to IS: 269

Rapid Hardening Portland Cement, conforming to IS: 8041

Ordinary Portland cement, 43 Grade, conforming to IS: 8112

Ordinary Portland cement, 53 Grade, conforming to IS: 12269

Sulphate Resistance Cement, Conforming to IS: 12330

The chloride content in cement shall in no case exceed 0.05 percent by mass of cement. Also, total sulphur content calculated as sulphuric anhydride (SO3) shall in no case exceed 2.5 percent and 3.0 percent when tri-calcium aluminates present by mass is upto 5 or greater than 5 respectively. Cement will be available at Imphal with an average lead of 107.5km from project Road.

Reinforcement

For plain and reinforced concrete (PCC and RCC) or pre-stressed concrete (PSC) works, the reinforcement/un-tensioned steel as the case may be shall consists of the following grades of reinforcing bars as shown in the table below. Steel will be available at Imphal with an average lead of 107.5km from project road has been considered.

Grade Designation	Bar Type conforming to governing IS Specification	Characteristic Strength f_y(MP_a)	Elastic Modulus GP_a
S 240	IS:432 Part I, Mild Steel Bar	240	200
S 500	IS:1786 High Yield Strength Deformed Bars (HYSD)	500	200

Table 5.1

Sl. No.	Material	Place	Av.Lead (Km.)
1	Local Sand (Fine)	Noney	60
2	Stone Metal	Barak	71
3	Stone Boulder	Barak	71
4	Stone Chips, Aggregate	Noney	60
5	Coarse Sand	Noney	60
6	Cement	Imphal	107.5
7	Steel	Imphal	107.5
8	Bitumen	Numaligarh Refinery, Assam	417
9	Structural Steel	Imphal	107.5

5.5 Analysis of unit rates

Rate Analysis for different items of works has been carried out considering **Manipur Schedule of Rates for National Highways-Works 2016**.

5.6 Abstract of Cost Estimate

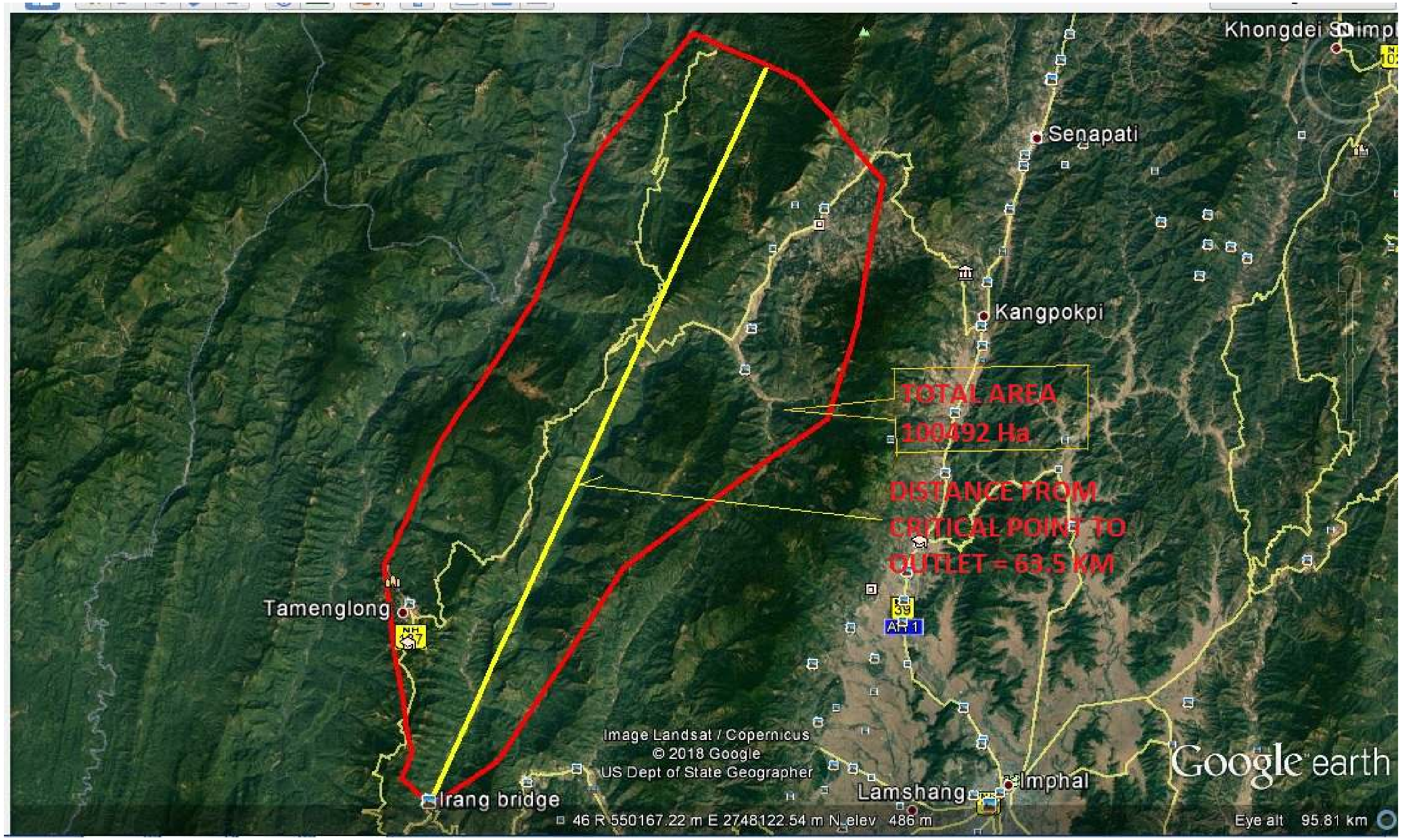
General Abstract of Cost is mentioned below in **Table 5.2**

Table 5.2

Sr No	Description of Bill Items	Amount (INR Crore)
A	**Road Portion (Approach road)**	
I	Cutting , Earth filling & Disposal	1.65
II	Sub base	0.24
III	Non-Bituminous Base Course	0.47
IV	Bituminous Base Course	0.25
V	Wearing Coat	0.14
	Sub Total A	**2.75**
B	**Culvert (Sub Total B)**	**0.38**
C	**Bridge**	
I	Foundation	9.10
II	Substructure	5.52
III	Superstructure	10.28
IV	Protection work	0.17
V	Miscellaneous	0.03
	Sub Total C	**25.1**
D	**Grand Total (A+B+C) (As per SOR 2016)**	**28.23**
E	Inflation @ 2.93%	0.83
F	Add GST @ 6%	1.69
G	Civil work without Maintenance **(D+E+F)**	**30.75**

HYDRAULICS OF IRANG RIVER
CH._95.500 KM

CH._95.500KM



TOTAL CATCHMENT AREA = 100492 HA

FALL IN LEVEL FROM THE CRITICAL POINT TO THE OUTLET = 1928 M

DISTANCE FROM THE CRITICAL POINT TO THE OUTLET = 63.5 KM

Detailed Hydraulic Calculations

Introduction:

The length of a bridge, its depth of foundation etc. are dependent on the maximum recorded quantum of water or flood discharge which has passed through the river or the channel over which the bridge is proposed and as such the design discharge is very important not only from economic consideration but also from safety or stability consideration. Therefore, the design discharge, which might be the recorded dischrge during the past 50-100 years, shall be ascertained very carefully.

There are various methods for the estimation of flood discharge like:

1. Catchment-Run-off Method from rainfall and other characteristics of the catchment by the use of empirical formulae or by Rational Method.
2. By using Empirical Formulae.
3. From hydraulic characteristics of the stream such as the conveyance factor and slope of the stream.
4. From area of cross-section and velocity as observed on the stream at the bridge site.
5. From recorded flood discharge near the bridge site.

The use of a particular method depends upon (i) the desired objective, (ii) the available data and (iii) the importance of the project. Further the Rational Method is found to be suitable for peak flow prediction in small catchments upto 50 km^2 in area. It finds considerable application in urban drainage designs and in the design of small bridges and culverts.

Below, the flood discharge is estimated only by the first two methods written above. The third method cannot be used for the hydraulic calculations as it is not possible to measure the velocity of the stream at the bridge site as the stream is dry now. The fourth or last method is not used as the recorded flood discharge near the bridge site is not available.

A) Rational Method to calculate Peak Run-off from Catchment:

Step I - Input data:-

Area of Catchment (A):	=	100492 Ha
Distance from the critical point to the Outlet (L):	=	63.5 km
Fall in level from the critical point to the Outlet (H):	=	1928 m

The values of H and L can be found from the contour map of the catchment area.

One - hour Rainfall (I_0):

Value of I_0 can be worked out if the total rainfall and the duration of the severest storm are known. If these data are not available for some place, for that Meteorological Department of the Government of India, have supplied the heaviest rainfall in mm/hour experienced by various places in India. For Gawhati, the I_0 is considered from IRC:SP-13-2004.

Hence, I_0 (ref. IRC:SP: 13-2004 Appendix- A) = 4.8 cm

Percentage coefficient of Run-off for the Catchment Characteristics (P):

Coefficient P depends on the (i) porosity of the soil, (ii) area, shape and the size of the catchment, (iii) vegetation cover, (iv) surface storage viz. existence of lakes and marshes, and (v) initial state of wetness of soil. The values of P for the various conditions of the catchment area are given below:

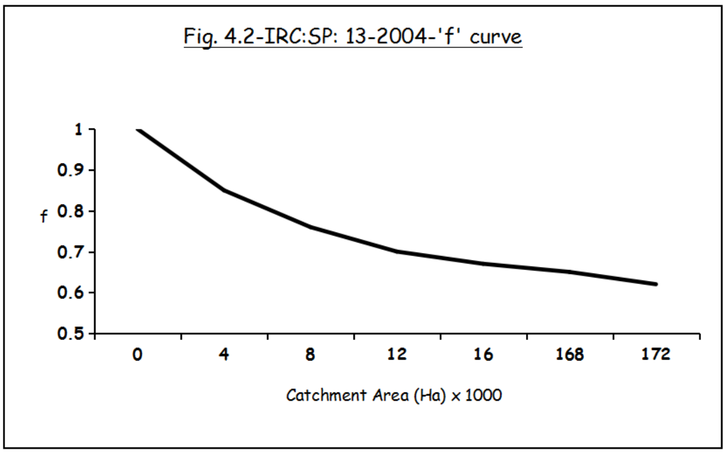
Table 4.1 (IRC:SP: 13-2004): Maximum Value of P :-

Sl. No.	Characteristics of the catchment	Value of P
1	Steep, bare rock and also city pavements	0.90
2	Rock, steep but wooded	0.80
3	Plateaus, lightly covered	0.70
4	Clayey soils, stiff and bare	0.60
5	Clayey soils, lightly covered	0.50
6	Loam lightly cultivated or covered	0.40
7	Loam largely covered	0.30
8	Sandy soil, light growth	0.20
9	Sandy soil, heavy brush	0.10

#N/A = 0.38

Fraction spread of storm over the catchment (f):

The mean intensity of the storm depends on the catchment area. The larger the area considered the smaller would be the mean intensity,i.e., the mean intensity is some inverse function of the size of the catchment. The relation of f with A can be represented by the curve given in Fig. 4.2 of IRC:SP: 13-2004, shown below:



Hence, from curve 'f' for catchment area of 100492 hectares = 0.53

Step II - Estimating the Concentration time of a Catchment (t_c):-

The time of concentration is to be calculated by the formula given below:

$t_c = (0.87 \times L^3/H)^{0.385}$ = (0.87 x 63.5^3 /1928)^0.385 = 6.23 hr

Step III - Calculation of Critical/ Design Intensity (I_c):-

The Design Intensity is given by the formula:

$I_c = 2 \times I_0 /(t_c + 1)$ = 2 x 4.8 /(6.23 + 1) = 1.4 cm/hr

Step IV - Calculation of Run-off (Q):-

$Q = 0.028 \times I_c \times f \times P \times A$ = 0.028 x 1.4 x 0.53 x 0.38 x 100492 = 793.37 m^3/s

Design Discharge = 794 m^3/s

Regime width = $4.8 \times (Q)^{0.5}$ = 135.26 m

Cross-Sectinal Area-Bed Slope Method

Since the Bridge is provided across a defined stream, we estimate flood discharge from the conveyance factor & slope of the stream applying Manning's Velocity Formula. As the flood rises above the bank line, the cross section of the river is divided in to three subsections and velocity is calculated as per Cl-5.7, SP-13:2004.

Step I - HFL Fixation:-

From the survey data, we fix the HFL = 223.078

Position	HFL	LBL	Chainage
At U/S	225.044	219.454	433.268
At D/S	221.324	215.396	350.784
At ± 0.0	223.078	217.847	0.0

Step II - Slope Calculation :-

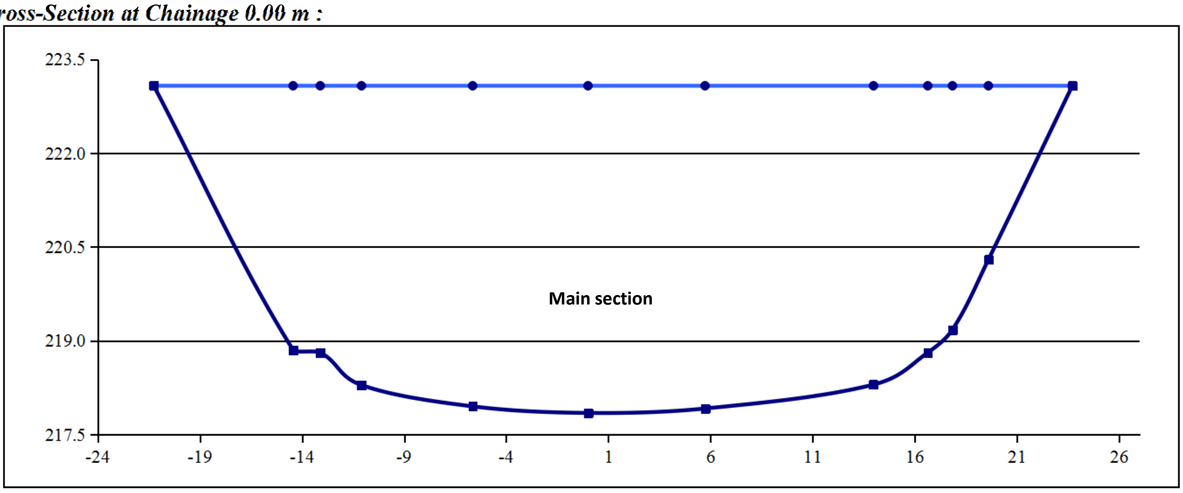
Average Slope is caculated as = 0.005

Position	LBL	Chainage
At U/S	219.454	433.268
At D/S	215.396	350.784

Use Slope,S : = 0.005

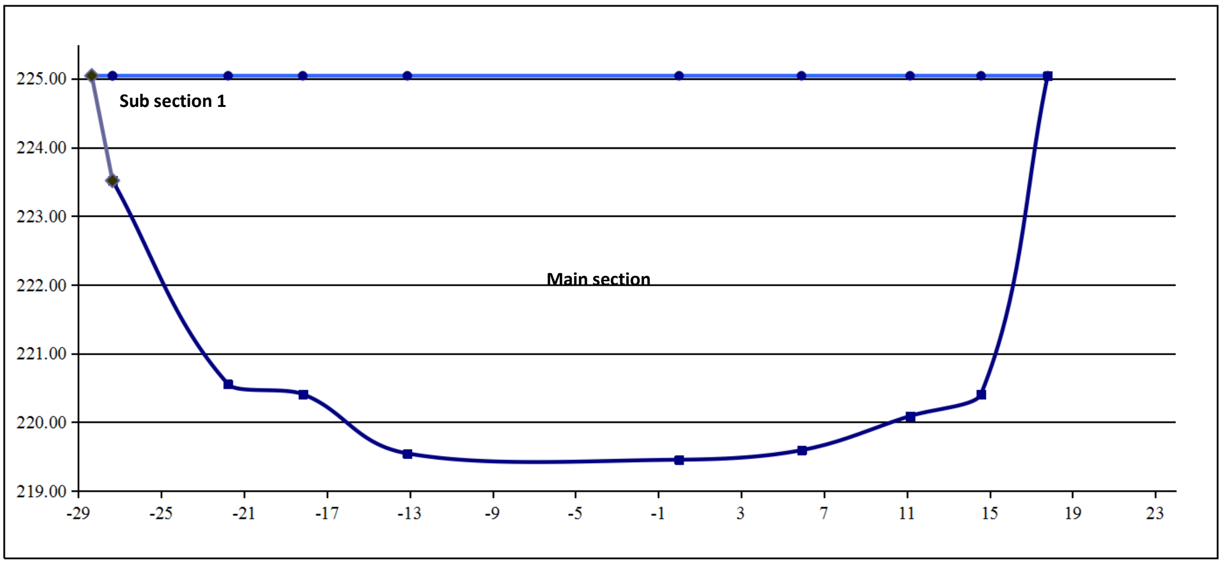
Rugosity co-efficient used= = 0.05

Cross-Section at Chainage 0.00 m :



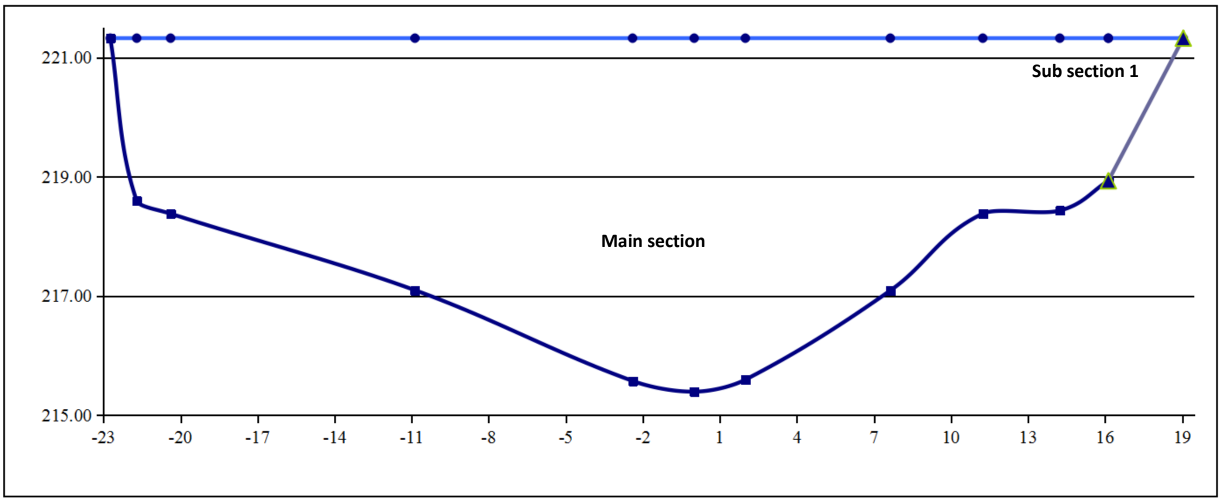
Points	Distance (m)	Level (m)	Level Difference with HFL(m)	Segment Length, L (m)	Avg. water depth below HFL for each segment(m)	Area of Cross Section, A (in sqm)	Wetted Perimeter,P (m)	Hydraulic radius,R= A/P (m)	Velocity,V= $(R^{2/3}xS^{1/2})/n$ (m/s)	Discharge,Q =VxA (m^3/s)
1	-21.254	223.078	0.000					3.885	3.495	645.309
2	-14.435	218.854	4.224	6.819	2.112	14.40	8.02			
3	-13.106	218.812	4.266	1.329	4.245	5.64	1.33			
4	-11.087	218.293	4.785	2.019	4.525	9.14	2.08			
5	-5.646	217.952	5.126	5.441	4.956	26.96	5.45			
6	0.000	217.847	5.231	5.646	5.179	29.24	5.65			
7	5.729	217.918	5.160	5.729	5.196	29.77	5.73			
8	13.975	218.300	4.778	8.246	4.969	40.97	8.25			
9	16.631	218.812	4.266	2.656	4.522	12.01	2.70			
10	17.839	219.179	3.899	1.208	4.083	4.93	1.26			
11	19.598	220.298	2.780	1.759	3.340	5.87	2.08			
12	23.708	223.078	0.000	4.110	1.390	5.71	4.96			
					Total	**184.65**	**47.53**	**Total discharge**		**645.309**

Cross-Section at Chainage 433.268 m U/S :



Points	Distance (m)	Level (m)	Level Difference with HFL(m)	Segment Length, L (m)	Avg. water depth below HFL for each segment(m)	Area of Cross Section, A (in sqm)	Wetted Perimeter,P (m)	Hydraulic radius,R= A/P (m)	Velocity,V= $(R^{2/3}xS^{1/2})/n$ (m/s)	Discharge,Q =VxA (m^3/s)
1	-28.354	225.044						0.419	0.792	0.606
2	-27.351	223.518	1.526	1.003	0.763	0.77	1.83			
Left bank					**Total**	**0.77**	**1.83**			
2	-27.351	223.518	1.526					4.446	3.824	823.390
3	-21.769	220.553	4.491	5.582	3.009	16.79	6.32			
4	-18.168	220.409	4.635	3.601	4.563	16.43	3.60			
5	-13.119	219.546	5.498	5.049	5.067	25.58	5.12			
6	0.000	219.454	5.590	13.119	5.544	72.73	13.12			
7	5.910	219.593	5.451	5.910	5.521	32.63	5.91			
8	11.152	220.090	4.954	5.242	5.203	27.27	5.27			
9	14.579	220.409	4.635	3.427	4.795	16.43	3.44			
10	17.797	225.044	0.000	3.218	2.318	7.46	5.64			
					Total	**215.32**	**48.43**	**Total discharge**		**823.996**

Cross-Section at Chainage 350.784 m D/S :



Points	Distance (m)	Level (m)	Level Difference with HFL(m)	Segment Length, L (m)	Avg. water depth below HFL for each segment(m)	Area of Cross Section, A (in sqm)	Wetted Perimeter,P (m)	Hydraulic radius,R= A/P (m)	Velocity,V= $(R^{2/3}xS^{1/2})/n$ (m/s)	Discharge,Q =VxA (m^3/s)
1	-22.735	221.324	0.000							
2	-21.706	218.596	2.728	1.029	1.364	1.40	2.92			
3	-20.388	218.381	2.943	1.318	2.836	3.74	1.34			
4	-10.873	217.098	4.226	9.515	3.585	34.11	9.60			
5	-2.399	215.576	5.748	8.474	4.987	42.26	8.61			
6	0.000	215.396	5.928	2.399	5.838	14.01	2.41	3.902	3.505	567.423
7	1.992	215.597	5.727	1.992	5.828	11.61	2.00			
8	7.636	217.092	4.232	5.644	4.980	28.10	5.84			
9	11.237	218.381	2.943	3.601	3.588	12.92	3.82			
10	14.235	218.433	2.891	2.998	2.917	8.75	3.00			
11	16.125	218.932	2.392	1.890	2.642	4.99	1.95			
Right Bank					**Total**	**161.88**	**41.49**			
11	16.125	218.932	2.392					0.925	1.343	4.685
12	19.043	221.324	0.000	2.918	1.196	3.49	3.77			
					Total	**3.49**	**3.77**	**Total discharge**		**572.109**

Discharge :-

At D/S	=	572.11 m^3/s
At U/S	=	824.00 m^3/s
At ± 0.0	=	645.31 m^3/s
Discharge to be taken (Ref.: Cl.-6.2.1,IRC:SP:13-2004)	=	824.00 m^3/s
Design Discharge	=	824.00 m^3/s

Fixing of Bridge Length:

1) From Hydraulic calculations:

i) Rational Method:

Discharge Calculated: = 794.00 m^3/s

i) Area-Slope Method:

Discharge Calculated: = 824.00 m^3/s

Calculated Regime width for discharge of **824.00** m^3/s = **137.79** m

Ref.: Cl.-6.2.1,IRC:SP:13-2004

Regime width considering a restriction of 30 % = **96.453 m**

2) Existing Bridge:

i) There is a 1x42.672 M _RCC T-Girder Bridge at the proposed Bridge location in bad condition

ii)Type of existing bridge = BOX

ii) Length of existing bridge = 42.672 m

3) Bank to bank distance:

Bank to bank distance: = **46** m

From the above three data and also considering the suitable abutment location and site condition,the proposed bridge has been fixed as the details given below:

Overall length of Bridge: = **123.04** m

Span Arrangement = **3x41 M _PSC T-GIRDER**

Total Bridge length (dirt wall inner to inner) = 123.040 m

Afflux Calculation with Orifice Formula:-

Discharge, Q	=	824.0 m^3/sec
Linear Waterway, L	=	113.680 m
Unobstructed Stream Width, W	=	137.790 m
Downstream Depth, D_d	=	5.928 m
Ratio, L/W	=	0.825
Coefficient 'C_0' from graph 5.2 of IRC:SP:82-2008	=	0.875
Coefficient 'e' from graph 5.3 of IRC:SP:82-2008	=	0.730
g	=	9.800

From IRC:SP:82-2008

$$Q = C0(2g)0.5\ L\ Dd\ [\ h + (1+e)\ U^2/2g]^{1/2}$$

or, 824.0 = 2610.527957 (h + 0.088265306 u2)^0.5

or, (h + 0.0882653 u2) - 0.09963071817 = 0.000

Also just upstream of the Bridge,

Q=W(Dd+h)U

823.9957639 = 137.790 x (Dd+h)u

5.980083924 = (5.928 +h) * u

h = (5.980 / u) - 5.9

u = 1.0070601 **ok**

h = 0.010 m **ok**

Actual Afflux = 0.01 m

Considering Afflux = 0.050 m

Fixation of formation level

Freeboard (vertical clearance)	=	1.200 m
Depth of deck slab with girder	=	2.775 m
Wearing course at middle	=	0.065 m
H.F.L	=	223.078

soffit level = H.F.L+Afflux+Vertical Clearance

= 223.078 + 0.05 + 1.2 = **224.328 m**

soffit level calculated based on HFL is less than the exisiting soffit level of bridge,
so consider the soffit level is = **237.160 m**

FRL = soffit level + depth of girder + wearing coat

237.16 + 2.775 + 0.065 = **240.0 m**

Scour Depth Calculation :

From IRC-78 , % increment of discharge with respect to catchment area	=	10	%
Maximum Discharge	=	793.37	
Total incremented discharge as per code (for scour calculation)	=	872.71	m^3/s

Scour Depth Calculation :

Effective Span (bearing center to center distance)	=	38.800	m
Distance bearing center to deck end	=	1.08	m
Nos. of span	=	3	Nos.
Expansion Gap between two adjacent span	=	0.04	m
No of Expansion Gap	=	4	Nos.
Total Bridge length (dirt wall inner to inner)	=	123.040	m
No. of Pier	=	2	
Thickness of Pier	=	3	m
No. of Abutment	=	2	
Thickness of Abutment	=	1.2	m
Deduction of linear waterway due to			
Abutment (1/2 of Abt.thk.)	=	1.680	m
Pier	=	3	m
Effective L_W	=	113.680	m
D_b = Design Discharge/ Eff. Span	=	7.677	m^3/sec/m
K_{sf} (Silt Factor)	=	1.000	
D_{sm} (Normal Scour Depth) = $1.34(D_b^2/K_{sf})^{1/3}$	=	5.215	m
Normal Scour Level	=	217.847	m
Max. scour depth at abutment location = (1.27 x D_{sm})	=	6.623	m
Max. scour depth at abutment location = (1.27 x D_{sm}) [From HFL-LBL Criteria]	=	5.231	m
Max. scour level at abutment location	=	216.455	m
Max. scour depth at pier location = (2.0 x D_{sm})	=	10.430	m
Max. scour level at pier location	=	212.648	m

SOIL RREPORT
CHAINAGE -95.500 KM
3X40.0M PSC T-GIRDER
IRANG RIVER

BRIDGE AT CH. KM. 95+500

1. SOIL PROFILE & PROPERTIES:

The subsoils in general are of good quality. It is characterized by a stiff to very stiff silty clay layer at top around BH-A2 location only. A boulder layer is encountered thereafter. Underlying the above a weathered rock layer of is encountered and that layer continued up to the terminating depth of all the boreholes except BH-04(A2) location where boulder layer continues upto terminating depth.

On the above basis the layer wise descriptions are presented below.

1.1. STRATUM – I :

The soil in this layer is characterized by stiff to very stiff, brownish/ yellowish grey, silty clay / clayey silt with sand mixture. Rock fragments are observed at lower reaches. The average "N" value of this layer is 18. The soil properties of this layer revealed from routine laboratory test on "UDS" as well as "SPT" samples as collected from this layer are presented below.

Bulk Density, gms/cc	1.99	Specific gravity	2.69
Dry Density, gms/cc	1.68	Natural Water Content, %	18
Liquid limit %	32	Void ratio	0.940
Plastic limit %	19	**GRAIN SIZE**	
		Gravel %	07
TRSH-UU:		Sand %	38
Cohesion kg/sqcm	1.19	Silt %	50
Friction angle °	06	Clay %	05

1.2. STRATUM – IA:

This is a sand layer consists of loose brownish grey silty sand with decomposed rock dust observed at top around BH-03 location only. Mica and clay binder have been observed in the layer. The average corrected "N" value of this layer is 19. No UDS could be collected from this layer. Grain size analysis of some "SPT" samples shows the following average properties.

Specific gravity	2.73
GRAIN SIZE	
Sand %	68
(Silt + Clay)%	32

1.3. STRATUM – II:

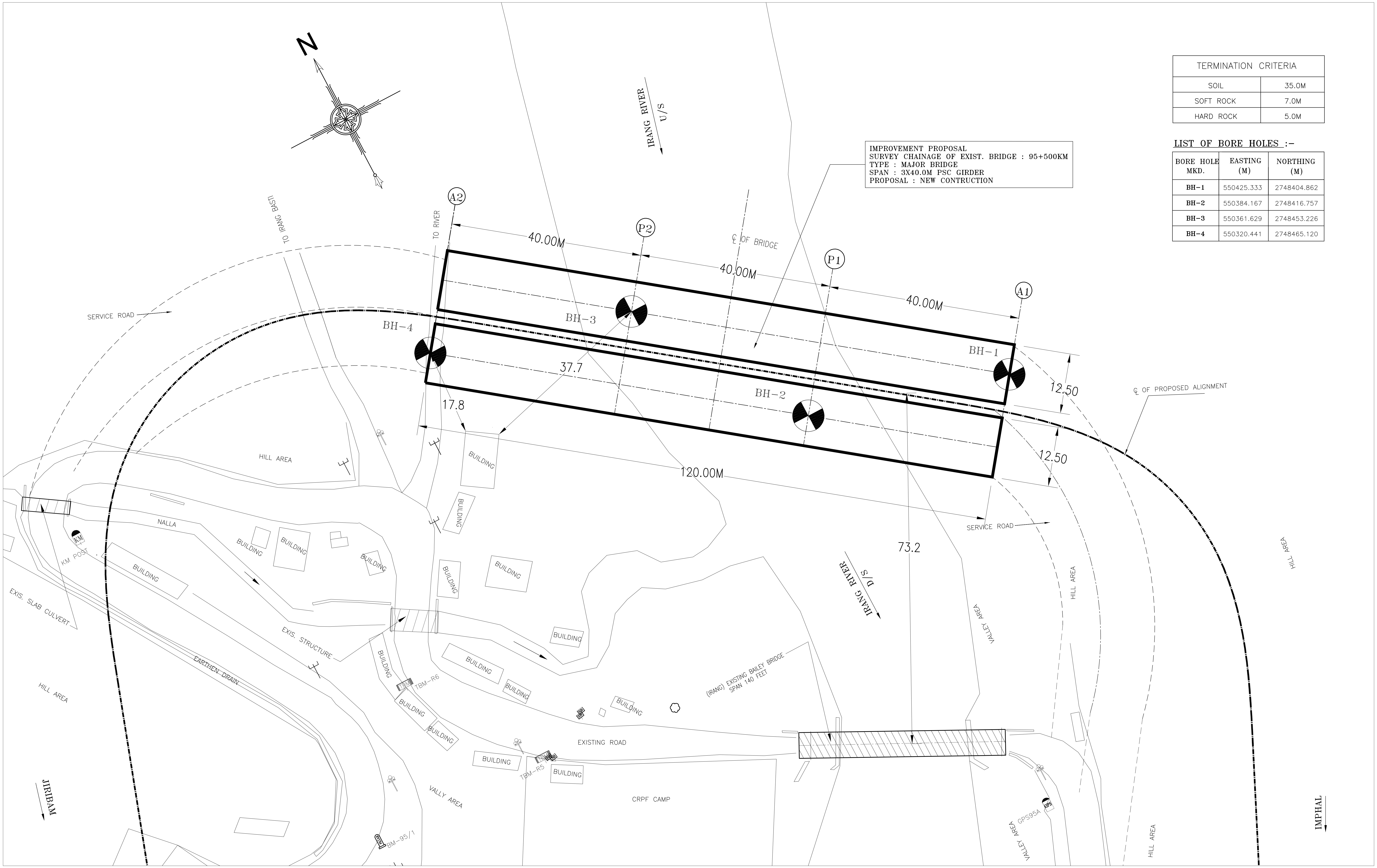
This is a boulder layer consists of different size & different type of colour boulder and the intermediate voids are filled up with silty sand. This layer continues upto the terminating depth around BH-04(A2) location. Drilling method has been adopted to go through the layer. The core recovery of this layer varies from 13% to 29% with RQD nil. The following one type

TERMINATION CRITERIA	
SOIL	35.0M
SOFT ROCK	7.0M
HARD ROCK	5.0M

LIST OF BORE HOLES :-

BORE HOLE MKD.	EASTING (M)	NORTHING (M)
BH-1	550425.333	2748404.862
BH-2	550384.167	2748416.757
BH-3	550361.629	2748453.226
BH-4	550320.441	2748465.120

IMPROVEMENT PROPOSAL
SURVEY CHAINAGE OF EXIST. BRIDGE : 95+500KM
TYPE : MAJOR BRIDGE
SPAN : 3X40.0M PSC GIRDER
PROPOSAL : NEW CONTRUCTION

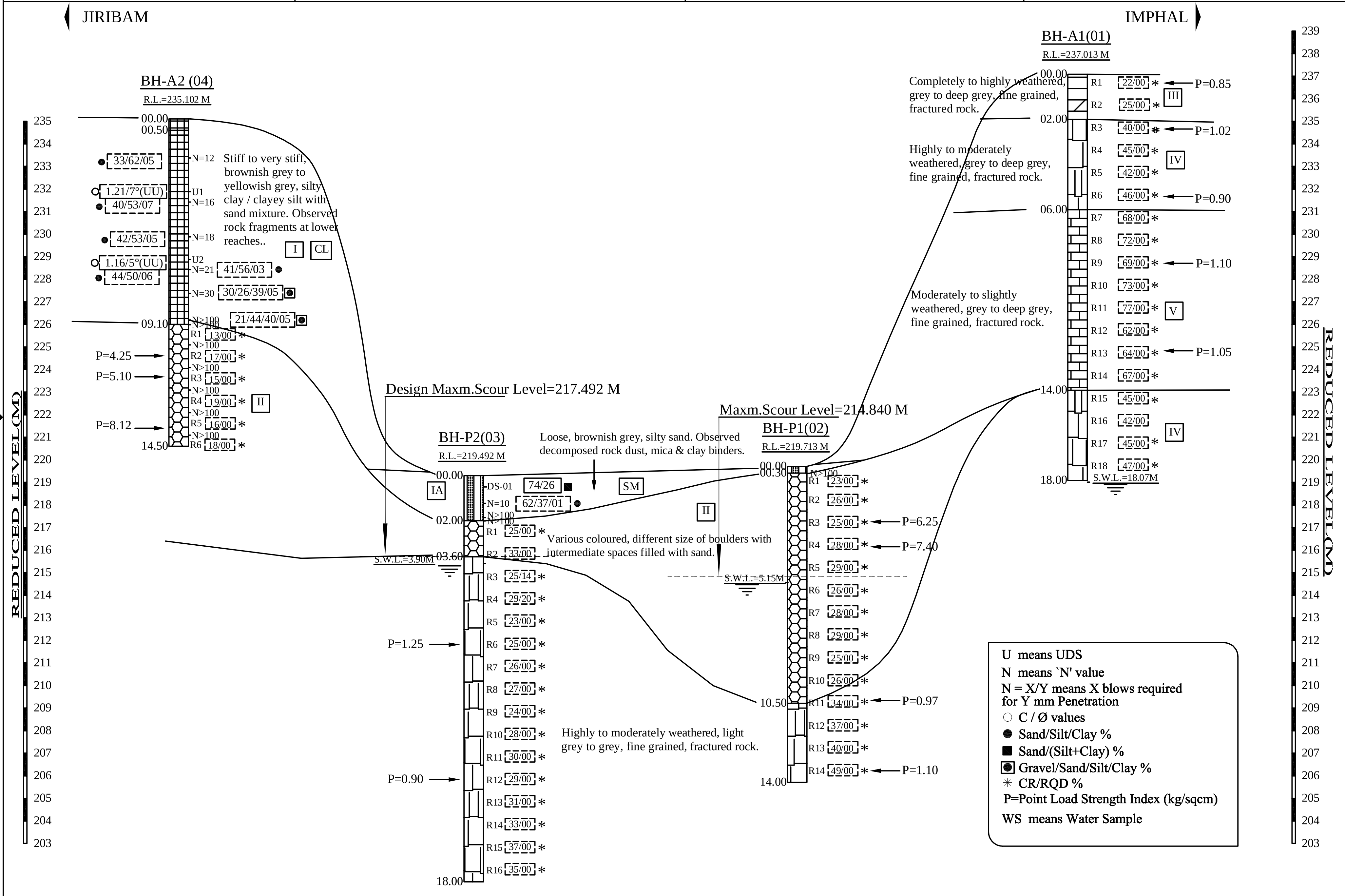


SCALE: 1:500	CLIENT: NATIONAL HIGHWAYS AND INFRASTRUCTURE DEVELOPMENT CORPORATION LTD. 4, Parliament Street, New Delhi - 110001	BORE HOLE PLAN
DATE: FEBRUARY, 2018	PROJECT: Consultancy Services for Preparation of Detailed Project Report and providing pre-construction services in respect of 4 Laning with Paved Shoulder of Imphal - Jiribam Section (length-220 Km) on NH-37 (NH-53) for proposed bridge over River Irang in the State of Manipur.	ROAD NAME:- IMPHAL TO JIRIBAM (NH-37)

MKD.	DATE	DESCRIPTION	CHKD.	APPRD.
REVISIONS				

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.

Job No : 4047	**Created by :** JRC	**Created on :**	**Sheet No:**



of test was carried out on the rock core sample.

1. Point Load index.

The average properties of this layer as revealed from the routine laboratory test are as follows.

Bulk Density, gms/cc	2.433
Dry Density, gms/cc	2.413
Water Content, %	0.829
Specific Gravity	2.543
Porosity %	5.140
Point Load index, kg/sqcm	6.22

1.4. STRATUM – III:

This layer consists of completely to highly weathered, deep grey/ grey, fine to medium grained rock. The core recovery of this layer ranges from 22% to 25% with nil RQD. The average properties of this layer as revealed from the routine laboratory test are as follows.

Bulk Density, gms/cc	2.410
Dry Density, gms/cc	2.385
Water Content, %	1.07
Specific Gravity	2.722
Porosity %	6.3750
Point Load index, kg/sqcm	0.85

1.5. STRATUM – IV:

This layer consists of moderately to slightly weathered, deep grey/ grey, fine to medium grained rock. The core recovery of this layer ranges from 62% to 73% with nil RQD. The average properties of this layer as revealed from the routine laboratory test are as follows.

Bulk Density, gms/cc	2.601
Dry Density, gms/cc	2.582
Water Content, %	0.736
Specific Gravity	2.919
Porosity %	5.4576
Point Load index, kg/sqcm	1.02

1.6. STRATUM – V:

Underlying the above we have a highly to moderately weathered light grey/ grey, fine grained fractured rock and that continued up to the terminating depth around all borehole location except BH-A2 location. The core recovery of this layer ranges from 26% to 49% with nil RQD. The average properties of this layer as revealed from the routine laboratory test are as follows.

Bulk Density, gms/cc	2.587
Dry Density, gms/cc	2.569
Water Content, %	0.700
Specific Gravity	2.801
Porosity %	6.8020
Point Load index, kg/sqcm	1.08

2. CHOICE OF FOUNDATION & FOUNDING LEVEL:

The proposed structure will be a Bridge. Considering the nature of the subsoil and the type of structures to be constructed at the present site, suitable type of foundation is suggested around abutment and pier locations.

3. CALCULATION OF SCOUR DEPTH & SCOUR LEVEL:

As per hydraulic calculation

- Maximum scour level around Pier locations = (+) 214.840 M
- Around Abutment location no scour is expected since it is 40m away from bank. Also around BH-A1(01) location weathered rock is encountered right from the beginning of the borehole.

Maximum scour level (214.840M) falls inside the rock layer around BH-P2(03) location. No scour is expected inside rock layer. Therefore we assume that, upto top of rock layer scour will take place.

Hence design Maximum scour level around BH-P2(03) = 219.492–3.60 = 215.892 M.

4. USE OF OPEN FOUNDATION:

Since scour is not expected around abutment location A2 it is suggested to place the foundation at a depth 3m below existing ground level.

4.1 <u>AROUND BH-A2(04) Location</u>

The founding level falls inside stratum I i.e. stiff to very stiff silty clay layer.

Average "N" in this layer = 21, So, estimated cohesion from N value =1.00 kg/sqcm[†].

From laboratory TRSH-UU test results average C = 1.19 kg/sqcm & Φ = 6°

Considering the predominant soil condition use C = 1.00 kg/sqcm & Φ = 0°

4.1.1. Evaluation of strength & deformation parameters:

STRATUM – I

Total soil modulus, E_s = 4.4 x N = 92.40 kg/sqcm

[Ref. to "History of Soil penetration testing" by B. B. Broms & N. Flodin in "Penetration Testing 1988", ISPOT-1: vol.1, p – 185]

Undrained Young's modulus, E_u = K x C = 500 x 1.00= 500 kg/sqcm

Again, $1/E_s = 1/E_u + 1/E_d$ giving drained young's modulus, E_d = 113.35 kg/sqcm

Now, we have, $E_d = E_u/3$ = 166.67 kg/sqcm

[Refer to "Cone Penetration Testing" by A.C.Meigh, pp. No. – 53]

Considering the above, let us use E_d = 140 kg/sqcm

From E_d, $m_{vc} = 1/G.E_d$ =0.0119 sqcm/kg [Geological Factor, G = 0.60 & μ = 0.35]

Again from SPT "N", m_{vc} = 1/5N = 0.0095 sqcm/kg

[Refer to "Standard Penetration Test, State-of-the-art-Report" by Ivan K. Nixon in "Penetration testing 1" Edited by A.Verrujt, F.L.beringen & E.H.De Leeuw, pp. No. 11]

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† **Relation between SPT "N" and Shear Strength**

Widely used relationship is due to Terzagi and Peck recommending C = N/16

However, it has been seen over the years with stiffness in clay the shear strength does not increase as rapidly as proposed by Terzagi.

Our experience also shows that for clays at medium to higher depth, the above relation does not hold good.

For Static Cone Penetration Tests, the recommendations for cone factor N_k generally are

$C = q_c/N_k$ where C = Cohesion in kg/sqcm and q_c = Cone resistance in Static Cone and

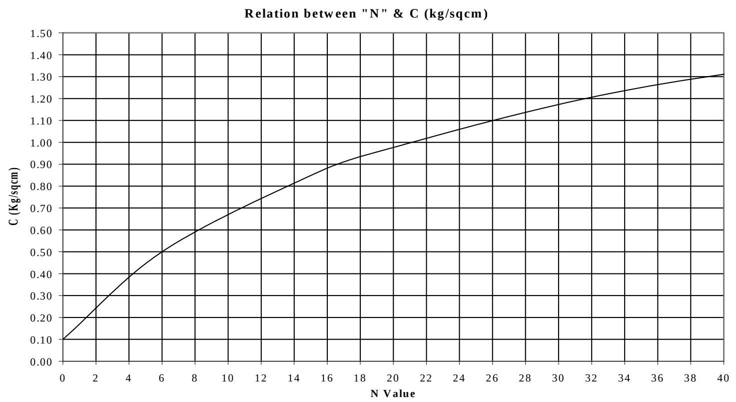
N_k = 17 , 21 & 27 for normally consolidated clay, partly over consolidated clay and heavily over consolidated clay respectively

[Ref. Meigh, A.C (1987) : Cone Penetration Testing Methods and Interpretation, Butterworths, London, pp-43-47]

Taveres, A.X [Penetration Testing 1988, ISOPT-1, Volume-I, J.De Ruiter Editor, pp-375-379] has shown very clearly that a better correlation can be obtained with stiffness of the clay. From his experimental results he obtained,

Range of SPT 'N'	K = N/C
N < 10	12.50
10 < N < 20	14.20
20 < N < 30	16.25
30 < N < 40	20.00

Over the years on the basis of the laboratory test results we have been using the following relations. However, for "N" value greater than 40, we use C = N/27



Now, let us consider the m_{vc} value for the pressure range between 0.50 to 2.00 kg/sqcm

Sample No.	0.50 – 1.00kg/sqcm	1.00 – 2.00kg/sqcm
BH04 / UDS02	0.0207	0.0159
Average weighted m_{vc}, sqcm/kg over entire pressure range	**0.0175**	

Giving more weightage to the laboratory test results,

Use m_{vc} = [2 x 0.0175+0.0095+0.0119]/4=0.0141 sqcm/kg

STRATUM – II

This is a boulder layer. Use lowest possible Young's modulus E_s = 750 kg/sqcm [corresponding to very dense sand layer]

4.1.2. DETERMINATION OF BEARING CAPACITY:

The Net Ultimate Bearing Capacity is given as:

$$q_{nu} = C.N_c.S_c.D_c + q.N_q.S_q.D_q + 0.5\gamma.B.N_\gamma.S_\gamma.D_\gamma - q$$

Where,

N_c, N_q and N_γ are bearing capacity factors,
S_c, S_q and S_γ are shape factors,
D_c, D_q and D_γ are depth factors,

And

C = Cohesion,
q = Overburden pressure,
B = Width of foundation,
γ = Effective density below foundation.

For (7.50mx14.00m) Isolated Footing

Cohesion, C = 10.00 t/sqm
Using ϕ = 0 degree, the bearing capacity factors are:

N_c = 5.14
N_q = 1.00
N_γ = 0.00

Use,

Depth of Foundation = D_f = 3.00 M(Below Existing Ground level)
Width of Foundation = B = 7.50 M
Length of Foundation = L = 14.00 M

Overburden Pressure = q =3.000 (Depth) x 0.90 (Submerged density) = 2.70 t/sqm (Assuming the ground water table is flushing with the ground level)

The Shape factors are [IS:6403 - 1981]

S_c = 1.11 S_q = 1.11 S_γ = 0.79

The Depth factors are [IS:6403 - 1981]

D_c = 1.08 D_q = 1.00 D_γ = 1.00

Computed Net Ultimate Bearing Capacity = 61.80 t/sqm

Using a factor of safety of 2.5, Net Safe Bearing Capacity = 24.72 t/sqm

The above bearing capacity should be checked against settlement criteria and this is shown below.

4.1.3. Settlement Calculation:

With reference to the above, the settlement is calculated and is presented below.‡

Depth of Foundation = 3.00 m below EGL
Foundation Width, B = 7.50 m
Foundation Length, L = 14.00 m
Net base Pressure, P = 2.40 kg/sqcm

Strata	From (M)	To (M)	Thickness of compressible layer (M)	Mid depth below Founding level, (M)	ΔP	Young's Modulus, kg/sqcm	Mvc, sqcm/kg	G	Si (cm)	Sc (cm)	St (cm)
I	3.00	5.00	2.00	1.00	1.976	500.00	0.0120	0.60	0.79	2.85	3.64
	5.00	7.00	2.00	3.00	1.412	500.00	0.0120	0.60	0.56	2.03	2.60
	7.00	9.10	2.10	5.05	1.054	500.00	0.0120	0.60	0.44	1.59	2.04
II	9.10	12.00	2.90	7.55	0.777	750.00	0.0000	1.00	0.30	0.00	0.30
	12.00	16.00	4.00	11.00	0.545	750.00	0.0000	1.00	0.29	0.00	0.29
	16.00	19.00	3.00	14.50	0.402	750.00	0.0000	1.00	0.16	0.00	0.16
								Total =	2.55	6.47	9.02

Fox's Depth correction Factor = 0.92
Hence, corrected Total Settlement = 8.32

Use a net allowable bearing capacity of 24 t/m^2

‡ **Use of m_{vc} value to Calculate Settlement:**

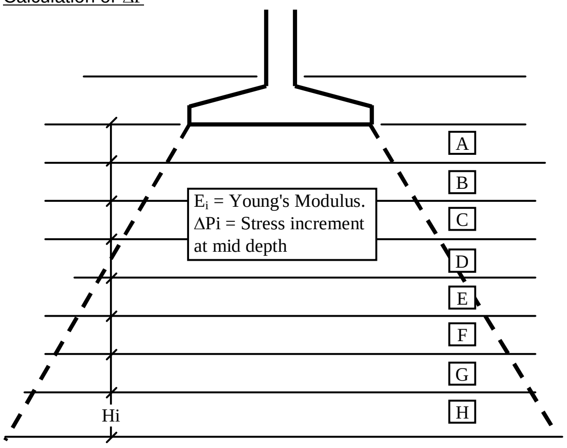
In our report we have calculated the settlement of soil assuming trapezoidal distribution of stress. In case of sandy soil, we are having only immediate settlement. Now the problem starts when the soil becomes silty clay / clayey silt.

There is no definite and particular formula to calculate settlement of this type of soil. The m_v value we get from the consolidation test results includes immediate part also. It is seen that for the present case about 20-31% of compression takes place just after application of test load. Hence, judicious use of m_{vc} (the consolidation part only and not the m_v) should be used. Thus m_v value obtained from the e-logP curves should not be used blindly.

It is seen that the settlement determined as per Clause 9.2.2.3 for computation as per IS 8009 Part-1 leads to more or less same value as calculated by us. Moreover, for sites like the present project where we have large numbers of consolidation test results, this type of approach is very very tedious. Since, this e-logP curve is totally undefined and can not be framed in any particular equation, it is not at all suitable for any computer programming. Thus determination of settlement through this procedure is not feasible for a day to day work basis job.

m_{vc} & Δp approach is very straight forward and the equation is similar to normal settlement calculation by theory of elasticity ($m_{vc} \propto 1/E$). We are in this field for the last 25 years and using this approach (m_{vc} & Δp) satisfactorily to calculate settlement for almost all of our Clients.

Calculation of ΔP



Note:
1) Ei = Young's Modulus values and Gi = Geological factor of the i[th] layer / sub layer,
2) ΔPi = Stress increment at the mid depth of the layer / sub layer due to dispersion of the load applied at the founding level using trapezoidal distribution of load (1H:2V)
3) Hi = Thickness of each layer / sub layer.
4) A thick layer is sometimes divided in to several sub layers (like A, B, C etc.) for more accurate prediction of settlement.
5) Si(immediate) = ΔPi x Hi / Ei
6) Si(consolidation) = ΔPi x Hi x m_{vc}i x Gi

Geological Factor: This is nothing but the consolidation settlement reduction factor, λ as given in IS 8009, Part I. This is termed as geological factor by M. J. Tomlinson as the value is dependent on geology and pore pressure history of the soil.

4.2 AROUND BH-A1 (01)

The founding level falls inside weathered rock layer

Based on RMR Method:

Calculated RMR = 08 as per IS: 13365(Part1)-1998, Annex B

a) The minimum value of point load strength index in this layer is taken as 0.85 kg/sqcm, corresponding rating = 00
b) RQD value, a minimum of 10% is considered as per codal provision (IS 13365, Part 1), corresponding rating = 03
c) Spacing of discontinuity taken as close. corresponding rating = 05
d) Condition of discontinuity considered as *5mm thick soft gauge 5mm wide continuous discontinuity,* corresponding rating = 00
e) Ground water condition taken as Wet. corresponding rating = 07
f) Dip Angle Joint Orientation taken as fair (based on geological formation), corresponding rating = (–)7

So, RMR = 00+03+05+00+07+(–)7 = 08

So, from Table-3 of IS: 12070 (Amendment 1, Nov. 2008) q_{ns} = 36 t/sqm (by interpolation) for 12mm settlement.

4.3. RECOMMENDATIONS:

Based on the above calculation, the following bearing capacity values are recommended. It is suggested to go for foundation with scour protection.

Borehole Location	Depth of foundation (m) below Existing Ground Level	Founding Layer	Recommended Net Allowable Bearing Capacity (t/sqm)
BH – A2 (04)	2.00	Very stiff silty Clay	23
	3.00		24
	4.00		25
BH – A1 (01)	2.00-4.00	Weathered Rock	35

Note:

1) Limiting settlement inside rock is considered as 12mm irrespective of foundation type.
2) The above bearing capacity inside rock is based on limiting the settlement; it should not be increased if the foundation is embedded further into the rock unless the rock quality improves.
3) In case any loose pocket is observed at the founding level, then the same should be excavated out and the same shall be filled up with PCC upto the founding level..

5. USE OF R.C. BORED PILE:

Around BH-P2(03) and BH-P1(02) location deep foundation in the form of pile is recommended. Bored cast in-situ piles are preferred due to typical geological formation, availability of construction agencies, ease of construction and less sound pollution. Such piles may be placed at a suitable depth below Maximum scour level / inside rock layer depending on structural requirement/ relevant codal provisions.

While calculating the pile capacity, let us assume that

a) Assumed Grade of Concrete = M35

b) Diameter of pile used = 1200mm.

Pile capacity is determined as per the following two Approaches:

5.1 DETERMINATION OF VERTICAL PILE CAPACITY

AROUND BH-P1(02) LOCATION :

<u>Approach – 1</u>

As per Appendix-5, Clause 709.3.1, Method 2 of IRC 78-2014

<u>Pile Capacity Calculation around BH-P2 (1200mm diameter Pile)</u>

Length of pile inside rock layer = 10m i.e. length of socket = 9.70m [Neglecting 0.30m as seating drive]

Refer to IRC: 78 - 2014

From the subsoil profile, it is seen that rock is available at 3.50 m below EGL

Let us neglect the soil upto top of rock for pile capacity calculation

Ultimate Pile capacity, $Q_u = R_e + R_{af} = C_{ub}.N_c.A_b + C_{us}.A_s$

Where, R_e = Ultimate End Bearing & R_{af} = Ultimate side socket shear

C_{ub} = Average shear strength below base of Pile based on "N" value

C_{us} = Ultimate shear strength along socket length based on "N" value

A_b = Base area of Pile

A_s = Socket side area

N_c = 9

<u>Calculation of End Bearing</u>

From field data, extrapolated "N" = 675, restricted to 300

By interpolation C_{ub} = 330t/sqm

Hence, ultimate base pressure = 330 x 9 = 2970t/sqm

Using a FOS of 3, safe base pressure = 2970 / 3 = 990t/sqm

As per code, restrict the safe base pressure to 500t/sqm

Pile Base Area, Ab = 1.131sqm

Hence, Safe End Resistance = 500 x 1.131 = 565.5T

Calculation of Socket Side Resistance

From field data, extrapolated "N" = 675, restricted to 300

By interpolation C_{us} = 330t/sqm

Restricting the C_{us} value to 300t/sqm assuming grade of concrete as M35

Length of Socket, L = 10m

Neglecting the top 300mm, effective length of socket, l = 7.20m
maximum value as per code

So, socket area, As = 27.143sqm

Hence Ultimate Socket Side Resistance = 300 x 27.143 = 8142.9T

Using a FOS of 6, safe socket side resistance = 8142.9 / 6 = 1357.15T

Hence total pile capacity = 565.5 + 1357.15 = 1922.65T

Maximum permissible stress in compression = 9 N/sqmm = 900 T/sqm for M35 grade concrete as per [Table-21, IS 456:2000, Page 81]

Pile area = 1.131 sqm

Maximum load permissible on pile = 1018 Ton

Approach – 2

As per Clause B-8 of IS 2911 (Part 1/ Sec 2) : 2010

As per IS 2911(Part 1/Sec 2): 2010, (Annex-B, cl.B-8) the allowable load on the pile,

$Q_a = c_{u1}N_c\ (\pi B^2/4F_s) + \alpha\ c_{u2}\ (\pi BL/F_s)$

Where,

C_{u1} = Shear strength of rock below the base of the pile, in kN/m^2

N_c = Bearing capacity factor taken as 9

F_s = Factor of safety usually taken as 3

α = 0.9 (recommended value)

C_{u2} = Average shear strength of rock in the socketed length of pile, in kN/m^2

B = Minimum width of pile shaft (diameter in case of circular piles), in m

L = socket length of pile, in m

Considering socket length, L = 9.70m [Neglecting 0.30m as seating drive]

D = diameter of pile in m = 1.20m

Use design C_{u1} = 400 kN/m^2 = 40t/m^2 and design C_{u2} = 400 kN/m^2 = 40t/m^2

Therefore, $Q_a = c_{u1}N_c\ (\pi B^2/4F_s) + \alpha\ c_{u2}\ (\pi BL/F_s)$

$= [40 \times 9 \times (3.14159 \times 1.20^2)/(4 \times 3) + 0.90 \times 40 \times (3.14159 \times 1.20 \times 9.70) / 3]$ T

= 574.53 T

However due to uncertain behavior of rock and wide variation of rock quality the vertical pile capacity value is restricted to 400 T

5.3. HORIZONTAL PILE CAPACITY

For 1200mm diameter Pile

Use design cohesion C = 4 kg/sqcm for lateral pile capacity calculation (As used above)

Refer to IS : 2911 (Part I/Sec 2) - 2010, Appendix - C
Modulus of Subgrade reaction, k_1 = 14.40 kg/cucm corresponding to Cohesion = 4.00 kg/sqcm
Now, K = (k_1/1.5) x (30/D) which is coming as 2.40 kg/cucm [D = Pile dia in cm]
Stiffness factor, R = $[EI/KD]^{1/4}$
Now, I = 10.18 x 10^6 cm^4 [for 1200mm dia pile]
E = 5000 x $(f_{ck})^{0.5}$ = 5000 x $(35)^{0.5}$ = 29580 N/sqmm = 2.96 x 10^5 kg/sqcm
Hence, R = 319.76 cm
From Graph (Fig.4), L_f = 1.95 x R = 623.54 cm [Assuming Fixed Head Piles]
Pile Head deflection, Y = H x L_f^3 / 12EI = 0.0671mm for 1T load
So, for 12mm horizontal deflection at maximum scour level Horizontal force at pile head is given by , H = 178.85T, restricted to 40 T
Now, Moment = [H x L_f/2] = [1 x 6.24/2] = 3.12t-m per T of thrust
The Reduction Factor for computation of Maximum Moment in Pile, m = 0.70
So, the corrected actual moment, M = 3.12 x 0.70 = 2.18t-m per T of thrust

5.4. RECOMMENDATION:

With reference to the above and considering subsoil condition, the recommended pile capacity values is presented below.

Pile Diameter = 1200mm

Borehole Location	Approximate Length of Pile* (m)	Recommended Vertical Pile Capacity (T)	Lateral Pile Capacity
BH-P2 (03)	11.50	400	H = 40T & 40 T for Overhang length of 0m & 2m Respectively. Corresponding M = 2.18 & 3.21t-m/t of thrust and L_f = 6.23m and 5.68m respectively.
BH-P1 (02)	14.50	400	

Note :

a. Pile load test should be conducted to ascertain the pile capacity as per IS: 2911,Part IV.
b. *Bottom of Pile Cap is considered 2m depth below Existing ground level.

6. SUMMARY & CONCLUSIONS

Based on the field and laboratory test results and the foregoing discussion, the following are summarised.

1. The subsoils in general are of good quality. It is characterized by a stiff to very stiff silty clay layer at top around BH-A2 location only. A boulder layer is encountered thereafter. Underlying the above a weathered rock layer of is encountered and that layer continued up to the terminating depth of all the boreholes except BH-04(A2) location where boulder layer continues upto terminating depth.

2. **Foundation System around BH-A2 (04) & A1 (01) location:**

   Use of Open Foundation:

   a) We have boulder deposit from 9.10m below EGL which continues upto the borehole terminating depth. Piling through the boulder layer is very difficult & thus avoided.

   b) In A1 location weathered rock layer is encountered right from the beginning of the borehole.

   c) Under this circumstances it is suggested to go for open foundation.

   d) The determination of bearing capacity and recommendations are presented in the previous section. However, for routine design, this is further presented below.

Borehole Location	Depth of foundation (m) below Existing Ground Level	Founding Layer	Recommended Net Allowable Bearing Capacity (t/sqm)
BH – A2 (04)	2.00	Very stiff silty Clay	23
	3.00		24
	4.00		25
BH – A1 (01)	2.00-4.00	Weathered Rock	35

Note:

1) Limiting settlement inside rock is considered as 12mm irrespective of foundation type.
2) The above bearing capacity inside rock is based on limiting the settlement; it should not be increased if the foundation is embedded further into the rock unless the rock quality improves.
3) In case any loose pocket is observed at the founding level, then the same should be excavated out and the same shall be filled up with PCC upto the founding level..
4) Foundation will be scour protected.

3. **Foundation System around BH-P2 (03) & P1(03)location:**

Use of Pile Foundation:

a) Deep foundation in form R.C. Bored pile is recommended.

b) Bored cast-in-situ piles are preferred due to availability of construction agencies, ease of construction, less sound pollution and typical geological formation.

c) The recommended pile capacity value is presented below.

Pile Diameter = 1200mm & Grade of concrete = M35

Borehole Location	Approximate Length of Pile* (m)	Recommended Vertical Pile Capacity (T)	Lateral Pile Capacity
BH-P2 (03)	11.50	400	H = 40T & 40 T for Overhang length of 0m & 2m Respectively. Corresponding M = 2.18 & 3.21t-m/t of thrust and L_f = 6.23m and 5.68m respectively.
BH-P1 (02)	14.50	400	

Note :

i. Pile load test should be conducted to ascertain the pile capacity as per IS: 2911,Part IV.

ii. *Bottom of Pile Cap is considered 2m depth below Existing ground level.

DESCRIPTION OF FEW CORE SAMPLES

BH: A1 (01)

R_{01} (00.00-01.00)m : Moderately weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{03} (02.00-03.00)m : Moderately weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{06} (05.00-06.00)m : Slightly weathered, dark grey, very fine grains are moderately compacted and thinly bedded, soft and weak **Shale**.

R_{09} (08.00-09.00)m : Slightly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{11} (10.00-11.00)m : Slightly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{13} (12.00-13.00)m : Slightly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

BH: A2 (04)

R_{02}(10.00-11.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{03} (11.00-12.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{05} (13.00-14.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

BH: P1 (02)

R_{03}(02.00-03.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{04}(03.00-04.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{06}(05.00-06.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{10}(09.00-10.00)m : Steel grey, fine grains are densely compacted, hard and strong **Siltstone** boulder.

R_{11}(10.00-11.00)m : Moderately weathered, dark grey, very fine grains are moderately compacted and thinly bedded, soft and weak **Shale**.

R_{14}(13.00-14.00)m : Moderately weathered, dark grey, very fine grains are moderately compacted and thinly bedded, soft and weak **Shale**.

BH: P2 (03)

R_{03}(04.00-05.00)m : Highly weathered, steel grey with brown patches, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{04} (05.00-06.00)m : Highly weathered, steel grey with brown patches, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{06} (07.00-08.00)m **:** Highly weathered, steel grey with brown patches, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{08} (09.00-10.00)m **:** Highly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{12} (13.00-14.00)m **:** Highly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

R_{16} (17.00-18.00)m **:** Highly weathered, steel grey, fine grains are moderately compacted and bedded, medium hard and weak **Siltstone**.

SUMMARY OF FIELD AND LABORATORY TEST RESULTS

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.
Bore Hole No. : 3 (P2) **Location :** E=550361.231 / N=2748453.528

Commencement Date : 10/1/2018 **Completion Date :** 15/01/2018

Level of Ground : 219.492 M **Standing Water Level :** 3.90 M

Job No : 4047 **Sheet No :**

Elevation in Metre	Depth in Mete below reference: From	Depth in Mete below reference: To	Depth of Sample below reference level	Sample Ref. No.	Level of Water table/L.W.L.	S.P.T. blows per 30cm: Depth in Metre	S.P.T. blows per 30cm: Value	Percent Core Recovery	Percent RQD	Size of Hole (mm)	Symbolic representation	Visual Description of Soil	Type of test conducted at Laboratory	%Gravel>72mm	%Sand 2.0-0.06mm	% Silt 0.06-0.002mm	% Clay<0.002mm	Dry Density in gms/cm^3	Specific Gravity	Void Ratio	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Shearing Strength Characteristics: Cohesion C (kg/cm^2)	Shearing Strength Characteristics: Angle of Shearing Resistance in Deg.	pH	Cl (%)	SO_4 (%)	Unconfined Compressive Strength of Rock (Kg/cm^2)	Remarks
219.4920					215.592					150/NX		0.00M																			
218.9920		0.50	0.50	DS-01								Loose, brownish grey, silty sand with decomposed rock dust. Obs. Mica & clay binders.			74	26 #															
218.4920	1.00	1.45	1.00	SPT-01		1.00	10								62	37	1		2.73												
217.7920		1.70	1.70	DS-02								2.00M																			
217.6420	1.85	1.88	1.85	*SPT-02		1.85	R																								
217.4920	2.00	2.02	2.00	*SPT-03		2.00	R					Various coloured, different size of boulders, with intermediate voids filled up by sand.																			
219.4920	2.00	3.00		R1				25	0																						
219.4920	3.00	4.00		R2				33	0			4.00M																			
219.4920	4.00	5.00		R3				25	14									2.496	3.573		1.20									--/--	
219.4920	5.00	6.00		R4				29	20									2.515	2.867		0.80									--/--	
219.4920	6.00	7.00		R5				23	0																						
219.4920	7.00	8.00		R6				25	0									2.721	2.791		0.52										
219.4920	8.00	9.00		R7				26	0																						
219.4920	9.00	10.00		R8				27	0									2.684	2.958		0.71										
219.4920	10.00	11.00		R9				24	0			Highly weathered, light grey to grey, fine grained, highly fractured rocks.																			
219.4920	11.00	12.00		R10				28	0																						
219.4920	12.00	13.00		R11				30	0																						
219.4920	13.00	14.00		R12				29	0									2.545	3.064		0.55										
219.4920	14.00	15.00		R13				31	0																						
219.4920	15.00	16.00		R14				33	0																						
219.4920	16.00	17.00		R15				37	0																						
219.4920	17.00	18.00		R16				35	0									2.515	2.710		0.62										

18.00M

Undisturbed (UDS) Penetrometer (SPT) Disturbed (DS) Water Sample (WS)

*** means sample could not be recovered**

 # means(Silt + clay) %

R = Refusal

@ 1)Note: Chemical Test results for Water Samples for Chloride & Sulphate is given as Mg/Litr & for soil samples SO_4 content is expressed as SO_3.

SUMMARY OF FIELD AND LABORATORY TEST RESULTS

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.
Bore Hole No. : 1 (A1) **Location :** E=550426.572 / N=2748403.922

Commencement Date : 18/01/2018
Completion Date : 24/01/2018

Level of Ground : 237.013 M
Standing Water Level : 18.07 M

Job No : 4047
Sheet No :

Elevation in Metre	Depth in Mete below reference: From	Depth in Mete below reference: To	Depth of Sample below reference level	Sample Ref. No.	Level of Water table/L.W.L.	S.P.T. blows per 30cm: Depth in Metre	S.P.T. blows per 30cm: Value	Percent Core Recovery	Percent RQD	Size of Hole (mm)	Symbolic representation	Visual Description of Soil	Type of test conducted at Laboratory	%Gravel>72mm	%Sand 2.0-0.06mm	% Silt 0.06-0.002mm	% Clay<0.002mm	Dry Density in gms/cm^3	Specific Gravity	Void Ratio	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Shearing Strength Characteristics: Cohesion C (kg/cm^2)	Shearing Strength Characteristics: Angle of Shearing Resistance in Deg.	pH	Cl (%)	SO_4 (%)	Unconfined Compressive Strength of Rock (Kg/cm^2)	Remarks
237.0130					218.943					NX		0.00M																			
236.0130	0.00	1.00	1.00	R1				22	0			Completely to highly weathered, grey to deep grey, fine grained, fractured rock.						2.385	2.722		1.07									P=0.85	
235.0130	1.00	2.00	2.00	R2				25	0																						
234.0130	2.00	3.00	3.00	R3				40	0			2.00M						2.721	2.948		0.66									P=1.02	
233.0130	3.00	4.00	4.00	R4				45	0			Highly to moderately weathered, grey to deep grey, fine grained, fractured rock.																			
232.0130	4.00	5.00	5.00	R5				42	0																						
231.0130	5.00	6.00	6.00	R6				46	0			6.00M						2.486	2.610		0.54									P=0.9	
230.0130	6.00	7.00	7.00	R7				68	0																						
229.0130	7.00	8.00	8.00	R8				72	0																						
228.0130	8.00	9.00	9.00	R9				69	0			Moderately to slightly weathered, grey to deep grey, fine grained, fractured rock.						2.491	2.665		0.66									P=1.1	
227.0130	9.00	10.00	10.00	R10				73	0																						
226.0130	10.00	11.00	11.00	R11				77	0									2.704	3.052		0.83									--/--	
225.0130	11.00	12.00	12.00	R12				62	0																						
224.0130	12.00	13.00	13.00	R13				64	0									2.511	2.686		0.58									P=1.05	
223.0130	13.00	14.00	14.00	R14				67	0			14.00M																			
222.0130	14.00	15.00	15.00	R15				45	0																						
221.0130	15.00	16.00	16.00	R16				42	0			Higly to moderately weathered,steel grey to grey, fine grained, fractured rock.																			
220.0130	16.00	17.00	17.00	R17				45	0																						
219.0130	17.00	18.00	18.00	R18				47	0			18.00M																			

Undisturbed (UDS) Penetrometer (SPT) Disturbed (DS) Water Sample (WS)

*** means sample could not be recovered**
 # means(Silt + clay) %

R = Refusal

@ 1)Note: Chemical Test results for Water Samples for Chloride & Sulphate is given as Mg/Litr & for soil samples SO_4 content is expressed as SO_3.

SUMMARY OF FIELD AND LABORATORY TEST RESULTS

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur. | **Commencement Date :** 6/2/2018 | **Level of Ground :** 219.713 M | **Job No :** 4047

Bore Hole No. : 2 (P1) | **Location :** E=550384.558 / N=2748416.460 | **Completion Date :** 12/2/2018 | **Standing Water Level :** 5.15 M | **Sheet No :**

Elevation in Metre	Depth in Mete below reference: From	Depth in Mete below reference: To	Depth of Sample below reference level	Sample Ref. No.	Level of Water table/L.W.L.	S.P.T. blows per 30cm: Depth in Metre	S.P.T. blows per 30cm: Value	Percent Core Recovery	Percent RQD	Size of Hole (mm)	Symbolic representation	Visual Description of Soil	Type of test conducted at Laboratory	%Gravel>72mm	%Sand 2.0-0.06mm	% Silt 0.06-0.002mm	% Clay<0.002mm	Dry Density in gms/cm^3	Specific Gravity	Void Ratio	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Shearing Strength Characteristics: Cohesion C (kg/cm^2)	Shearing Strength Characteristics: Angle of Shearing Resistance in Deg.	pH	Cl (%)	SO_4 (%)	Unconfined Compressive Strength of Rock (Kg/cm^2)	Remarks
219.7130					214.563					150/NX		0.00M																			
219.5130		0.20	0.20	DS-01								Brownish grey, silty clay with sand mixture. Observed boulders.																			
219.4130	0.30	0.32	0.30	*SPT-01		0.30	R					0.30M																			
219.7130	0.30	1.00		R1				23	0																						
219.7130	1.00	2.00		R2				26	0																						
219.7130	2.00	3.00		R3				25	0									2.265	2.446		0.75									P=6.25	
219.7130	3.00	4.00		R4				28	0									2.221	2.396		0.99									P=7.4	
219.7130	4.00	5.00		R5				29	0			Various coloured, different size of boulders with intermediate space filled with sand.																			
219.7130	5.00	6.00		R6				26	0									2.364	2.456		0.65									--/--	
219.7130	6.00	7.00		R7				28	0																						
219.7130	7.00	8.00		R8				29	0																						
219.7130	8.00	9.00		R9				25	0																						
219.7130	9.00	10.00		R10				26	0									2.521	2.608		0.58									--/--	
219.7130	10.00	11.00		R11				34	0			10.50M						2.571	2.948		0.97									P=0.97	
219.7130	11.00	12.00		R12				37	0																						
219.7130	12.00	13.00		R13				40	0			Highly to moderately weathered, light grey to grey, fine grained, fractured rock.																			
219.7130	13.00	14.00		R14				49	0			14.00M						2.569	2.721		0.67									P=1.1	

Undisturbed (UDS) Penetrometer (SPT) Disturbed (DS) Water Sample (WS)

*** means sample could not be recovered**

 # means(Silt + clay) %

R = Refusal

@ 1)Note: Chemical Test results for Water Samples for Chloride & Sulphate is given as Mg/Litr & for soil samples SO_4 content is expressed as SO_3.

SUMMARY OF FIELD AND LABORATORY TEST RESULTS

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.
Bore Hole No. : 04 (A2) **Location :** E=550319.246 / N=2748466.027

Commencement Date : 23/12/2017
Completion Date : 7/1/2018

Level of Ground : 235.102 M
Standing Water Level : Not found M

Job No : 4047
Sheet No :

Elevation in Metre	Depth in Mete below reference: From	Depth in Mete below reference: To	Depth of Sample below reference level	Sample Ref. No.	Level of Water table/L.W.L.	S.P.T. blows per 30cm: Depth in Metre	S.P.T. blows per 30cm: Value	Percent Core Recovery	Percent RQD	Size of Hole (mm)	Symbolic representation	Visual Description of Soil	Type of test conducted at Laboratory	%Gravel>72mm	%Sand 2.0-0.06mm	% Silt 0.06-0.002mm	% Clay<0.002mm	Dry Density in gms/cm^3	Specific Gravity	Void Ratio	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Shearing Strength Characteristics: Cohesion C (kg/cm^2)	Shearing Strength Characteristics: Angle of Shearing Resistance in Deg.	pH	Cl (%)	SO_4 (%)	Unconfined Compressive Strength of Rock (Kg/cm^2)	Remarks
235.1020										150/NX		0.00M																			
234.6020		0.50	0.50	DS-01																											
234.1020		1.00	1.00	DS-02																											
233.6020	1.50	1.95	1.50	SPT-01		1.50	12								33	62	5		2.70												
232.6020		2.50	2.50	DS-03																											
232.1020	3.00	3.45	3.00	UDS-01								Stiff to Very stiff, brownish grey to yellowish grey, silty clay / clayey silt with sand mixture. Observed rock pieces at lower reaches.	UU		40	53	7	1.68	2.66		17	30	18		1.21	7					
231.6520	3.45	3.90	3.45	SPT-02		3.45	16																								
230.6020		4.50	4.50	DS-04																											
230.1020	5.00	5.45	5.00	SPT-03		5.00	18								42	53	5		2.67												
229.4020		5.70	5.70	DS-05																											
229.1020	6.00	6.45	6.00	UDS-02									UU		44	50	6	1.69	2.68	0.940	19	32	20		1.16	5					
228.6520	6.45	6.90	6.45	SPT-04		6.45	21								41	56	3		2.71												
227.9020		7.20	7.20	DS-06																		33	20								
227.6020	7.50	7.95	7.50	SPT-05		7.50	30							30	26	39	5		2.74												
226.6020		8.50	8.50	DS-07																											
226.3020	8.80	9.00	8.80	SPT-06		8.80	>100							21	44	30	5		2.70												
226.0020	9.10	9.13	9.10	*SPT-07		9.10	R					9.10M																			
235.1020	9.1	10.00		R1				13	0																						
225.1020	10.00	10.02	10.00	*SPT-08		10.00	R																								
235.1020	10.00	11.00		R2				17	0									2.494	2.601		1.65									P=4.25	
224.1020	11.00	11.03	11.00	*SPT-09		11.00	R																								
235.1020	11.00	12.00		R3				15	0									2.542	2.696		0.58									P=5.1	
223.1020	12.00	12.02	12.00	*SPT-10		12.00	R					Various coloured, different size of boulders with intermediate voids filled up with sand.																			
235.1020	12.00	13.00		R4				19	0																						
222.1020	13.00	13.03	13.00	*SPT-11		13.00	R																								
235.1020	13.00	14.00		R5				16	0									2.483	2.596		0.65									P=8.12	
221.1020	14.00	14.02	14.00	*SPT-12		14.00	R																								

SUMMARY OF FIELD AND LABORATORY TEST RESULTS

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.
Bore Hole No. : 04 (A2) **Location :** E=550319.246 / N=2748466.027
Commencement Date : 23/12/2017
Completion Date : 7/1/2018
Level of Ground : 235.102 M
Standing Water Level : Not found M
Job No : 4047
Sheet No :

Elevation in Metre	Depth in Mete below reference		Depth of Sample below reference level	Sample Ref. No.	Level of Water table/L.W.L.	S.P.T. blows per 30cm		Percent Core Recovery	Percent RQD	Size of Hole (mm)	Symbolic representation	Visual Description of Soil	Type of test conducted at Laboratory	%Gravel>72mm	%Sand 2.0-0.06mm	% Silt 0.06-0.002mm	% Clay<0.002mm	Dry Density in gms/cm^3	Specific Gravity	Void Ratio	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Shearing Strength Characteristics		pH	Cl (%)	SO_4 (%)	Unconfined Compressive Strength of Rock (Kg/cm^2)	Remarks
	From	To				Depth in Metre	Value																		Cohesion C (kg/cm^2)	Angle of Shearing Resistance in Deg.					
235.1020	14.00	15.50		R6				18	0			15.50M																			

Undisturbed (UDS) Penetrometer (SPT) Disturbed (DS) Water Sample (WS)

*** means sample could not be recovered**
 # means(Silt + clay) %

R = Refusal

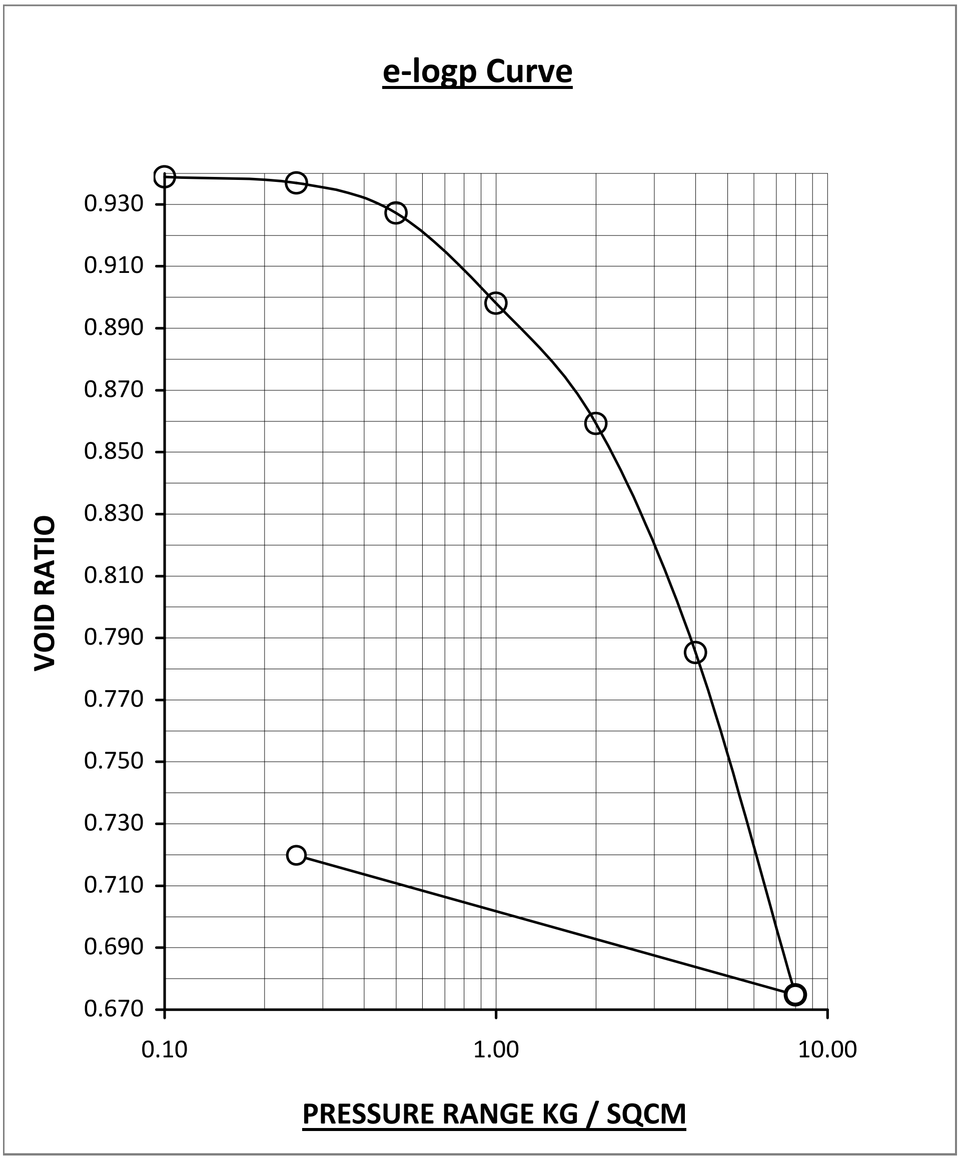
@ 1)Note: Chemical Test results for Water Samples for Chloride & Sulphate is given as Mg/Litr & for soil samples SO_4 content is expressed as SO_3.

Project : Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.	
Job No. 4047	Sheet No.

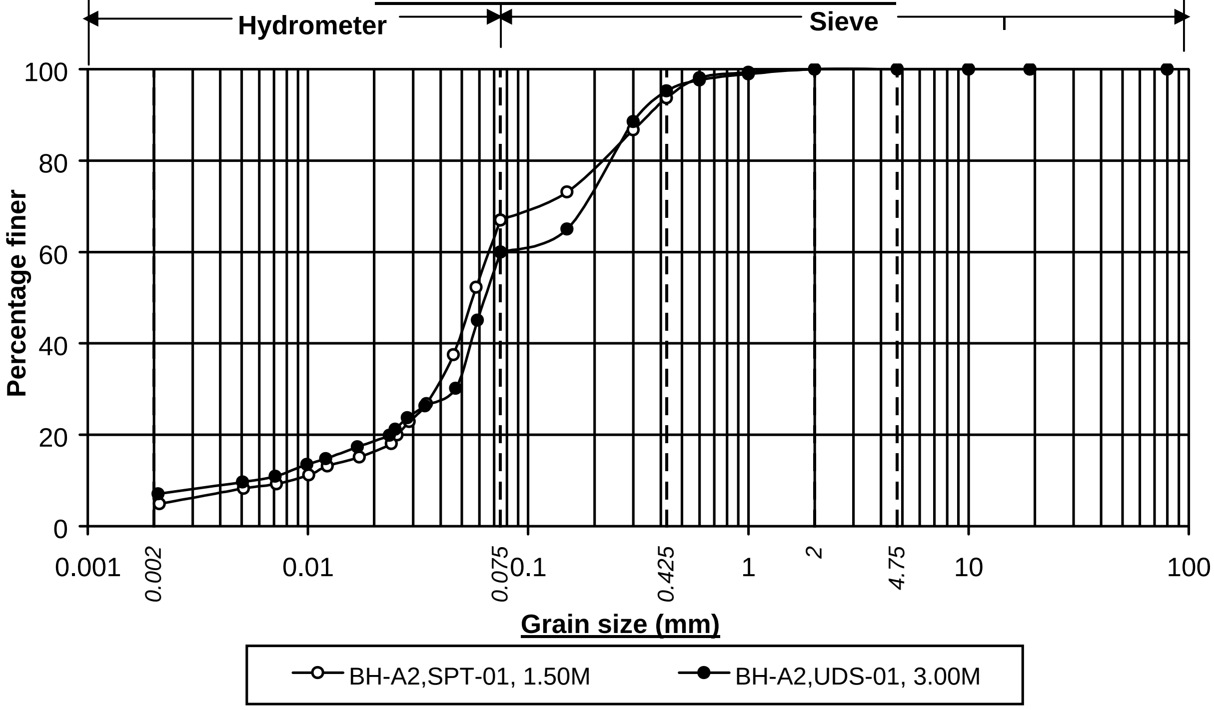
CONSOLIDATION TEST RESULTS

Sample Number:BH-01/UDS-02 Depth :6-6.45 meters
Description :Yellowish grey silty clay with sand mixture.
Water content:Initial=19.7% Final =22.4% Initial Void Ratio =0.94

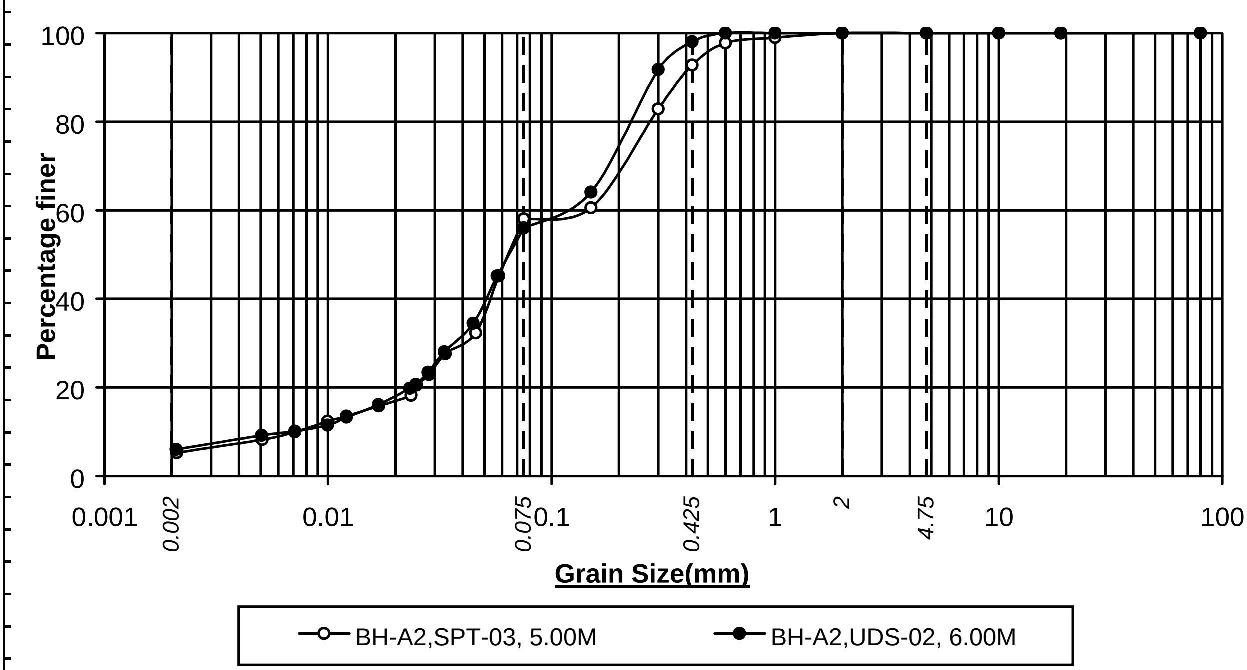
P_1-P_2 Kg/Sqcm	Dial Change	Void Ratio	M_v Sqcm/kg	Comprn %	M_{vc} sqcm/kg	T90 Sec	1000.C_v sqcm/sec
0.00 - 0.10	7	0.939	0.0070				
0.10 - 0.25	10	0.937	0.0066	20.00	0.0053	82.1	10.352
0.25 - 0.50	50	0.927	0.0200	24.00	0.0152	109.4	7.679
0.50 - 1.00	151	0.898	0.0303	31.50	0.0208	103.6	7.781
1.00 - 2.00	201	0.859	0.0205	22.00	0.0160	164.8	4.545
2.00 - 4.00	382	0.785	0.0199	25.80	0.0147	123.4	5.340
4.00 - 8.00	572	0.675	0.0155	30.20	0.0108	202.3	2.590
8.00 - 0.25	233	0.720	0.0035				



GRAIN SIZE DISTRIBUTION CURVES



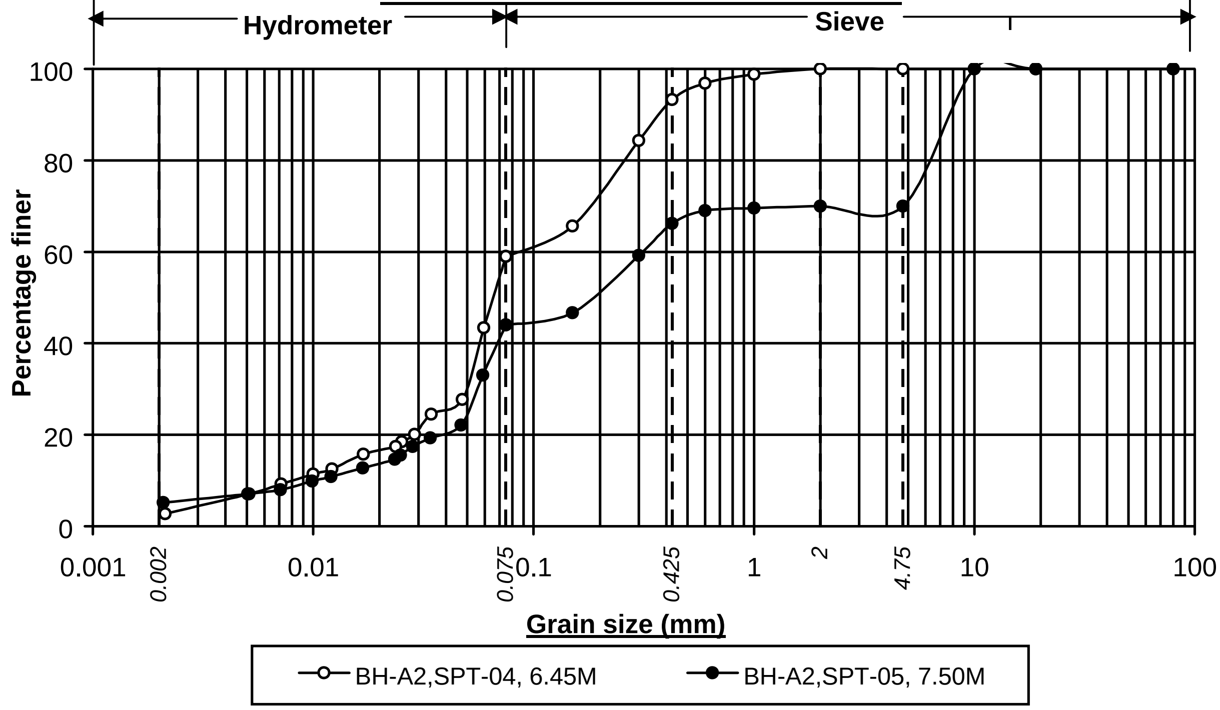
Grain size (mm)	*<0.002*	*0.002-0.075*	*0.075-0.425*	*0.425-2.00*	*2.0-4.75*	**Total sand**	**Weighted mean dia (mm)**	*>4.75*
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**
BH-A2,SPT-01, 1.50M	4.7	62.3	26.7	6.3	0.0	33.0		0.0
BH-A2,UDS-01, 3.00M	6.9	53.1	35.2	4.8	0.0	40.0		0.0



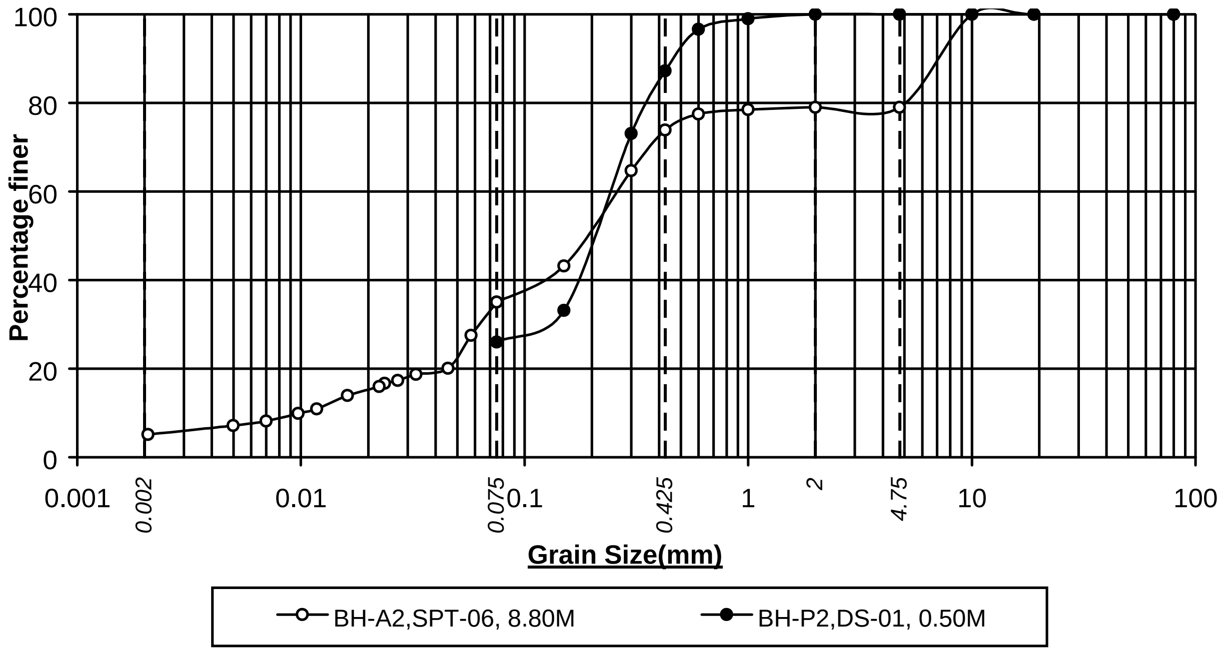
Grain size (mm)	*<0.002*	*0.002-0.075*	*0.075-0.425*	*0.425-2.00*	*2.0-4.75*	**Total sand**	**Weighted mean dia (mm)**	*>4.75*
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**
BH-A2,SPT-03, 5.00M	5.1	52.9	34.8	7.2	0.0	42.0		0.0
BH-A2,UDS-02, 6.00M	5.8	50.2	42.1	1.9	0.0	44.0		0.0

Project:- Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.	**Job No. 4047**	

GRAIN SIZE DISTRIBUTION CURVES



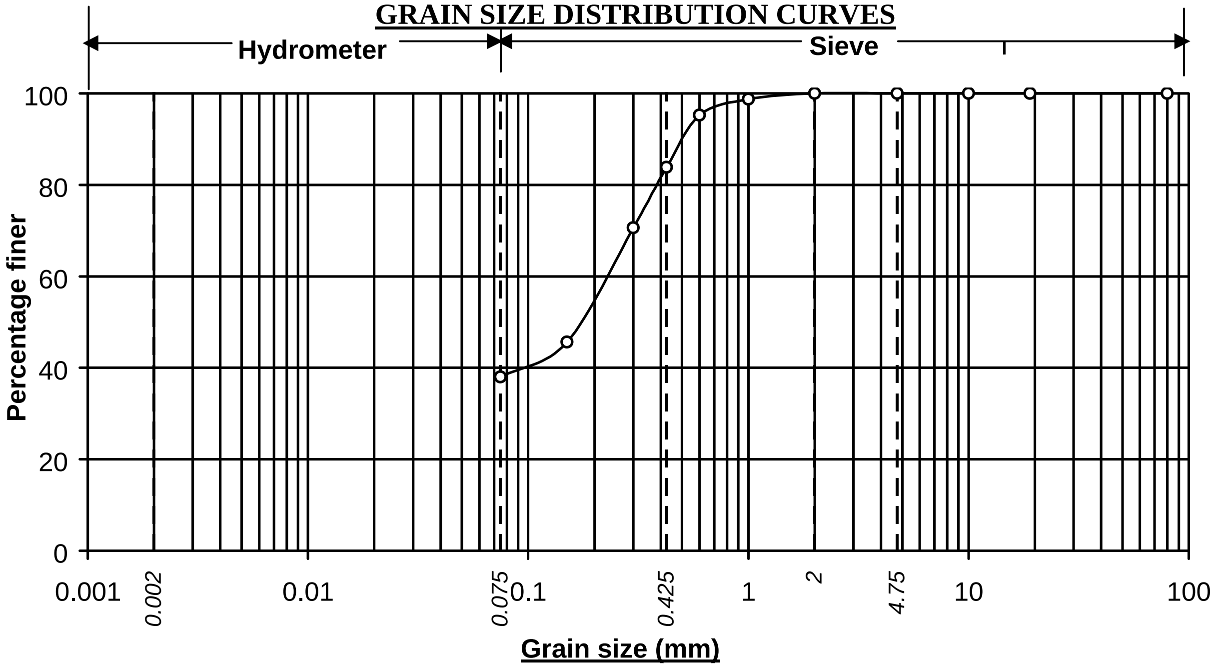
Grain size (mm)	<0.002	0.002-0.075	0.075-0.425	0.425-2.00	2.0-4.75	Total sand	Weighted mean dia (mm)	>4.75
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**
BH-A2,SPT-04, 6.45M	2.4	56.6	34.3	6.7	0.0	41.0		0.0
BH-A2,SPT-05, 7.50M	5.1	38.9	22.2	3.8	0.0	26.0		30.0



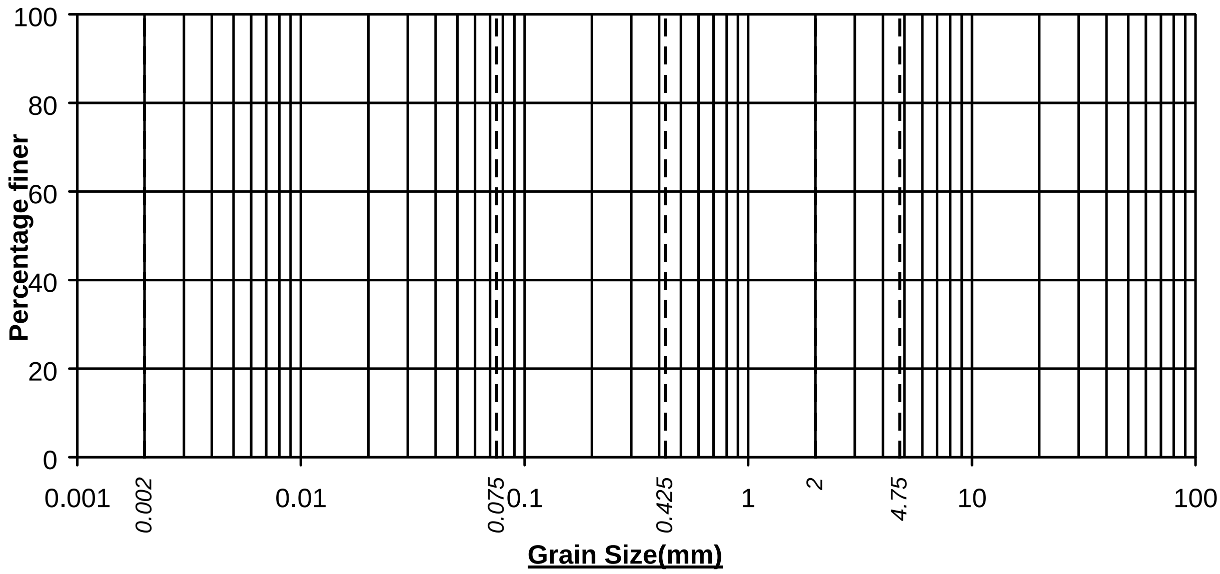
Grain size (mm)	<0.002	0.002-0.075	0.075-0.425	0.425-2.00	2.0-4.75	Total sand	Weighted mean dia (mm)	>4.75
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**
BH-A2,SPT-06, 8.80M	5.0	30.0	38.9	5.1	0.0	44.0		21.0
BH-P2,DS-01, 0.50M		26.0	61.2	12.8	0.0	74.0	0.244	0.0

Project:- Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur. | **Job No. 4047**

GRAIN SIZE DISTRIBUTION CURVES



Grain size (mm)	<0.002	0.002-0.075	0.075-0.425	0.425-2.00	2.0-4.75	Total sand	Weighted mean dia (mm)	>4.75
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**
BH-P2,SPT-01, 1.00M		38.0	45.9	16.1	0.0	62.0		0.0



Grain size (mm)	<0.002	0.002-0.075	0.075-0.425	0.425-2.00	2.0-4.75	Total sand	Weighted mean dia (mm)	>4.75
Sample No.	**Clay (%)**	**Silt (%)**	**Fine sand (%)**	**Medium sand (%)**	**Coarse sand (%)**			**Gravel (%)**

Project:- Geotech Inv. work for Irang Bridge at Taoban Bazar in Manipur.	**Job No. 4047**	

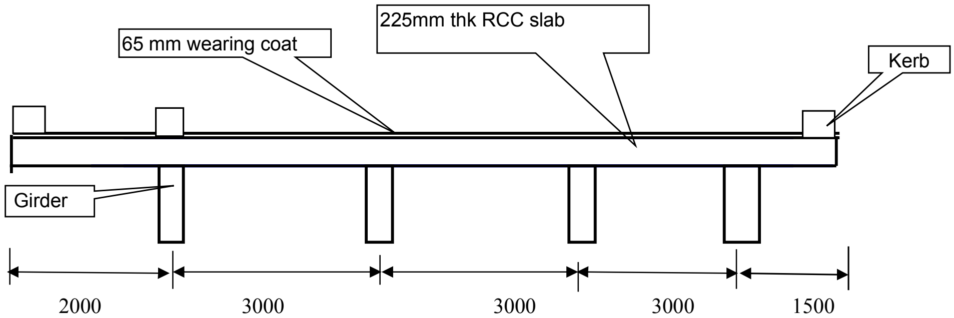
SUPER STRUCTURE DESIGN OF IRANG BRIDGE
CH._95.500 KM

2. LOAD CALCULATION

a. Impact Factor for Deck Slab

Live Load Type	Span L0 = 2.7m	
	RCC	Multiplying Factor
Class A	0.517	1.517
Class 70R Tracked	0.250	1.250

b. Properties of Deck Slab



C/C span between main girder = 3000 mm
Total no.of main girders= 4
Total deck width= 12500
Thickness of longitudinal girder = 300 mm
Thickness of cross girder = 300 mm
C/C of the expansion joint of the bridge= 41000 mm
C/C of the expansion joint to c/c bearing= (1080+20) = 1100 mm
No. of cross girder= 5
Distance between two cross girder = (41000-(2*1100))/(5-1)= 9700 mm
Assume Thickness of Deck Slab (edge)= t_s = 225 mm
Thickness of Deck Slab (centre)= 225 mm
Use clear Cover for Deck slab = 40 mm
Bar dia used= 12 mm
Wearing coat thickness = t_{wc} = 65 mm
No of internal Girder = 2
No of outer Girder = 2
Deck slab is monolithic with main girder and cross girder

Shorter span = l_0 = 3-0.3 2.7 m
Longer span = b = 9.7-0.3 9.4 m

K = (9.4/2.7)= 3.481 > 2
Hence the slab can be designed as one way slab

c. General considerations

Hence the deck slab is designed with the following conditions

i) The slab is continuous over the long girders
ii) The action of the slab is one way
iii) The slab is to be designed for bending moment for critical position of loads with one way bending and punching shear.
iv) For simplicity, slab is considered as 3 span continuously supported on main beam & free at ends

d. Moment & Shear Calculation due to DL, Surfacing & SIDL

Mid Span & Support:

The load calculation for deck slab is being done for unit width.

Dead Load:

Self wt of slab = 0.225*25 = 5.63 KN/m per unit width

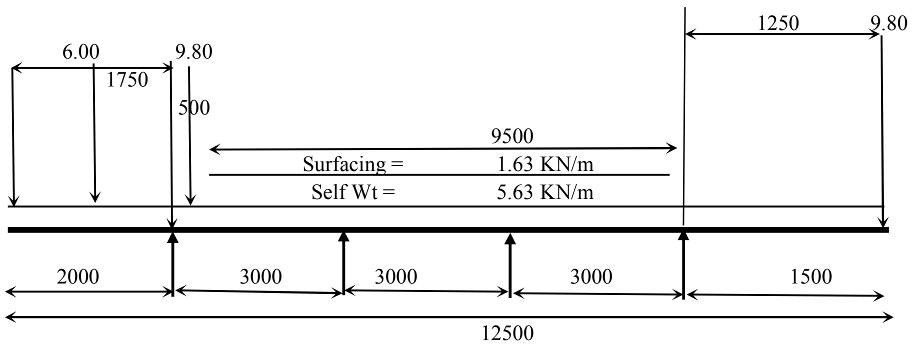
Surfacing:

Self wt of WC = 0.065 x 25 = 1.63 KN/m per unit width

SIDL (Superimposed Dead Load)

	Thickness (M)	Width (M)	L (M)	Density (KN/m^3)	Total (KN)
Kerb (Left)	0.225	0.5	1.00	25.00	2.81
Crash Barrier (Left)	0.392		1.00	25.00	9.80
Crash Barrier (Right)	0.392		1.00	25.00	9.80
Footpath Live load (Left)	-	-	1.00	-	6.00
				Total =	**19.60**

Loading System for Deck Slab



e. Live Load Analysis

Spacing of Girder=	3 m
Width of web=	0.3 m
As the deck slab is continuous over support, effective span= clear span=l_0=	2.7 m
Thickness of wearing course=	0.065 m
b/l_0= 9.4/2.7=	3.481
α=	2.600

b_e= Dispersion width across the span of deck= $\alpha*a*[1-(a/l_0)]+b_1$

Load Details:

Class-A Train vehicle:

Maximum Axle Load=	11.4 T=	114 KN	
Maximum Wheel Load	=	57 KN	
Spacing between axles along the traffic direction	=	1.2 m	
Tyre dimension along the traffic direction =l_w=	=	0.25 m	
Tyre dimension across the traffic direction =b_w=	=	0.5 m	
C/C spacing of wheels across the traffic direction	=	1.8 m	
Edge to edge spacing of two vehicles across the traffic direction	=	1.2 m	
Edge clearence	=	0.15 m	
Impact Factor= 1+[4.5/(6+2.7)]=	=	1.52	***Ref-Cl-208, IRC-6:2010***
l_e= Dispersion width along the span of deck= $b_w+2*(t_s+t_{wc})$=	=	1.08 m	
$b_1=l_w+2*t_{wc}$	=	0.38 m	

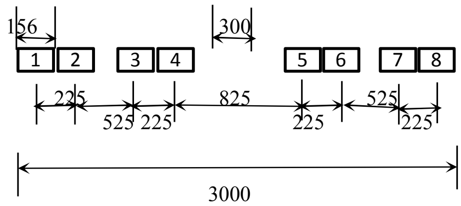
Class 70R:

Tracked

Maximum Axle Load=	70 T=	700 KN	
Maximum Wheel Load	=	350 KN	
Length along the traffic direction	=	4.57 m	
Tyre dimension along the traffic direction =l_w=	=	4.57 m	
Tyre dimension across the traffic direction =b_w=	=	0.84 m	
C/C spacing of wheels across the traffic direction	=	2.06 m	
Edge clearence	=	1.2 m	
Impact Factor=	=	1.25	***Ref-Cl-208, IRC-6:2010***
l_e= Dispersion width along the span of deck= $b_w+2*(t_s+t_{wc})$=	=	1.42 m	
$b_1=l_w+2*t_{wc}$	=	4.7 m	

Special Vehicle

Maximum Axle Load=	9 T=	90 KN
Maximum Wheel Load	=	22.5 KN
Spacing between axles along the traffic direction	=	1.5 m
Tyre dimension along the traffic direction =l_w=	=	0.274
Tyre dimension across the traffic direction =b_w=	=	0.156
l_e= Dispersion width along the span of deck= $b_w+2*(t_s+t_{wc})$=	=	0.736
$b_1=l_w+2*t_{wc}$	=	0.404
Impact Factor	=	1



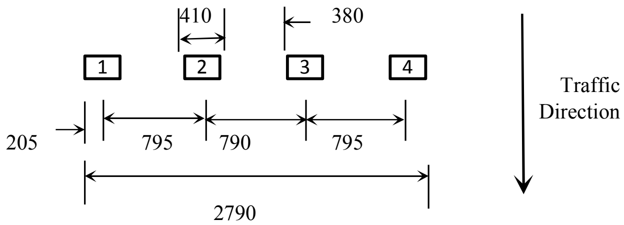
C/C spacing of wheels across the traffic direction between 1 & 2	=	0.225 m
C/C spacing of wheels across the traffic direction between 2 & 3	=	0.525 m
C/C spacing of wheels across the traffic direction between 3 & 4	=	0.225 m
C/C spacing of wheels across the traffic direction between 4 & 5	=	0.825 m
C/C spacing of wheels across the traffic direction between 5 & 6	=	0.225 m
C/C spacing of wheels across the traffic direction between 6 & 7	=	0.525 m
C/C spacing of wheels across the traffic direction between 7 & 8	=	0.225 m

Impact Factor =	=	1	**Ref-Amendment No.1/IRC:6-2014**
l_e= Dispersion width along the span of deck= $b_w+2*(t_s+t_{wc})$=		0.736	

l_{e1}= Dispersion width for SV Load 1 =	(0.225/2+0.404/2)=	0.315 m
l_{e2}= Dispersion width for SV Load 2 =	(0.225/2+0.404/2)=	0.315 m
l_{e3}= Dispersion width for SV Load 3 =	(0.404/2+0.225/2)=	0.315 m
l_{e4}= Dispersion width for SV Load 4 =	(0.225/2+0.404/2)=	0.315 m
l_{e5}= Dispersion width for SV Load 5 =	(0.404/2+0.225/2)=	0.315 m
l_{e6}= Dispersion width for SV Load 6 =	(0.225/2+0.404/2)=	0.315 m
l_{e7}= Dispersion width for SV Load 7 =	(0.404/2+0.225/2)=	0.315 m
l_{e8}= Dispersion width for SV Load 8 =	(0.225/2+0.404/2)=	0.315 m

Class 40R :

Boggie



Wheel Plan

Maximum Axle Load=	20 T=	200 KN
Maximum Wheel Load	=	50 KN
Spacing between axles along the traffic direction	=	1.22 m
Tyre dimension along the traffic direction =l_w=	=	0.36 m
Tyre dimension across the traffic direction =b_w=	=	0.263 m
C/C spacing of wheels across the traffic direction between 1 & 2	=	0.795 m
C/C spacing of wheels across the traffic direction between 2 & 3	=	0.79 m
C/C spacing of wheels across the traffic direction between 3 & 4	=	0.795 m
Edge clearence	=	1.2 m
Impact Factor=	=	1.25 ***Ref-Cl-208, IRC-6:2010***
l_e= Dispersion width along the span of deck= $b_w+2*(t_s+t_{wc})$=	=	0.843 m

As le is greater than the wheel distance, dispersion along the span will be as follows:

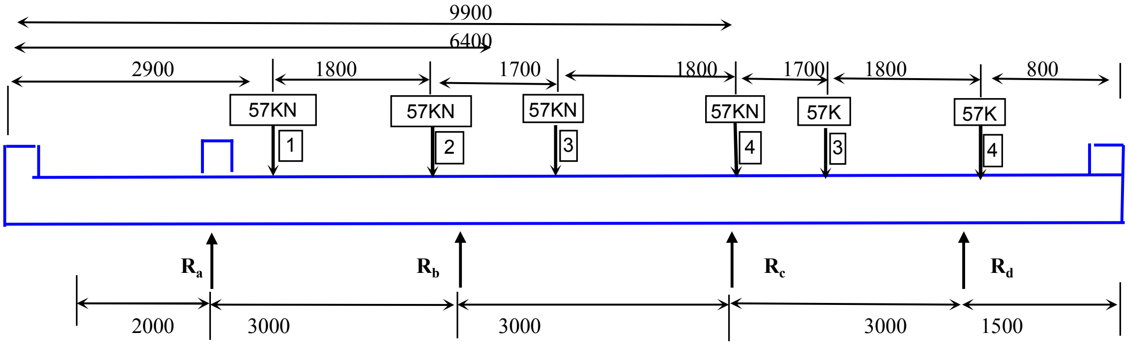
l_{e1}= Dispersion length for load 1=	(0.843/2+0.795/2)=	=	0.819 m
l_{e2}= Dispersion length for load 2=	(0.79/2+0.795/2)=	=	0.7925 m
l_{e3}= Dispersion length for load 3=	(0.79/2+0.795/2)=	=	0.7925 m
l_{e4}= Dispersion length for load 4=	(0.843/2+0.795/2)=	=	0.819 m
$b_1=l_w+2*t_{wc}$		=	0.393 m

CASE-I: Place Class A load 3 Lane load minimum clearance from inner kerb at 1st

Centre of the first wheel of 1st lane load from left edge for first span= 2900 mm

Centre of the first wheel of 2nd lane load from left edge = 6400 mm

Centre of the first wheel of 3nd lane load from left edge = 9900 mm



Further the loads are distributed on certain areas

Thickness of Wearing Coat =	65 mm
α=	2.600
l_0=	2.7 m
b_1=	0.38 m
l_e= Dispersion width along the span of deck=	1.08 m

For load 1

Distance from the support = a = 0.9 m

b_e = Dispersion width across the span of deck = α*a*[1-(a/l_0)]+b_1 = 1.94 m

Therefore, load on 1m span = 44.66 KN

For load 2

Distance from the support = b = 0.3 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.073 m

Therefore, load on 1m span = 80.75 KN

For load 3

Distance from the support = c = 1.4 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.133 m

Therefore, load on 3rd span = 40.62 KN

For load 4

Distance from the support= d = 0.2 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 0.861 m

Therefore, load on 3rd span = 100.63 KN

For load 5

Distance from the support = a = 1.10 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.075 m

Therefore, load on Span-4 = 41.75 KN

For load 6

Distance from the support= a = 0.70 m

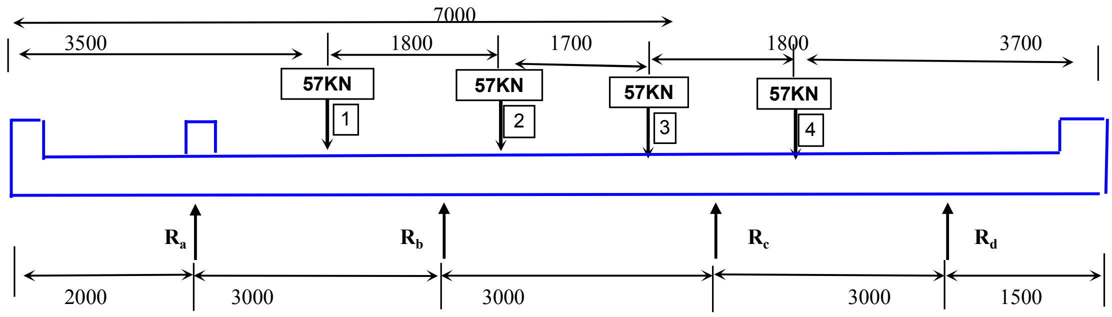
b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.728 m

Therefore, load on Span-5 = 50.14 KN

CASE-II: Place Class A load at the middle of second span

Centre of the first wheel of 1st lane load from left edge for first span= 3500 mm

Centre of the first wheel of 2nd lane load from left edge = 7000 mm



For load 1

Distance from the support= a = 1.5 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.113 m

Therefore, load on 1m span = 41.00 KN

For load 2

Distance from the support= b= 0.3 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.073 m

Therefore, load on 1m span = 80.75 KN

For load 3

Distance from the support= c = 1 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.017 m

Therefore, load on 3rd span = 42.95 KN

For load 4

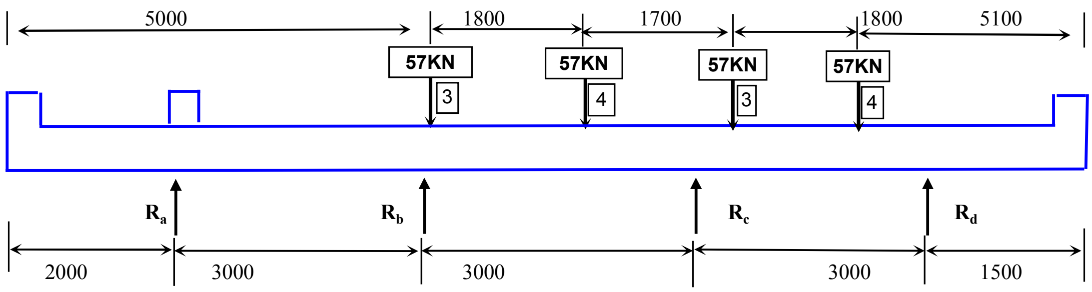
Distance from the support= c = 0.8 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.844 m

Therefore, load on 3rd span = 46.98 KN

CASE-III: Place Class A load at support B

Centre of the first wheel of 1st lane load from left edge for first span= 5000 mm



For load 1

Distance from the support= b = 0 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 0.38 m

Therefore, load on 1m span = 228.00 KN

For load 2

Distance from the support= c = 1.2 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.113 m

Therefore, load on 1m span = 41.00 KN

For load 3

Distance from the support= b =

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 0.50 m

Therefore, load on 1m span 1.439 m

60.21 KN

For load 4

Distance from the support= c = 0.70 m

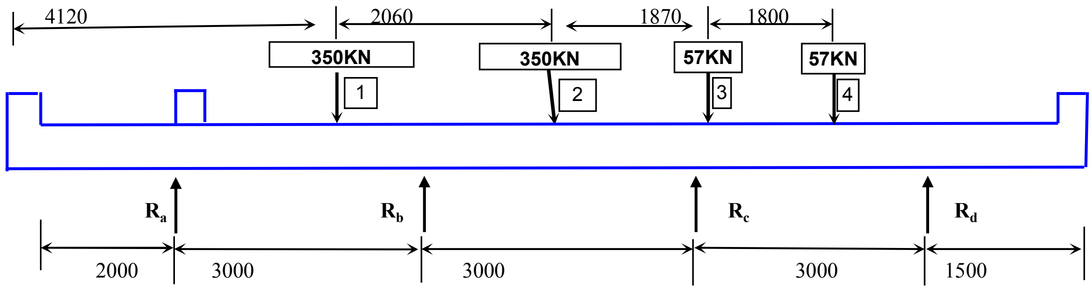
b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.728 m

Therefore, load on 1m span 50.14 KN

CASE-IV: Here Load 1 & Load 2 represents two wheels of Class 70R load. Place load 1 centrally at first span.

Centre of the first wheel of 1st lane load from left edge for first span= 4120 mm

Centre of the first wheel of 2nd lane load from left edge = 6180 mm



Further the loads are distributed on certain areas

Thickness of Wearing Coat =	65 mm
α=	2.600 m
l_0=	2.7 m
b_1 for Class 70R=	4.7 m
l_e= Dispersion width along the span of deck for Class70R=	1.42 m

For load 1

Distance from the support=b= 0.88 m

b_e= Dispersion width across the span of deck= $\alpha*a*[1-(a/l_0)]+b_1$= 6.242 m

Therefore, load on 1m span = 70.09 KN

For load 2

Distance from the support=b= 1.18 m

b_e= Dispersion width across the span of deck= $\alpha*a*[1-(a/l_0)]+b_1$= 6.427 m

Therefore, load on 1m span = 68.07 KN

For load 3

Distance from the support= b = 0.05 m

b_e= Dispersion width across the span of deck= $\alpha*a*[1-(a/l_0)]+b_1$= 0.508 m

Therefore, load on 1m span 170.55 KN

For load 4

Distance from the support= c = 1.15 m

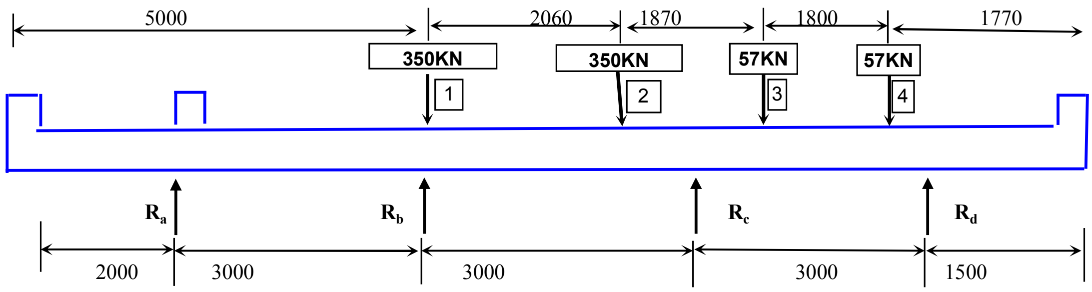
b_e= Dispersion width across the span of deck= $\alpha*a*[1-(a/l_0)]+b_1$= 2.096 m

Therefore, load on 1m span 41.34 KN

CASE-V: Here Load 1 & Load 2 represents two wheels of Class 70R load. Place load 1 centrally at first span.

Centre of the first wheel of 1st lane load from left edge for first span= 5000 mm

Centre of the first wheel of 2nd lane load from left edge = 7060 mm



Further the loads are distributed on certain areas

Thickness of Wearing Coat =	65 mm
α=	2.600 m
l_0=	2.7 m
b_1 for Class 70R=	4.7 m
l_e= Dispersion width along the span of deck for Class70R=	2.06 m

For load 1

Distance from the support=b= 0 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 4.7 m

Therefore, load on 1m span = 93.09 KN

For load 2

Distance from the support=b= 0.94 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 6.293 m

Therefore, load on 1m span = 69.52 KN

For load 3

Distance from the support= b = 0.93 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.965 m

Therefore, load on 1m span 44.09 KN

For load 4

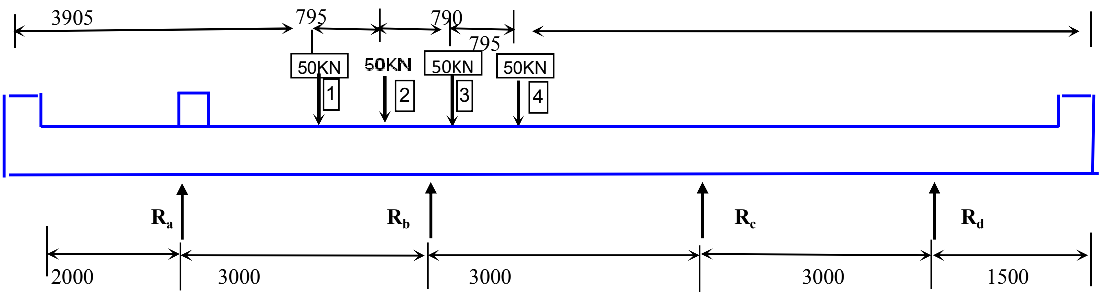
Distance from the support= c = 0.27 m

b_e= Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.012 m

Therefore, load on 1m span 85.61 KN

CASE-VI: Here Load 1 to 4 represents four wheels of 40T Bogie load. Place load 20T axle load at minimum distance from inner kerb of 1st span.

Centre of the first wheel of 1st lane load from left edge for first span= 3905 mm



Further the loads are distributed on certain areas

Thickness of Wearing Coat = 65 mm

α= 2.600 m

l_0= 2.7 m

b_1 for Class 40T= 0.393 m

l_{e1}= Dispersion width along the span of deck for Class40T 1st Load= 0.819 m

l_{e2}= Dispersion width along the span of deck for Class40T 2nd Load= 0.7925 m

l_{e3}= Dispersion width along the span of deck for Class 40T 3rd Load= 0.7925 m

l_{e4}= Dispersion width along the span of deck for Class40T 4th Load= 0.819 m

For load 1

Distance from the support=b= 1.095 m

Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.085 m >1.22m. Dispersion will be restricted

be= = 1.6525 m

Therefore, load on 1m span = 37.82 KN

For load 2

Distance from the support=b= 0.3 m

Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.086 m <1.22m. Dispersion will not be

be= = 1.086 m

Therefore, load on 1m span = 57.55 KN

For load 3

Distance from the support=b= 0.49 m

Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 1.436 m >1.22m. Dispersion will be restricted

be= = 1.328 m

Therefore, load on 1m span = 47.06 KN

For load 4

Distance from the support=c= 1.285 m

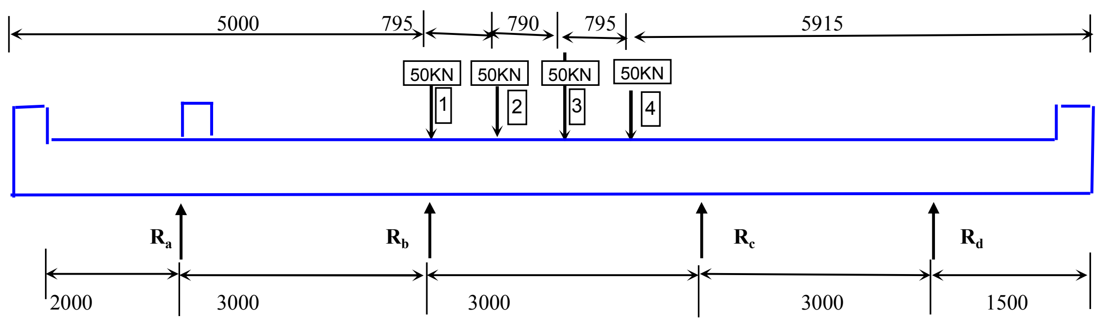
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1= 2.144 m >1.22m. Dispersion will be restricted

be= = 1.682 m

Therefore, load on 1m span = 37.16 KN

CASE-VII: Here Load 1 to 4 represents four wheels of 40T load. Place load 40T load centrally at support.

Centre of the first wheel of 1st lane load from left edge for first span= 5000 mm

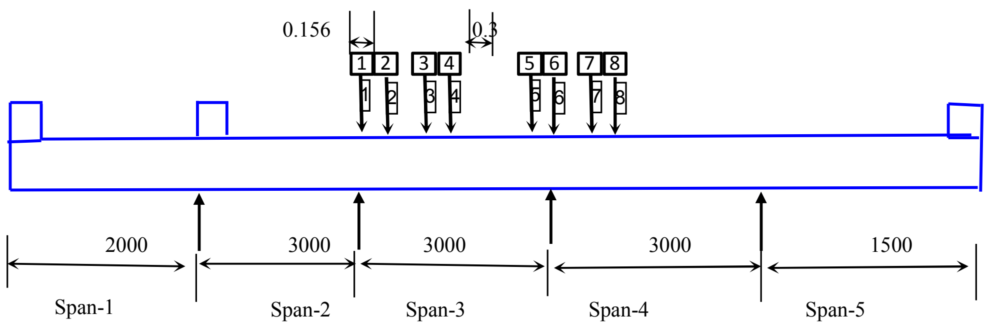


Further the loads are distributed on certain areas

Thickness of Wearing Coat =	65 mm
α=	2.600 m
l_0=	2.7 m
b_1 for Class 40T=	0.393 m
l_{e1}= Dispersion width along the span of deck for Class40T 1st Load=	0.819 m
l_{e2}= Dispersion width along the span of deck for Class40T 2nd Load=	0.7925 m
l_{e3}= Dispersion width along the span of deck for Class 40T 3rd Load=	0.7925 m
l_{e4}= Dispersion width along the span of deck for Class40T 4th Load=	0.819 m

For load 1			
Distance from the support=b=		0 m	
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		0.393 m	<1.22m. Dispersion will not be
be=	=	0.393 m	
Therefore, load on 1m span	=	159.03 KN	
For load 2			
Distance from the support=b=		0.795 m	
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.851 m	>1.22m. Dispersion will be restricted
be=	=	1.5355 m	
Therefore, load on 1m span	=	40.70 KN	
For load 3			
Distance from the support=c=		1.415 m	
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		2.144 m	>1.22m. Dispersion will be restricted
be=	=	1.682 m	
Therefore, load on 1m span	=	37.16 KN	
For load 4			
Distance from the support=c=		0.79 m	
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.846 m	>1.22m. Dispersion will be restricted
be=	=	1.533 m	
Therefore, load on 1m span	=	40.77 KN	

CASE-VII: Here Load 1 & Load 2 represents two wheels of Special Vehicle , Placed at the centre of carriage way.



Further the loads are distributed on certain areas

Thickness of Wearing Coat =	65 mm
α=	2.60
l_0=	2.700 m
b_1=	0.40 m
l_e= Dispersion width along the span of deck=	0.74 m
l_{e1}= Dispersion width along the span of deck for SV 1st Load=	0.315 m
l_{e2}= Dispersion width along the span of deck for SV 2nd Load=	0.315 m
l_{e3}= Dispersion width along the span of deck for SV 3rd Load=	0.315 m
l_{e4}= Dispersion width along the span of deck for SV 4th Load=	0.315 m
l_{e5}= Dispersion width along the span of deck for SV 5th Load=	0.315 m
l_{e6}= Dispersion width along the span of deck for SV 6th Load=	0.315 m
l_{e7}= Dispersion width along the span of deck for SV 7th Load=	0.315 m
l_{e8}= Dispersion width along the span of deck for SV 8th Load=	0.315 m

For load 1		
Distance from the support=a=		1.313 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		2.16 m
be=	=	1.22 m
Therefore, load on 3rd span		18.51 KN
For load 2		
Distance from the support=a=		1.162 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=	=	2.12 m
be=		1.20 m
Therefore, load on 3rd span		18.76 KN
For load 3		
Distance from the support=b=		0.6370 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.67 m
be=		0.97 m
Therefore, load on 3rd span		23.15 KN
For load 4		
Distance from the support=b=		0.4120 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.31 m
be=		0.79 m
Therefore, load on 3rd span		28.38 KN
For load 5		
Distance from the support=a=		0.4130 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.31 m
be=		0.79 m
Therefore, load on 4th span		28.35 KN
For load 6		
Distance from the support=a=		0.6380 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		1.67 m
be=		0.97 m
Therefore, load on 4th span		23.14 KN
For load 7		
Distance from the support=b=		1.1630 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		2.13 m
be=		1.20 m
Therefore, load on 4th span		18.76 KN
For load 8		
Distance from the support=b=		1.3880 m
Dispersion width across the span of deck= α*a*[1-(a/l_0)]+b_1=		2.16 m
be=		1.22 m
Therefore, load on 4th span		18.51 KN

f. Analysis Output

The analysis of deck slab has been carried by SAP 2000, considering the deck slab as a beam of unit width. The output at various location has been tabulated below.

Sl No.	Load Case	Exterior support mts.(Left) (KN-m)	Exterior support mts(Right) (KN-m)	Interior support Moment (KN-m)	Mid-span (KN-m)	Exterior Support Reaction (Left)_KN	Exterior Support Reaction (Right)_KN	Interior Support Reaction (KN)
1	DL	-11.650	-6.55	-4.270	3.870	11.760	9.5	9.290
2	FPLL	-4.500	-0.1	1.180	3.100	6.000	0.1	0.490
3	Surfacing	-0.810	-0.81	-1.460	1.040	1.210	2.3	2.870
4	SIDL	-9.420	-6.84	-6.840	4.360	8.810	5.62	11.620
5	Case-I	-35.100	-35.1	-27.390	26.840	30.206	50.14	95.000
6	Case-II	0.000	0.000	-25.510	17.890	29.000	4.71	87.680
7	Case-III	0.000	0.000	-30.800	9.550	0.000	38.31	33.970
10	Case-IV	0.000	0.000	-32.290	20.060	9.800	20.61	60.290
11	Case-V	0.000	0.000	-34.960	18.600	2.010	79.92	57.370
12	Case-VI	0.000	0.000	-35.970	17.150	7.570	2.14	87.800
	Case-VII	0.000	0.000	-25.380	35.660	6.740	7.22	61.520
13	Case-VIII	0.000	0.000	-38.950	13.560	2.226	11.68	73.070
LOAD SUMMARY								
1	DL	-11.65	-6.55	-4.27	3.87	11.76	9.5	9.29
2	FPLL	-4.5	-0.1	1.18	3.1	6	0.1	0.49
3	Surfacing	-0.81	-0.81	-1.46	1.04	1.21	2.3	2.87
4	SIDL	-9.42	-6.84	-6.84	4.36	8.810	5.62	11.62
5	LL Maximum Moment	0.000	0.000	-25.380	35.660	6.740	7.220	61.520

g. Combination of Moments

Ultimate Limit State (Basic Combination) *Ref.: Table 3.2, IRC-6:2010*

Sl. No.	Load Case	PSF	Cantilever Support Mts. (Left) (-ve) (KN-m)	Cantilever Support Mts.(Right) (-ve)KN-m	Mid Support (-ve) (KN-m)	Mid Span (+ve) (KN-m)	Exterior Support Reaction (Left)(KN)	Exterior Support Reaction (Right)(KN)	Interior Support Reaction (KN)
1	DL	1.350	-15.728	-8.843	-5.765	5.225	15.876	12.825	12.542
2	FPLL	1.500	-6.750	-0.150	1.770	4.650	9.000	0.150	0.735
3	Surfacing	1.750	-1.418	-1.418	-2.555	1.820	2.118	4.025	5.023
4	SIDL	1.350	-12.717	-9.234	-9.234	5.886	11.894	7.587	15.687
5	LL Maximum Moment	1.500	0.000	0.000	-38.070	53.490	10.110	10.830	92.280
6	Support Displacement	1.000	0.000	0	0.000	0.000	0.000	0.000	0.000
		Total	**-36.6**	**-19.6**	**-53.9**	**71.1**	**49.0**	**35.4**	**126.3**

Servicibilty Limit State (Rare Combination) *Ref.: Table 3.3, IRC-6:2010*

Sl. No	Load Case	PSF	Cantilever Support Mts. (Left) (-ve) (KN-m)	Cantilever Support Mts.(Right) (-ve)KN-m	Mid Support (-ve)_KN-m	Mid Span (+ve) (KN-m)	Exterior Support Reaction (Left)(KN)	Exterior Support Reaction (Right)(KN)	Interior Support Reaction (KN)
1	DL	1.000	-11.650	-6.550	-4.270	3.870	11.760	9.500	9.290
2	FPLL	1.000	-4.500	-0.100	1.180	3.100	6.000	0.100	0.490
3	Surfacing	1.000	-0.810	-0.810	-1.460	1.040	1.210	2.300	2.870
4	SIDL	1.000	-9.420	-6.840	-6.840	4.360	8.810	5.620	11.620
5	LL Maximum Moment	1.000	0.000	0.000	-25.380	35.660	6.740	7.220	61.520
6	Support Displacement	0.750	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Total	**-26.4**	**-14.3**	**-36.8**	**48.0**	**34.5**	**24.7**	**85.8**

f. Check for effective depth

Section-1: **At Mid span**

Consider the section as balanced section.

Depth of NA=$X_u$bal= 0.46 * d *Ref.: Note, Cl-38,IS-456:2000*

Mu= $0.36*f_{ck}*b*X_u bal*(d-0.42*x_u bal)$= 71.1 KN-m *Ref.: Annexure-G,IS-456:2000*

d_{bal}= 115 mm
Clear cover= 40 mm
Provided dia of reinforcement= 12 mm
Required overall depth=D_{reqd}= 161 mm
Provided overall depth=$D_{provided}$= 225 mm **Hence OK**

Section-2: **At Mid support**

Consider the section as balanced section.

Depth of NA=$X_u$bal= 0.46 * d *Ref.: Note, Cl-38,IS-456:2000*

Mu= $0.36*f_{ck}*b*X_u bal*(d-0.42*x_u bal)$= 53.9 KN-m *Ref.: Annexure-G,IS-456:2000*

d_{bal}= 100 mm
Clear cover= 40 mm
Provided dia of reinforcement= 12 mm
Required overall depth=D_{reqd}= 146 mm
Provided overall depth=$D_{provided}$= 225 mm **Hence OK**

Section-3: **At Cantilever part**

Consider the section as balanced section.

Depth of NA=$X_u$bal= 0.46 * d *Ref.: Note, Cl-38,IS-456:2000*

Mu= $0.36*f_{ck}*b*X_u bal*(d-0.42*x_u bal)$= 36.6 KN-m *Ref.: Annexure-G,IS-456:2000*

d_{bal}= 83 mm
Clear cover= 40 mm
Provided dia of reinforcement= 16 mm
Required overall depth=D_{reqd}= 131 mm
Provided overall depth=$D_{provided}$= 225 mm **Hence OK**

g. Calculation of Reinforcement

Section-1: **At Mid span**

Calculation of main steel

Effective depth of Slab Provided=d_{eff}= 179 mm *Ref.: Annexure-G,IS-456:2000*

A_{st}reqd= $0.5* f_{ck}/f_y*[1-sqrt\{1-0.46*M_u/(f_{ck}*b*d^2)\}]*b*d$= 981 mm^2

Maximum spacing allowed= 300 mm *Ref.: Cl-16.6.1.1.(4), IRC-112:2011*

Provide **12** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 1130 mm^2 **Hence OK**

Calculation of distribution steel

Distribution steel will be maximum of the followings.

i) 20% of A_{st}provided= 226 mm^2 *Ref.: Cl-16.6.1.1.(3), IRC-112:2011*

ii) $0.0013b_t*d_{eff}$= 233 mm^2 *Ref.: Cl-16.5.1.1, IRC-112:2011*

iii) $0.26*f_{ctm}/f_{yk}*b_t*d$= 279 mm^2

A_{st}reqd= 279 mm^2

Maximum spacing allowed= 400 mm *Ref.: Cl-16.6.1.1.(4), IRC-112:2011*

Provide **10** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 785 mm^2 **Hence OK**

Section-2: **At Mid support**

Calculation of main steel

Effective depth of Slab Provided=d_{eff}= 179 mm ***Ref.: Annexure-G,IS-456:2000***

A_{st}reqd= $0.5* f_{ck}/f_y*[1-sqrt\{1-0.46*M_u/(f_{ck}*b*d^2)\}]*b*d$= 730 mm^2

Maximum spacing allowed= 300 mm ***Ref.: Cl-16.6.1.1.(4), IRC-112:2011***

Provide **12** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 1130 mm^2 **Hence OK**

Calculation of distribution steel

Distribution steel will be maximum of the followings.

i) 20% of A_{st}provided= 226 mm^2 ***Ref.: Cl-16.6.1.1.(3), IRC-112:2011***

ii) $0.0013b_t*d_{eff}$= 233 mm^2 ***Ref.: Cl-16.5.1.1, IRC-112:2011***

iii) $0.26*f_{ctm}/f_{yk}*b_t*d$= 279 mm^2

A_{st}reqd= 233 mm^2

Maximum spacing allowed= 400 mm ***Ref.: Cl-16.6.1.1.(4), IRC-112:2011***

Provide **10** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 785 mm^2 **Hence OK**

Section-3: **At Cantilever part**

Calculation of main steel

Effective depth of Slab Provided=d_{eff}= 177 mm ***Ref.: Annexure-G,IS-456:2000***

A_{st}reqd= $0.5* f_{ck}/f_y*[1-sqrt\{1-0.46*M_u/(f_{ck}*b*d^2)\}]*b*d$= 493 mm^2

Maximum spacing allowed= 250 mm ***Ref.: Cl-16.6.1.1.(4), IRC-112:2011***

Provide **16** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 2010 mm^2 **Hence OK**

Calculation of distribution steel

Distribution steel will be maximum of the followings.

i) 20% of A_{st}provided= 402 mm^2 ***Ref.: Cl-16.6.1.1.(3), IRC-112:2011***

ii) $0.0013b_t*d_{eff}$= 230 mm^2 ***Ref.: Cl-16.5.1.1, IRC-112:2011***

iii) $0.26*f_{ctm}/f_{yk}*b_t*d$= 276 mm^2

A_{st}reqd= 402 mm^2

Maximum spacing allowed= 400 mm ***Ref.: Cl-16.6.1.1.(4), IRC-112:2011***

Provide **10** mm Tor bar @ **100** mm c/c spacing.

A_{st}provided= 785 mm^2 **Hence OK**

h. Minimum Reinforcement for crack control: ***Ref.: Cl-12.3.3,IRC-112:2011***

Required Minimum Reinforcement for crack control= $A_{s,min}=k_c*k*f_{ct,eff}*A_{ct}/\sigma_s$

A_{ct}= Concrete area within the tensile zone= 179000 mm^2

σ_s= Maximum tensile stress permitted= 240 Mpa ***Ref.: Table-12.2,IRC-112:2011***

$f_{ct,eff}$= Mean value of the tensile strength of concrete (f_{ctm})= 3 Mpa

k=Coefficient which allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces= 1 for flange less than 300mm

k_c=Coefficient which takes account of the stress distribution within the section just prior to cracking and of the change of the lever arm= 0.40

$=0.4 \times [1-\{\sigma_c/(k_1 \times (h/h^*)*f_{ct,eff}\}]$=

σ_c= compressive stress in the concrete= 0 Mpa

k_1= Coefficient considering the effects of axial force in the stress distribution= 1

h= 179 mm

h*= 179 mm

A_{smin}= Minimum area of within the tensile zone = 895 mm^2 **Hence OK**

i. **Calculation of Crack Width** ***Ref.: Cl-12.3.3,IRC-112:2011***

Section-1: **At Mid span**

Moment= 48.00 KN-m

Neutral axis depth: Xu= $(0.87*f_y*A_{stprov})/(0.36*f_{ck}*b)$= 34 mm

d_{eff}= 179 mm

Diameter of bar= 12 mm

Spacing of bar= 100 mm

A_{st}provided= 1130 mm^2

m (lomgterm)= 12.5

Equating the moments of areas about the centroidal axis, $b*Y_t^2=m*A_{stprov}*(d-Y_t)$

Solving thae equation, Y_t= $[-(m*A_{stprov})\pm\{(-m*A_{stprov})^2-4*(-m*A_{stprov})*d_{eff}*0.5*b\}^{0.5}]/(2*0.5*b)$

= 58 mm

-87 (Negetive value is neglected)

Distance of CG from reinforcing steel, Y_s= 121 mm

Inertia of the section, $I_{cr}=m*A_{tprov}*Y_s^2+b*Y_t^3$= 2.72E+08 mm^4

Section modulus=Z_t= I_{cr}/Y_t= 4.66E+06 mm^3

σ_{sc} = stress in the tension reinforcement = 266 Mpa

Maximum permissible stress in tensile steel=$0.8*f_{yk}$= 400 Mpa ***Ref.:-Cl- 12.2.2, IRC:112-2011***

Hence OK

f_c = stress in the concrete = 10.3 Mpa

Maximum allowable stress in concrete = $0.48f_{ck}$ = 19.2 Mpa ***Ref.:-Cl- 12.2.1(1), IRC:112-2011***

Hence OK

Calculation of crack width

Crack width= $Wk=S_{r\,max}*(\varepsilon_{sm}-\varepsilon_{cm})$

Where, $S_{r.max}$ = Maximum crack spacing

™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

™$_{cm}$ = mean strain in the concrete between cracks.

Now,

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

Ref.:- Eq-12.6, IRC:112-2011

$\alpha_e = E_s/E_{cm}$ = 12.5

$f_{ct.eff}$ = mean value of tensile strength of concrete = 3 Mpa

$\rho_{p.eff} = A_s/A_{c.eff}$= 0.0100

$A_{c.eff}$= Effective area of concrete in tension, surrounding the reinforcement= $b*h_{ceff}$= 112500 mm^2

h_{ceff}= 112.5 mm lesser of i) 2.5*(D-d)= 115 mm

ii) D-x/3= 214 mm

iii) D/2= 113 mm

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+f/2)), the maximum crack spacing,

$$Sr.max=3.4c+\frac{0.425k_1k_2\phi}{\rho_{p.eff}}$$

Ref;-. Eq-12.8, IRC:112-2011

f = diameter of bar = 12 mm **The Equation is valid**

c = clear cover = 40 mm

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 339 mm

And, ™$_{sm}$-™$_{cm}$ = 0.000491

Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.000798

So, governing value of ™$_{sm}$ - ™$_{cm}$ = **0.000798**

So, crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.271 mm

Maximum crack width = 0.3 mm ***Ref.:-Table 12.1, IRC:112-2011, page-122***

Crack width within permissible limit

Section-3: **At Cantilever part**

Moment= 26.40 KN-m

Neutral axis depth: Xu= $(0.87*f_y*A_{stprov})/(0.36*f_{ck}*b)$= 61 mm

d_{eff}= 177 mm

Diameter of bar= 16 mm

Spacing of bar= 100 mm

A_{st}provided= 2010 mm^2

m (lomgterm)= 12.5

Equating the moments of areas about the centroidal axis, $b*Y_t^2=m*A_{stprov}*(d-Y_t)$

Solving thae equation, Y_t= $[-(m*A_{stprov})\pm\{(-m*A_{stprov})^2-4*(-m*A_{stprov})*d_{eff}*0.5*b\}^{0.5}]/(2*0.5*b)$

= 72 mm

-123 (Negetive value is neglected)

Distance of CG from reinforcing steel, Y_s= 105 mm

Inertia of the section, $I_{cr}=m*A_{tprov}*Y_s^2+b*Y_t^3$= 4.01E+08 mm^4

Section modulus=$Z_t= I_{cr}/Y_t$= 5.54E+06 mm^3

σ_{sc} = stress in the tension reinforcement = 86 Mpa

Maximum permissible stress in tensile steel=$0.8*f_{yk}$= 400 Mpa ***Ref.:-Cl- 12.2.2, IRC:112-2011***

Hence OK

f_c = stress in the concrete = 4.8 Mpa

Maximum allowable stress in concrete = $0.48f_{ck}$ = 19.2 Mpa ***Ref.:-Cl- 12.2.1(1), IRC:112-2011***

Hence OK

Calculation of crack width

Crack width= $Wk=S_{r\,max}*(\varepsilon_{sm}-\varepsilon_{cm})$

Where, $S_{r.max}$ = Maximum crack spacing

™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

™$_{cm}$ = mean strain in the concrete between cracks.

Now,

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

Ref.:- Eq-12.6, IRC:112-2011

$\alpha_e = E_s/E_{cm}$ = 12.5

$f_{ct.eff}$ = mean value of tensile strength of concrete = 0.8 Mpa

$\rho_{\rho.eff} = A_s/A_{c.eff}$= 0.0179

$A_{c.eff}$= Effective area of concrete in tension, surrounding the reinforcement= $b*h_{ceff}$= 112500 mm^2

h_{ceff}= 112.5 mm lesser of i) 2.5*(D-d)= 120 mm

ii) D-x/3= 205 mm

iii) D/2= 113 mm

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+f/2)), the maximum crack spacing,

$$Sr.max = 3.4c + \frac{0.425k_1k_2\phi}{\rho_{\rho.eff}}$$

Ref:-. Eq-12.8, IRC:112-2011

f = diameter of bar = 16 mm **The Equation is valid**

c = clear cover = 40 mm

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 288 mm

And, ™$_{sm}$-™$_{cm}$ = 0.000293

Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.000258

So, governing value of ™$_{sm}$ - ™$_{cm}$ = **0.000293**

So, crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.084 mm

Maximum crack width = 0.3 mm ***Ref.:-Table 12.1, IRC:112-2011***

Crack width within permissible limit

j. <u>Shear Check</u> ***Ref.: Cl-10.3.2,IRC-112:2011***

<u>Section-3:</u> **At Cantilever part**

Design Shear Force = 49.000 KN

The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

$=[0.12K(80\rho_1.f_{ck})^{0.33}+0.15\sigma_{cp}]b_w.d$ ***Ref:-. Eq-10.1, IRC:112-2011***

Where, $K = 1+\sqrt{(200/d)}<=2.0$

So, K = 2.000

$\rho_1 = A_{sl}/b_w.d$

Where A_{sl} = Area of steel provided = 2010 mm^2

b_w = Width of section = 1000 mm

d = 177 mm

ρ_1 = 0.0114

$\sigma_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0

A_c = Cross Sectional area of concrete

σ_{cp} = 0

So,$V_{Rd.c}$ = 139.02 KN

Now, $V_{Rd.c}$ minimum = $(\nu_{min}+0.15\sigma_{cp})b_w.d$

where ν_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.555

So, $V_{Rd.c}$ minimum = 98.154 KN

So, governing shear resistance = 139.02 KN **OK, Safe in shear**

k. <u>Summary of Reinforcement</u>

<u>Section-1:</u> **<u>At Mid span</u>**

<u>Main Steel</u>

Provide 12 mm Tor bar @ 100 mm c/c spacing.

<u>Distribution Steel</u>

Provide 10 mm Tor bar @ 100 mm c/c spacing.

<u>Section-2:</u> **<u>At Mid support</u>**

<u>Main Steel</u>

Provide 12 mm Tor bar @ 100 mm c/c spacing.

<u>Distribution Steel</u>

Provide 10 mm Tor bar @ 100 mm c/c spacing.

<u>Section-3:</u> **<u>At Cantilever part</u>**

<u>Main Steel</u>

Provide 16 mm Tor bar @ 100 mm c/c spacing.

<u>Distribution Steel</u>

Provide 10 mm Tor bar @ 100 mm c/c spacing.

l. <u>Deflection check:</u>

Permissible deflection in cantilever part= span/300 6.67 mm

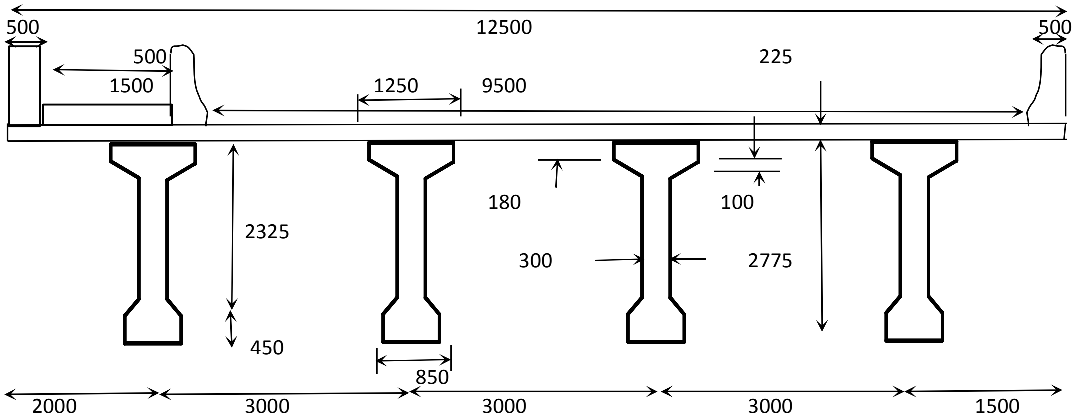
Permissible deflection in intermediate span= span/800 3.75 mm

Developed deflection in cantilever part= 0.693 mm **Ok**

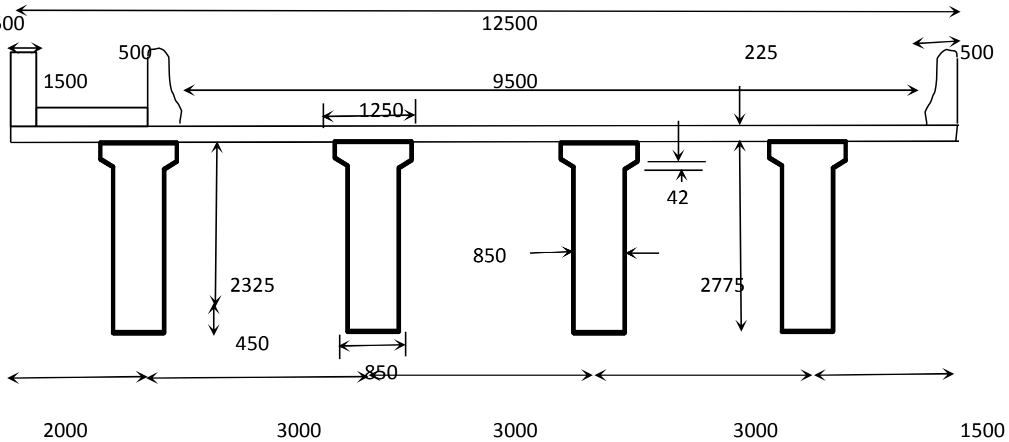
Developed deflection in intermediate span= 1.7 mm **Ok**

DESIGN OF PSC T-GIRDER WITH 38.8M SPAN (C/C OF BEARING) [INNER GIRDER]

A. GEOMETRIC PROPERTIES OF THE GIRDER

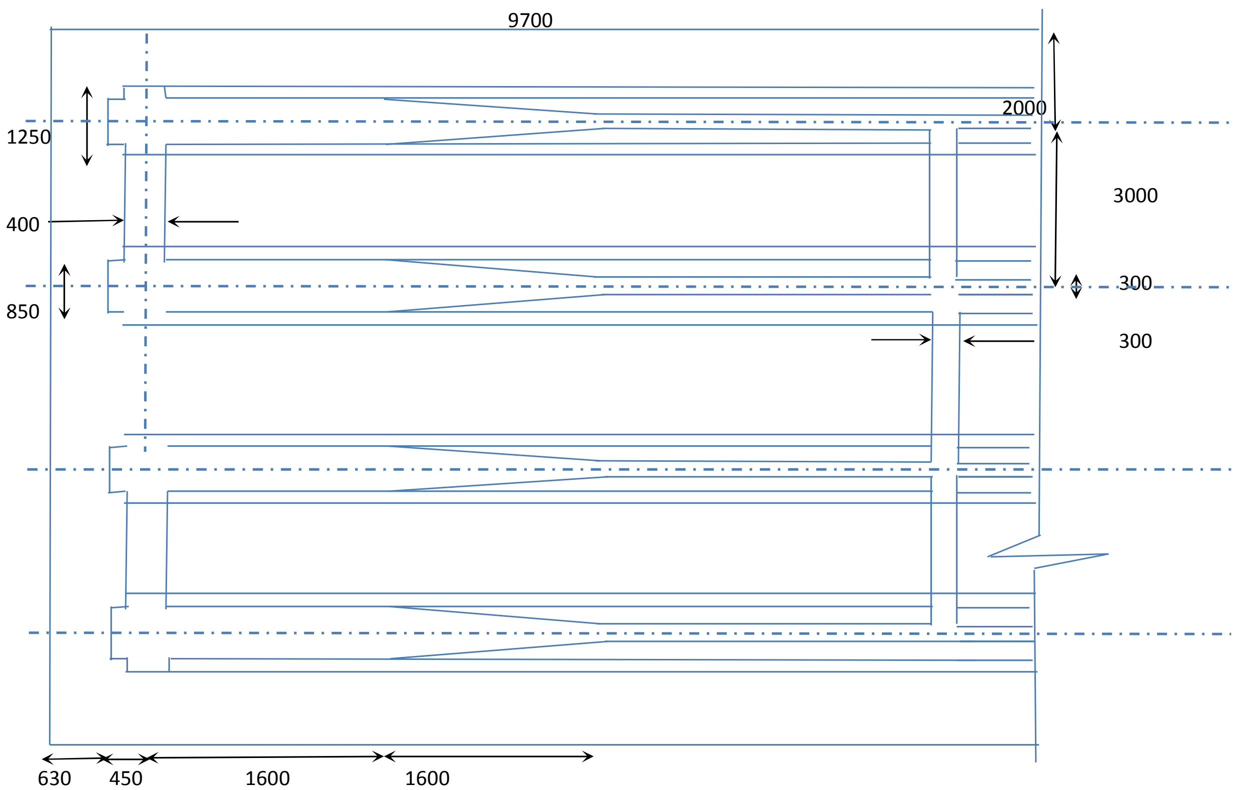


C/S AT MIDDLE



C/S AT END

Thickness of web at end = 850 mm



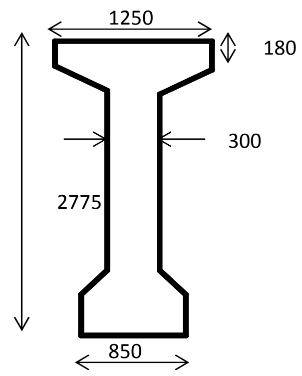
SECTIONAL PLAN THROUGH WEB GIRDER

B. PROPERTIES OF GIRDER SECTION

Precast Section :

For middle portion

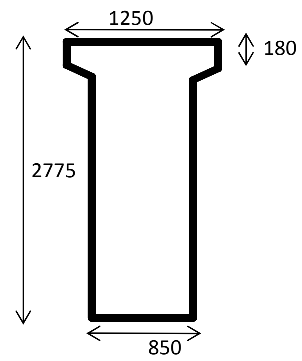
Total girder depth	D =	2775 mm		
Web width of T-Girder	b_w =	300 mm		
Flange width of T-Girder	b_f =	1250 mm		
Flange depth of T-Girder	D_f =	180 mm		
Girder Bulb Width	b_{gb} =	850 mm		
Girder Bulb Depth (Straight)	D_{gb} =	250 mm		
Haunch in Girder Bulb	H : V =	275 mm	:	200 mm
Haunch in Girder Flange to Web	H : V =	475 mm	:	150 mm
Area of Girder (inner portion)	Ac =	1.267 m^2		



Pre Cast Girder Section at Mid Span

For end portion of girder having length 1.6 m

Total girder depth	D =	2775 mm		
Web width of T-Girder	b_w =	850 mm		
Flange width of T-Girder	b_f =	1250 mm		
Flange depth of T-Girder	D_f =	180 mm		
Haunch in Girder Flange to Web	H : V =	200 mm	:	63.16 mm
Area of Girder (end thickened portion)	Ac =	2.443 m^2		



Pre Cast Girder Section near Support

For Precast Girder at mid span:

cg of section from bottom of girder will be as follows :

Y_{bp} = [(0.18x1.25)x(2.685)+2x(0.5x0.475x0.15)x(2.55)+(2.345)x(1.6225)+(2x0.5x(0.275x0.2)x(66.92)+(0.85x0.25)x(0.13)] /1.267
1.444 m

cg of section from top of girder will be as follows :

Y_{tp} = 2.775-1.444 = 1.331 m

Moment of Inertia of precast girder :

$I_{precast}$ = [{1.25x0.005832/12+0.225x1.68350625}+{(0.475x0.003375)/18}+{0.07125x1.34+(0.3x12.9)/12}+ {(0.275x0.008/18)+0.055x1.15}+{(0.85x0.015625/12)+0.2125x1.59}]

1.2 m^4

Z_{tp} = 0.902 m^3 Z_{bp} = 0.831 m^3

For Composite Girder :

Edge Girder

Effective flange width = 1.5+1.75 = 3.25 m

Area of girder = [(3.25x0.233] +1.267= 2.024 m^2

cg from bottom of girder = [{(3.25x0.233)x(2.775+0.117)}+{1.267x1.444}]/2.024 = 1.986 m

Y_{bg} = 1.986 m Y_{tg} = 0.789 m Y_{ts} = 1.022 m

$I_{composite}$ = [(3.25x0.013/12)+(3.25x0.054)]+[1.2+1.267x(0.542^2)] = 2.278 m^4

Z_{ts} = 2.229 m^3 Z_{tg} = Z_{bs} = 2.887 m^3 Z_{bg} = 1.147 m^3

inner Girder

Effective flange width = 1.5+1.5 = 3 m

Area of girder = [(3x0.233] +1.267= 1.966 m^2

cg from bottom of girder = [{(3x0.233)x(2.775+0.117)}+{1.267x1.444}]/1.966 = 1.959 m

Y_{bg} = 1.959 m **Y_{tg} =** 0.816 m **Y_{ts} =** 1.049 m

$I_{composite}$ = [(3x0.013/12)+(3x0.054)]+[1.2+1.267x(0.515^2)] = 2.209 m^4

Z_{ts} = 2.106 m^3 **Z_{tg} = Z_{bs} =** 2.707 m^3 **Z_{bg} =** 1.128 m^3

C. DEAD LOAD

Calculation of loads and moments at different sections of girder :

Dead Load

1. Precast Girder

a Area of Girder (inner portion) Ac = 1.267 m^2

Loading = 1.267x25 = 31.675 kN/m

b Area of Girder (end thickened portion) Ac = 2.443 m^2

Loading = 2.443x25 = 61.075 kN/m

2. Self weight of diaphragms

i) Intermediate diaphragm 300 mm Thickness No in each girder = 3

a. Precast portion

Area of each diaphragm (per girder) = 0.5 x(1.25+0.85) x2.345- 0.5 x (1.25+0.3) x0.15- 0.5 x (0.3+0.85) x 0.2- (0.3x1.995)

= 2.83 m^2

Loading = 2.83x25x0.3 = 21.225 kN (on each girder)

b. In Situ portion

Area of each diaphragm = 0.5 x(1.75+2.15) x 2.325 = 4.534 m^2

Loading = 4.534x25x0.3 = 34.005 kN (at each location)

Hence , load on end girder = 8.501 kN (at each location)

Load on inner girder = 17.003 kN (at each location)

ii) Exterior diaphragm 400 mm Thickness No in each girder = 2

a. Precast portion

Area of each diaphragm (per girder) = (1.25x2.775) -2.443-{(1.25-0.85) x0.35} = 0.88575 m^2

Loading = 0.88575x25x0.4 = 8.858 kN (on each girder)

b. In Situ portion

Area of each diaphragm = (3-1.25)x(2.775-0.35) = = 4.24375 m^2

Loading = 4.24375x25x0.4 = 42.4375 kN (at each location)

Hence , load on end girder = 10.609 kN (at each location)

Load on inner girder = 21.219 kN (at each location)

3. Self weight of deck slab

Total weight of deck slab =	12.5x0.225x25 =	70.3125 kN/m
Loading on each end girder =	(1.5+1.5)x0.225x25 =	16.875 kN/m
Loading on each intermediate girder =	(1.5+1.5)x0.225x25 =	16.875 kN/m

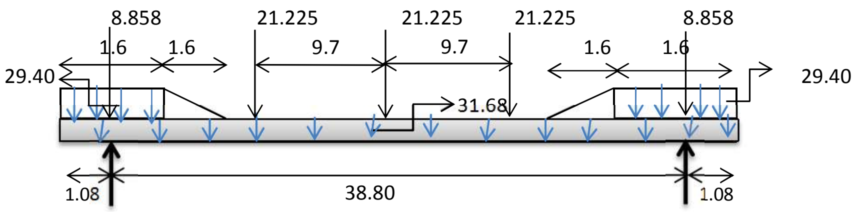
4. Superimposed dead load (crash barrier/safety kerb/wearing coat)

Superimposed dead load will be placed on deck slab after composite action starts.

<u>i)Load of crash barrier</u>		
Wt. of each crash barrier =	0.329 x 25 =	0 kN/m
load for two sides =		0.000 kN/m
<u>ii)Load of safety kerb & Foot path & Railing</u>		
Wt. of each safety kerb =	0.5 x 0.225 x 25 + 1.5 =	0.000 kN/m
load for one side =		0.000 kN/m
<u>iii)Load of wearing coat</u>		
Wt. of wearing coat =	0.065 x 9.5 x 22 =	13.585 kN/m
Total superimposed dead load =	0+0+13.585 =	13.585 kN/m
Load per Longitudinal girder =	13.585/4 =	3.39625 kN/m

D. <u>DESIGN MOMENTS & SHEARS :</u>

1. <u>DUE TO S/W OF PRECAST GIRDERS & PRECAST PORTION OF DIAPHRAGMS :</u>



Reaction on each support = [(38.8+1.08+1.08)x31.675/2]+[29.4x1.6]+[29.4x1.6x0.5]+[8.858]+[21.225X3/2] =

= 648.704 + 47.04 + 23.52 + 8.858 + 31.838

= 759.960 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support

BM= [759.9595X19.4]-[31.675X20.48^2/2]-[47.04X(19.4+1.08-1.6/2)]-[23.52X(19.4+1.08-1.6-0.533)]

-[8.858x19.4]-[21.225x(19.4-9.7)]

= 6365.5 kN-m

At 3/8th section: 14.55 m from support

BM= [759.9595X14.55]-[31.675X15.63^2/2]-[47.04X(14.55+1.08-1.6/2)]-[23.52X(14.55+1.08-1.6-0.533)]

-[8.858x14.55]-[21.225x(14.55-9.7)]

= 5941.49 kN-m

At 1/4th section: 9.7 m from support

BM= [759.9595X9.7]-[31.675X10.78^2/2]-[47.04X(9.7+1.08-1.6/2)]-[23.52X(9.7+1.08-1.6-0.533)]

-[8.858x9.7]

= 4772.41 kN-m

At 1/8th section: 4.85 m from support

BM= [759.9595X4.85]-[31.675X5.93^2/2]-[47.04X(4.85+1.08-1.6/2)]-[23.52X(4.85+1.08-1.6-0.533)]

-[8.858x4.85]

= 2755.31 kN-m

section: 2.120 m from support

BM= [759.9595X2.12]-[31.675X3.2^2/2]-[47.04X(2.12+1.08-1.6/2)]-[23.52X(2.12+1.08-1.6-0.533)]

-[8.858x2.12]

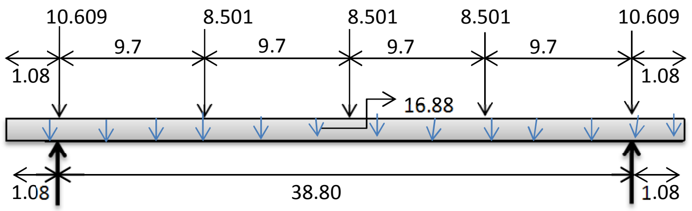
= 1292.18 kN-m

SHEAR AT DIFFERENT SECTIONS OF T GIRDER DUE TO DEAD LOAD:

section: 2.120 m from support
SF= 759.9595-[31.675X(2.12+1.08)]-47.04-23.52-8.858
= 579.182 kN
At 1/8th section: 4.850 m from support
SF= 759.9595-[31.675X(4.85+1.08)]-47.04-23.52-8.858
= 492.709 kN
At 1/4th section: 9.700 m from support
SF= 759.9595-[31.675X(9.7+1.08)]-47.04-23.52-8.858
= 339.085 kN
At 3/8th section: 14.550 m from support
SF= 759.9595-[31.675X(14.55+1.08)]-47.04-23.52-8.858-21.225
= 164.236 kN
At mid section: 19.4 m from support
SF= 759.9595-[31.675X(19.4+1.08)]-47.04-23.52-8.858-21.225
= 10.613 kN

2. DUE TO S/W OF DECK SLAB & CAST IN SITU PORTION OF DIAPHRAGMS :

i)For Edge Girder



Reaction on each support = [(38.8+1.08+1.08)x16.875/2]+[10.609]+[8.50125X3/2] =
= 345.600 + 10.609 + 12.752
= 368.961 kN

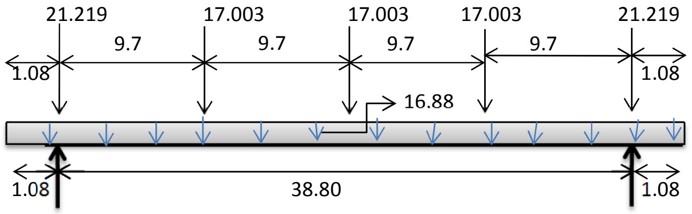
MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support
BM= [368.960875X19.4]-[16.875X20.48^2/2]-[10.609x19.4]-[8.50125x(19.4-9.7)]
= 3330.62 kN-m
At 3/8th section: 14.55 m from support
BM= [368.960875X14.55]-[16.875X15.63^2/2]-[10.609x14.55]-[8.50125x(14.55-9.7)]
= 3111.53 kN-m
At 1/4th section: 9.7 m from support
BM= [368.960875X9.7]-[16.875X10.78^2/2]-[10.609x9.7]
= 2495.5 kN-m
At 1/8th section: 4.85 m from support
BM= [368.960875X4.85]-[16.875X5.93^2/2]-[10.609x4.85]
= 1441.3 kN-m
section: 2.120 m from support
BM= [368.960875X2.12]-[16.875X3.2^2/2]-[10.609x2.12]
= 673.306 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.12 m from support
SF= 368.960875-[16.875X(2.12+1.08)]-10.609
= 304.352 kN
At 1/8th section: 4.85 m from support
SF= 368.960875-[16.875X(4.85+1.08)]-10.609
= 258.283 kN
At 1/4th section: 9.7 m from support
SF= 368.960875-[16.875X(9.7+1.08)]-10.609
= 176.439 kN
At 3/8th section: 14.55 m from support
SF= 368.960875-[16.875X(14.55+1.08)]-10.609-8.50125
= 86.0944 kN
At mid section: 19.4 m from support
SF= 368.960875-[16.875X(19.4+1.08)]-10.609-8.50125
= 4.251 kN

ii) For inner Girder



Reaction on each support = [(38.8+1.08+1.08)x16.875/2]+[21.219]+[17.0025X3/2] =

= 345.600 + 21.219 + 25.504

= 392.323 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support

BM= [392.32275X19.4]-[16.875X20.48^2/2]-[21.219x19.4]-[17.0025x(19.4-9.7)]

= 3495.54 kN-m

At 3/8th section: 14.55 m from support

BM= [392.32275X14.55]-[16.875X15.63^2/2]-[21.219x14.55]-[17.0025x(14.55-9.7)]

= 3255.84 kN-m

At 1/4th section: 9.7 m from support

BM= [392.32275X9.7]-[16.875X10.78^2/2]-[21.219x9.7]

= 2619.2 kN-m

At 1/8th section: 4.85 m from support

BM= [392.32275X4.85]-[16.875X5.93^2/2]-[21.219x4.85]

= 1503.15 kN-m

section: 2.120 m from support

BM= [392.32275X2.12]-[16.875X3.2^2/2]-[21.219x2.12]

= 700.34 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.12 m from support

SF= 392.32275-[16.875X(2.12+1.08)]-21.219

= 317.104 kN

At 1/8th section: 4.85 m from support

SF= 392.32275-[16.875X(4.85+1.08)]-21.219

= 271.035 kN

At 1/4th section: 9.7 m from support

SF= 392.32275-[16.875X(9.7+1.08)]-21.219

= 189.191 kN

At 3/8th section: 14.55 m from support

SF= 392.32275-[16.875X(14.55+1.08)]-21.219-17.0025

= 90.345 kN

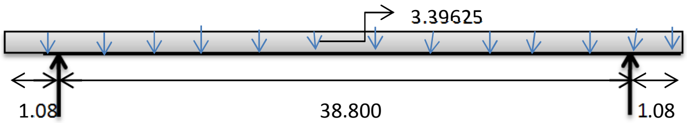
At mid section: 19.4 m from support

SF= 392.32275-[16.875X(19.4+1.08)]-21.219-17.0025

= 8.501 kN

3. DUE TO SUPERIMPOSED DEAD LOAD :

Intensity of load on each girder = 3.39625 kN/m



Reaction on each support = [(38.8+1.08+1.08)x3.39625/2] = 69.555 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support
BM= [69.555X19.4]-[3.39625X20.48^2/2]
= 637.122 kN-m

At 3/8th section: 14.550 m from support
BM= [69.555X14.55]-[3.39625X15.63^2/2]
= 597.179 kN-m

At 1/4th section: 9.700 m from support
BM= [69.555X9.7]-[3.39625X10.78^2/2]
= 477.347 kN-m

At 1/8th section: 4.850 m from support
BM= [69.555X4.85]-[3.39625X5.93^2/2]
= 277.627 kN-m

section: 2.120 m from support
BM= [69.555X2.12]-[3.39625X3.2^2/2]
= 130.068 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.120 m from support
SF= 69.555-[3.39625X(2.12+1.08)]
= 58.687 kN

At 1/8th section: 4.850 m from support
SF= 69.555-[3.39625X(4.85+1.08)]
= 49.4152 kN

At 1/4th section: 9.700 m from support
SF= 69.555-[3.39625X(9.7+1.08)]
= 32.9434 kN

At 3/8th section: 14.550 m from support
SF= 69.555-[3.39625X(14.55+1.08)]
= 16.4716 kN

At mid section: 19.400 m from support
SF= 69.555-[3.39625X(19.4+1.08)]
= 0.000 kN

E. LOAD TABLES

1. TABLE SHOWING MAX. BM AT DIFERENT SECTION AND CORRESPONDING SHEAR FORCE:

SECTION	MOMENT /SHEAR	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD	DUE TO LIVE LOAD	DUE TO CROWD LOAD
			INNER GIRDER	CENTRAL GIRDER		INNER GIRDER	INNER GIRDER
MID	MOMENT(T-M)	636.55	349.55	349.55	63.71	441.38	0.00
	SHEAR(T)	1.06	0.85	0.85	0.00	25.74	0.00
3/8 TH	MOMENT(T-M)	594.15	325.58	325.58	59.72	387.93	0.00
	SHEAR(T)	16.42	9.03	9.03	1.65	24.42	0.00
1/4 TH	MOMENT(T-M)	477.24	261.92	261.92	47.73	340.62	0.00
	SHEAR(T)	33.91	18.92	18.92	3.29	36.38	0.00
1/8 TH	MOMENT(T-M)	275.53	150.31	150.31	27.76	179.85	0.00
	SHEAR(T)	49.27	27.10	27.10	4.94	36.95	0.00
WEB. TH	MOMENT(T-M)	129.22	70.03	70.03	13.01	152.43	0.00
	SHEAR(T)	57.92	31.71	31.71	5.87	37.54	0.00

2. TABLE SHOWING MAX. SHEAR FORCE AT DIFERENT SECTION AND CORRESPONDING BM:

SECTION	MOMENT /SHEAR	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD	DUE TO LIVE LOAD	DUE TO CROWD LOAD
			INNER GIRDER	CENTRAL GIRDER		INNER GIRDER	INNER GIRDER
WEB. TH	SHEAR(T)	57.92	31.71	31.71	5.87	40.55	0.00
	MOMENT(T-M)	129.22	70.03	70.03	13.01	157.99	0.00
1/8 TH	SHEAR(T)	49.27	27.10	27.10	4.94	38.27	0.00
	MOMENT(T-M)	275.53	150.31	150.31	27.76	184.04	0.00
1/4 TH	SHEAR(T)	33.91	18.92	18.92	3.29	45.06	0.00
	MOMENT(T-M)	477.24	261.92	261.92	47.73	311.62	0.00
3/8 TH	SHEAR(T)	16.42	9.03	9.03	1.65	28.85	0.00
	MOMENT(T-M)	594.15	325.58	325.58	59.72	354.42	0.00
MID	SHEAR(T)	1.06	0.85	0.85	0.00	35.96	0.00
	MOMENT(T-M)	636.55	349.55	349.55	63.71	413.31	0.00

3. TABLE SHOWING MAX. BENDING MOMENTS AND STRESSES AT DIFERENT SECTION

Properties of precast girder		Properties of composite girder(Edge)		Properties of composite girder(Centra	
Area =	1.267 m^2	Area =	2.024 m^2	Area =	1.966 m^2
Y_{bg} =	1.444 m	Y_{bg} =	1.986 m	Y_{bg} =	1.959 m
Z_{tg} =	0.902 m^3	Z_{tg} =	2.887 m^3	Z_{tg} =	2.707 m^3
Z_{bg} =	0.831 m^3	Z_{bg} =	1.147 m^3	Z_{bg} =	1.128 m^3
		Z_{ts} =	2.229 m^3	Z_{ts} =	2.106 m^3

SECTION	LOCATION	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD		DUE TO LIVE LOAD	DUE TO CROWD LOAD
			Inner Girder	Central Girder	Inner Girder	Central Girder	Inner Girder	Inner Girder
MID	MOMENT(T-M)	636.55	349.55	349.55	63.71	63.71	441.38	0.00
	stress at top of deck slab (T/m2)	-	-	-	28.58	30.25	198.02	0
	stress at bottom of deck slab (T/m2)	-	-	-	22.07	23.54	152.89	0
	stress at top of precast girder (T/m2)	705.71	387.53	387.53	70.63	70.63	152.89	0
	stress at bottom of precast girder (T/m2)	-766.00	-420.64	-420.64	-76.67	-76.67	-384.81	0.00
3/8 TH	MOMENT(T-M)	594.15	325.58	325.58	59.72	59.72	387.93	0.00
	stress at top of deck slab (T/m2)	-	-	-	26.79	28.36	174.04	0
	stress at bottom of deck slab (T/m2)	-	-	-	20.69	22.06	134.37	0
	stress at top of precast girder (T/m2)	658.70	360.96	360.96	66.21	66.21	134.37	0
	stress at bottom of precast girder (T/m2)	-714.98	-391.80	-391.80	-71.86	-71.86	-338.21	0.00
1/4 TH	MOMENT(T-M)	477.24	261.92	261.92	47.73	47.73	340.62	0.00
	stress at top of deck slab (T/m2)	-	-	-	21.42	22.67	152.81	0
	stress at bottom of deck slab (T/m2)	-	-	-	16.53	17.63	117.98	0
	stress at top of precast girder (T/m2)	529.09	290.38	290.38	52.92	52.92	117.98	0
	stress at bottom of precast girder (T/m2)	-574.30	-315.19	-315.19	-57.44	-57.44	-296.97	0.00
1/8 TH	MOMENT(T-M)	275.53	150.31	150.31	27.76	27.76	179.85	0.00
	stress at top of deck slab (T/m2)	-	-	-	12.46	13.18	80.69	0
	stress at bottom of deck slab (T/m2)	-	-	-	9.62	10.26	62.30	0
	stress at top of precast girder (T/m2)	305.47	166.65	166.65	30.78	30.78	62.30	0
	stress at bottom of precast girder (T/m2)	-331.57	-180.88	-180.88	-33.41	-33.41	-156.80	0.00

WEB. TH	MOMENT(T-M)	129.22	70.03	70.03	13.01	13.01	152.43	0.00
	stress at top of deck slab (T/m2)	-	-	-	5.84	6.18	68.38	0
	stress at bottom of deck slab (T/m2)	-	-	-	4.51	4.80	52.80	0
	stress at top of precast girder (T/m2)	143.26	77.64	77.64	14.42	14.42	52.80	0
	stress at bottom of precast girder (T/m2)	-155.50	-84.28	-84.28	-15.65	-15.65	-132.89	0.00

F. **<u>PRESTRESSING</u>**

Prestressing cables shall be 19 strand cables conforming to IS 14268-1995 class II with minimum breaking load = 18.371 Ton
for 12.7 mm dia ,7 ply strand.

Duct dia shall be= 90 mm.
Nominal steel area of each strand is 98.8 mm2
Area of each cable= 18.772 cm^2
Ultimate force in one cable(U.T.S) = 349.000 t
Taking maximum jack pull to be applied at jack end = 70% of U.T.S = 244.3 t

No of cable on each side for each girder= 6 nos

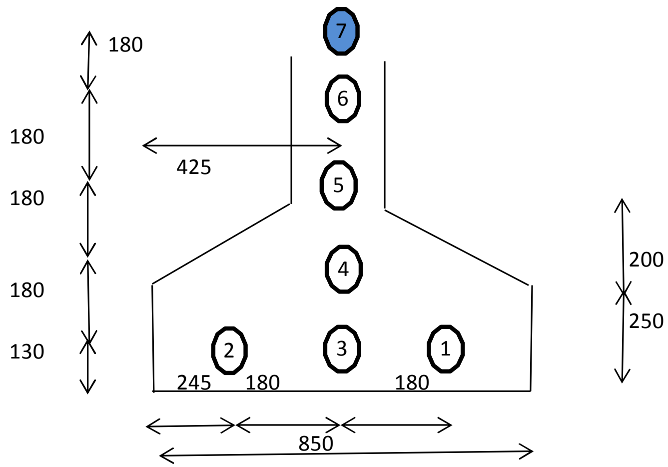
1. CABLE AT MID SECTION

No of rows of cable= 3
Vertical distance between two rows of cable (1st row to 2nd row) = 180 mm
Vertical distance between two rows of cable (2nd row to 3rd row) = 180 mm
Vertical distance between two rows of cable (3rdd row to 4th row) = 180 mm
Horizontal distance between two cable= 180 mm
Distance between cable centre & edges of T-Girder= 245 mm

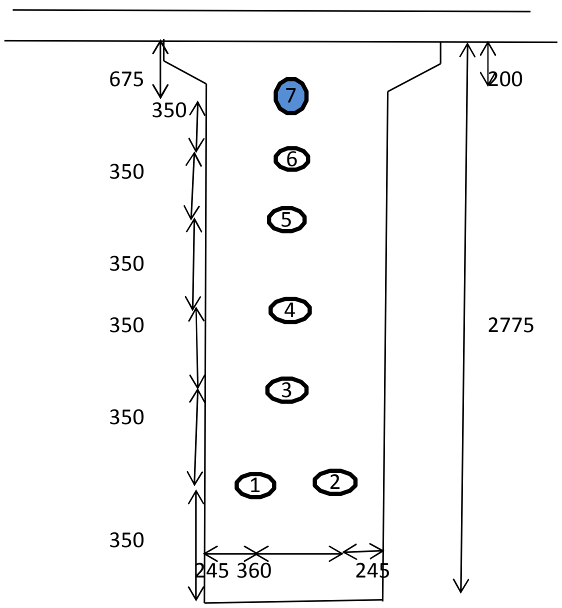
2. CABLE AT END SECTION

Vertical distance between two rows of cable (1st row to 2nd row) = 350 mm
Vertical distance between two rows of cable (2nd row to 3rd row) = 350 mm
Vertical distance between two rows of cable (3rd row to 4th row) = 350 mm
Vertical distance between two rows of cable (4th row to 5th row) = 350 mm
Distance of lowest cable centre from bottom of T-Girder= 350 mm

Half length of cable = 19720 mm



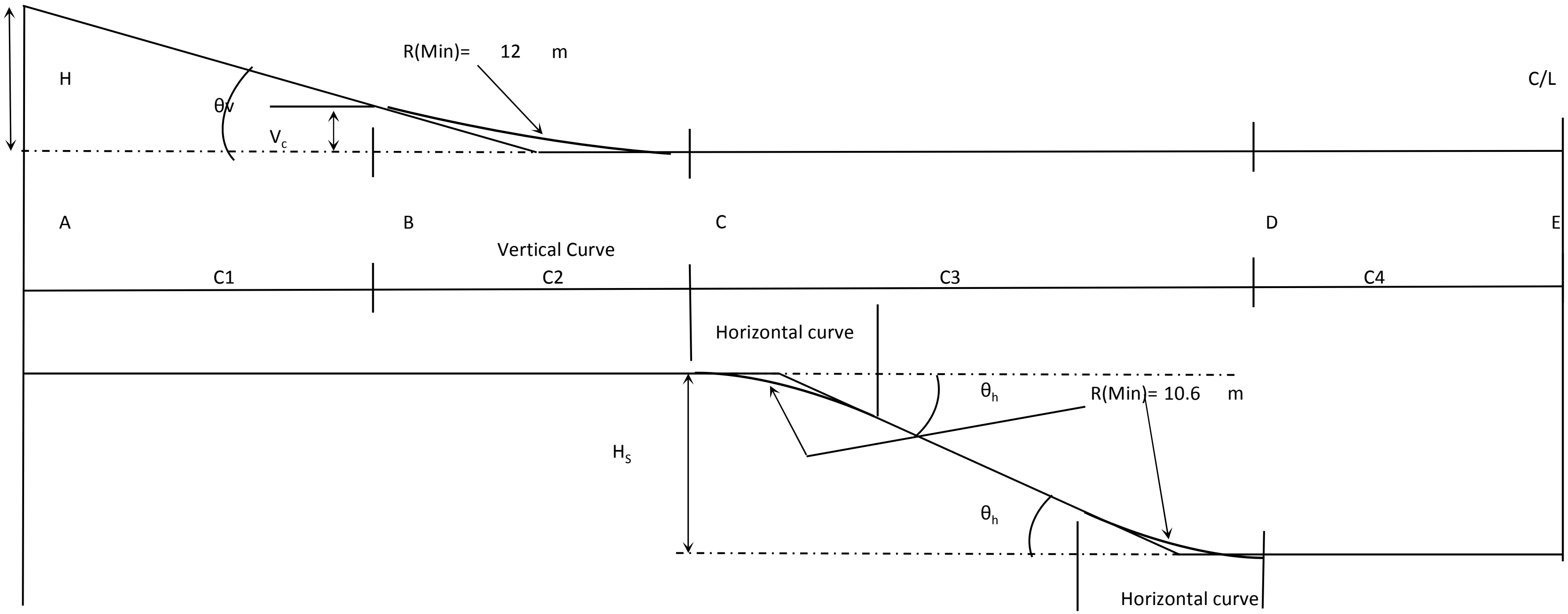
POSITION OF CABLE AT MID SECTION



POSITION OF CABLE AT END SECTION

3. Typical arrangement of cables:

Typical profile of cable in elevation & plan will be as follows:



Horrizontal splay to be given in cables 1 in 10, i.e. θH= $\tan^{-1} 1/10$= 5.71 °

LHc= 10.6X0.0997= 1.056 m

C3= $10xH_s+2xLHcX0.5$

C1+C2= 19.72 -(C3+C4) **Eqn-1**

$\tan\theta_v$= H/(C1+C2x0.5)

=>C1+0.5xC2=H/tanθv **Eqn-2**

Vc= $(C2/2)x\tan\theta_v$

LVc= $C2x\theta_v/(2xCos\theta_v xTan(0.5x\theta_v)$

Eqn-1-Eqn-2

=>C2= [{19.72 -(C3+C4)}-H/tanθv]x2 **Eqn-3**

Substitute the value of C2 in **Eqn-1**,

=>C1= 19.72 -(C3+C4+C2) **Eqn-4**

Cable No.	Length (C1) in m.	Length (C2) in m.	Length (C3) in m.	Length (C4) in m.	Lift height (H) in m	Height of vertical curve (V_c) in m	Vertical angle (θv) in deg.	Length of vertical curve (LV_c) in m	Horizontal sway (H_s) in m	Horizontal angle (θ_h) in deg.	Length of horizontal curve (LH_c) in m	Length 'AB' (m)
1	1.767	2.758	0.000	15.195	0.220	0.096	4.000	2.764	0.000	0.000	0.000	1.771
2	1.767	2.758	0.000	15.195	0.220	0.096	4.000	2.764	0.000	0.000	0.000	1.771
3	1.960	5.365	0.000	12.395	0.570	0.329	7.000	5.399	0.000	0.000	0.000	1.975
4	3.834	4.386	0.000	11.500	0.740	0.269	7.000	4.413	0.000	0.000	0.000	3.863
5	2.271	6.949	0.000	10.500	0.910	0.550	9.000	7.021	0.000	0.000	0.000	2.299
6	3.918	5.802	0.000	10.000	1.080	0.459	9.000	5.862	0.000	0.000	0.000	3.967
7	4.458	5.262	0.000	10.000	1.250	0.464	10.000	5.330	0.000	0.000	0.000	4.527

4. Force diagram of each cable after anchorage slip will be as shown follow:

According to IRC-112,2011,

The steel stress at jacking end= $\sigma_{po}=\sigma_{px}.e(kx+\mu\theta)$

σ_{po}= Applied force

$\sigma_{po}(x)$= Force at any place in cable

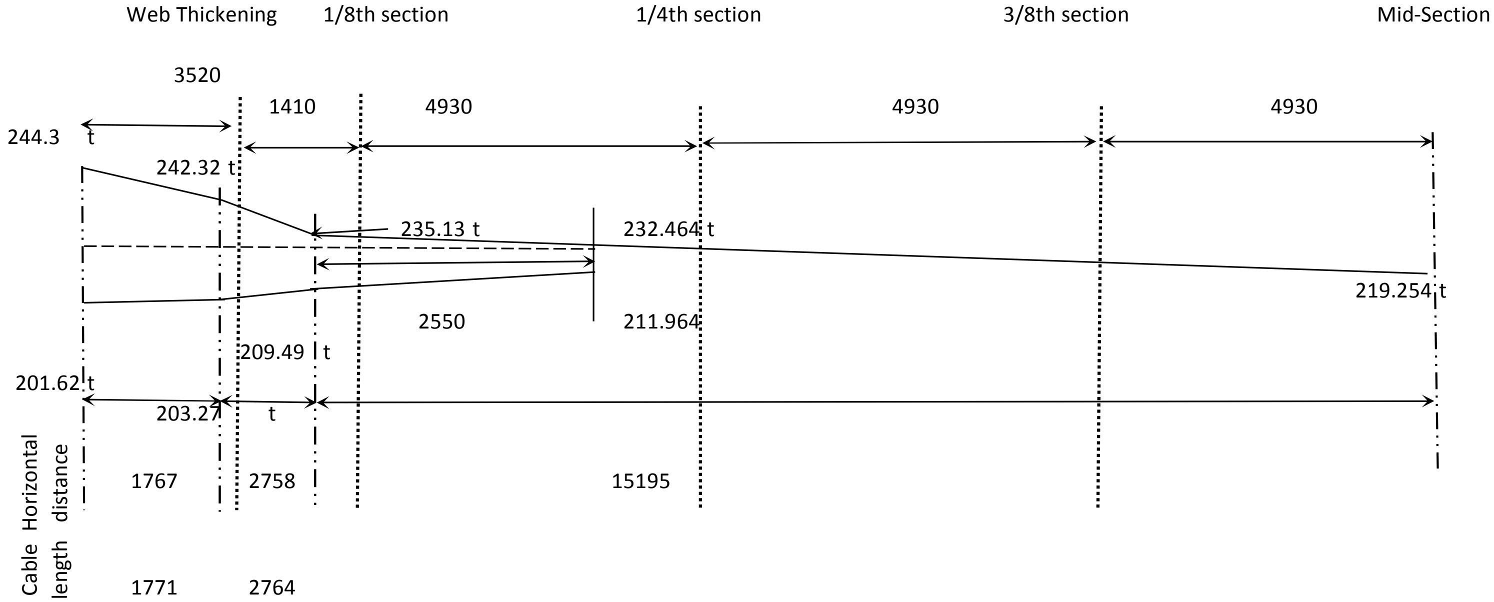
μ= Friction co-efficient = 0.25 for bright metal stress

k= Wooble co-efficient = 0.0046 relieved strand

Say slip loss= 6 mm

Modulus of elasticity of material of cable= 1.95E+06 Kg/sqcm

For cable 1&2:

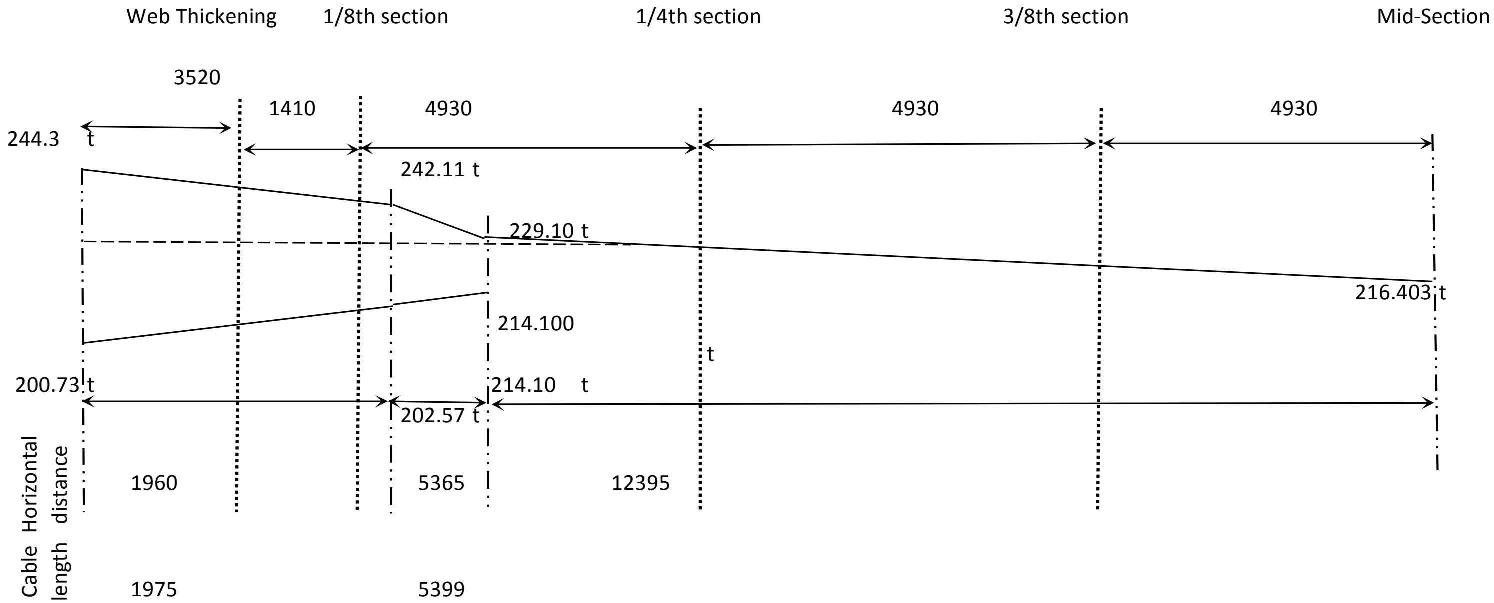


Area of the diagram

=0.5x(42.683+39.056)x1.771+0.5x(39.056+25.636)x2.764+0.5x(25.636+20.5)x2.55= 220.606 t-m

Slip of anchorage= (220.606x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 3:

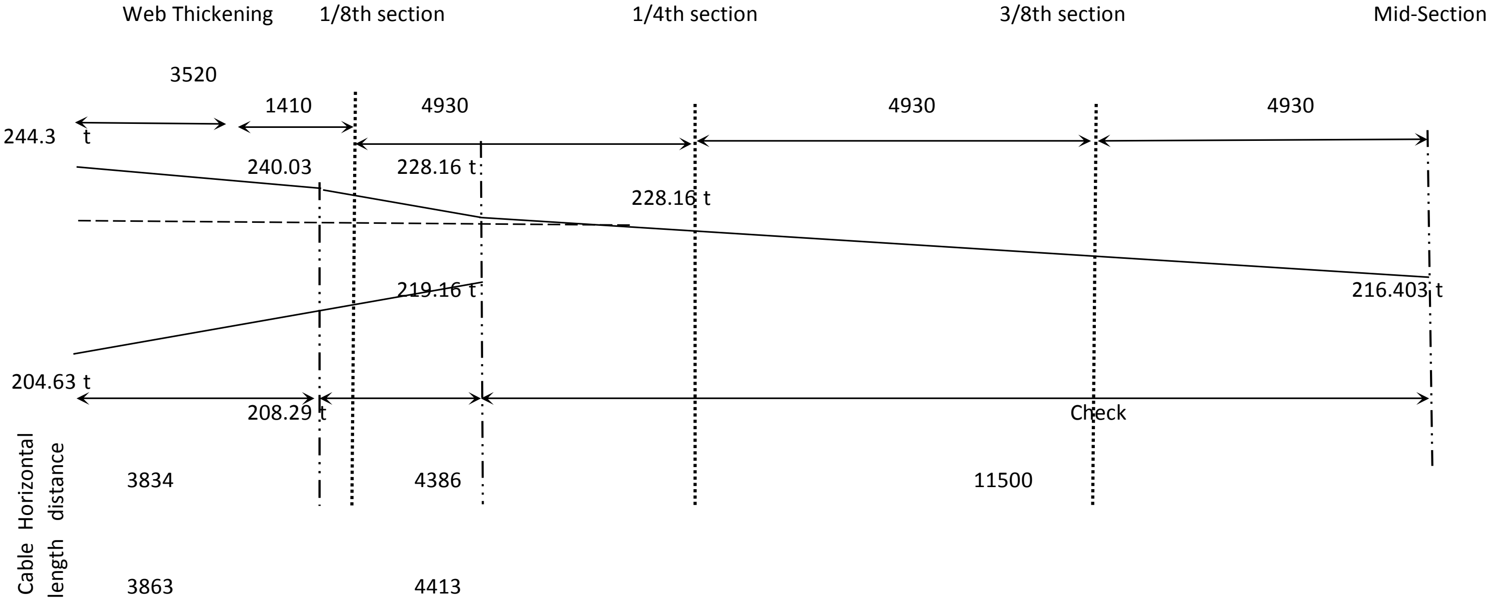


Area of the diagram

=0.5x(43.566+39.542)x1.975+0.5x(39.542+15)x5.399+0.5x(15+-214.1)x0= 229.304 t-m

Slip of anchorage= (229.304x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 4:

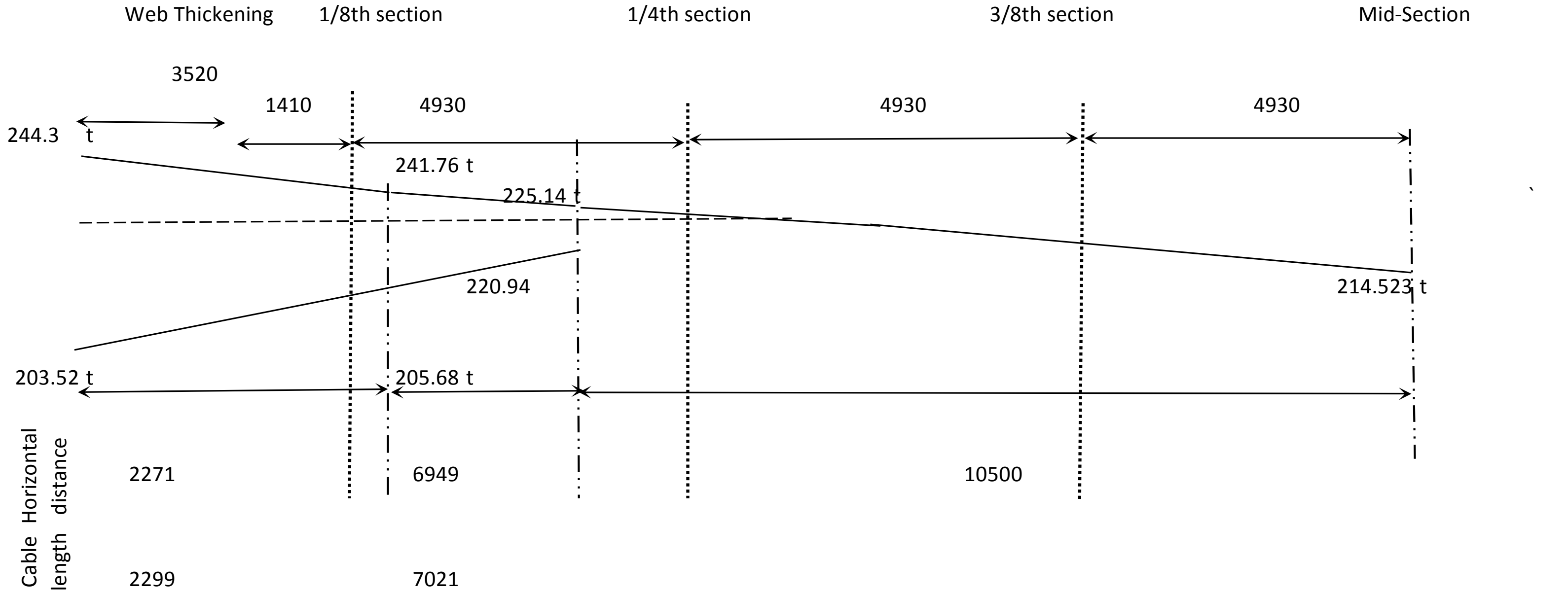


Area of the diagram

=0.5x(39.674+31.735)x3.863+0.5x(31.735+9)x4.413+0.5x(9+228.159)x0)= 227.807 t-m

Slip of anchorage= (227.807x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 5:

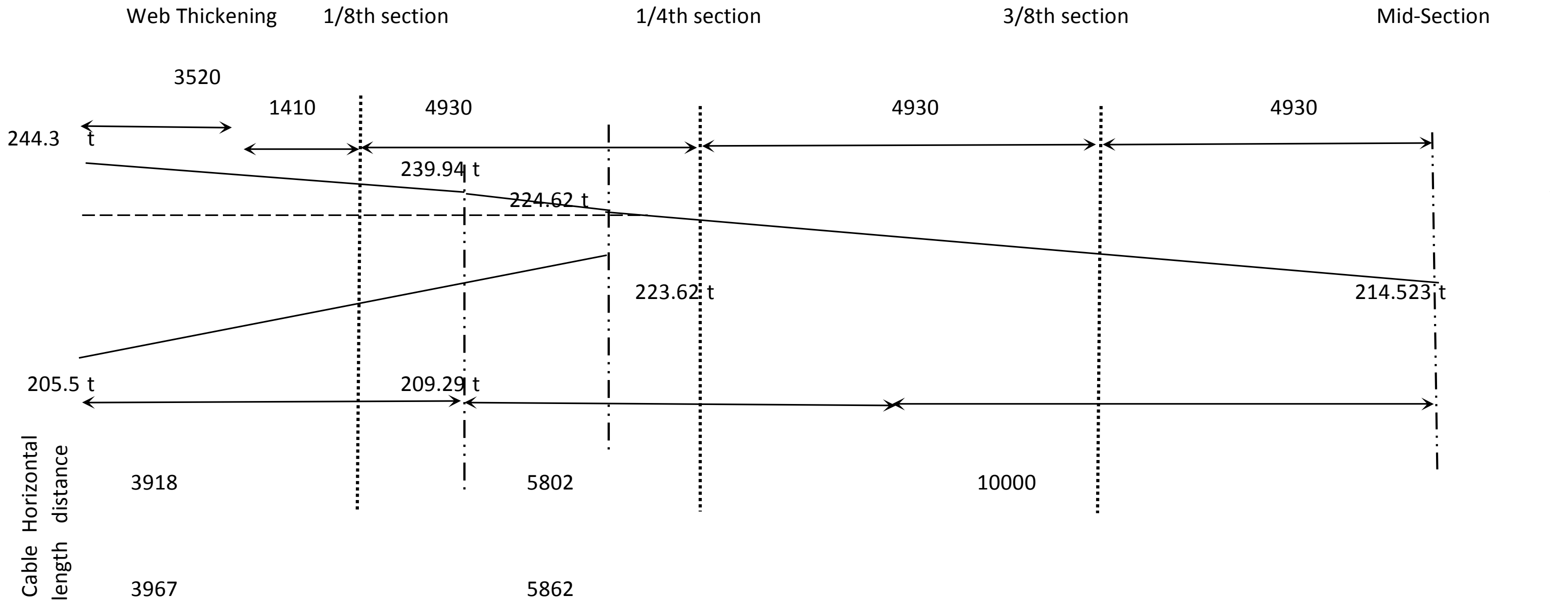


Area of the diagram

227.242 t-m

Slip of anchorage= (227.24220467294x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 6:



Area of the diagram = 227.845 t-m

Slip of anchorage= (227.845257798206x100000/((1950000)x18.772))x10= 6.0 **OK**

5. **Stages of Prestressing**

First Stage: 4 cables in each girder at 14 days or the

First stage prestressing will be on precast girder when the girder concrete attains a strength at least equal to 0.9 of its 28 days compressive strength or the concrete is 14 days old whichever is later. Using grade of girder and deck concrete as M45, strength of girder concrete at the time of stressing will be at least 40.5 Mpa. Cable no. 1,2,3 & 6 will be stressed during the first stage.

Second tag: 2 cables in each girder at 28 days after casting of deck.

Second stage stressing will be done after casting of deck slab and after the deck concrete have attained its 28 days strength. The deck will be cast after 7 days from the date of first stage prestress, i.e. when the girder concrete is 21 days old. Hence girder concrete will be 49 days old at the time of second stage presress and full composite action is obtained. Cable no. 4 & 5 will be stressed at this stage

Kerb, crash barrier, wearing course will be laid, when the girders are 60 days old.

6. **PROPERTIES OF GIRDER SECTION:**

Location	PREACAST GIRDER				COMPOSITE GIRDER				
	Area (A_p) in m^2	CG from bottom of girder Y_{bp} (m)	Section modulus of top of girder Z_{tp} (m^3)	Section modulus of bottom of girder Z_{bp} (m^3)	Area (A_c) in m^2	CG from bottom of girder Y_{bg} (m)	Section modulus of top of girder Z_{tgc} (m^3)	Section modulus of bottom of girder Z_{bgc} (m^3)	Section modulus of top of slab Z_{ts} (m^3)
End Girder	1.267	1.444	0.902	0.831	2.024	1.986	2.887	1.147	2.229
Central Girder	1.267	1.444	0.902	0.831	1.966	1.959	2.707	1.128	2.106

7. **PROPERTIES OF CONCRETE WITH AGE:**

For the purpose of calculation of loss, maturity of concrete at different days and their properties are taken as follows:

Age of concrete (days)		14	21	49	60
βcc(t)=exp{.25*[1-(28/(t/1))^.5]}		0.9016279	0.9620632	1.0629178	1.08243971
fcm(t)=βcc(t).fcm (T/m2)		4057.3255	4329.2844	4783.1301	4870.97871
Ecm= 100*5000*fcm^0.5, fcm in Mpa (T/m2)		3295999.3	3360779.33	3462811.3	3481769.63
Shrinkage co-efficient,εs=		0.0003285	0.00034737	0.0003545	0.00035675
Creep co-efficient,εc=		0	0.97306564	1.5583585	1.6838354
Permissible Temporary Stress (T/m^2)	Tensile	298	317	234	-
	Compressive	2160	2160	2160	-
Permissible Stress during service (T/m^2)	Tensile	-	-	-	0
	Compressive	-	-	-	1754

8. INITIAL PRESTRESS AT DIFFERENT SECTIONS

i). END GIRDER:

a).MID SECTION: 19.72 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.13	0	0	219.25	0	219.254	1.314	288.100	(869.434/1.267)-(1026.594/0.902)=	(869.434/1.267)+(1026.594/0.831)=	-
	2	0.13	0	0	219.25	0	219.254	1.314	288.100			
	3	0.13	0	0	216.40	0	216.403	1.314	284.353	-451.916	1921.586	-
	6	0.67	0	0	214.52	0	214.523	0.774	166.040			
	Total	1.06				0	869.434		1026.594			
2nd Stage at 28 Days	4	0.31	0	0	216.40	0	216.403	1.676	362.691	(430.925/2.024)-(683.617/2.887)=	(430.925/2.024)+(683.617/1.147)=	(430.925/2.024)-(683.617/2.229)=
	5	0.49	0	0	214.52	0	214.523	1.496	320.926			
	Total	0.8				0	430.925		683.617	-23.884	808.912	-93.784

CG of the first stage cable from soffit of girder= 0.265 m

CG of the second stage cable from soffit of girder= 0.400 m

CG of the all cables from soffit of girder= 0.310 m

b).3/8TH SECTION: 14.79 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.13	0	0	216.41	0	216.412	1.314	284.365	(867.319/1.267)-(1021.392/0.902)=	(867.319/1.267)+(1021.392/0.831)=	-
	2	0.13	0	0	216.41	0	216.412	1.314	284.365			
	3	0.13	0	0	215.49	0	215.487	1.314	283.150			
	6	0.67	0	0	219.01	0	219.008	0.774	169.512	-447.819	1913.658	-
	Total	1.06				0	867.319		1021.392			
2nd Stage at 28 Days	4	0.31	0	0	217.58	0	217.584	1.676	364.671	(435.119/2.024)-(690.104/2.887)=	(435.119/2.024)+(690.104/1.147)=	(435.119/2.024)-(690.104/2.229)=
	5	0.49	0	0	217.53	0	217.535	1.496	325.432			
	Total	0.8				0	435.119		690.104	-24.058	816.639	-94.622

CG of the first stage cable from soffit of girder= 0.265 m

CG of the second stage cable from soffit of girder= 0.400 m

CG of the all cables from soffit of girder= 0.310 m

c).1/4TH SECTION: 9.86 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.13	0	0	213.57	0	213.569	1.314	280.630	(865.204/1.267)-(1016.191/0.902)=	(865.204/1.267)+(1016.191/0.831)=	-
	2	0.13	0	0	213.57	0	213.569	1.314	280.630			
	3	0.13	0	0	214.57	0	214.571	1.314	281.946			
	6	0.67	0	0	223.49	0	223.494	0.774	172.984	-443.722	1905.729	-
	Total	1.06				0	865.204		1016.191			
2nd Stage at 28 Days	4	0.31	0	0	218.77	0	218.766	1.676	366.651	(439.313/2.024)-(696.59/2.887)=	(439.313/2.024)+(696.59/1.147)=	(439.313/2.024)-(696.59/2.229)=
	5	0.49	0	0	220.55	0	220.547	1.496	329.939			
	Total	0.8				0	439.313		696.590	-24.233	824.367	-95.461

CG of the first stage cable from soffit of girder= 0.265 m
CG of the second stage cable from soffit of girder= 0.400 m
CG of the all cables from soffit of girder= 0.310 m

d).1/8TH SECTION: 4.93 M

Stage of Prestressing	Cable No.	CG form soffit of Girder (ŷ) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-ŷ) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.130	0.000	0	209.88	0	209.885	1.314	275.788	(837.373/1.267)-(969.948/0.902)=	(837.373/1.267)+(969.948/0.831)=	-
	2	0.130	0.000	0	209.88	0	209.885	1.314	275.788			
	3	0.130	0.122	0	208.95	25.465	207.394	1.314	272.515			
	6	0.750	0.122	0	211.79	25.811	210.211	0.694	145.856	-414.420	1828.116	-
	Total	1.140				51.275	837.373		969.948			
2nd Stage at 28 Days	4	0.377	0.122	0	211.01	25.716	209.437	1.609	336.924	(418.35/2.024)-(605.467/2.887)=	(418.35/2.024)+(605.467/1.147)=	(418.35/2.024)-(605.467/2.229)=
	5	0.701	0.157	0	211.52	33.089	208.914	1.285	268.544			
	Total	1.078				58.804	418.350		605.467	-3.027	734.565	-64.937

CG of the first stage cable from soffit of girder= 0.285 m

CG of the second stage cable from soffit of girder= 0.539 m

CG of the all cables from soffit of girder= 0.370 m

e).WEB-THICKENING SECTION: 3.52 M

Stage of Prestressing	Cable No.	CG form soffit of Girder (ŷ) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-ŷ) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.130	0.070	0	207.22	14.455178	206.719	1.314	271.628	(825.17/1.267)-(837.134/0.902)=	(825.17/1.267)+(837.134/0.831)=	-
	2	0.130	0.070	0	207.22	14.455178	206.719	1.314	271.628			
	3	0.226	0.122	0	205.92	25.095	204.385	1.218	248.987			
	6	1.228	0.122	0	208.90	25.459	207.348	0.216	44.891	-276.808	1658.661	-
	Total	1.713				79.465	825.170		837.134			
2nd Stage at 28 Days	4	0.742	0.122	0	207.99	25.348	206.444	1.244	256.777	(412.3/2.024)-(544.376/2.887)=	(412.3/2.024)+(544.376/1.147)=	(412.3/2.024)-(544.376/2.229)=
	5	0.589	0.157	0	208.42	32.604	205.856	1.397	287.599			
	Total	1.331				57.952	412.300		544.376	15.144	678.314	-40.519

CG of the first stage cable from soffit of girder= 0.428 m
CG of the second stage cable from soffit of girder= 0.666 m
CG of the all cables from soffit of girder= 0.507 m

G. **ELONGATION CALCULATION**

Area of each cable= 1877.2 sq.mm
Modulus of elasticity 1.95E+05 Mpa

Cable No.	Segment	Length (mm)	Force at Start (T)	Force at End (T)	Elongation (mm)	Total elongation in each side (mm)
1&2	L_1	1767	201.62	203.27	10	114
	L_2	2758	203.3	209.49	16	
	L_3	2550	209.5	211.96	15	
	L_4	12645	211.96	219.254	74	
3	L_1	1960	200.73	202.57	11	114
	L_2	5365	202.6	214.10	31	
	L_3	0	214.1	214.10	0	
	L_4	12395	214.10	216.403	73	
4	L_1	3834	204.63	208.29	22	116
	L_2	4386	208.3	219.16	26	
	L_3	0	219.2	219.16	0	
	L_4	11500	219.16	216.403	68	
5	L_1	2271	203.52	205.68	13	116
	L_2	6949	205.7	220.94	40	
	L_3	0	220.9	225.14	0	
	L_4	10500	225.14	214.523	63	
6	L_1	3918	205.50	209.29	22	116
	L_2	5802	209.3	223.62	34	
	L_3	0	223.6	223.62	0	
	L_4	10000	223.62	214.523	60	

The elongation length calculated only for the cable between the midspan and end faces.
Additional length for attaching the jack may be added in consultation with the system manufacturer.
Extra elongation may be added @ 7mm/m for portion between end face and gripping point of jack.

H. LOSSES IN PRESTRESS

1. END GIRDER:

i). Stage-1: Between 14 days to 21 days

Average stress at CG of 1st stage cable= (Ref.: Stress tables)
=(2x(1250.539+1331.641+1212.439+1104.347)+1069.465)/9
1207.488 T/m^2

Elastic shortening:

Loss in Prestressing force due to elastic shortening=
(0.5x1207.488x19500000x0.0018772x(4-1)/3295999)=
= 20.116 t

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	20.116x 100/(869.434)=	2.31 %
ii) At 3/8 th section=	20.116x 100/(867.319)=	2.32 %
iii) At 1/4 th section=	20.116x 100/(865.204)=	2.33 %
iv) At 1/8 th section=	20.116x 100/(837.373)=	2.40 %
v) At web thickening section=	20.116x 100/(825.17)=	2.44 %

Relaxation loss in 1st stage cable:

Average stresses in 1st stage cables at differen sections, just after seating of anchorage will be as follows :

i) At mid-section=	(1000x(869.434x0.977))/(4x18.772)=	11310.97 Kg/cm^2
ii) At 3/8 th section=	(1000x(867.319x0.977))/(4x18.772)=	11282.80 Kg/cm^2
iii) At 1/4 th section=	(1000x(865.204x0.977))/(4x18.772)=	11254.63 Kg/cm^2
iv) At 1/8 th section=	(1000x(837.373x0.976))/(4x18.772)=	10883.99 Kg/cm^2
v) At web thickening section=	(1000x(825.17x0.976))/(4x18.772)=	10721.47 Kg/cm^2

Average stress in 1st stage cables:
=(2x(10721.472+10883.993+11254.631+11282.798)+11310.966)/9
= 11066.306 Kg/cm^2
= 0.595 of Ultimate tensile stress
Ultimate tensile stress= (349x1000)/18.772= 18592 Kg/cm^2

Ref: Table- 6.2, IRC-112:2011
1000 hour relaxation loss in 1 st stage cables= 1.190 %
Final (0.5x10^6 hours) relaxation loss in 1 st stage cables= 3.571 %

Loss due to shrinkage and creep in 1st stage cable:

Shrinkage strain= (0.000347365891535569-0.000328469167354467)= 1.89E-05
Creep strain between 14 days and 21 days = 3.28E-04

Average stress at CG of 1st stage cables at 14 days just after seating of anchorages is: [Ref: stress Tables]
=(2x(1217.387+1293.258+1173.347+1065.193)+1030.25)/9
1169.847 T/m^2 = 11.698 Mpa

Assumed loss in different sections due to creep and shrinkage as follows:

i) At mid-section=	5.85 %
ii) At 3/8 th section=	5.86 %
iii) At 1/4 th section=	5.88 %
iv) At 1/8 th section=	6.07 %
v) At web thickening section=	6.16 %

Average stress at CG of 1st stage cables at 21 days with 1000 hour relaxation loss will be as follows:
[Ref: stress Tables]
=(2x(1116.383+1176.206+1053.513+945.113)+909.925)/9
1054.706 T/m^2 = 10.547 Mpa
Average stress along CG of 1st stage cables during 14 days and 21 days will be
(0.5x(10.547+11.69847)= 11.123 Mpa
Creep strain during this period= 3.28E-04
Loss due to creep and shrinkage=
(0.0000188967241811016+0.000328)x(1950000x4x18.772)/1000= 50.849 T
Percentage loss:

i) At mid-section=	(50.849X100)/(869.434)=	5.85 % Hence OK
ii) At 3/8 th section=	(50.849X100)/(867.319)=	5.86 % Hence OK
iii) At 1/4 th section=	(50.849X100)/(865.204)=	5.88 % Hence OK
iv) At 1/8 th section=	(50.849X100)/(837.373)=	6.07 % Hence OK
v) At web thickening section=	(50.849X100)/(825.17)=	6.16 % Hence OK

ii). Stage-2: Between 21 days to 49 days

Loss due to shrinkage and creep :

Shrinkage strain= (0.000354499693215269-0.000347365891535569)= 7.13E-06
Creep strain between 21 days and 49 days = 1.43E-04

Average stress at CG of 1st stage cables at 14 days just after seating of anchorages is: [Ref: stress Tables]
=(2x(1057.099+1031.019+796.155+625.2)+566.459)/9
842.823 T/m^2 = 8.428 Mpa

Assumed loss in different sections due to creep and shrinkage betweeen 21 days to 49 as follows:

i) At mid-section=	2.53 %
ii) At 3/8 th section=	2.54 %
iii) At 1/4 th section=	2.54 %
iv) At 1/8 th section=	2.63 %
v) At web thickening section=	2.67 %

Average stress at CG of 1st stage cables at 21 days with 1000 hour relaxation loss will be as follows:
[Ref: stress Tables]
=(2x(1020.822+989.019+753.381+582.357)+523.549)/9
801.634 T/m^2 = 8.016 Mpa
Average stress along CG of 1st stage cables during 14 days and 21 days will be
(0.5x(8.016+8.42823)= 8.222 Mpa
Creep strain during this period= 1.43E-04
Loss due to creep and shrinkage=
(0.0000071338016796995+0.000143)x(1950000x4x18.772)/1000= 22.011 T
Percentage loss:

i) At mid-section=	(22.011X100)/(869.434)=	2.53 % Hence OK
ii) At 3/8 th section=	(22.011X100)/(867.319)=	2.54 % Hence OK
iii) At 1/4 th section=	(22.011X100)/(865.204)=	2.54 % Hence OK
iv) At 1/8 th section=	(22.011X100)/(837.373)=	2.63 % Hence OK
v) At web thickening section=	(22.011X100)/(825.17)=	2.67 % Hence OK

iii). Stage-3: Between 49 days to 60 days

Additional stress at CG of 1st & 2nd stage cables due to 2nd stage prestressing are as follows:

Total depth of precast girder:	2.775 m		
a) At mid section:			
Stress at top= f_{tg}=	-23.884 T/m^2		
Stress at bottom= f_{bg}=	808.912 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(808.912-(808.912--23.884)x0.265/2.775)=	729.384 T/m^2
Stress at CG of 2nd stage cables=		(808.912-(808.912--23.884)x0.4/2.775)=	688.869 T/m^2
b) At 3/8 th section:			
Stress at top= f_{tg}=	-24.058 T/m^2		
Stress at bottom= f_{bg}=	816.639 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(816.639-(816.639--24.058)x0.265/2.775)=	736.356 T/m^2
Stress at CG of 2nd stage cables=		(816.639-(816.639--24.058)x0.4/2.775)=	695.457 T/m^2
c) At 1/4 th section:			
Stress at top= f_{tg}=	-24.233 T/m^2		
Stress at bottom= f_{bg}=	824.367 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(824.367-(824.367--24.233)x0.265/2.775)=	743.33 T/m^2
Stress at CG of 2nd stage cables=		(824.367-(824.367--24.233)x0.4/2.775)=	702.046 T/m^2
d) At 1/8 th section:			
Stress at top= f_{tg}=	-3.027 T/m^2		
Stress at bottom= f_{bg}=	734.565 T/m^2		
CG of 1st stage cables=	0.285 m		
CG of 2nd stage cables=	0.539 m		
Stress at CG of 1st stage cables=		(734.565-(734.565--3.027)x0.285035631701054/2.7	658.803 T/m^2
Stress at CG of 2nd stage cables=		(734.565-(734.565--3.027)x0.538929026393131/2.7	591.318 T/m^2

e) Web thickening section:

Stress at top= f_{tg} =	15.144 T/m^2		
Stress at bottom= f_{bg} =	678.314 T/m^2		
CG of 1st stage cables=	0.428 m		
CG of 2nd stage cables=	0.666 m		
Stress at CG of 1st stage cables=		(678.314-(678.314-15.144)x0.42831826434307/2.77	575.954 T/m^2
Stress at CG of 2nd stage cables=		(678.314-(678.314-15.144)x0.665549784066415/2.7	519.261 T/m^2

Average stress at CG of 1st stage cables=
(2x(575.954+658.803+743.33+736.356)+729.384)/9= 684.252 T/m^2
Average stress at CG of 2nd stage cables=
(2x(519.261+591.318+702.046+695.457)+688.869)/9= 633.893 T/m^2

<u>Elastic shortening:</u>

No. of cable stressed= 2 in each web

Loss in 1st stage cables due to second stage prestressing=
(2x684.252x19500000x0.0018772/3462811)= 14.466 T

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	14.466x 100/(869.434)=	1.66 %
ii) At 3/8 th section=	14.466x 100/(867.319)=	1.67 %
iii) At 1/4 th section=	14.466x 100/(439.313)=	1.67 %
iv) At 1/8 th section=	14.466x 100/(837.373)=	1.73 %
v) At web thickening section=	14.466x 100/(825.17)=	1.75 %

Loss in 2nd stage cables due to second stage prestressing=
(1x633.893x19500000x0.0018772/(2x3462811))= 3.35 T

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	3.35x 100/(430.925)=	0.78 %
ii) At 3/8 th section=	3.35x 100/(435.119)=	0.77 %
iii) At 1/4 th section=	3.35x 100/(439.313)=	0.76 %
iv) At 1/8 th section=	3.35x 100/(418.35)=	0.80 %
v) At web thickening section=	3.35x 100/(412.3)=	0.81 %

<u>Relaxation loss in 2nd stage cable:</u>

Average stresses in 1st stage cables at differen sections, just after seating of anchorage will be as follows :

i) At mid-section=	(1000x(430.925x0.992))/(2x18.772)=	11388.65 Kg/cm^2
ii) At 3/8 th section=	(1000x(435.119x0.992))/(2x18.772)=	11500.35 Kg/cm^2
iii) At 1/4 th section=	(1000x(439.313x0.992))/(2x18.772)=	11612.06 Kg/cm^2
iv) At 1/8 th section=	(1000x(418.35x0.992))/(2x18.772)=	11053.71 Kg/cm^2
v) At web thickening section=	(1000x(412.3x0.992))/(2x18.772)=	10892.54 Kg/cm^2

Average stress in 1st stage cables:
=(2x(10892.54+11053.707+11612.056+11500.353)+11388.649)/9
= 11278.44 Kg/cm^2
= 0.607 of Ultimate tensile stress
Ultimate tensile stress= (349x1000)/18.772= 18592 Kg/cm^2

Ref: Table- 6.2, IRC-112:2011
1000 hour relaxation loss in 1 st stage cables= 1.333 %
Final (0.5x10^6 hours) relaxation loss in 1 st stage cables= 3.999 %

Loss due to shrinkage and creep in 2nd stage cable:

Shrinkage strain= (0.000356751507507038-0.000354499693215269)= 0.000002

Creep strain between 28 days and 40 days = 5.22E-05

Average stress at CG of all cables at 49 days , just after seating of 2nd stage anchorage will be as follows:

[Ref: stress Tables]

=(2x(1549.941+1603.754+1485.706+1314.446)+1251.119)/9

1462.09 T/m^2 = 14.621 Mpa

Assumed loss in different sections at 1st stage cables after 2nd stage prestressing due to creep and shrinkage as follows:

i) At mid-section=	0.91 %
ii) At 3/8 th section=	0.91 %
iii) At 1/4 th section=	0.92 %
iv) At 1/8 th section=	0.95 %
v) At web thickening section=	0.96 %

Assumed loss in different sections at 2nd stage cables after 2nd stage prestressing due to creep and shrinkage as follows:

i) At mid-section=	0.92 %
ii) At 3/8 th section=	0.91 %
iii) At 1/4 th section=	0.90 %
iv) At 1/8 th section=	0.95 %
v) At web thickening section=	0.96 %

Average stress at CG of all cables at 60 days before completion of W.C., Railing, Crash Barrier: [Ref: stress Tables]

=(2x(1524.619+1574.758+1454.334+1283.141)+1219.881)/9

1432.62 T/m^2 = 14.326 Mpa

Average stress along CG of 1st stage cables during 49 days and 60 days will be

(0.5x(14.6209+14.3262)= 14.474 Mpa

Creep strain during this period= (0.0000521601619669419X1.447355)= 0.000052

Loss due to creep and shrinkage in first stage cables=

(0.000002+0.000052)x(1950000x4x18.772)/1000= 7.930 T

Loss due to creep and shrinkage in second stage cables=

(0.000002+0.000052)x(1950000x4x18.772)/1000= 3.965 T

Percentage loss in different sections will be as follows:

a) For first stage prestress:		
i) At mid-section=	(7.93X100)/(869.434)=	0.91 % Hence OK
ii) At 3/8 th section=	(7.93X100)/(867.319)=	0.91 % Hence OK
iii) At 1/4 th section=	(7.93X100)/(865.204)=	0.92 % Hence OK
iv) At 1/8 th section=	(7.93X100)/(837.373)=	0.95 % Hence OK
v) At web thickening section=	(7.93X100)/(825.17)=	0.96 % Hence OK
a) For second stage prestress:		
i) At mid-section=	(3.965X100)/(430.925)=	0.92 % Hence OK
ii) At 3/8 th section=	(3.965X100)/(435.119)=	0.91 % Hence OK
iii) At 1/4 th section=	(3.965X100)/(439.313)=	0.90 % Hence OK
iv) At 1/8 th section=	(3.965X100)/(418.35)=	0.95 % Hence OK
v) At web thickening section=	(3.965X100)/(412.3)=	0.96 % Hence OK

iv). Stage-4: Between 60 days to end

Loss due to shrinkage and creep in 2nd stage cable:

Shrinkage strain=	1.32E-05
Creep strain between 60 days to infinity =	7.80E-04

Average stress at CG of all cables at 60 days , after completion of WC, Railing, Crash barrier:
[Ref: stress Tables]
=(2x(1514.465+1549.9+1409.221+1226.702)+1159.668)/9
1395.583 T/m^2 = 13.956 Mpa

Assumed loss in different sections at 1st stage cables after 3rd stage casting due to creep and shrinkage as follows:

i) At mid-section=	13.36 %
ii) At 3/8 th section=	13.39 %
iii) At 1/4 th section=	13.42 %
iv) At 1/8 th section=	13.87 %
v) At web thickening section=	14.07 %

Assumed loss in different sections at 2nd stage cables after 3rd stage casting due to creep and shrinkage as follows:

i) At mid-section=	13.48 %
ii) At 3/8 th section=	13.35 %
iii) At 1/4 th section=	13.22 %
iv) At 1/8 th section=	13.88 %
v) At web thickening section=	14.08 %

Average stress at CG of all cables after final loss: [Ref: stress Tables]
=(2x(1252.36+1249.452+1092.195+933.029)+841.924)/9
1099.555 T/m^2 = 10.996 Mpa
Average stress along CG of 1st stage cables during 7 days and 21 days will be
(0.5x(13.95583+10.99555)= 12.476 Mpa
Creep strain during this period= 7.80E-04
Loss due to creep and shrinkage in first stage cables=
(0.0000132+0.00078)x(1950000x4x18.772)/1000= 116.142 T
Loss due to creep and shrinkage in second stage cables=
(0.0000132+0.00078)x(1950000x2x18.772)/1000= 58.071 T
Percentage loss in different sections will be as follows:

a) For first stage prestress:

i) At mid-section=	(116.142X100)/(869.434)=	13.36 % Hence OK
ii) At 3/8 th section=	(116.142X100)/(867.319)=	13.39 % Hence OK
iii) At 1/4 th section=	(116.142X100)/(865.204)=	13.42 % Hence OK
iv) At 1/8 th section=	(116.142X100)/(837.373)=	13.87 % Hence OK
v) At web thickening section=	(116.142X100)/(825.17)=	14.07 % Hence OK

a) For second stage prestress:

i) At mid-section=	(58.071X100)/(430.925)=	13.48 % Hence OK
ii) At 3/8 th section=	(58.071X100)/(435.119)=	13.35 % Hence OK
iii) At 1/4 th section=	(58.071X100)/(439.313)=	13.22 % Hence OK
iv) At 1/8 th section=	(58.071X100)/(418.35)=	13.88 % Hence OK
v) At web thickening section=	(58.071X100)/(412.3)=	14.08 % Hence OK

v). Total percentage loss in each different section will be as follows:

Stages of Prestressing	% LOSS → / SECTIONS ↓	Elastic shortening 1st stage	Final Relaxation in 1st stage cables	Creep & shrinkage between 10 days & 21 days	Creep & shrinkage between 21 days & 49 days	Elasting shortening 2nd stage	Relaxation 2nd stage cables	Creep and shrinkage 49 days to 60 days	Final creep and shrinkage	Total	20% Higher time dependent loss (creep, shrinkage and relaxation)
1st stage cables	mid section	2.314	3.571	5.849	2.532	1.664		0.912	13.358	30.199	5.244
	3/8 th section	2.319	3.571	5.863	2.538	1.668		0.914	13.391	30.264	5.255
	1/4 th section	2.325	3.571	5.877	2.544	1.672		0.917	13.424	30.330	5.267
	1/8 th section	2.402	3.571	6.072	2.629	1.728		0.947	13.870	31.219	5.418
	Web thk section	2.438	3.571	6.162	2.667	1.753		0.961	14.075	31.628	5.487
2nd stage cables	mid section					0.777	3.999	0.920	13.476	19.173	3.679
	3/8 th section					0.770	3.999	0.911	13.346	19.026	3.651
	1/4 th section					0.763	3.999	0.903	13.219	18.883	3.624
	1/8 th section					0.801	3.999	0.948	13.881	19.629	3.766
	Web thk section					0.813	3.999	0.962	14.085	19.858	3.809

I. STRESS TABLES

1. END GIRDER:

i). SECTION AT 0.5 L

Description of load and prestress	Stress at top Of Girder (T/m²)		Stress at bottom of Girder(T/m²)		Stress at top of Deck Slab (T/m²)		Stress at bottom of Deck Slab(T/m²)		Stress at C.G of 1st stage cables (T/m²)	Stress at C.G of all stage cables (T/m²)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-451.916	-451.916	1921.586	1921.586						
Self weight of Precast girder	705.709	253.793	-766.004	1155.582					1069.465	
Loss due to elastic shortenings during 1st stage cables	10.456	264.249	-44.460	1111.122					1030.250	
Relaxation loss of 1st stage cables	5.380	258.869	-22.875	1088.247						
Final Relaxation loss for 1st stage cables	16.139		-68.625							
Creep and shrinkage loss between 10 days & 21 days	26.428	285.298	-112.375	975.872					909.925	
Weight of Deck slab, cast-in-situ diaphragms	387.533	672.830	-420.643	555.228					566.459	
Creep & Shrinkage loss between 21 days & 49	11.441	684.271	-48.648	506.581					523.549	
2nd stage prestress	-23.884	648.947	808.912	1364.141	-93.784	-93.784	-23.884	-23.884		
Loss due to elastic shortenings during 1st stage cables	7.519	656.466	-31.972	1332.168						
Loss due to elastic shortenings during 2nd stage cables	0.186	656.651	-6.288	1325.880	0.729	-93.055	0.186	-23.698		1251.119
Relaxation loss of 2nd stage cables	0.318	656.970	-10.783	1315.097	1.250	-91.805	0.318	-24.016		
Final Relaxation loss for 2nd stage cables	0.955		-32.350		3.751		0.955			
Creep and shrinkage loss between 49 days & 60 days	4.342	661.311	-24.969	1290.127	0.863	-90.942	0.220	-23.797		1219.881
Self weight of hand rail and wearing course	70.634	731.946	-76.669	1213.458	28.583	-62.359	22.069	-1.728		1159.668
Creep and shrinkage loss from 60 days to infinity	63.587	795.533	-365.700	847.758	12.638	-49.721	3.219	1.491		841.924
Additional loss due to full relaxation	11.396	806.929	-67.317	780.441	2.500	-47.220	0.637	2.127		
Carriage way live load and footpath live load	152.885	959.814	-384.813	395.629	198.017	150.797	152.885	155.013		
20 % higher time dependent loss	24.5789	984.393	-130.5353	265.093	3.4504	154.247	0.8787	155.891		

Compressive Stress on 14 days	=	1155.582	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	684.271	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1364.141	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-93.784	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	959.814	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	984.393	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-447.819	-447.819	1913.658	1913.658						
Self weight of Precast girder	658.702	210.883	-714.981	1198.677					1104.347	
Loss due to elastic shortenings during 1st stage cables	10.386	221.269	-44.384	1154.293					1065.193	
Relaxation loss of 1st stage cables	5.331	215.938	-22.781	1131.512						
Final Relaxation loss for 1st stage cables	15.993		-68.342							
Creep and shrinkage loss between 10 days & 21 days	26.253	242.191	-112.187	1019.326					945.113	
Weight of Deck slab, cast-in-situ diaphragms	360.958	603.149	-391.798	627.528					625.200	
Creep & Shrinkage loss between 21 days & 49	11.365	614.514	-48.565	578.962					582.357	
2nd stage prestress	-24.058	579.091	816.639	1444.167	-94.622	-94.622	-24.058	-24.058		
Loss due to elastic shortenings during 1st stage cables	7.469	586.560	-31.918	1412.249						
Loss due to elastic shortenings during 2nd stage cables	0.185	586.745	-6.287	1405.962	0.729	-93.894	0.185	-23.873		1314.446
Relaxation loss of 2nd stage cables	0.321	587.066	-10.886	1395.076	1.261	-92.633	0.321	-24.194		
Final Relaxation loss for 2nd stage cables	0.962		-32.659		3.784		0.962			
Creep and shrinkage loss between 49 days & 60 days	4.314	591.380	-24.938	1370.137	0.862	-91.770	0.219	-23.975		1283.141
Self weight of hand rail and wearing course	66.206	657.586	-71.863	1298.274	26.791	-64.979	20.685	-3.290		1226.702
Creep and shrinkage loss from 60 days to infinity	63.178	720.764	-365.245	933.029	12.628	-52.351	3.211	-0.079		933.029
Additional loss due to full relaxation	11.303	732.067	-67.334	865.696	2.523	-49.828	0.641	0.563		
Carriage way live load and footpath live load	134.371	866.438	-338.213	527.483	174.038	124.210	134.371	134.934		
20 % higher time dependent loss	24.4132	890.852	-130.3886	397.094	3.4549	127.665	0.8784	135.812		

Compressive Stress on 14 days	=	1198.677	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	614.514	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1444.167	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-94.622	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	866.438	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	890.852	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iii). <u>SECTION AT 1/4TH L</u>

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-443.722	-443.722	1905.729	1905.729						
Self weight of Precast girder	529.091	85.370	-574.297	1331.432					1212.439	
Loss due to elastic shortenings during 1st stage cables	10.317	95.686	-44.308	1287.124					1173.347	
Relaxation loss of 1st stage cables	5.282	90.404	-22.686	1264.438						
Final Relaxation loss for 1st stage cables	15.847		-68.059							
Creep and shrinkage loss between 10 days & 21 days	26.076	116.481	-111.995	1152.443					1053.513	
Weight of Deck slab, cast-in-situ diaphragms	290.377	406.857	-315.186	837.257					796.155	
Creep & Shrinkage loss between 21 days & 49	11.288	418.146	-48.482	788.774					753.381	
2nd stage prestress	-24.233	382.624	824.367	1661.623	-95.461	-95.461	-24.233	-24.233		
Loss due to elastic shortenings during 1st stage cables	7.419	390.043	-31.863	1629.760						
Loss due to elastic shortenings during 2nd stage cables	0.185	390.228	-6.286	1623.474	0.728	-94.733	0.185	-24.048		1485.706
Relaxation loss of 2nd stage cables	0.323	390.551	-10.989	1612.485	1.273	-93.460	0.323	-24.371		
Final Relaxation loss for 2nd stage cables	0.969		-32.968		3.818		0.969			
Creep and shrinkage loss between 49 days & 60 days	4.286	394.836	-24.907	1587.577	0.862	-92.598	0.219	-24.153		1454.334
Self weight of hand rail and wearing course	52.921	447.757	-57.442	1530.135	21.415	-71.183	16.534	-7.618		1409.221
Creep and shrinkage loss from 60 days to infinity	62.767	510.524	-364.788	1165.347	12.619	-58.565	3.203	-4.415		1092.195
Additional loss due to full relaxation	11.210	521.735	-67.351	1097.996	2.545	-56.020	0.646	-3.769		
Carriage way live load and footpath live load	117.984	639.719	-296.966	801.030	152.813	96.793	117.984	114.215		
20 % higher time dependent loss	24.2469	663.966	-130.2412	670.788	3.4595	100.253	0.8782	115.093		

Compressive Stress on 14 days	=	1331.432	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	788.774	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1661.623	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-95.461	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	801.030	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	670.788	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). **SECTION AT 1/8TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-414.420	-414.420	1828.116	1828.116						
Self weight of Precast girder	305.466	-108.954	-331.565	1496.551					1331.641	
Loss due to elastic shortenings during 1st stage cables	9.956	-98.998	-43.916	1452.635					1293.258	
Relaxation loss of 1st stage cables	4.933	-103.932	-21.762	1430.872						
Final Relaxation loss for 1st stage cables	14.800		-65.287							
Creep and shrinkage loss between 10 days & 28 days	25.164	-78.768	-111.004	1319.868					1176.206	
Weight of Deck slab, cast-in-situ diaphragms	166.646	87.878	-180.884	1138.983					1031.019	
Creep & Shrinkage loss between 21 days & 49	10.894	98.772	-48.055	1090.929					989.019	
2nd stage prestress	-3.027	84.851	734.565	1873.548	-64.937	-64.937	-3.027	-3.027		
Loss due to elastic shortenings during 1st stage cables	7.159	92.011	-31.582	1841.967						
Loss due to elastic shortenings during 2nd stage cables	0.024	92.035	-5.882	1836.085	0.520	-64.417	0.024	-3.003		1603.754
Relaxation loss of 2nd stage cables	0.040	92.075	-9.792	1826.293	0.866	-63.551	0.040	-3.043		
Final Relaxation loss for 2nd stage cables	0.121		-29.376		2.597		0.121			
Creep and shrinkage loss between 49 days & 60 days	3.953	96.029	-24.274	1802.018	0.615	-62.936	0.029	-3.014		1574.758
Self weight of hand rail and wearing course	30.779	126.808	-33.409	1768.609	12.455	-50.481	9.616	6.602		1549.900
Creep and shrinkage loss from 60 days to infinity	57.899	184.707	-355.521	1413.089	9.014	-41.467	0.420	7.022		1249.452
Additional loss due to full relaxation	9.947	194.654	-63.109	1349.980	1.731	-39.735	0.081	7.103		
Carriage way live load and footpath live load	62.297	256.951	-156.800	1193.179	80.686	40.951	62.297	69.399		
20 % higher time dependent loss	22.5665	279.517	-126.7047	1066.475	2.4452	43.396	0.1140	69.513		

Compressive Stress on 14 days	=	1496.551	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-108.954	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1090.929	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1873.548	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-64.937	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1193.179	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1066.475	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). **SECTION AT WEB THICKENING**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-276.808	-276.808	1658.661	1658.661						
Self weight of Precast girder	143.257	-133.552	-155.496	1503.164					1250.539	
Loss due to elastic shortenings during 1st stage cables	6.748	-126.804	-40.435	1462.729					1217.387	
Relaxation loss of 1st stage cables	3.295	-130.099	-19.745	1442.984						
Final Relaxation loss for 1st stage cables	9.886		-59.235							
Creep and shrinkage loss between 10 days & 21 days	17.057	-113.042	-102.205	1340.779					1116.383	
Weight of Deck slab, cast-in-situ diaphragms	77.643	-35.399	-84.277	1256.502					1057.099	
Creep & Shrinkage loss between 21 days & 49 days	7.384	-28.015	-44.246	1212.257					1020.822	
2nd stage prestress	15.144	-20.255	678.314	1934.816	-40.519	-40.519	15.144	15.144		
Loss due to elastic shortenings during 1st stage cables	4.853	-15.402	-29.078	1905.738						
Loss due to elastic shortenings during 2nd stage cables	-0.123	-15.525	-5.511	1900.227	0.329	-40.190	-0.123	15.021		1549.941
Relaxation loss of 2nd stage cables	-0.202	-15.727	-9.042	1891.184	0.540	-39.650	-0.202	15.223		
Final Relaxation loss for 2nd stage cables	-0.606		-27.127		1.620		-0.606			
Creep and shrinkage loss between 49 days & 60 days	2.515	-13.213	-22.463	1868.721	0.390	-39.260	-0.146	15.077		1524.619
Self weight of hand rail and wearing course	14.420	1.207	-15.652	1853.069	5.835	-33.425	4.505	19.583		1514.465
Creep and shrinkage loss from 60 days to infinity	36.828	38.035	-328.993	1524.076	5.707	-27.718	-2.133	17.450		1252.360
Additional loss due to full relaxation	6.187	44.221	-57.575	1466.501	1.080	-26.637	-0.404	17.046		
Carriage way live load and footpath live load	52.799	97.020	-132.895	1333.607	68.385	41.747	52.799	69.845		
20 % higher time dependent loss	14.6127	111.633	-116.8547	1216.752	1.5434	43.291	-0.5769	69.268		

Compressive Stress on 14 days	=	1503.164	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-133.552	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1212.257	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	-28.015	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1934.816	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-40.519	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1333.607	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1216.752	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

I. A) STRESS TABLES (Servicibility checking with 1.1 times prestressing force) **cl-7.9.5(6), IRC-112:2011**

1. END GIRDER:

i). SECTION AT 0.5 L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-497.108	-497.108	2113.745	2113.745						
Self weight of Precast girder	705.709	208.601	-766.004	1347.740					1238.958	
Loss due to elastic shortenings during 1st stage cables	11.502	220.103	-48.905	1298.835					1195.821	
Relaxation loss of 1st stage cables	5.918	214.185	-25.163	1273.672						
Final Relaxation loss for 1st stage cables	17.753		-75.488							
Creep and shrinkage loss between 10 days & 21 days	29.071	243.256	-123.613	1150.059					1063.464	
Weight of Deck slab, cast-in-situ diaphragms	387.533	630.789	-420.643	729.416					719.998	
Creep & Shrinkage loss between 21 days & 49	12.585	643.374	-53.513	675.904					672.797	
2nd stage prestress	-26.272	604.517	889.803	1619.219	-103.163	-103.163	-26.272	-26.272		
Loss due to elastic shortenings during 1st stage cables	8.271	612.788	-35.169	1584.050						
Loss due to elastic shortenings during 2nd stage cables	0.204	612.992	-6.917	1577.133	0.802	-102.361	0.204	-26.068		1469.427
Relaxation loss of 2nd stage cables	0.350	613.343	-11.862	1565.271	1.375	-100.986	0.350	-26.418		
Final Relaxation loss for 2nd stage cables	1.051		-35.585		4.126		1.051			
Creep and shrinkage loss between 49 days & 60 days	4.776	618.118	-27.466	1537.805	0.949	-100.036	0.242	-26.176		1435.065
Self weight of hand rail and wearing course	70.634	688.753	-76.669	1461.136	28.583	-71.453	22.069	-4.108		1374.851
Creep and shrinkage loss from 60 days to infinity	69.946	758.699	-402.270	1058.865	13.902	-57.551	3.540	-0.567		1025.333
Additional loss due to full relaxation	12.536	771.234	-74.048	984.817	2.750	-54.801	0.700	0.133		
Carriage way live load and footpath live load	152.885	924.120	-384.813	600.004	198.017	143.216	152.885	153.019		
20 % higher time dependent loss	27.0368	951.156	-143.5889	456.416	3.7954	147.012	0.9666	153.985		

Compressive Stress on 14 days	=	1347.740	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	675.904	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1619.219	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-103.163	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	924.120	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	951.156	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-492.601	-492.601	2105.023	2105.023						
Self weight of Precast girder	658.702	166.101	-714.981	1390.043					1273.162	
Loss due to elastic shortenings during 1st stage cables	11.425	177.526	-48.822	1341.220					1230.093	
Relaxation loss of 1st stage cables	5.864	171.662	-25.059	1316.162						
Final Relaxation loss for 1st stage cables	17.592		-75.176							
Creep and shrinkage loss between 10 days & 21 days	28.878	200.540	-123.405	1192.756					1098.004	
Weight of Deck slab, cast-in-situ diaphragms	360.958	561.498	-391.798	800.958					778.091	
Creep & Shrinkage loss between 21 days & 49	12.501	574.000	-53.422	747.536					730.964	
2nd stage prestress	-26.464	535.034	898.303	1699.262	-104.085	-104.085	-26.464	-26.464		
Loss due to elastic shortenings during 1st stage cables	8.216	543.250	-35.110	1664.152						
Loss due to elastic shortenings during 2nd stage cables	0.204	543.454	-6.916	1657.236	0.801	-103.283	0.204	-26.261		1532.813
Relaxation loss of 2nd stage cables	0.353	543.807	-11.975	1645.261	1.388	-101.896	0.353	-26.613		
Final Relaxation loss for 2nd stage cables	1.058		-35.925		4.163		1.058			
Creep and shrinkage loss between 49 days & 60 days	4.745	548.552	-27.432	1617.829	0.948	-100.947	0.241	-26.372		1498.378
Self weight of hand rail and wearing course	66.206	614.758	-71.863	1545.966	26.791	-74.156	20.685	-5.687		1441.939
Creep and shrinkage loss from 60 days to infinity	69.496	684.253	-401.770	1144.196	13.891	-60.265	3.532	-2.155		1144.196
Additional loss due to full relaxation	12.434	696.687	-74.067	1070.129	2.775	-57.490	0.706	-1.450		
Carriage way live load and footpath live load	134.371	831.058	-338.213	731.916	174.038	116.548	134.371	132.922		
20 % higher time dependent loss	26.8545	857.913	-143.4275	588.489	3.8004	120.348	0.9663	133.888		

Compressive Stress on 14 days	=	1390.043	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	747.536	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1699.262	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-104.085	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	831.058	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	857.913	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iii). **SECTION AT 1/4TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-488.094	-488.094	2096.302	2096.302						
Self weight of Precast girder	529.091	40.998	-574.297	1522.005					1380.576	
Loss due to elastic shortenings during 1st stage cables	11.348	52.346	-48.739	1473.266					1337.575	
Relaxation loss of 1st stage cables	5.810	46.535	-24.955	1448.311						
Final Relaxation loss for 1st stage cables	17.431		-74.865							
Creep and shrinkage loss between 10 days & 21 days	28.684	75.219	-123.194	1325.117					1205.757	
Weight of Deck slab, cast-in-situ diaphragms	290.377	365.596	-315.186	1009.931					948.400	
Creep & Shrinkage loss between 21 days & 49	12.417	378.013	-53.330	956.600					901.348	
2nd stage prestress	-26.657	338.940	906.803	1916.734	-105.007	-105.007	-26.657	-26.657		
Loss due to elastic shortenings during 1st stage cables	8.161	347.100	-35.050	1881.684						
Loss due to elastic shortenings during 2nd stage cables	0.203	347.304	-6.915	1874.770	0.801	-104.206	0.203	-26.453		1704.134
Relaxation loss of 2nd stage cables	0.355	347.659	-12.088	1862.681	1.400	-102.806	0.355	-26.809		
Final Relaxation loss for 2nd stage cables	1.066		-36.265		4.199		1.066			
Creep and shrinkage loss between 49 days & 60 days	4.714	352.373	-27.398	1835.283	0.948	-101.858	0.241	-26.568		1669.625
Self weight of hand rail and wearing course	52.921	405.294	-57.442	1777.841	21.415	-80.443	16.534	-10.034		1624.511
Creep and shrinkage loss from 60 days to infinity	69.044	474.338	-401.267	1376.574	13.880	-66.563	3.524	-6.510		1275.784
Additional loss due to full relaxation	12.331	486.669	-74.086	1302.488	2.800	-63.763	0.711	-5.799		
Carriage way live load and footpath live load	117.984	604.653	-296.966	1005.522	152.813	89.050	117.984	112.185		
20 % higher time dependent loss	26.6716	631.325	-143.2654	862.256	3.8055	92.855	0.9660	113.151		

Compressive Stress on 14 days	=	1522.005	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	956.600	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1916.734	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-105.007	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1005.522	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	862.256	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). **SECTION AT 1/8TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-455.862	-455.862	2010.927	2010.927						
Self weight of Precast girder	305.466	-150.396	-331.565	1679.362					1491.418	
Loss due to elastic shortenings during 1st stage cables	10.951	-139.445	-48.308	1631.054					1449.197	
Relaxation loss of 1st stage cables	5.427	-144.872	-23.939	1607.116						
Final Relaxation loss for 1st stage cables	16.280		-71.816							
Creep and shrinkage loss between 10 days & 28 days	27.680	-117.191	-122.105	1485.011					1320.440	
Weight of Deck slab, cast-in-situ diaphragms	166.646	49.455	-180.884	1304.127					1175.252	
Creep & Shrinkage loss between 21 days & 49	11.983	61.438	-52.860	1251.266					1129.052	
2nd stage prestress	-3.330	46.125	808.022	2112.148	-71.431	-71.431	-3.330	-3.330		
Loss due to elastic shortenings during 1st stage cables	7.875	54.000	-34.740	2077.409						
Loss due to elastic shortenings during 2nd stage cables	0.027	54.027	-6.470	2070.938	0.572	-70.859	0.027	-3.303		1802.259
Relaxation loss of 2nd stage cables	0.044	54.072	-10.771	2060.167	0.952	-69.906	0.044	-3.348		
Final Relaxation loss for 2nd stage cables	0.133		-32.314		2.857		0.133			
Creep and shrinkage loss between 49 days & 60 days	4.349	58.420	-26.702	2033.465	0.677	-69.229	0.032	-3.316		1770.363
Self weight of hand rail and wearing course	30.779	89.199	-33.409	2000.056	12.455	-56.774	9.616	6.301		1745.505
Creep and shrinkage loss from 60 days to infinity	63.689	152.889	-391.073	1608.983	9.915	-46.859	0.462	6.763		1415.012
Additional loss due to full relaxation	10.942	163.831	-69.420	1539.564	1.904	-44.954	0.089	6.852		
Carriage way live load and footpath live load	62.297	226.127	-156.800	1382.763	80.686	35.732	62.297	69.148		
20 % higher time dependent loss	24.8232	250.950	-139.3751	1243.388	2.6898	38.422	0.1254	69.273		

Compressive Stress on 14 days	=	1679.362	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-150.396	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1251.266	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	2112.148	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-71.431	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1382.763	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1243.388	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). SECTION AT WEB THICKENING

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-304.489	-304.489	1824.527	1824.527						
Self weight of Precast girder	143.257	-161.233	-155.496	1669.030					1386.531	
Loss due to elastic shortenings during 1st stage cables	7.423	-153.810	-44.478	1624.552					1350.064	
Relaxation loss of 1st stage cables	3.625	-157.435	-21.720	1602.832						
Final Relaxation loss for 1st stage cables	10.874		-65.159							
Creep and shrinkage loss between 10 days & 21 days	18.762	-138.672	-112.426	1490.407					1238.960	
Weight of Deck slab, cast-in-situ diaphragms	77.643	-61.029	-84.277	1406.130					1179.675	
Creep & Shrinkage loss between 21 days & 49 days	8.122	-52.907	-48.670	1357.460					1139.771	
2nd stage prestress	16.659	-44.371	746.145	2152.275	-44.571	-44.571	16.659	16.659		
Loss due to elastic shortenings during 1st stage cables	5.338	-39.033	-31.986	2120.289						
Loss due to elastic shortenings during 2nd stage cables	-0.135	-39.168	-6.063	2114.227	0.362	-44.209	-0.135	16.523		1720.489
Relaxation loss of 2nd stage cables	-0.222	-39.390	-9.947	2104.280	0.594	-43.615	-0.222	16.745		
Final Relaxation loss for 2nd stage cables	-0.666		-29.840		1.782		-0.666			
Creep and shrinkage loss between 49 days & 60 days	2.766	-36.624	-24.709	2079.571	0.429	-43.186	-0.160	16.585		1692.635
Self weight of hand rail and wearing course	14.420	-22.204	-15.652	2063.919	5.835	-37.351	4.505	21.090		1682.481
Creep and shrinkage loss from 60 days to infinity	40.510	18.306	-361.893	1702.026	6.278	-31.073	-2.346	18.744		1394.166
Additional loss due to full relaxation	6.805	25.112	-63.332	1638.694	1.188	-29.885	-0.444	18.300		
Carriage way live load and footpath live load	52.799	77.910	-132.895	1505.799	68.385	38.500	52.799	71.099		
20 % higher time dependent loss	16.0739	93.984	-128.5402	1377.259	1.6978	40.198	-0.6345	70.464		

Compressive Stress on 14 days	=	1669.030	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-161.233	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1357.460	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	-52.907	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	2152.275	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-44.571	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1505.799	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1377.259	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

I. B) STRESS TABLES (Servicibility checking with 0.9 times prestressing force) **cl-7.9.5(6), IRC-112:2011**

1. END GIRDER:

i). SECTION AT 0.5 L

Description of load and prestress	Stress at top Of Girder (T/m²)		Stress at bottom of Girder(T/m²)		Stress at top of Deck Slab (T/m²)		Stress at bottom of Deck Slab(T/m²)		Stress at C.G of 1st stage cables (T/m²)	Stress at C.G of all stage cables (T/m²)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-406.724	-406.724	1729.428	1729.428						
Self weight of Precast girder	705.709	298.985	-766.004	963.423					899.972	
Loss due to elastic shortenings during 1st stage cables	9.410	308.395	-40.014	923.410					864.678	
Relaxation loss of 1st stage cables	4.842	303.553	-20.588	902.822						
Final Relaxation loss for 1st stage cables	14.525		-61.763							
Creep and shrinkage loss between 10 days & 21 days	23.785	327.339	-101.138	801.684					756.386	
Weight of Deck slab, cast-in-situ diaphragms	387.533	714.871	-420.643	381.041					412.920	
Creep & Shrinkage loss between 21 days & 49	10.297	725.168	-43.783	337.258					374.302	
2nd stage prestress	-21.495	693.376	728.021	1109.062	-84.406	-84.406	-21.495	-21.495		
Loss due to elastic shortenings during 1st stage cables	6.767	700.143	-28.775	1080.287						
Loss due to elastic shortenings during 2nd stage cables	0.167	700.311	-5.660	1074.627	0.656	-83.750	0.167	-21.328		1032.812
Relaxation loss of 2nd stage cables	0.287	700.597	-9.705	1064.922	1.125	-82.625	0.287	-21.615		
Final Relaxation loss for 2nd stage cables	0.860		-29.115		3.376		0.860			
Creep and shrinkage loss between 49 days & 60 days	3.907	704.505	-22.473	1042.450	0.777	-81.848	0.198	-21.417		1004.697
Self weight of hand rail and wearing course	70.634	775.139	-76.669	965.781	28.583	-53.265	22.069	0.652		944.484
Creep and shrinkage loss from 60 days to infinity	57.228	832.367	-329.130	636.650	11.374	-41.890	2.897	3.548		658.514
Additional loss due to full relaxation	10.257	842.624	-60.585	576.065	2.250	-39.640	0.573	4.121		
Carriage way live load and footpath live load	152.885	995.509	-384.813	191.253	198.017	158.377	152.885	157.007		
20 % higher time dependent loss	22.1210	1017.630	-117.4818	73.771	3.1053	161.482	0.7908	157.798		

Compressive Stress on 14 days	=	963.423	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	725.168	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1109.062	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-84.406	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	995.509	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1017.630	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-403.037	-403.037	1722.292	1722.292						
Self weight of Precast girder	658.702	255.665	-714.981	1007.311					935.532	
Loss due to elastic shortenings during 1st stage cables	9.348	265.012	-39.946	967.366					900.294	
Relaxation loss of 1st stage cables	4.798	260.214	-20.503	946.863						
Final Relaxation loss for 1st stage cables	14.394		-61.508							
Creep and shrinkage loss between 10 days & 21 days	23.628	283.842	-100.968	845.895					792.222	
Weight of Deck slab, cast-in-situ diaphragms	360.958	644.800	-391.798	454.097					472.308	
Creep & Shrinkage loss between 21 days & 49	10.228	655.029	-43.709	410.388					433.750	
2nd stage prestress	-21.653	623.148	734.976	1189.072	-85.160	-85.160	-21.653	-21.653		
Loss due to elastic shortenings during 1st stage cables	6.722	629.870	-28.726	1160.346						
Loss due to elastic shortenings during 2nd stage cables	0.167	630.037	-5.659	1154.688	0.656	-84.505	0.167	-21.486		1096.078
Relaxation loss of 2nd stage cables	0.289	630.325	-9.798	1144.890	1.135	-83.369	0.289	-21.775		
Final Relaxation loss for 2nd stage cables	0.866		-29.393		3.406		0.866			
Creep and shrinkage loss between 49 days & 60 days	3.882	634.208	-22.445	1122.446	0.776	-82.593	0.197	-21.577		1067.904
Self weight of hand rail and wearing course	66.206	700.414	-71.863	1050.583	26.791	-55.802	20.685	-0.892		1011.465
Creep and shrinkage loss from 60 days to infinity	56.860	757.274	-328.721	721.862	11.365	-44.437	2.890	1.998		721.862
Additional loss due to full relaxation	10.173	767.447	-60.600	661.262	2.270	-42.166	0.577	2.575		
Carriage way live load and footpath live load	134.371	901.818	-338.213	323.049	174.038	131.872	134.371	136.946		
20 % higher time dependent loss	21.9719	923.790	-117.3498	205.699	3.1094	134.981	0.7906	137.737		

Compressive Stress on 14 days	=	1007.311	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	655.029	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1189.072	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-85.160	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	901.818	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	923.790	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iii). SECTION AT 1/4TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-399.350	-399.350	1715.156	1715.156						
Self weight of Precast girder	529.091	129.742	-574.297	1140.860					1044.302	
Loss due to elastic shortenings during 1st stage cables	9.285	139.027	-39.877	1100.982					1009.120	
Relaxation loss of 1st stage cables	4.754	134.273	-20.418	1080.564						
Final Relaxation loss for 1st stage cables	14.262		-61.253							
Creep and shrinkage loss between 10 days & 21 days	23.469	157.742	-100.795	979.769					901.269	
Weight of Deck slab, cast-in-situ diaphragms	290.377	448.118	-315.186	664.583					643.911	
Creep & Shrinkage loss between 21 days & 49	10.160	458.278	-43.634	620.949					605.414	
2nd stage prestress	-21.810	426.308	741.930	1406.513	-85.914	-85.914	-21.810	-21.810		
Loss due to elastic shortenings during 1st stage cables	6.677	432.985	-28.677	1377.836						
Loss due to elastic shortenings during 2nd stage cables	0.166	433.152	-5.658	1372.178	0.655	-85.259	0.166	-21.644		1267.278
Relaxation loss of 2nd stage cables	0.291	433.443	-9.890	1362.288	1.145	-84.114	0.291	-21.934		
Final Relaxation loss for 2nd stage cables	0.872		-29.671		3.436		0.872			
Creep and shrinkage loss between 49 days & 60 days	3.857	437.300	-22.416	1339.871	0.775	-83.339	0.197	-21.737		1239.044
Self weight of hand rail and wearing course	52.921	490.221	-57.442	1282.429	21.415	-61.923	16.534	-5.203		1193.930
Creep and shrinkage loss from 60 days to infinity	56.490	546.711	-328.309	954.119	11.357	-50.567	2.883	-2.320		908.607
Additional loss due to full relaxation	10.089	556.800	-60.616	893.503	2.291	-48.276	0.581	-1.739		
Carriage way live load and footpath live load	117.984	674.784	-296.966	596.537	152.813	104.537	117.984	116.245		
20 % higher time dependent loss	21.8222	696.607	-117.2171	479.320	3.1136	107.650	0.7904	117.036		

Compressive Stress on 14 days	=	1140.860	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	620.949	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1406.513	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-85.914	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	674.784	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	696.607	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). **SECTION AT 1/8TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-372.978	-372.978	1645.304	1645.304						
Self weight of Precast girder	305.466	-67.512	-331.565	1313.739					1171.863	
Loss due to elastic shortenings during 1st stage cables	8.960	-58.552	-39.525	1274.215					1137.319	
Relaxation loss of 1st stage cables	4.440	-62.992	-19.586	1254.628						
Final Relaxation loss for 1st stage cables	13.320		-58.758							
Creep and shrinkage loss between 10 days & 28 days	22.647	-40.344	-99.904	1154.724					1031.972	
Weight of Deck slab, cast-in-situ diaphragms	166.646	126.302	-180.884	973.840					886.785	
Creep & Shrinkage loss between 21 days & 49	9.804	136.106	-43.249	930.591					848.985	
2nd stage prestress	-2.724	123.577	661.109	1634.949	-58.443	-58.443	-2.724	-2.724		
Loss due to elastic shortenings during 1st stage cables	6.443	130.021	-28.423	1606.525						
Loss due to elastic shortenings during 2nd stage cables	0.022	130.043	-5.294	1601.231	0.468	-57.975	0.022	-2.703		1405.250
Relaxation loss of 2nd stage cables	0.036	130.079	-8.813	1592.418	0.779	-57.196	0.036	-2.739		
Final Relaxation loss for 2nd stage cables	0.109		-26.439		2.337		0.109			
Creep and shrinkage loss between 49 days & 60 days	3.558	133.637	-21.847	1570.571	0.554	-56.642	0.026	-2.713		1379.153
Self weight of hand rail and wearing course	30.779	164.416	-33.409	1537.163	12.455	-44.187	9.616	6.903		1354.295
Creep and shrinkage loss from 60 days to infinity	52.109	216.525	-319.969	1217.194	8.112	-36.075	0.378	7.282		1083.892
Additional loss due to full relaxation	8.953	225.478	-56.798	1160.396	1.558	-34.516	0.073	7.354		
Carriage way live load and footpath live load	62.297	287.775	-156.800	1003.596	80.686	46.170	62.297	69.651		
20 % higher time dependent loss	20.3099	308.085	-114.0342	889.561	2.2007	48.371	0.1026	69.753		

Compressive Stress on 14 days	=	1313.739	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-67.512	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	930.591	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1634.949	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-58.443	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1003.596	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	889.561	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). **SECTION AT WEB THICKENING**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-249.128	-249.128	1492.795	1492.795						
Self weight of Precast girder	143.257	-105.871	-155.496	1337.298					1114.547	
Loss due to elastic shortenings during 1st stage cables	6.073	-99.798	-36.391	1300.907					1084.710	
Relaxation loss of 1st stage cables	2.966	-102.763	-17.771	1283.136						
Final Relaxation loss for 1st stage cables	8.897		-53.312							
Creep and shrinkage loss between 10 days & 21 days	15.351	-87.412	-91.985	1191.151					993.807	
Weight of Deck slab, cast-in-situ diaphragms	77.643	-9.769	-84.277	1106.875					934.522	
Creep & Shrinkage loss between 21 days & 49 days	6.646	-3.124	-39.821	1067.054					901.873	
2nd stage prestress	13.630	3.860	610.482	1717.357	-36.467	-36.467	13.630	13.630		
Loss due to elastic shortenings during 1st stage cables	4.367	8.228	-26.170	1691.187						
Loss due to elastic shortenings during 2nd stage cables	-0.111	8.117	-4.960	1686.227	0.296	-36.171	-0.111	13.519		1379.392
Relaxation loss of 2nd stage cables	-0.182	7.935	-8.138	1678.089	0.486	-35.685	-0.182	13.701		
Final Relaxation loss for 2nd stage cables	-0.545		-24.414		1.458		-0.545			
Creep and shrinkage loss between 49 days & 60 days	2.263	10.198	-20.217	1657.872	0.351	-35.334	-0.131	13.570		1356.603
Self weight of hand rail and wearing course	14.420	24.618	-15.652	1642.220	5.835	-29.499	4.505	18.075		1346.449
Creep and shrinkage loss from 60 days to infinity	33.145	57.763	-296.094	1346.126	5.136	-24.362	-1.920	16.155		1110.555
Additional loss due to full relaxation	5.568	63.331	-51.817	1294.309	0.972	-23.390	-0.363	15.792		
Carriage way live load and footpath live load	52.799	116.130	-132.895	1161.414	68.385	44.995	52.799	68.591		
20 % higher time dependent loss	13.1514	129.281	-105.1692	1056.245	1.3891	46.384	-0.5192	68.071		

Compressive Stress on 14 days	=	1337.298	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-105.871	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1067.054	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	-3.124	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1717.357	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-36.467	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1161.414	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1056.245	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

J. CHECK FOR ULTIMATE STRENGTH:

Minimum Area of longitudinal reinforcement				0.18%
Area at mid section=				1.267 m^2
Required area of steel				2280.6 sqmm
Provide	**12 mm dia**	**35**	**no. bar.**	**OK**
Area at end section=				2.443 m^2
Required area of steel				4397.4 sqmm
Provide	**12 mm dia**	**59**	**no. bar.**	**OK**

Checking as Non-Prestressed high tensile reinforcement

Mu=M1+M2 — 1.35DL+1.75 SIDL+1.5LL+1.15FPLL

Mu=Design Moment= 2104.81 T-m

M1=.9*f_p*Asp*db1 **5632.86** T-m

f_p= Ultimate tensile strength of steel= 185915 t/m^2

No. of cables at mid section = 6

Total area of cable=Asp= 112.632 cm^2 = 1.13E-02 m^2

d_b = the depth of beam from the maximum compression edge to the centre of gravity of tendons

= 2.69 m for composite section

No extra reinforcement is required.

Checking as Crushing of Concrete

Mult= $.176*b*d^2*f_{ck}+(2/3)*.8*(B_f-b)*(d_b-t/2)*t*f_{ck}$

= **3808.6 T-m**

b=Width of web=	300 mm
d= Total depth=	3000 mm
f_{ck}=	45 Mpa
B_f=	1500 mm
t=	225 mm

No extra reinforcement is required.

N. DESIGN OF SHEAR :

Concrete strength = **45** Mpa

Strength of HYSD bar = **500** Mpa

i) **At Support Section**

Design shear force, V_{ED}= = 1.35DL+1.75 SIDL+1.5LL+1.15FPLL

= 3080.28 KN

VRds=VNs = 3080.28 KN

f_{cp}= Stress at composite centroid due to prestress

= 0.6837X[651.279-(928.087X0.542/1.331)]+0.0801X203.705

= 203.206 t/m^2

= 2.032 Mpa — 0.10 f_{cd}

f_{cd}= .67*fck/1.5= = 20.1 Mpa — **PN-87,IRC-112:2011**

Maximum allowable shear force, taking,θ=45° — **Eq-10.8,IRC-112:2011**

V_{Rdmax}= $\alpha_{cw}*b_w*z*v_1*(f_{cd}/(\cot\theta+\tan\theta))$ = **10159** — **The section is safe in shear**

α_{cw}= = 1.10

b_w= = 850 mm

z=lever arm = 0.6 d

= 1800 mm

v1= strength reduction factor for concrete cracked in shear 0.6

θ= 45 °

<u>Allowable shear force without shear reinforcement</u>				**Cl-10.3.2(2),IRC-112:2011**
V_{Rdc}=	$[0.12*K*(80*\rho_1*f_{ck})^{0.33}+0.15*\sigma_{cp}]*b_w*d$	=	**1724**	**Shear Reinforcement is required.**
$V_{rd.c\ min}$ =	$(V_{min}+0.15\sigma_{cp})b_w*d$	=	1513 KN	
V_{min} =	$0.031*K^{3/2}f_{ck}^{1/2}$	=	0.293	
K =		=	1.26	
ρ_1 =		=	0.0044	
σ_{cp} =		=	2.000 Mpa	

<u>Calculation of Reinforcement</u> **Eq-10.7,IRC-112:2011**

$V_{Rds}=A_{sw}/s*z*f_{ywd}*\cot\theta$

Provide 4 L 16 mm dia stp@ 150 mm c/c.

A_{sw}=	=	804 sqmm	
$f_{ywd}=.8*f_{yk}/\gamma_m$=	=	347.83 Mpa	
θ=	=	21.8 °	
S=	=	408.49288 mm	**Hence OK**
Reinforcement Ratio=	=	0.0021015	**Hence OK**
Minimum shear reinforcement ratio	=	0.000966	**Eq-10.20,IRC-112:2011**

N. DESIGN OF INTERFACE SHEAR : (Cl-10.3.4, IRC-112:2011)

Section	Formula	Support	Web-Thickening	L/8	L/4	3L/8	Mid-Section
V_{EDi}=Interface Shear stress, Mpa	$\beta*V_{ED}/z*bi$	1.71	1.07	0.94	0.80	0.45	0.31
β=	Conservatively	1	1	1	1	1	1
V_{ED}=in KN		3080	1921	1692	1447	805	565
z=in mm	=.6d for PSC	1800	1800	1800	1800	1800	1800
b_i=		1000	1000	1000	1000	1000	1000
V_{Rdi}=in KN	Resistance capacity	3.36	2.66	2.02	1.87	1.49	1.34
	$\mu*\sigma n+\rho*f_{yd}*[\mu*\sin\alpha+\cos\alpha]$=	3.355	2.660	2.020	1.873	1.488	1.344
	$0.5*v*f_{cd}$	6.03	6.03	6.03	6.03	6.03	6.03
μ=		0.6	0.6	0.6	0.6	0.6	0.6
σn=	$<.6*f_{cd}$	3.08	1.92	1.69	1.45	0.81	0.57
f_{yd}=in Mpa	$.8*f_{yd}$	400	400	400	400	400	400
α		90	90	90	90	90	90
No. of leg		**2**	**2**	**2**	**2**	**2**	**2**
Dia		**16**	**16**	**16**	**16**	**16**	**16**
Spacing		**100**	**100**	**150**	**150**	**150**	**150**
Area of steel		**4019**	**4019**	**2679**	**2679**	**2679**	**2679**
No. of leg		**2**	**2**	**2**	**2**	**2**	**2**
Dia		**12**	**12**	**12**	**12**	**12**	**12**
Spacing		**100**	**100**	**150**	**150**	**150**	**150**
Area of steel		**2261**	**2261**	**1507**	**1507**	**1507**	**1507**
As=		6280	6280	4187	4187	4187	4187
Asmin=	=.15% of Aj=	1500	1500	1500	1500	1500	1500
Check for minimum reinforcement		OK	OK	OK	OK	OK	OK
ρ=	As/Aj	0.0063	0.0063	0.0042	0.0042	0.0042	0.0042
v		0.6	0.6	0.6	0.6	0.6	0.6
Check for shear capacity		OK	OK	OK	OK	OK	OK

O. DESIGN OF END BLOCK FOR BURSTING TENSILE FORCE: (Cl-13.5.1, IRC-112:2011)

Prestressing force applied at cable-1=P_k= 317.59 T

Load Factor= 1.3 1025

Side of equivalent square of bearing plate,$2Y_{p0}$= 177 mm

Side of loaded area,$2Y_0$= 350 mm 350

Y_{p0}/Y_0= 0.5

F_{bst}/P_k= 0.16

Bursting tensile force=F_{bst}= 508.14 KN 350

Tensile strength for mild steel, Fe250= 217.5 Mpa 350

Reinforcement required= 2336 mm^2 2775

Provide 1 no 20 dia spiral @ 40 mm pitch. 350

Steel provided= 2747.5 mm^2

Hence Ok 350

245 245

360

K. DEFLECTION CHECK:

Deflection Calculation: Long term

Properties of composite girder(Edge)

Area = 2.024 m^2

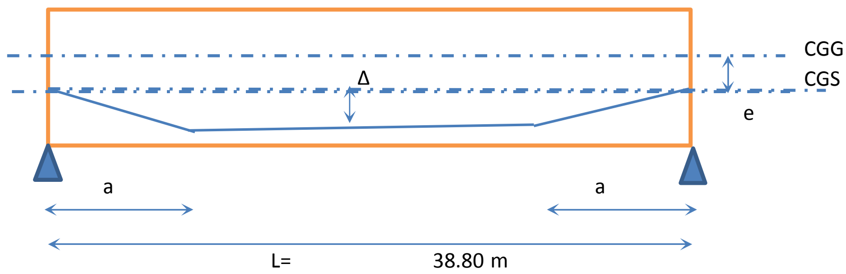
Y_{bg} = 1.986 m

Z_{tg} = 2.887 m^3

Z_{bg} = 1.147 m^3

Z_{ts} = 2.229 m^3

I_g = 2.278 m^4



Deflection due to prestress=δ_{ps}=$P.L^2/8EI\ [e+\Delta-4\Delta a^2/3L^2]$

E= 3481769.63 T/m^2

Location	Prestressing force after anchorage slip(T)	% loss at service	Effective prestressing force (T)	a (m)	Δ (in m)	e (m)	Upward Deflection (in m)
Cable-1	219.25	30.20	153.0	4.525	0.220	1.636	6.72
Cable-2	219.25	30.20	153.0	4.525	0.220	1.636	6.72
Cable-3	216.4	30.20	151.1	7.325	0.570	1.286	6.55
Cable-4	216.4	30.20	151.1	8.22	0.740	0.936	5.85
Cable-5	214.5	19.17	173.4	9.22	0.910	0.586	5.87
Cable-6	214.5	19.17	173.4	9.72	1.080	0.236	5.04

Total upward deflection= 36.77 mm

Downward deflection:

Dead load deflection		2.53 mm	From SAP
SIDL		5.97 mm	2000
Live load deflection		15.5 mm	output
	Total	**24 mm**	

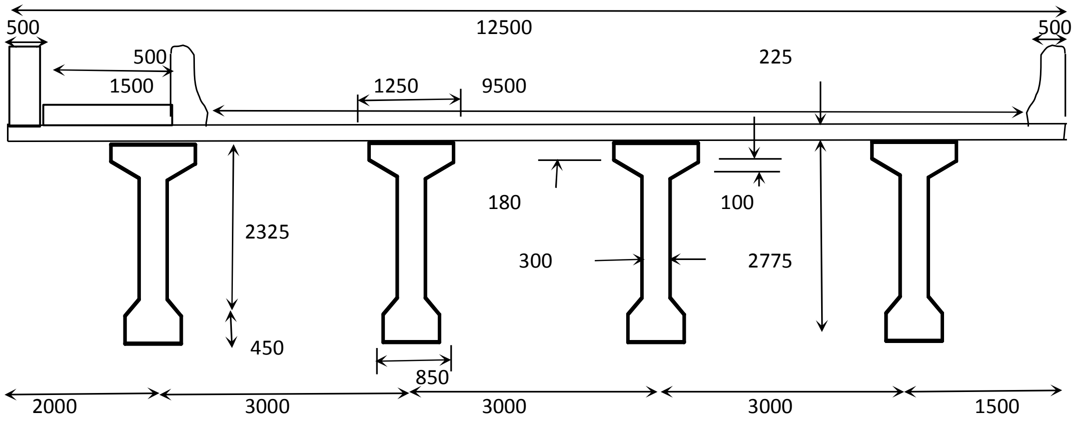
Net deflection= -12.77 mm **Upward**

Allowable deflection=L/600 **64.67 mm**

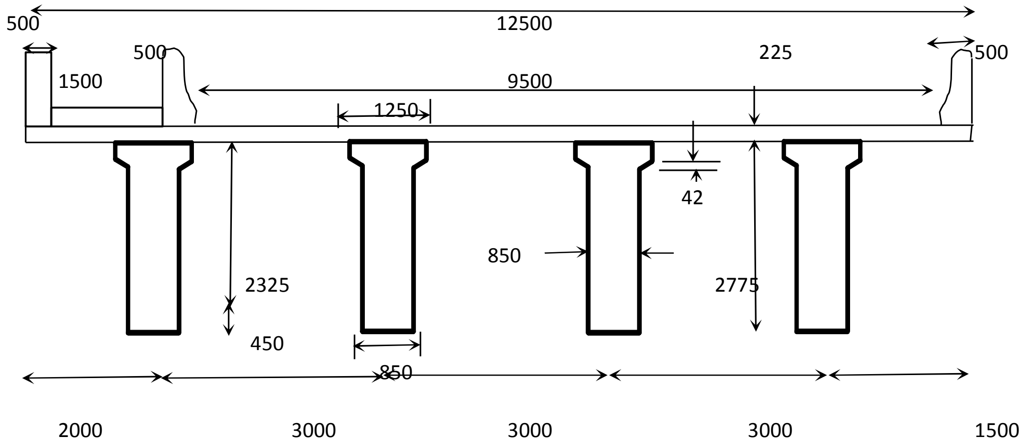
(Cl-12.4.1, IRC-112:2011) **Hence OK**

DESIGN OF PSC T-GIRDER WITH 38.8M SPAN (C/C OF BEARING) [OUTER GIRDER]

A. GEOMETRIC PROPERTIES OF THE GIRDER

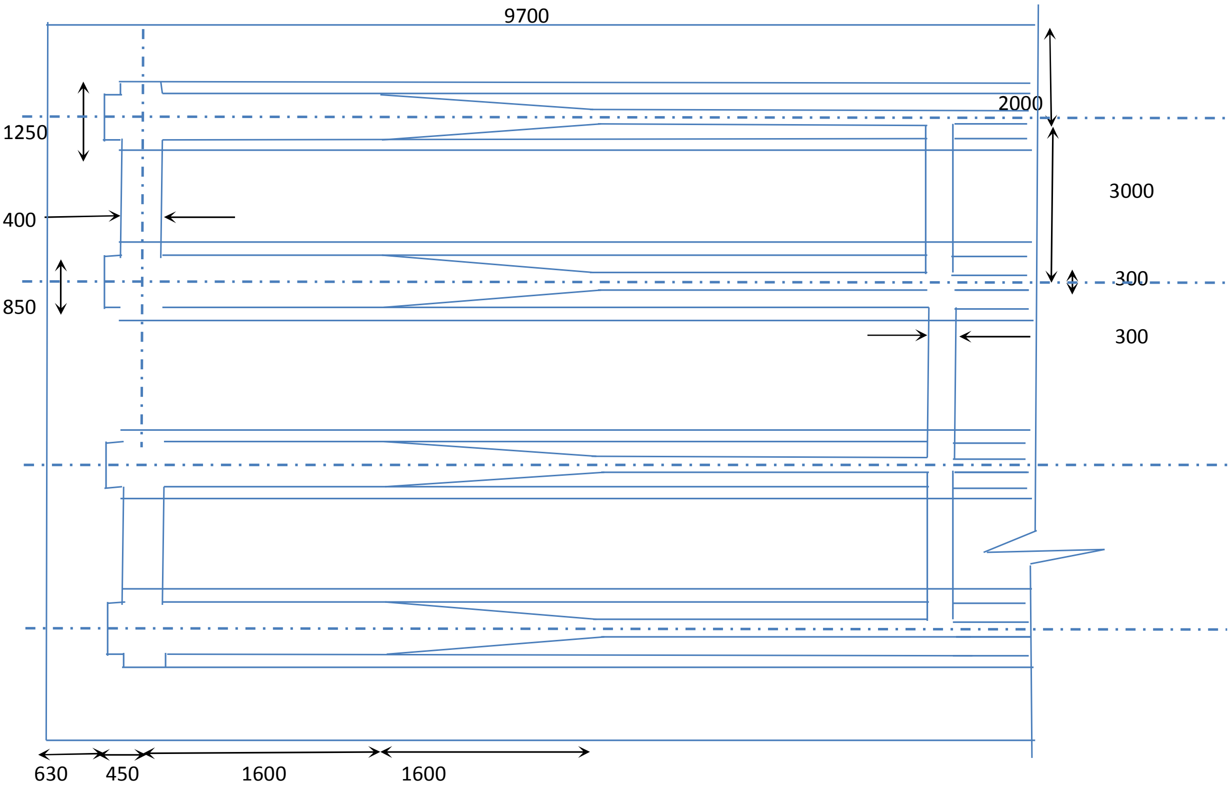


C/S AT MIDDLE



C/S AT END

Thickness of web at end = 850 mm



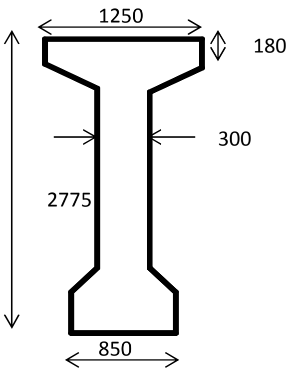
SECTIONAL PLAN THROUGH WEB GIRDER

B. PROPERTIES OF GIRDER SECTION

Precast Section :

For middle portion

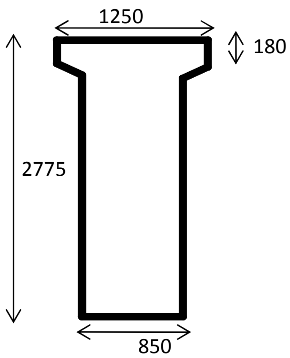
Total girder depth	D =	2775 mm		
Web width of T-Girder	b_w =	300 mm		
Flange width of T-Girder	b_f =	1250 mm		
Flange depth of T-Girder	D_f =	180 mm		
Girder Bulb Width	b_{gb} =	850 mm		
Girder Bulb Depth (Straight)	D_{gb} =	250 mm		
Haunch in Girder Bulb	H : V =	275 mm	:	200 mm
Haunch in Girder Flange to Web	H : V =	475 mm	:	150 mm
Area of Girder (central portion)	Ac =	1.267 m^2		



Pre Cast Girder Section at Mid Span

For end portion of girder having length 1.6 m

Total girder depth	D =	2775 mm		
Web width of T-Girder	b_w =	850 mm		
Flange width of T-Girder	b_f =	1250 mm		
Flange depth of T-Girder	D_f =	180 mm		
Haunch in Girder Flange to Web	H : V =	200 mm	:	63.16 mm
Area of Girder (end thickened portion)	Ac =	2.443 m^2		



Pre Cast Girder Section near Support

For Precast Girder at mid span:

cg of section from bottom of girder will be as follows :

Y_{bp} = [(0.18x1.25)x(2.685)+2x(0.5x0.475x0.15)x(2.55)+(2.345)x(1.6225)+(2x0.5x(0.275x0.2)x(66.92)+(0.85x0.25)x(0.13)] /1.267
1.444 m

cg of section from top of girder will be as follows :

Y_{tp} = 2.775-1.444 = 1.331 m

Moment of Inertia of precast girder :

$I_{precast}$ = [{1.25x0.005832/12+0.225x1.68350625}+{(0.475x0.003375)/18}+{0.07125x1.34+(0.3x12.9)/12}+
{(0.275x0.008/18)+0.055x1.15}+{(0.85x0.015625/12)+0.2125x1.59}]

1.2 m^4

Z_{tp} = 0.902 m^3 Z_{bp} = 0.831 m^3

For Composite Girder :

Edge Girder

Effective flange width = 1.5+1.75 = 3.25 m

Area of girder = [(3.25x0.233] +1.267= 2.024 m^2

cg from bottom of girder = [{(3.25x0.233)x(2.775+0.117)}+{1.267x1.444}]/2.024 = 1.986 m

Y_{bg} = 1.986 m Y_{tg} = 0.789 m Y_{ts} = 1.022 m

$I_{composite}$ = [(3.25x0.013/12)+(3.25x0.054)]+[1.2+1.267x(0.542^2)] = 2.278 m^4

Z_{ts} = 2.229 m^3 Z_{tg} = Z_{bs} = 2.887 m^3 Z_{bg} = 1.147 m^3

Central Girder

Effective flange width = 1.5+1.5 = 3 m

Area of girder = [(3x0.233] +1.267= 1.966 m^2

cg from bottom of girder = [{(3x0.233)x(2.775+0.117)}+{1.267x1.444}]/1.966 = 1.959 m

Y_{bg} = 1.959 m Y_{tg} = 0.816 m Y_{ts} = 1.049 m

$I_{composite}$ = [(3x0.013/12)+(3x0.054)]+[1.2+1.267x(0.515^2)] = 2.209 m^4

Z_{ts} = 2.106 m^3 Z_{tg} = Z_{bs} = 2.707 m^3 Z_{bg} = 1.128 m^3

C. DEAD LOAD

Calculation of loads and moments at different sections of girder :

Dead Load

1. Precast Girder

a Area of Girder (central portion) Ac = 1.267 m^2

Loading = 1.267x25 = 31.675 kN/m

b Area of Girder (end thickened portion) Ac = 2.443 m^2

Loading = 2.443x25 = 61.075 kN/m

2. Self weight of diaphragms

i) Intermediate diaphragm 300 mm Thickness No in each girder = 3

a. Precast portion

Area of each diaphragm (per girder) = 0.5 x(1.25+0.85) x2.345- 0.5 x (1.25+0.3) x0.15- 0.5 x (0.3+0.85) x 0.2- (0.3x1.995)

= 2.83 m^2

Loading = 2.83x25x0.3 = 21.225 kN (on each girder)

b. In Situ portion

Area of each diaphragm = 0.5 x(1.75+2.15) x 2.325 = 4.534 m^2

Loading = 4.534x25x0.3 = 34.005 kN (at each location)

Hence , load on end girder = 8.501 kN (at each location)

Load on central girder = 17.003 kN (at each location)

ii) Exterior diaphragm 400 mm Thickness No in each girder = 2

a. Precast portion

Area of each diaphragm (per girder) = (1.25x2.775) -2.443-{(1.25-0.85) x0.35} = 0.88575 m^2

Loading = 0.88575x25x0.4 = 8.858 kN (on each girder)

b. In Situ portion

Area of each diaphragm = (3-1.25)x(2.775-0.35) = = 4.24375 m^2

Loading = 4.24375x25x0.4 = 42.4375 kN (at each location)

Hence , load on end girder = 10.609375 kN (at each location)

Load on central girder = 21.21875 kN (at each location)

3. Self weight of deck slab

Total weight of deck slab =	12.5x0.225x25 =	70.3125 kN/m
Loading on each end girder =	(1.5+1.5)x0.225x25 =	16.875 kN/m
Loading on each intermediate girder =	(1.5+1.5)x0.225x25 =	16.875 kN/m

4. Superimposed dead load (crash barrier/safety kerb/wearing coat)

Superimposed dead load will be placed on deck slab after composite action starts.

<u>i)Load of crash barrier</u>

Wt. of each crash barrier =	0.329 x 25 =	8.225 kN/m
load for two sides =		16.450 kN/m

<u>ii)Load of safety kerb & Foot path & Railing</u>

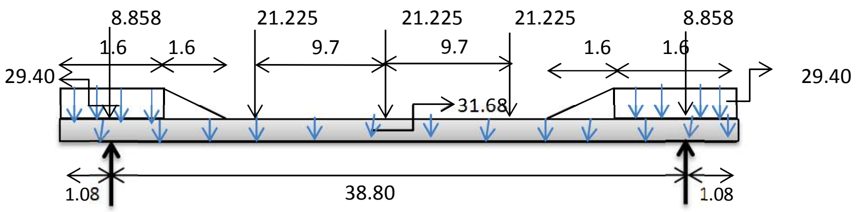
Wt. of each safety kerb =	0.5 x 0.225 x 25 + 1.5 =	0.000 kN/m
load for one side =		0.000 kN/m

<u>iii)Load of wearing coat</u>

Wt. of wearing coat =	0.065 x 9.5 x 22 =	13.585 kN/m
Total superimposed dead load =	16.45+0+13.585 =	30.035 kN/m
Load per Longitudinal girder =		9.074 kN/m

D. <u>DESIGN MOMENTS & SHEARS :</u>

1. <u>DUE TO S/W OF PRECAST GIRDERS & PRECAST PORTION OF DIAPHRAGMS :</u>



Reaction on each support = [(38.8+1.08+1.08)x31.675/2]+[29.4x1.6]+[29.4x1.6x0.5]+[8.858]+[21.225X3/2] =

= 648.704 + 47.04 + 23.52 + 8.858 + 31.838

= 759.960 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support

BM= [759.9595X19.4]-[31.675X20.48^2/2]-[47.04X(19.4+1.08-1.6/2)]-[23.52X(19.4+1.08-1.6-0.533)]

-[8.858x19.4]-[21.225x(19.4-9.7)]

= 6365.5 kN-m

At 3/8th section: 14.55 m from support

BM= [759.9595X14.55]-[31.675X15.63^2/2]-[47.04X(14.55+1.08-1.6/2)]-[23.52X(14.55+1.08-1.6-0.533)]

-[8.858x14.55]-[21.225x(14.55-9.7)]

= 5941.49 kN-m

At 1/4th section: 9.7 m from support

BM= [759.9595X9.7]-[31.675X10.78^2/2]-[47.04X(9.7+1.08-1.6/2)]-[23.52X(9.7+1.08-1.6-0.533)]

-[8.858x9.7]

= 4772.41 kN-m

At 1/8th section: 4.85 m from support

BM= [759.9595X4.85]-[31.675X5.93^2/2]-[47.04X(4.85+1.08-1.6/2)]-[23.52X(4.85+1.08-1.6-0.533)]

-[8.858x4.85]

= 2755.31 kN-m

section: 2.120 m from support

BM= [759.9595X2.12]-[31.675X3.2^2/2]-[47.04X(2.12+1.08-1.6/2)]-[23.52X(2.12+1.08-1.6-0.533)]

-[8.858x2.12]

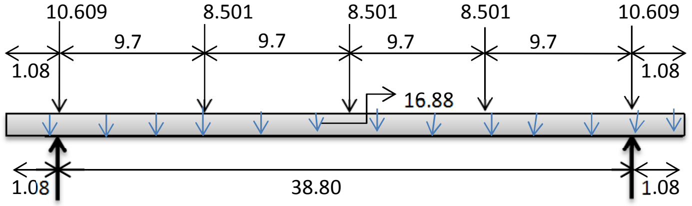
= 1292.18 kN-m

SHEAR AT DIFFERENT SECTIONS OF T GIRDER DUE TO DEAD LOAD:

section: 2.120 m from support
SF= 759.9595-[31.675X(2.12+1.08)]-47.04-23.52-8.858
= 579.182 kN
At 1/8th section: 4.850 m from support
SF= 759.9595-[31.675X(4.85+1.08)]-47.04-23.52-8.858
= 492.709 kN
At 1/4th section: 9.700 m from support
SF= 759.9595-[31.675X(9.7+1.08)]-47.04-23.52-8.858
= 339.085 kN
At 3/8th section: 14.550 m from support
SF= 759.9595-[31.675X(14.55+1.08)]-47.04-23.52-8.858-21.225
= 164.236 kN
At mid section: 19.4 m from support
SF= 759.9595-[31.675X(19.4+1.08)]-47.04-23.52-8.858-21.225
= 10.613 kN

2. DUE TO S/W OF DECK SLAB & CAST IN SITU PORTION OF DIAPHRAGMS :

i)For Edge Girder



Reaction on each support = [(38.8+1.08+1.08)x16.875/2]+[10.609]+[8.50125X3/2] =
= 345.600 + 10.609 + 12.752
= 368.961 kN

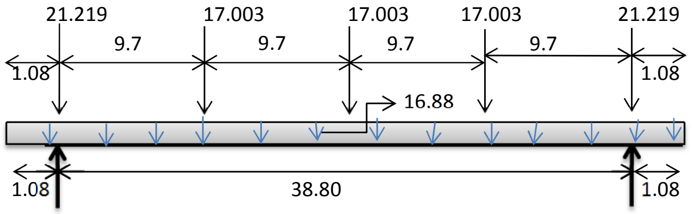
MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support
BM= [368.960875X19.4]-[16.875X20.48^2/2]-[10.609x19.4]-[8.50125x(19.4-9.7)]
= 3330.62 kN-m
At 3/8th section: 14.55 m from support
BM= [368.960875X14.55]-[16.875X15.63^2/2]-[10.609x14.55]-[8.50125x(14.55-9.7)]
= 3111.53 kN-m
At 1/4th section: 9.7 m from support
BM= [368.960875X9.7]-[16.875X10.78^2/2]-[10.609x9.7]
= 2495.5 kN-m
At 1/8th section: 4.85 m from support
BM= [368.960875X4.85]-[16.875X5.93^2/2]-[10.609x4.85]
= 1441.3 kN-m
section: 2.120 m from support
BM= [368.960875X2.12]-[16.875X3.2^2/2]-[10.609x2.12]
= 673.306 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.12 m from support
SF= 368.960875-[16.875X(2.12+1.08)]-10.609
= 304.352 kN
At 1/8th section: 4.85 m from support
SF= 368.960875-[16.875X(4.85+1.08)]-10.609
= 258.283 kN
At 1/4th section: 9.7 m from support
SF= 368.960875-[16.875X(9.7+1.08)]-10.609
= 176.439 kN
At 3/8th section: 14.55 m from support
SF= 368.960875-[16.875X(14.55+1.08)]-10.609-8.50125
= 86.0944 kN
At mid section: 19.4 m from support
SF= 368.960875-[16.875X(19.4+1.08)]-10.609-8.50125
= 4.251 kN

ii) For Central Girder



Reaction on each support = [(38.8+1.08+1.08)x16.875/2]+[21.219]+[17.0025X3/2] =

= 345.600 + 21.219 + 25.504

= 392.323 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support

BM= [392.32275X19.4]-[16.875X20.48^2/2]-[21.219x19.4]-[17.0025x(19.4-9.7)]

= 3495.54 kN-m

At 3/8th section: 14.55 m from support

BM= [392.32275X14.55]-[16.875X15.63^2/2]-[21.219x14.55]-[17.0025x(14.55-9.7)]

= 3255.84 kN-m

At 1/4th section: 9.7 m from support

BM= [392.32275X9.7]-[16.875X10.78^2/2]-[21.219x9.7]

= 2619.2 kN-m

At 1/8th section: 4.85 m from support

BM= [392.32275X4.85]-[16.875X5.93^2/2]-[21.219x4.85]

= 1503.15 kN-m

section: 2.120 m from support

BM= [392.32275X2.12]-[16.875X3.2^2/2]-[21.219x2.12]

= 700.34 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.12 m from support

SF= 392.32275-[16.875X(2.12+1.08)]-21.219

= 317.104 kN

At 1/8th section: 4.85 m from support

SF= 392.32275-[16.875X(4.85+1.08)]-21.219

= 271.035 kN

At 1/4th section: 9.7 m from support

SF= 392.32275-[16.875X(9.7+1.08)]-21.219

= 189.191 kN

At 3/8th section: 14.55 m from support

SF= 392.32275-[16.875X(14.55+1.08)]-21.219-17.0025

= 90.345 kN

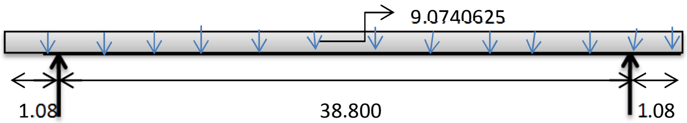
At mid section: 19.4 m from support

SF= 392.32275-[16.875X(19.4+1.08)]-21.219-17.0025

= 8.501 kN

3. **DUE TO SUPERIMPOSED DEAD LOAD :**

Intensity of load on each girder = 9.0740625 kN/m



Reaction on each support = [(38.8+1.08+1.08)x9.0740625/2] = 185.837 kN

MOMENTS AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

At mid section: 19.4 m from support
BM= [185.837X19.4]-[9.0740625X20.48^2/2]
= 1702.27 kN-m

At 3/8th section: 14.550 m from support
BM= [185.837X14.55]-[9.0740625X15.63^2/2]
= 1595.55 kN-m

At 1/4th section: 9.700 m from support
BM= [185.837X9.7]-[9.0740625X10.78^2/2]
= 1275.38 kN-m

At 1/8th section: 4.850 m from support
BM= [185.837X4.85]-[9.0740625X5.93^2/2]
= 741.765 kN-m

section: 2.120 m from support
BM= [185.837X2.12]-[9.0740625X3.2^2/2]
= 347.515 kN-m

SHEAR AT DIFFERENT SECTIONS OF T-GIRDER DUE TO DEAD LOAD:

section: 2.120 m from support
SF= 185.837-[9.0740625X(2.12+1.08)]
= 156.8 kN

At 1/8th section: 4.850 m from support
SF= 185.837-[9.0740625X(4.85+1.08)]
= 132.028 kN

At 1/4th section: 9.700 m from support
SF= 185.837-[9.0740625X(9.7+1.08)]
= 88.0186 kN

At 3/8th section: 14.550 m from support
SF= 185.837-[9.0740625X(14.55+1.08)]
= 44.0094 kN

At mid section: 19.400 m from support
SF= 185.837-[9.0740625X(19.4+1.08)]
= 0.000 kN

F. CROWD LOAD:

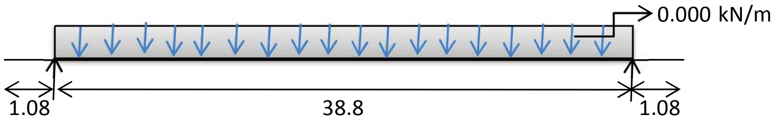
Crowd Load on safety kerb

Width of footpath = 0 mm

Live Load for foot way area (P') = 400.00 Kg/m^2

Intensity of Live Load = ((P'-260+(4800/L))x(16.5-W)/15) = 290.082 kg/m^2

Load per m run for one sides = 2.901 x 0 = 0.000 kN/m



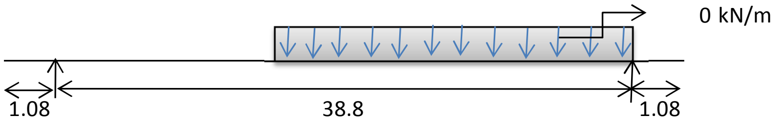
Ra = Rb = 0.000 kN

Max. Bending Moment at different locations of girder :

At mid section:	0.00 kN-m
At 3/8th section:	0.00 kN-m
At 1/4th section:	0.00 kN-m
At 1/8th section:	0.00 kN-m
At web thickening section:	0.00 kN-m

Calculation of shear force at different locations for maximum Bending Moment

At mid section:	0 kN
At 3/8th section:	0.000 kN
At 1/4th section:	0.000 kN
At 1/8th section:	0.000 kN
At web thickening section:	0.000 kN



Ra = Rb = 0.000 kN

Max. Shear Force at different locations of girder :

At mid section:	0.000 kN
At 3/8th section:	0.000 kN
At 1/4th section:	0.000 kN
At 1/8th section:	0.000 kN
At web thickening section:	0.000 kN

Calculation of Bending Moment at different locations for maximum shear force

At mid section:	Reaction=	0.000 kN
	Moment=	0.000 kN-m
At 3/8th section:	Reaction=	0.000 kN
	Moment=	0.000 kN-m
At 1/4th section:	Reaction=	0.000 kN
	Moment=	0.000 kN-m
At 1/8th section:	Reaction=	0.000 kN
	Moment=	0.000 kN-m
\t web thickening section:	Reaction=	0.000 kN
	Moment=	0.000 kN-m

E. LOAD TABLES

1. TABLE SHOWING MAX. BM AT DIFERENT SECTION AND CORRESPONDING SHEAR FORCE:

SECTION	MOMENT /SHEAR	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD	DUE TO LIVE LOAD	DUE TO CROWD LOAD
			END GIRDER	CENTRAL GIRDER		END GIRDER	END GIRDER
MID	MOMENT(T-M)	636.55	333.06	349.55	170.23	629.70	0.00
	SHEAR(T)	1.06	0.43	0.85	0.00	36.92	0.00
3/8 TH	MOMENT(T-M)	594.15	311.15	325.58	159.55	605.70	0.00
	SHEAR(T)	16.42	8.61	9.03	4.40	39.26	0.00
1/4 TH	MOMENT(T-M)	477.24	249.55	261.92	127.54	476.68	0.00
	SHEAR(T)	33.91	17.64	18.92	8.80	52.86	0.00
1/8 TH	MOMENT(T-M)	275.53	144.13	150.31	74.18	304.18	0.00
	SHEAR(T)	49.27	25.83	27.10	13.20	58.38	0.00
WEB. TH	MOMENT(T-M)	129.22	67.33	70.03	34.75	256.85	0.00
	SHEAR(T)	57.92	30.44	31.71	15.68	60.67	0.00

2. TABLE SHOWING MAX. SHEAR FORCE AT DIFERENT SECTION AND CORRESPONDING BM:

SECTION	MOMENT /SHEAR	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD	DUE TO LIVE LOAD	DUE TO CROWD LOAD
			END GIRDER	CENTRAL GIRDER		END GIRDER	END GIRDER
WEB. TH	SHEAR(T)	57.92	30.44	31.71	15.68	65.95	0.00
	MOMENT(T-M)	129.22	67.33	70.03	34.75	250.60	0.00
1/8 TH	SHEAR(T)	49.27	25.83	27.10	13.20	62.59	0.00
	MOMENT(T-M)	275.53	144.13	150.31	74.18	297.19	0.00
1/4 TH	SHEAR(T)	33.91	17.64	18.92	8.80	63.00	0.00
	MOMENT(T-M)	477.24	249.55	261.92	127.54	433.00	0.00
3/8 TH	SHEAR(T)	16.42	8.61	9.03	4.40	42.96	0.00
	MOMENT(T-M)	594.15	311.15	325.58	159.55	589.77	0.00
MID	SHEAR(T)	1.06	0.43	0.85	0.00	47.76	0.00
	MOMENT(T-M)	636.55	333.06	349.55	170.23	622.90	0.00

3. **TABLE SHOWING MAX. BENDING MOMENTS AND STRESSES AT DIFERENT SECTION**

Properties of precast girder		**Properties of composite girder(Edge)**		**Properties of composite girder(Centra**	
Area =	1.267 m^2	Area =	2.024 m^2	Area =	1.966 m^2
Y_{bg} =	1.444 m	Y_{bg} =	1.986 m	Y_{bg} =	1.959 m
Z_{tg} =	0.902 m^3	Z_{tg} =	2.887 m^3	Z_{tg} =	2.707 m^3
Z_{bg} =	0.831 m^3	Z_{bg} =	1.147 m^3	Z_{bg} =	1.128 m^3
		Z_{ts} =	2.229 m^3	Z_{ts} =	2.106 m^3

SECTION	LOCATION	DUE TO SELF WT OF PRECAST GIRDER & DIAPHRAGM	DUE TO SELF WT OF DECK SLAB & CAST IN SITU DIAPHRAGM		SUPER IMPOSED DEAD LOAD		DUE TO LIVE LOAD	DUE TO CROWD LOAD
			Edge Girder	Central Girder	Edge Girder	Central Girder	Edge Girder	Edge Girder
MID	MOMENT(T-M)	636.55	333.06	349.55	170.23	170.23	629.70	0.00
	stress at top of deck slab (T/m2)	-	-	-	76.37	80.83	282.50	0
	stress at bottom of deck slab (T/m2)	-	-	-	58.96	62.88	218.12	0
	stress at top of precast girder (T/m2)	705.71	369.25	387.53	188.72	188.72	218.12	0
	stress at bottom of precast girder (T/m2)	-766.00	-400.80	-420.64	-204.85	-204.85	-549.00	0.00
3/8 TH	MOMENT(T-M)	594.15	311.15	325.58	159.55	159.55	605.70	0.00
	stress at top of deck slab (T/m2)	-	-	-	71.58	75.76	271.74	0
	stress at bottom of deck slab (T/m2)	-	-	-	55.27	58.94	209.80	0
	stress at top of precast girder (T/m2)	658.70	344.96	360.96	176.89	176.89	209.80	0
	stress at bottom of precast girder (T/m2)	-714.98	-374.43	-391.80	-192.00	-192.00	-528.07	0.00
1/4 TH	MOMENT(T-M)	477.24	249.55	261.92	127.54	127.54	476.68	0.00
	stress at top of deck slab (T/m2)	-	-	-	57.22	60.56	213.85	0
	stress at bottom of deck slab (T/m2)	-	-	-	44.18	47.11	165.11	0
	stress at top of precast girder (T/m2)	529.09	276.66	290.38	141.39	141.39	165.11	0
	stress at bottom of precast girder (T/m2)	-574.30	-300.30	-315.19	-153.48	-153.48	-415.59	0.00
1/8 TH	MOMENT(T-M)	275.53	144.13	150.31	74.18	74.18	304.18	0.00
	stress at top of deck slab (T/m2)	-	-	-	33.28	35.22	136.46	0
	stress at bottom of deck slab (T/m2)	-	-	-	25.69	27.40	105.36	0
	stress at top of precast girder (T/m2)	305.47	159.79	166.65	82.24	82.24	105.36	0
	stress at bottom of precast girder (T/m2)	-331.57	-173.44	-180.88	-89.26	-89.26	-265.20	0.00

WEB. TH	MOMENT(T-M)	129.22	67.33	70.03	34.75	34.75	256.85	0.00
	stress at top of deck slab (T/m2)	-	-	-	15.59	16.50	115.23	0
	stress at bottom of deck slab (T/m2)	-	-	-	12.04	12.84	88.97	0
	stress at top of precast girder (T/m2)	143.26	74.65	77.64	38.53	38.53	88.97	0
	stress at bottom of precast girder (T/m2)	-155.50	-81.02	-84.28	-41.82	-41.82	-223.93	0.00

F. PRESTRESSING

Prestressing cables shall be 19 strand cables conforming to IS 14268-1995 class II with minimum breaking load = 18.371 Ton
for 12.7 mm dia ,7 ply strand.

Duct dia shall be= 90 mm.
Nominal steel area of each strand is 98.8 mm2
Area of each cable= 18.772 cm^2
Ultimate force in one cable(U.T.S) = 349.000 t
Taking maximum jack pull to be applied at jack end = 70% of U.T.S = 244.3 t

No of cable on each side for each girder= 6 nos

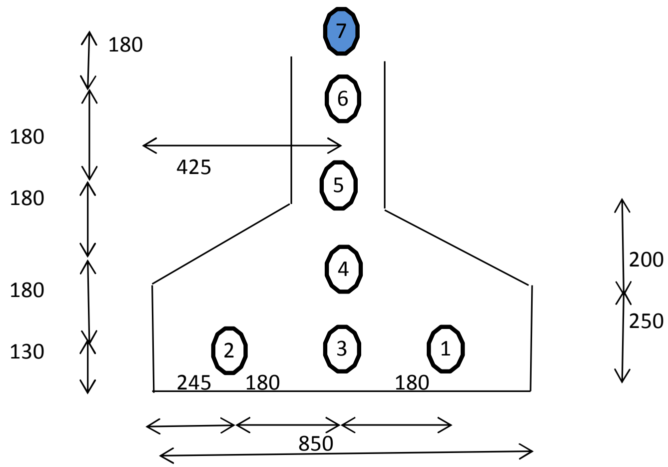
1. CABLE AT MID SECTION

No of rows of cable= 3
Vertical distance between two rows of cable (1st row to 2nd row) = 180 mm
Vertical distance between two rows of cable (2nd row to 3rd row) = 180 mm
Vertical distance between two rows of cable (3rdd row to 4th row) = 180 mm
Horizontal distance between two cable= 180 mm
Distance between cable centre & edges of T-Girder= 245 mm

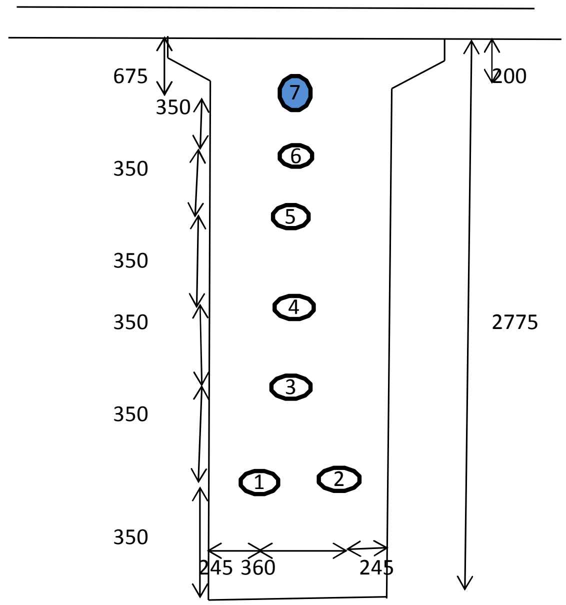
2. CABLE AT END SECTION

Vertical distance between two rows of cable (1st row to 2nd row) = 350 mm
Vertical distance between two rows of cable (2nd row to 3rd row) = 350 mm
Vertical distance between two rows of cable (3rd row to 4th row) = 350 mm
Vertical distance between two rows of cable (4th row to 5th row) = 350 mm
Distance of lowest cable centre from bottom of T-Girder= 350 mm

Half length of cable = 19720 mm



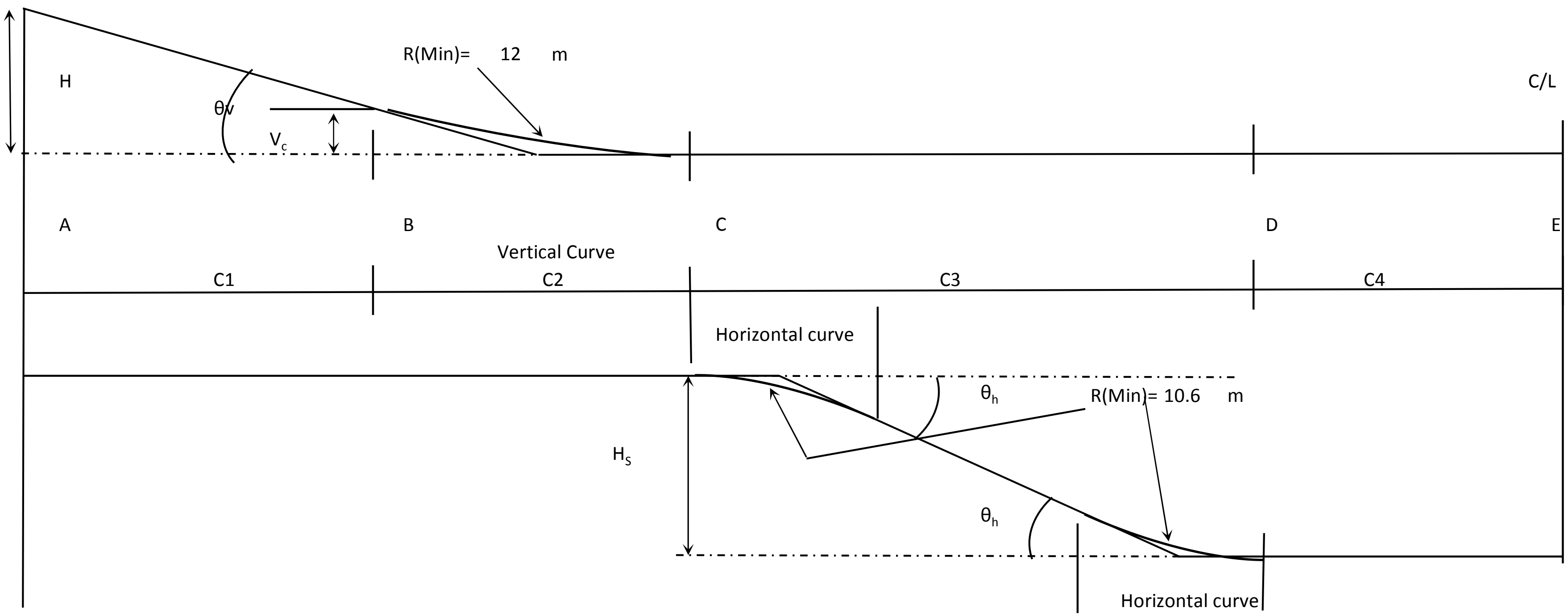
POSITION OF CABLE AT MID SECTION



POSITION OF CABLE AT END SECTION

3. Typical arrangement of cables:

Typical profile of cable in elevation & plan will be as follows:



Horrizontal splay to be given in cables 1 in 10, i.e. θH= $\tan^{-1}1/10$= 5.71 °

LHc= 10.6X0.0997= 1.056 m

C3= $10xH_s$+2xLHcX0.5

C1+C2= 19.72 -(C3+C4) **Eqn-1**

$\tan\theta_v$= H/(C1+C2x0.5)

=>C1+0.5xC2=H/tanθv **Eqn-2**

Vc= $(C2/2)x\tan\theta_v$

LVc= $C2x\theta_v/(2xCos\theta_v xTan(0.5x\theta_v)$

Eqn-1-Eqn-2

=>C2= [{19.72 -(C3+C4)}-H/tanθv]x2 **Eqn-3**

Substitute the value of C2 in **Eqn-1**,

=>C1= 19.72 -(C3+C4+C2) **Eqn-4**

Cable No.	Length (C1) in m.	Length (C2) in m.	Length (C3) in m.	Length (C4) in m.	Lift height (H) in m	Height of vertical curve (V_c) in m	Vertical angle (θv) in deg.	Length of vertical curve (LV_c) in m	Horizontal sway (H_s) in m	Horizontal angle (θ_h) in deg.	Length of horizontal curve (LH_c) in m	Length 'AB' (m)
1	1.767	2.758	0.000	15.195	0.220	0.096	4.000	2.764	0.000	0.000	0.000	1.771
2	1.767	2.758	0.000	15.195	0.220	0.096	4.000	2.764	0.000	0.000	0.000	1.771
3	1.960	5.365	0.000	12.395	0.570	0.329	7.000	5.399	0.000	0.000	0.000	1.975
4	3.834	4.386	0.000	11.500	0.740	0.269	7.000	4.413	0.000	0.000	0.000	3.863
5	2.271	6.949	0.000	10.500	0.910	0.550	9.000	7.021	0.000	0.000	0.000	2.299
6	3.918	5.802	0.000	10.000	1.080	0.459	9.000	5.862	0.000	0.000	0.000	3.967
7	4.458	5.262	0.000	10.000	1.250	0.464	10.000	5.330	0.000	0.000	0.000	4.527

4. <u>Force diagram of each cable after anchorage slip will be as shown follow:</u>

According to IRC-112,2011,

The steel stress at jacking end= $\sigma_{po}=\sigma_{px}.e(kx+\mu\theta)$

σ_{po}= Applied force

$\sigma_{po}(x)$= Force at any place in cable

μ= Friction co-efficient = 0.25 for bright metal stress

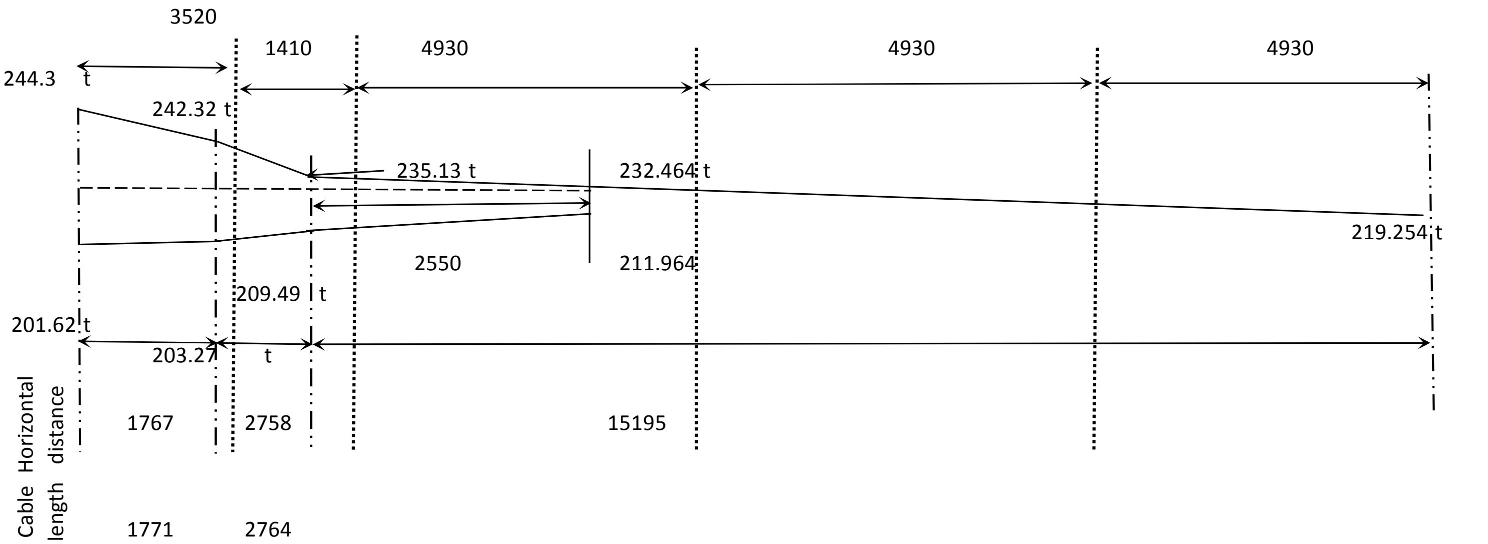
k= Wooble co-efficient = 0.0046 relieved strand

Say slip loss= 6 mm

Modulus of elasticity of material of cable= 1.95E+06 Kg/sqcm

For cable 1&2:

Web Thickening 1/8th section 1/4th section 3/8th section Mid-Section



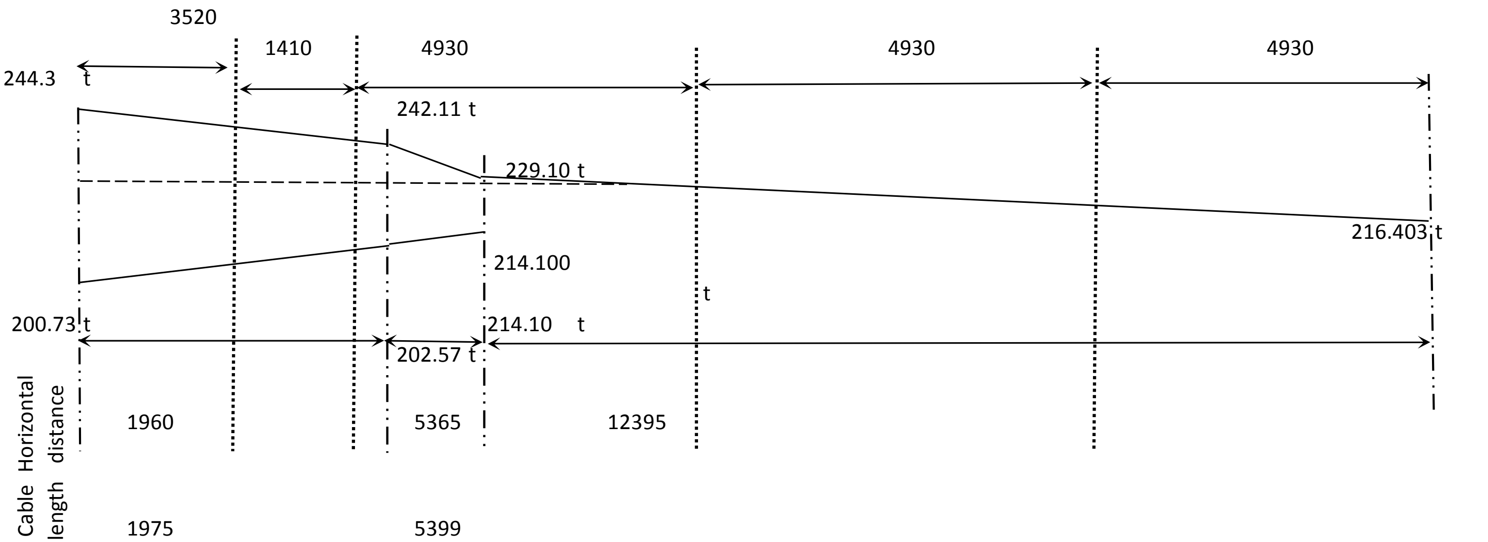
Area of the diagram

=0.5x(42.683+39.056)x1.771+0.5x(39.056+25.636)x2.764+0.5x(25.636+20.5)x2.55= 220.606 t-m

Slip of anchorage= (220.606x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 3:

Web Thickening 1/8th section 1/4th section 3/8th section Mid-Section

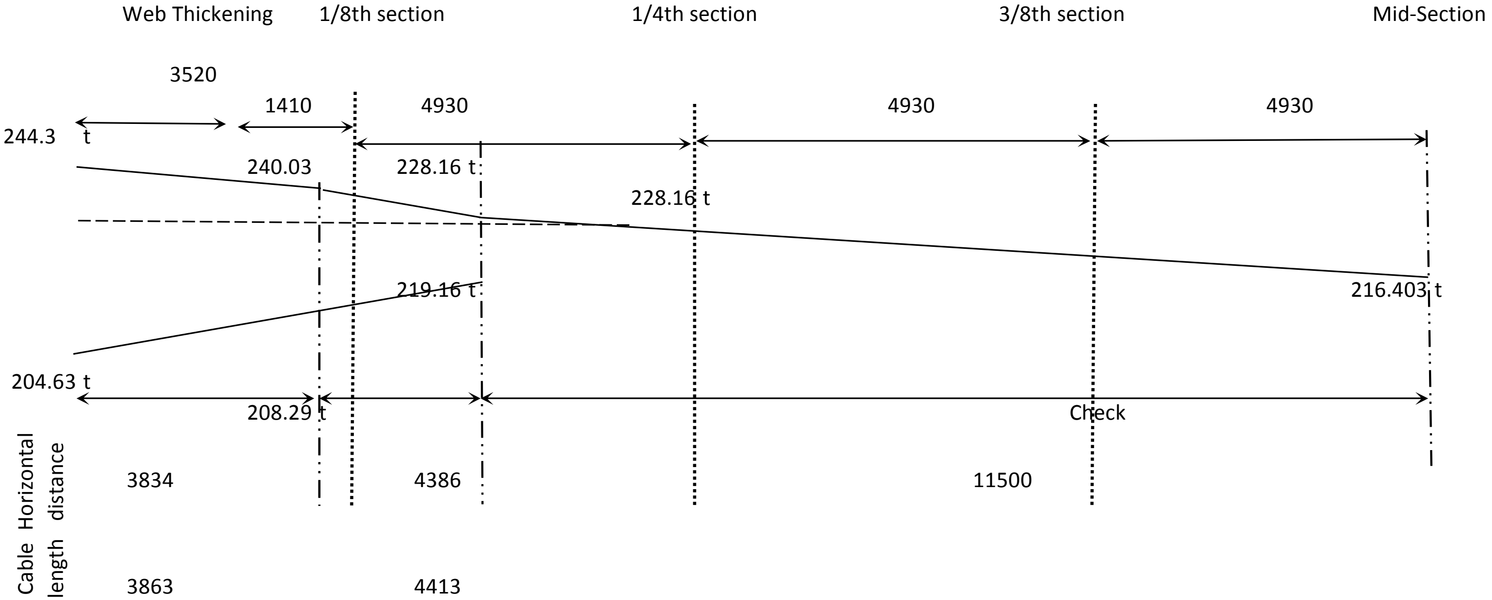


Area of the diagram

=0.5x(43.566+39.542)x1.975+0.5x(39.542+15)x5.399+0.5x(15+-214.1)x0= 229.304 t-m

Slip of anchorage= (229.304x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 4:

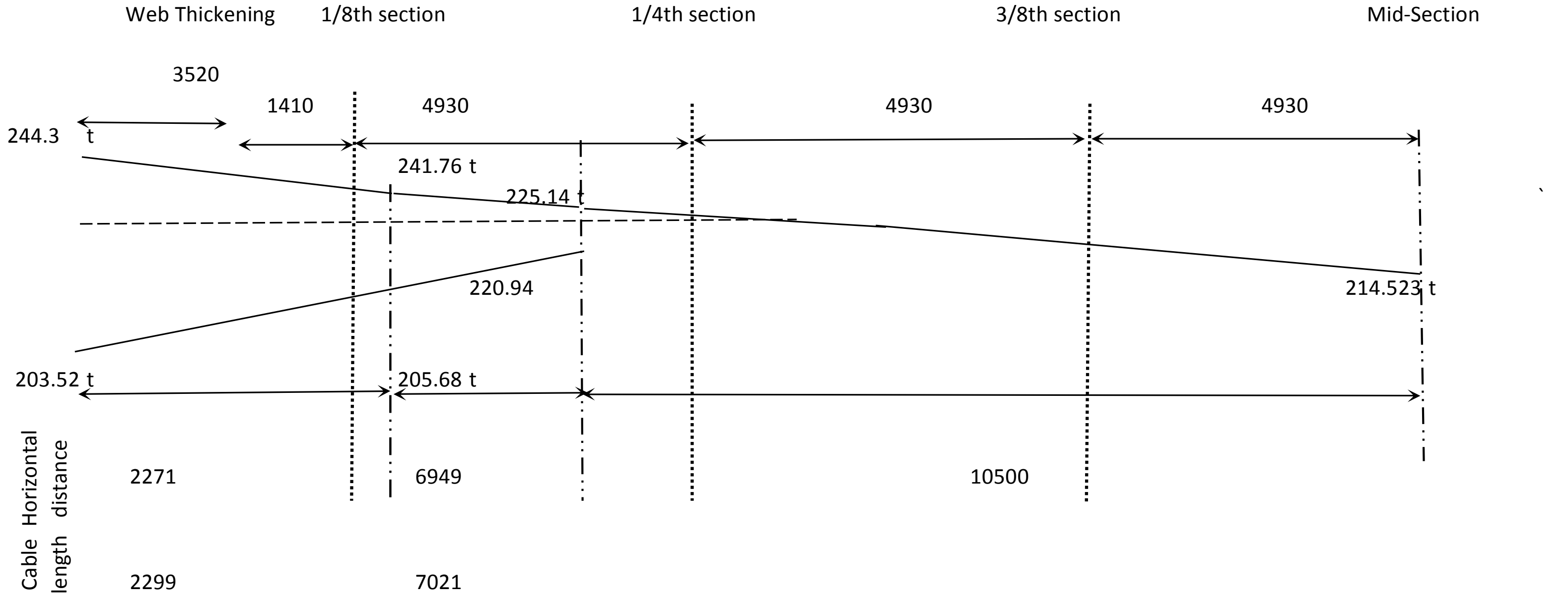


Area of the diagram

=0.5x(39.674+31.735)x3.863+0.5x(31.735+9)x4.413+0.5x(9+228.159)x0)= 227.807 t-m

Slip of anchorage= (227.807x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 5:

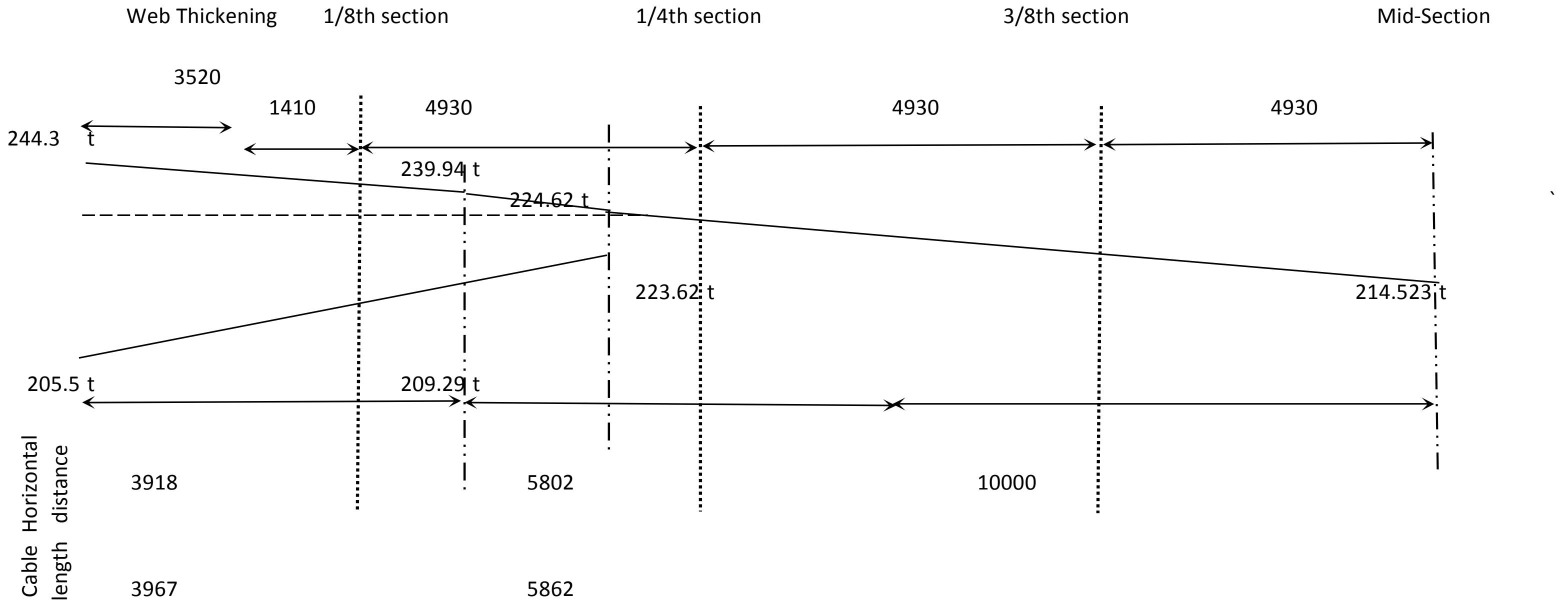


Area of the diagram

227.242 t-m

Slip of anchorage= (227.24220467294x100000/((1950000)x18.772))x10= 6.0 **OK**

For cable 6:



Area of the diagram = 227.845 t-m

Slip of anchorage= (227.845257798206x100000/((1950000)x18.772))x10= 6.0 **OK**

5. **Stages of Prestressing**

First Stage: 4 cables in each girder at 14 days or the

First stage prestressing will be on precast girder when the girder concrete attains a strength at least equal to 0.9 of its 28 days compressive strength or the concrete is 14 days old whichever is later. Using grade of girder and deck concrete as M45, strength of girder concrete at the time of stressing will be at least 40.5 Mpa. Cable no. 1,2,3 & 6 will be stressed during the first stage.

Second tag: 2 cables in each girder at 28 days after casting of deck.

Second stage stressing will be done after casting of deck slab and after the deck concrete have attained its 28 days strength. The deck will be cast after 7 days from the date of first stage prestress, i.e. when the girder concrete is 21 days old. Hence girder concrete will be 49 days old at the time of second stage presress and full composite action is obtained. Cable no. 4 & 5 will be stressed at this stage

Kerb, crash barrier, wearing course will be laid, when the girders are 60 days old.

6. **PROPERTIES OF GIRDER SECTION:**

Location	PREACAST GIRDER				COMPOSITE GIRDER				
	Area (A_p) in m^2	CG from bottom of girder Y_{bp} (m)	Section modulus of top of girder Z_{tp} (m^3)	Section modulus of bottom of girder Z_{bp} (m^3)	Area (A_c) in m^2	CG from bottom of girder Y_{bg} (m)	Section modulus of top of girder Z_{tgc} (m^3)	Section modulus of bottom of girder Z_{bgc} (m^3)	Section modulus of top of slab Z_{ts} (m^3)
End Girder	1.267	1.444	0.902	0.831	2.024	1.986	2.887	1.147	2.229
Central Girder	1.267	1.444	0.902	0.831	1.966	1.959	2.707	1.128	2.106

7. **PROPERTIES OF CONCRETE WITH AGE:**

For the purpose of calculation of loss, maturity of concrete at different days and their properties are taken as follows:

Age of concrete (days)		14	21	49	60
βcc(t)=exp{.25*[1-(28/(t/1))^.5]}		0.9016279	0.9620632	1.0629178	1.08243971
fcm(t)=βcc(t).fcm (T/m2)		4057.3255	4329.2844	4783.1301	4870.97871
Ecm= 100*5000*fcm^0.5, fcm in Mpa (T/m2)		3295999.3	3360779.33	3462811.3	3481769.63
Shrinkage co-efficient,εs=		0.0003285	0.00034737	0.0003545	0.00035675
Creep co-efficient,εc=		0	0.97306564	1.5583585	1.6838354
Permissible Temporary Stress (T/m^2)	Tensile	298	317	234	-
	Compressive	2160	2160	2160	-
Permissible Stress during service (T/m^2)	Tensile	-	-	-	0
	Compressive	-	-	-	1754

8. INITIAL PRESTRESS AT DIFFERENT SECTIONS

i). END GIRDER:

a).MID SECTION: 19.72 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {Σ P.Cosθ_v/A-Σ P.Cosθ_v. e/Z_{tp}} (t/m²)	Stress at Bottom of girder [σ_{pbg}= {Σ P.Cosθ_v/A+ ΣP.Cosθ_v. e/Z_{bp}} (t/m²)	Stress at top of slab [σ_{pts}= {Σ P.Cosθ_v/A+ ΣP.Cosθ_v. e/Z_{ts}} (t/m²)
1st Stage at 14 Days	1	0.13	0	0	219.25	0	219.254	1.314	288.100	(869.434/1.267)-(1026.594/0.902)=	(869.434/1.267)+(1026.594/0.831)=	-
	2	0.13	0	0	219.25	0	219.254	1.314	288.100			
	3	0.13	0	0	216.40	0	216.403	1.314	284.353			
	6	0.67	0	0	214.52	0	214.523	0.774	166.040	-451.916	1921.586	-
	Total	1.06				0	869.434		1026.594			
2nd Stage at 28 Days	4	0.31	0	0	216.40	0	216.403	1.676	362.691	(430.925/2.024)-(683.617/2.887)=	(430.925/2.024)+(683.617/1.147)=	(430.925/2.024)-(683.617/2.229)=
	5	0.49	0	0	214.52	0	214.523	1.496	320.926			
	Total	0.8				0	430.925		683.617	-23.884	808.912	-93.784

CG of the first stage cable from soffit of girder= 0.265 m

CG of the second stage cable from soffit of girder= 0.400 m

CG of the all cables from soffit of girder= 0.310 m

b).3/8TH SECTION: 14.79 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.13	0	0	216.41	0	216.412	1.314	284.365	(867.319/1.267)-(1021.392/0.902)=	(867.319/1.267)+(1021.392/0.831)=	-
	2	0.13	0	0	216.41	0	216.412	1.314	284.365			
	3	0.13	0	0	215.49	0	215.487	1.314	283.150	-447.819	1913.658	-
	6	0.67	0	0	219.01	0	219.008	0.774	169.512			
	Total	1.06				0	867.319		1021.392			
2nd Stage at 28 Days	4	0.31	0	0	217.58	0	217.584	1.676	364.671	(435.119/2.024)-(690.104/2.887)=	(435.119/2.024)+(690.104/1.147)=	(435.119/2.024)-(690.104/2.229)=
	5	0.49	0	0	217.53	0	217.535	1.496	325.432			
	Total	0.8				0	435.119		690.104	-24.058	816.639	-94.622

CG of the first stage cable from soffit of girder= 0.265 m

CG of the second stage cable from soffit of girder= 0.400 m

CG of the all cables from soffit of girder= 0.310 m

c).1/4TH SECTION: 9.86 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.13	0	0	213.57	0	213.569	1.314	280.630	(865.204/1.267)-(1016.191/0.902)=	(865.204/1.267)+(1016.191/0.831)=	-
	2	0.13	0	0	213.57	0	213.569	1.314	280.630			
	3	0.13	0	0	214.57	0	214.571	1.314	281.946			
	6	0.67	0	0	223.49	0	223.494	0.774	172.984	-443.722	1905.729	-
	Total	1.06				0	865.204		1016.191			
2nd Stage at 28 Days	4	0.31	0	0	218.77	0	218.766	1.676	366.651	(439.313/2.024)-(696.59/2.887)=	(439.313/2.024)+(696.59/1.147)=	(439.313/2.024)-(696.59/2.229)=
	5	0.49	0	0	220.55	0	220.547	1.496	329.939			
	Total	0.8				0	439.313		696.590	-24.233	824.367	-95.461

CG of the first stage cable from soffit of girder= 0.265 m
CG of the second stage cable from soffit of girder= 0.400 m
CG of the all cables from soffit of girder= 0.310 m

d).1/8TH SECTION: 4.93 M

Stage of Prestressing	Cable No.	CG form soffit of Girder ($\hat{y}$) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-$\hat{y}$) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {Σ P.Cosθ_v/A-Σ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {Σ P.Cosθ_v/A+ ΣP.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {Σ P.Cosθ_v/A+ ΣP.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.130	0.000	0	209.88	0	209.885	1.314	275.788	(837.373/1.267)-(969.948/0.902)=	(837.373/1.267)+(969.948/0.831)=	-
	2	0.130	0.000	0	209.88	0	209.885	1.314	275.788			
	3	0.130	0.122	0	208.95	25.465	207.394	1.314	272.515			
	6	0.750	0.122	0	211.79	25.811	210.211	0.694	145.856	-414.420	1828.116	-
	Total	1.140				51.275	837.373		969.948			
2nd Stage at 28 Days	4	0.377	0.122	0	211.01	25.716	209.437	1.609	336.924	(418.35/2.024)-(605.467/2.887)=	(418.35/2.024)+(605.467/1.147)=	(418.35/2.024)-(605.467/2.229)=
	5	0.701	0.157	0	211.52	33.089	208.914	1.285	268.544			
	Total	1.078				58.804	418.350		605.467	-3.027	734.565	-64.937

CG of the first stage cable from soffit of girder= 0.285 m

CG of the second stage cable from soffit of girder= 0.539 m

CG of the all cables from soffit of girder= 0.370 m

e).WEB-THICKENING SECTION: 3.52 M

Stage of Prestressing	Cable No.	CG form soffit of Girder (ŷ) (m)	Vertical angle (θ_v)(rad)	Horizontal angle (θ_h)(rad)	Pull in Cable(P) (ton)	Vertical pull (P. Sinθ_v) (ton)	Horizontal pull (P. Cosθ_v) (ton)	Eccentricity from CG of section (Y_b-ŷ) (m)	M=(P. Cosθ_v) x e	Stress at Top of girder [σ_{ptg}= {∑ P.Cosθ_v/A-∑ P.Cosθ_v. e/Z_{tp}} (t/m^2)	Stress at Bottom of girder [σ_{pbg}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{bp}} (t/m^2)	Stress at top of slab [σ_{pts}= {∑ P.Cosθ_v/A+ ∑P.Cosθ_v. e/Z_{ts}} (t/m^2)
1st Stage at 14 Days	1	0.130	0.070	0	207.22	14.455178	206.719	1.314	271.628	(825.17/1.267)-(837.134/0.902)=	(825.17/1.267)+(837.134/0.831)=	-
	2	0.130	0.070	0	207.22	14.455178	206.719	1.314	271.628			
	3	0.226	0.122	0	205.92	25.095	204.385	1.218	248.987			
	6	1.228	0.122	0	208.90	25.459	207.348	0.216	44.891	-276.808	1658.661	-
	Total	1.713				79.465	825.170		837.134			
2nd Stage at 28 Days	4	0.742	0.122	0	207.99	25.348	206.444	1.244	256.777	(412.3/2.024)-(544.376/2.887)=	(412.3/2.024)+(544.376/1.147)=	(412.3/2.024)-(544.376/2.229)=
	5	0.589	0.157	0	208.42	32.604	205.856	1.397	287.599			
	Total	1.331				57.952	412.300		544.376	15.144	678.314	-40.519

CG of the first stage cable from soffit of girder= 0.428 m
CG of the second stage cable from soffit of girder= 0.666 m
CG of the all cables from soffit of girder= 0.507 m

G. ELONGATION CALCULATION

Area of each cable= 1877.2 sq.mm

Modulus of elasticity 1.95E+05 Mpa

Cable No.	Segment	Length (mm)	Force at Start (T)	Force at End (T)	Elongation (mm)	Total elongation in each side (mm)
1&2	L_1	1767	201.62	203.27	10	114
	L_2	2758	203.3	209.49	16	
	L_3	2550	209.5	211.96	15	
	L_4	12645	211.96	219.254	74	
3	L_1	1960	200.73	202.57	11	114
	L_2	5365	202.6	214.10	31	
	L_3	0	214.1	214.10	0	
	L_4	12395	214.10	216.403	73	
4	L_1	3834	204.63	208.29	22	116
	L_2	4386	208.3	219.16	26	
	L_3	0	219.2	219.16	0	
	L_4	11500	219.16	216.403	68	
5	L_1	2271	203.52	205.68	13	116
	L_2	6949	205.7	220.94	40	
	L_3	0	220.9	225.14	0	
	L_4	10500	225.14	214.523	63	
6	L_1	3918	205.50	209.29	22	116
	L_2	5802	209.3	223.62	34	
	L_3	0	223.6	223.62	0	
	L_4	10000	223.62	214.523	60	

The elongation length calculated only for the cable between the midspan and end faces.
Additional length for attaching the jack may be added in consultation with the system manufacturer.
Extra elongation may be added @ 7mm/m for portion between end face and gripping point of jack.

H. LOSSES IN PRESTRESS

1. END GIRDER:

i). Stage-1: Between 14 days to 21 days

Average stress at CG of 1st stage cable= (Ref.: Stress tables)
=(2x(1250.539+1331.641+1212.439+1104.347)+1069.465)/9
1207.488 T/m^2

Elastic shortening:

Loss in Prestressing force due to elastic shortening=
(0.5x1207.488x19500000x0.0018772x(4-1)/3295999)=
= 20.116 t

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	20.116x 100/(869.434)=	2.31 %
ii) At 3/8 th section=	20.116x 100/(867.319)=	2.32 %
iii) At 1/4 th section=	20.116x 100/(865.204)=	2.33 %
iv) At 1/8 th section=	20.116x 100/(837.373)=	2.40 %
v) At web thickening section=	20.116x 100/(825.17)=	2.44 %

Relaxation loss in 1st stage cable:

Average stresses in 1st stage cables at differen sections, just after seating of anchorage will be as follows :

i) At mid-section=	(1000x(869.434x0.977))/(4x18.772)=	11310.97 Kg/cm^2
ii) At 3/8 th section=	(1000x(867.319x0.977))/(4x18.772)=	11282.80 Kg/cm^2
iii) At 1/4 th section=	(1000x(865.204x0.977))/(4x18.772)=	11254.63 Kg/cm^2
iv) At 1/8 th section=	(1000x(837.373x0.976))/(4x18.772)=	10883.99 Kg/cm^2
v) At web thickening section=	(1000x(825.17x0.976))/(4x18.772)=	10721.47 Kg/cm^2

Average stress in 1st stage cables:
=(2x(10721.472+10883.993+11254.631+11282.798)+11310.966)/9
= 11066.306 Kg/cm^2
= 0.595 of Ultimate tensile stress
Ultimate tensile stress= (349x1000)/18.772= 18592 Kg/cm^2

Ref: Table- 6.2, IRC-112:2011
1000 hour relaxation loss in 1 st stage cables= 1.190 %
Final (0.5x10^6 hours) relaxation loss in 1 st stage cables= 3.571 %

Loss due to shrinkage and creep in 1st stage cable:

Shrinkage strain= (0.000347365891535569-0.000328469167354467)= 1.89E-05
Creep strain between 14 days and 21 days = 3.28E-04

Average stress at CG of 1st stage cables at 14 days just after seating of anchorages is: [Ref: stress Tables]
=(2x(1217.387+1293.258+1173.347+1065.193)+1030.25)/9
1169.847 T/m^2 = 11.698 Mpa

Assumed loss in different sections due to creep and shrinkage as follows:

i) At mid-section=	5.85 %
ii) At 3/8 th section=	5.86 %
iii) At 1/4 th section=	5.88 %
iv) At 1/8 th section=	6.07 %
v) At web thickening section=	6.16 %

Average stress at CG of 1st stage cables at 21 days with 1000 hour relaxation loss will be as follows:
[Ref: stress Tables]
=(2x(1116.378+1176.2+1053.508+945.109)+909.919)/9
1054.701 T/m^2 = 10.547 Mpa
Average stress along CG of 1st stage cables during 14 days and 21 days will be
(0.5x(10.547+11.69847)= 11.123 Mpa
Creep strain during this period= 3.28E-04
Loss due to creep and shrinkage=
(0.0000188967241811016+0.000328)x(1950000x4x18.772)/1000= 50.849 T
Percentage loss:

i) At mid-section=	(50.849X100)/(869.434)=	5.85 % Hence OK
ii) At 3/8 th section=	(50.849X100)/(867.319)=	5.86 % Hence OK
iii) At 1/4 th section=	(50.849X100)/(865.204)=	5.88 % Hence OK
iv) At 1/8 th section=	(50.849X100)/(837.373)=	6.07 % Hence OK
v) At web thickening section=	(50.849X100)/(825.17)=	6.16 % Hence OK

ii). Stage-2: Between 21 days to 49 days

Loss due to shrinkage and creep :

Shrinkage strain= (0.000354499693215269-0.000347365891535569)= 7.13E-06
Creep strain between 21 days and 49 days = 1.45E-04

Average stress at CG of 1st stage cables at 14 days just after seating of anchorages is: [Ref: stress Tables]
=(2x(1059.382+1036.986+808.305+639.375)+582.658)/9
852.306 T/m^2 = 8.523 Mpa

Assumed loss in different sections due to creep and shrinkage betweeen 21 days to 49 as follows:

i) At mid-section=	2.56 %
ii) At 3/8 th section=	2.57 %
iii) At 1/4 th section=	2.57 %
iv) At 1/8 th section=	2.66 %
v) At web thickening section=	2.70 %

Average stress at CG of 1st stage cables at 21 days with 1000 hour relaxation loss will be as follows:
[Ref: stress Tables]
=(2x(1022.726+994.521+765.069+596.059)+539.286)/9
810.671 T/m^2 = 8.107 Mpa
Average stress along CG of 1st stage cables during 14 days and 21 days will be
(0.5x(8.107+8.52306)= 8.315 Mpa
Creep strain during this period= 1.45E-04
Loss due to creep and shrinkage=
(0.0000071338016796995+0.000145)x(1950000x4x18.772)/1000= 22.248 T
Percentage loss:

i) At mid-section=	(22.248X100)/(869.434)=	2.56 % Hence OK
ii) At 3/8 th section=	(22.248X100)/(867.319)=	2.57 % Hence OK
iii) At 1/4 th section=	(22.248X100)/(865.204)=	2.57 % Hence OK
iv) At 1/8 th section=	(22.248X100)/(837.373)=	2.66 % Hence OK
v) At web thickening section=	(22.248X100)/(825.17)=	2.70 % Hence OK

iii). **Stage-3: Between 49 days to 60 days**

Additional stress at CG of 1st & 2nd stage cables due to 2nd stage prestressing are as follows:

Total depth of precast girder:	2.775 m		
a) At mid section:			
Stress at top=f_{tg}=	-23.884 T/m^2		
Stress at bottom=f_{bg}=	808.912 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(808.912-(808.912--23.884)x0.265/2.775)=	729.384 T/m^2
Stress at CG of 2nd stage cables=		(808.912-(808.912--23.884)x0.4/2.775)=	688.869 T/m^2
b) At 3/8 th section:			
Stress at top=f_{tg}=	-24.058 T/m^2		
Stress at bottom=f_{bg}=	816.639 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(816.639-(816.639--24.058)x0.265/2.775)=	736.356 T/m^2
Stress at CG of 2nd stage cables=		(816.639-(816.639--24.058)x0.4/2.775)=	695.457 T/m^2
c) At 1/4 th section:			
Stress at top=f_{tg}=	-24.233 T/m^2		
Stress at bottom=f_{bg}=	824.367 T/m^2		
CG of 1st stage cables=	0.265 m		
CG of 2nd stage cables=	0.400 m		
Stress at CG of 1st stage cables=		(824.367-(824.367--24.233)x0.265/2.775)=	743.33 T/m^2
Stress at CG of 2nd stage cables=		(824.367-(824.367--24.233)x0.4/2.775)=	702.046 T/m^2
d) At 1/8 th section:			
Stress at top=f_{tg}=	-3.027 T/m^2		
Stress at bottom=f_{bg}=	734.565 T/m^2		
CG of 1st stage cables=	0.285 m		
CG of 2nd stage cables=	0.539 m		
Stress at CG of 1st stage cables=		(734.565-(734.565--3.027)x0.285035631701054/2.7	658.803 T/m^2
Stress at CG of 2nd stage cables=		(734.565-(734.565--3.027)x0.538929026393131/2.7	591.318 T/m^2

e) Web thickening section:

Stress at top= f_{tg}=	15.144 T/m^2		
Stress at bottom= f_{bg}=	678.314 T/m^2		
CG of 1st stage cables=	0.428 m		
CG of 2nd stage cables=	0.666 m		
Stress at CG of 1st stage cables=		(678.314-(678.314-15.144)x0.42831826434307/2.77	575.954 T/m^2
Stress at CG of 2nd stage cables=		(678.314-(678.314-15.144)x0.665549784066415/2.7	519.261 T/m^2

Average stress at CG of 1st stage cables=
(2x(575.954+658.803+743.33+736.356)+729.384)/9= 684.252 T/m^2
Average stress at CG of 2nd stage cables=
(2x(519.261+591.318+702.046+695.457)+688.869)/9= 633.893 T/m^2

Elastic shortening:

No. of cable stressed= 2 in each web

Loss in 1st stage cables due to second stage prestressing=
(2x684.252x19500000x0.0018772/3462811)= 14.466 T

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	14.466x 100/(869.434)=	1.66 %
ii) At 3/8 th section=	14.466x 100/(867.319)=	1.67 %
iii) At 1/4 th section=	14.466x 100/(439.313)=	1.67 %
iv) At 1/8 th section=	14.466x 100/(837.373)=	1.73 %
v) At web thickening section=	14.466x 100/(825.17)=	1.75 %

Loss in 2nd stage cables due to second stage prestressing=
(1x633.893x19500000x0.0018772/(2x3462811))= 3.35 T

Percentage of loss in prestress in different sections will be as follows:

i) At mid-section=	3.35x 100/(430.925)=	0.78 %
ii) At 3/8 th section=	3.35x 100/(435.119)=	0.77 %
iii) At 1/4 th section=	3.35x 100/(439.313)=	0.76 %
iv) At 1/8 th section=	3.35x 100/(418.35)=	0.80 %
v) At web thickening section=	3.35x 100/(412.3)=	0.81 %

Relaxation loss in 2nd stage cable:

Average stresses in 1st stage cables at differen sections, just after seating of anchorage will be as follows :

i) At mid-section=	(1000x(430.925x0.992))/(2x18.772)=	11388.65 Kg/cm^2
ii) At 3/8 th section=	(1000x(435.119x0.992))/(2x18.772)=	11500.35 Kg/cm^2
iii) At 1/4 th section=	(1000x(439.313x0.992))/(2x18.772)=	11612.06 Kg/cm^2
iv) At 1/8 th section=	(1000x(418.35x0.992))/(2x18.772)=	11053.71 Kg/cm^2
v) At web thickening section=	(1000x(412.3x0.992))/(2x18.772)=	10892.54 Kg/cm^2

Average stress in 1st stage cables:
=(2x(10892.54+11053.707+11612.056+11500.353)+11388.649)/9
= 11278.44 Kg/cm^2
= 0.607 of Ultimate tensile stress
Ultimate tensile stress= (349x1000)/18.772= 18592 Kg/cm^2

Ref: Table- 6.2, IRC-112:2011
1000 hour relaxation loss in 1 st stage cables= 1.333 %
Final (0.5x10^6 hours) relaxation loss in 1 st stage cables= 3.999 %

Loss due to shrinkage and creep in 2nd stage cable:

Shrinkage strain= (0.000356751507507038-0.000354499693215269)= 0.000002
Creep strain between 28 days and 40 days = 5.25E-05

Average stress at CG of all cables at 49 days , just after seating of 2nd stage anchorage will be as follows:
[Ref: stress Tables]
=(2x(1552.046+1609.286+1497.391+1328.08)+1266.7)/9
1471.145 T/m^2 = 14.712 Mpa

Assumed loss in different sections at 1st stage cables after 2nd stage prestressing due to creep and shrinkage as follows:

i) At mid-section=	0.92 %
ii) At 3/8 th section=	0.92 %
iii) At 1/4 th section=	0.92 %
iv) At 1/8 th section=	0.95 %
v) At web thickening section=	0.97 %

Assumed loss in different sections at 2nd stage cables after 2nd stage prestressing due to creep and shrinkage as follows:

i) At mid-section=	0.93 %
ii) At 3/8 th section=	0.92 %
iii) At 1/4 th section=	0.91 %
iv) At 1/8 th section=	0.95 %
v) At web thickening section=	0.97 %

Average stress at CG of all cables at 60 days before completion of W.C., Railing, Crash Barrier: [Ref: stress Tables]
=(2x(1526.616+1580.165+1465.891+1296.642)+1235.329)/9
1441.551 T/m^2 = 14.416 Mpa
Average stress along CG of 1st stage cables during 49 days and 60 days will be
(0.5x(14.7115+14.4155)= 14.564 Mpa
Creep strain during this period= (0.0000524843261539538X1.45635)= 0.000052
Loss due to creep and shrinkage in first stage cables=
(0.000002+0.000052)x(1950000x4x18.772)/1000= 7.978 T
Loss due to creep and shrinkage in second stage cables=
(0.000002+0.000052)x(1950000x4x18.772)/1000= 3.989 T
Percentage loss in different sections will be as follows:

a) For first stage prestress:

i) At mid-section=	(7.978X100)/(869.434)=	0.92 % Hence OK
ii) At 3/8 th section=	(7.978X100)/(867.319)=	0.92 % Hence OK
iii) At 1/4 th section=	(7.978X100)/(865.204)=	0.92 % Hence OK
iv) At 1/8 th section=	(7.978X100)/(837.373)=	0.95 % Hence OK
v) At web thickening section=	(7.978X100)/(825.17)=	0.97 % Hence OK

a) For second stage prestress:

i) At mid-section=	(3.989X100)/(430.925)=	0.93 % Hence OK
ii) At 3/8 th section=	(3.989X100)/(435.119)=	0.92 % Hence OK
iii) At 1/4 th section=	(3.989X100)/(439.313)=	0.91 % Hence OK
iv) At 1/8 th section=	(3.989X100)/(418.35)=	0.95 % Hence OK
v) At web thickening section=	(3.989X100)/(412.3)=	0.97 % Hence OK

iv). Stage-4: Between 60 days to end

Loss due to shrinkage and creep in 2nd stage cable:

Shrinkage strain= 1.32E-05

Creep strain between 60 days to infinity = 7.49E-04

Average stress at CG of all cables at 60 days , after completion of WC, Railing, Crash barrier:

[Ref: stress Tables]

=(2x(1499.488+1513.749+1345.356+1145.849)+1074.449)/9

1342.593 T/m^2 = 13.426 Mpa

Assumed loss in different sections at 1st stage cables after 3rd stage casting due to creep and shrinkage as follows:

i)	At mid-section=	12.84 %
ii)	At 3/8 th section=	12.87 %
iii)	At 1/4 th section=	12.90 %
iv)	At 1/8 th section=	13.33 %
v)	At web thickening section=	13.52 %

Assumed loss in different sections at 2nd stage cables after 3rd stage casting due to creep and shrinkage as follows:

i)	At mid-section=	12.95 %
ii)	At 3/8 th section=	12.82 %
iii)	At 1/4 th section=	12.70 %
iv)	At 1/8 th section=	13.34 %
v)	At web thickening section=	13.53 %

Average stress at CG of all cables after final loss: [Ref: stress Tables]

=(2x(1247.627+1225.044+1040.721+844.372)+769.124)/9

1053.85 T/m^2 = 10.539 Mpa

Average stress along CG of 1st stage cables during 7 days and 21 days will be

(0.5x(13.42593+10.5385)= 11.982 Mpa

Creep strain during this period= 7.49E-04

Loss due to creep and shrinkage in first stage cables=

(0.0000132+0.000749)x(1950000x4x18.772)/1000= 111.603 T

Loss due to creep and shrinkage in second stage cables=

(0.0000132+0.000749)x(1950000x2x18.772)/1000= 55.801 T

Percentage loss in different sections will be as follows:

a) For first stage prestress:

i)	At mid-section=	(111.603X100)/(869.434)=	12.84 % Hence OK
ii)	At 3/8 th section=	(111.603X100)/(867.319)=	12.87 % Hence OK
iii)	At 1/4 th section=	(111.603X100)/(865.204)=	12.90 % Hence OK
iv)	At 1/8 th section=	(111.603X100)/(837.373)=	13.33 % Hence OK
v)	At web thickening section=	(111.603X100)/(825.17)=	13.52 % Hence OK

a) For second stage prestress:

i)	At mid-section=	(55.801X100)/(430.925)=	12.95 % Hence OK
ii)	At 3/8 th section=	(55.801X100)/(435.119)=	12.82 % Hence OK
iii)	At 1/4 th section=	(55.801X100)/(439.313)=	12.70 % Hence OK
iv)	At 1/8 th section=	(55.801X100)/(418.35)=	13.34 % Hence OK
v)	At web thickening section=	(55.801X100)/(412.3)=	13.53 % Hence OK

v). Total percentage loss in each different section will be as follows:

Stages of Prestressing	% LOSS → / SECTIONS ↓	Elastic shortening 1st stage	Final Relaxation in 1st stage cables	Creep & shrinkage between 10 days & 21 days	Creep & shrinkage between 21 days & 49 days	Elasting shortening 2nd stage	Relaxation 2nd stage cables	Creep and shrinkage 49 days to 60 days	Final creep and shrinkage	Total	20% Higher time dependent loss (creep, shrinkage and relaxation)
1st stage cables	mid section	2.314	3.571	5.849	2.559	1.664		0.918	12.836	29.710	5.147
	3/8 th section	2.319	3.571	5.863	2.565	1.668		0.920	12.868	29.774	5.157
	1/4 th section	2.325	3.571	5.877	2.571	1.672		0.922	12.899	29.838	5.168
	1/8 th section	2.402	3.571	6.072	2.657	1.728		0.953	13.328	30.711	5.316
	Web thk section	2.438	3.571	6.162	2.696	1.753		0.967	13.525	31.112	5.384
2nd stage cables	mid section					0.777	3.999	0.926	12.949	18.651	3.575
	3/8 th section					0.770	3.999	0.917	12.824	18.510	3.548
	1/4 th section					0.763	3.999	0.908	12.702	18.372	3.522
	1/8 th section					0.801	3.999	0.954	13.338	19.092	3.658
	Web thk section					0.813	3.999	0.968	13.534	19.313	3.700

I. **STRESS TABLES**

1. END GIRDER:

i). **SECTION AT 0.5 L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-451.916	-451.916	1921.586	1921.586						
Self weight of Precast girder	705.709	253.793	-766.004	1155.582					1069.465	
Loss due to elastic shortenings during 1st stage cables	10.456	264.249	-44.460	1111.122					1030.250	
Relaxation loss of 1st stage cables	5.380	258.869	-22.875	1088.247						
Final Relaxation loss for 1st stage cables	16.139		-68.625							
Creep and shrinkage loss between 10 days & 21 days	26.430	285.299	-112.382	975.865					909.919	
Weight of Deck slab, cast-in-situ diaphragms	369.248	654.548	-400.797	575.068					582.658	
Creep & Shrinkage loss between 21 days & 49	11.564	666.112	-49.172	525.896					539.286	
2nd stage prestress	-23.884	630.664	808.912	1383.980	-93.784	-93.784	-23.884	-23.884		
Loss due to elastic shortenings during 1st stage cables	7.519	638.183	-31.972	1352.008						
Loss due to elastic shortenings during 2nd stage cables	0.186	638.369	-6.288	1345.720	0.729	-93.055	0.186	-23.698		1266.700
Relaxation loss of 2nd stage cables	0.318	638.687	-10.783	1334.936	1.250	-91.805	0.318	-24.016		
Final Relaxation loss for 2nd stage cables	0.955		-32.350		3.751		0.955			
Creep and shrinkage loss between 49 days & 60 days	4.368	643.056	-25.123	1309.814	0.868	-90.937	0.221	-23.795		1235.329
Self weight of hand rail and wearing course	188.722	831.777	-204.846	1104.968	76.369	-14.568	58.963	35.168		1074.449
Creep and shrinkage loss from 60 days to infinity	61.102	892.879	-351.407	753.561	12.144	-2.424	3.093	38.261		769.124
Additional loss due to full relaxation	11.396	904.275	-67.317	686.244	2.500	0.077	0.637	38.897		
Carriage way live load and footpath live load	218.116	1122.391	-548.997	137.247	282.503	282.580	218.116	257.013		
20 % higher time dependent loss	24.1117	1146.503	-127.8117	9.435	3.3526	285.933	0.8538	257.867		

Compressive Stress on 14 days	=	1155.582	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	666.112	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1383.980	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-93.784	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1122.391	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1146.503	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-447.819	-447.819	1913.658	1913.658						
Self weight of Precast girder	658.702	210.883	-714.981	1198.677					1104.347	
Loss due to elastic shortenings during 1st stage cables	10.386	221.269	-44.384	1154.293					1065.193	
Relaxation loss of 1st stage cables	5.331	215.938	-22.781	1131.512						
Final Relaxation loss for 1st stage cables	15.993		-68.342							
Creep and shrinkage loss between 10 days & 21 days	26.254	242.192	-112.191	1019.321					945.109	
Weight of Deck slab, cast-in-situ diaphragms	344.959	587.152	-374.432	644.889					639.375	
Creep & Shrinkage loss between 21 days & 49	11.491	598.642	-49.103	595.786					596.059	
2nd stage prestress	-24.058	563.093	816.639	1461.528	-94.622	-94.622	-24.058	-24.058		
Loss due to elastic shortenings during 1st stage cables	7.469	570.562	-31.918	1429.610						
Loss due to elastic shortenings during 2nd stage cables	0.185	570.748	-6.287	1423.323	0.729	-93.894	0.185	-23.873		1328.080
Relaxation loss of 2nd stage cables	0.321	571.068	-10.886	1412.437	1.261	-92.633	0.321	-24.194		
Final Relaxation loss for 2nd stage cables	0.962		-32.659		3.784		0.962			
Creep and shrinkage loss between 49 days & 60 days	4.340	575.409	-25.092	1387.345	0.867	-91.765	0.221	-23.973		1296.642
Self weight of hand rail and wearing course	176.890	752.298	-192.003	1195.342	71.581	-20.184	55.267	31.293		1145.849
Creep and shrinkage loss from 60 days to infinity	60.709	813.007	-350.970	844.372	12.135	-8.049	3.085	34.379		844.372
Additional loss due to full relaxation	11.303	824.311	-67.334	777.038	2.523	-5.526	0.641	35.020		
Carriage way live load and footpath live load	209.803	1034.113	-528.073	248.965	271.736	266.210	209.803	244.823		
20 % higher time dependent loss	23.9491	1058.062	-127.6683	121.297	3.3572	269.567	0.8536	245.676		

Compressive Stress on 14 days	=	1198.677	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	598.642	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1461.528	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-94.622	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1034.113	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1058.062	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iii). **SECTION AT 1/4TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-443.722	-443.722	1905.729	1905.729						
Self weight of Precast girder	529.091	85.370	-574.297	1331.432					1212.439	
Loss due to elastic shortenings during 1st stage cables	10.317	95.686	-44.308	1287.124					1173.347	
Relaxation loss of 1st stage cables	5.282	90.404	-22.686	1264.438						
Final Relaxation loss for 1st stage cables	15.847		-68.059							
Creep and shrinkage loss between 10 days & 21 days	26.078	116.482	-112.000	1152.438					1053.508	
Weight of Deck slab, cast-in-situ diaphragms	276.664	393.145	-300.301	852.136					808.305	
Creep & Shrinkage loss between 21 days & 49	11.410	404.555	-49.004	803.132					765.069	
2nd stage prestress	-24.233	368.912	824.367	1676.503	-95.461	-95.461	-24.233	-24.233		
Loss due to elastic shortenings during 1st stage cables	7.419	376.331	-31.863	1644.640						
Loss due to elastic shortenings during 2nd stage cables	0.185	376.516	-6.286	1638.353	0.728	-94.733	0.185	-24.048		1497.391
Relaxation loss of 2nd stage cables	0.323	376.839	-10.989	1627.364	1.273	-93.460	0.323	-24.371		
Final Relaxation loss for 2nd stage cables	0.969		-32.968		3.818		0.969			
Creep and shrinkage loss between 49 days & 60 days	4.311	381.150	-25.056	1602.308	0.867	-92.593	0.220	-24.151		1465.891
Self weight of hand rail and wearing course	141.394	522.544	-153.475	1448.833	57.217	-35.376	44.177	20.025		1345.356
Creep and shrinkage loss from 60 days to infinity	60.314	582.858	-350.531	1098.302	12.125	-23.250	3.078	23.103		1040.721
Additional loss due to full relaxation	11.210	594.069	-67.351	1030.951	2.545	-20.705	0.646	23.749		
Carriage way live load and footpath live load	165.113	759.181	-415.588	615.363	213.854	193.148	165.113	188.862		
20 % higher time dependent loss	23.7858	782.967	-127.5243	487.839	3.3619	196.510	0.8534	189.715		

Compressive Stress on 14 days	=	1331.432	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	803.132	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1676.503	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-95.461	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	759.181	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	782.967	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). **SECTION AT 1/8TH L**

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-414.420	-414.420	1828.116	1828.116						
Self weight of Precast girder	305.466	-108.954	-331.565	1496.551					1331.641	
Loss due to elastic shortenings during 1st stage cables	9.956	-98.998	-43.916	1452.635					1293.258	
Relaxation loss of 1st stage cables	4.933	-103.932	-21.762	1430.872						
Final Relaxation loss for 1st stage cables	14.800		-65.287							
Creep and shrinkage loss between 10 days & 28 days	25.166	-78.766	-111.012	1319.860					1176.200	
Weight of Deck slab, cast-in-situ diaphragms	159.790	81.023	-173.442	1146.418					1036.986	
Creep & Shrinkage loss between 21 days & 49	11.014	92.038	-48.586	1097.832					994.521	
2nd stage prestress	-3.027	77.996	734.565	1880.983	-64.937	-64.937	-3.027	-3.027		
Loss due to elastic shortenings during 1st stage cables	7.159	85.156	-31.582	1849.402						
Loss due to elastic shortenings during 2nd stage cables	0.024	85.180	-5.882	1843.520	0.520	-64.417	0.024	-3.003		1609.286
Relaxation loss of 2nd stage cables	0.040	85.220	-9.792	1833.728	0.866	-63.551	0.040	-3.043		
Final Relaxation loss for 2nd stage cables	0.121		-29.376		2.597		0.121			
Creep and shrinkage loss between 49 days & 60 days	3.977	89.198	-24.421	1809.306	0.619	-62.932	0.029	-3.014		1580.165
Self weight of hand rail and wearing course	82.236	171.433	-89.262	1720.045	33.278	-29.654	25.693	22.679		1513.749
Creep and shrinkage loss from 60 days to infinity	55.637	227.070	-341.625	1378.419	8.662	-20.993	0.404	23.083		1225.044
Additional loss due to full relaxation	9.947	237.017	-63.109	1315.310	1.731	-19.261	0.081	23.163		
Carriage way live load and footpath live load	105.362	342.379	-265.196	1050.114	136.465	117.203	105.362	128.525		
20 % higher time dependent loss	22.1422	364.521	-124.0585	926.055	2.3755	119.579	0.1107	128.636		

Compressive Stress on 14 days	=	1496.551	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-108.954	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1097.832	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1880.983	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-64.937	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1050.114	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	926.055	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). **SECTION AT WEB THICKENING**

Description of load and prestress	Stress at top Of Girder (T/m²)		Stress at bottom of Girder(T/m²)		Stress at top of Deck Slab (T/m²)		Stress at bottom of Deck Slab(T/m²)		Stress at C.G of 1st stage cables (T/m²)	Stress at C.G of all stage cables (T/m²)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-276.808	-276.808	1658.661	1658.661						
Self weight of Precast girder	143.257	-133.552	-155.496	1503.164					1250.539	
Loss due to elastic shortenings during 1st stage cables	6.748	-126.804	-40.435	1462.729					1217.387	
Relaxation loss of 1st stage cables	3.295	-130.099	-19.745	1442.984						
Final Relaxation loss for 1st stage cables	9.886		-59.235							
Creep and shrinkage loss between 10 days & 21 days	17.058	-113.041	-102.211	1340.773					1116.378	
Weight of Deck slab, cast-in-situ diaphragms	74.646	-38.395	-81.024	1259.749					1059.382	
Creep & Shrinkage loss between 21 days & 49 days	7.461	-30.934	-44.708	1215.041					1022.726	
2nd stage prestress	15.144	-23.251	678.314	1938.063	-40.519	-40.519	15.144	15.144		
Loss due to elastic shortenings during 1st stage cables	4.853	-18.398	-29.078	1908.985						
Loss due to elastic shortenings during 2nd stage cables	-0.123	-18.521	-5.511	1903.474	0.329	-40.190	-0.123	15.021		1552.046
Relaxation loss of 2nd stage cables	-0.202	-18.723	-9.042	1894.432	0.540	-39.650	-0.202	15.223		
Final Relaxation loss for 2nd stage cables	-0.606		-27.127		1.620		-0.606			
Creep and shrinkage loss between 49 days & 60 days	2.530	-16.194	-22.599	1871.832	0.392	-39.258	-0.147	15.076		1526.616
Self weight of hand rail and wearing course	38.527	22.334	-41.819	1830.013	15.591	-23.667	12.037	27.114		1499.488
Creep and shrinkage loss from 60 days to infinity	35.388	57.722	-316.135	1513.878	5.484	-18.183	-2.050	25.064		1247.627
Additional loss due to full relaxation	6.187	63.909	-57.575	1456.304	1.080	-17.103	-0.404	24.660		
Carriage way live load and footpath live load	88.968	152.876	-223.932	1232.372	115.231	98.128	88.968	113.628		
20 % higher time dependent loss	14.3438	167.220	-114.4055	1117.966	1.4993	99.628	-0.5604	113.068		

Compressive Stress on 14 days	=	1503.164	T/m²	**Hence OK**
Tensile Stress on 14 days	=	-133.552	T/m²	**Hence OK**
Compressive Stress on 21 days	=	1215.041	T/m²	**Hence OK**
Tensile Stress on 21 days	=	-30.934	T/m²	**Hence OK**
Compressive Stress on 49 days	=	1938.063	T/m²	**Hence OK**
Tensile Stress on 49 days	=	-40.519	T/m²	**Hence OK**
Compressive Stress on 60 days	=	1232.372	T/m²	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m²	**Hence OK**
Compressive Stress on infinity	=	1117.966	T/m²	**Hence OK**
Tensile stress on infinity	=	0.000	T/m²	**Hence OK**

I. A) STRESS TABLES (Servicibility checking with 1.1 times prestressing force) **cl-7.9.5(6), IRC-112:2011**

1. END GIRDER:

i). SECTION AT 0.5 L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-497.108	-497.108	2113.745	2113.745						
Self weight of Precast girder	705.709	208.601	-766.004	1347.740					1238.958	
Loss due to elastic shortenings during 1st stage cables	11.502	220.103	-48.905	1298.835					1195.821	
Relaxation loss of 1st stage cables	5.918	214.185	-25.163	1273.672						
Final Relaxation loss for 1st stage cables	17.753		-75.488							
Creep and shrinkage loss between 10 days & 21 days	29.073	243.258	-123.621	1150.052					1063.457	
Weight of Deck slab, cast-in-situ diaphragms	369.248	612.507	-400.797	749.255					736.196	
Creep & Shrinkage loss between 21 days & 49	12.721	625.227	-54.089	695.166					688.487	
2nd stage prestress	-26.272	586.235	889.803	1639.058	-103.163	-103.163	-26.272	-26.272		
Loss due to elastic shortenings during 1st stage cables	8.271	594.506	-35.169	1603.889						
Loss due to elastic shortenings during 2nd stage cables	0.204	594.710	-6.917	1596.972	0.802	-102.361	0.204	-26.068		1485.007
Relaxation loss of 2nd stage cables	0.350	595.060	-11.862	1585.110	1.375	-100.986	0.350	-26.418		
Final Relaxation loss for 2nd stage cables	1.051		-35.585		4.126		1.051			
Creep and shrinkage loss between 49 days & 60 days	4.805	599.865	-27.635	1557.475	0.955	-100.031	0.243	-26.175		1450.499
Self weight of hand rail and wearing course	188.722	788.587	-204.846	1352.629	76.369	-23.662	58.963	32.788		1289.619
Creep and shrinkage loss from 60 days to infinity	67.212	855.799	-386.548	966.081	13.359	-10.303	3.402	36.190		953.762
Additional loss due to full relaxation	12.536	868.335	-74.048	892.033	2.750	-7.552	0.700	36.891		
Carriage way live load and footpath live load	218.116	1086.451	-548.997	343.036	282.503	274.951	218.116	255.007		
20 % higher time dependent loss	26.5229	1112.974	-140.5929	202.443	3.6879	278.639	0.9392	255.946		

Compressive Stress on 14 days	=	1347.740	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	695.166	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1639.058	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-103.163	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1086.451	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1112.974	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-492.601	-492.601	2105.023	2105.023						
Self weight of Precast girder	658.702	166.101	-714.981	1390.043					1273.162	
Loss due to elastic shortenings during 1st stage cables	11.425	177.526	-48.822	1341.220					1230.093	
Relaxation loss of 1st stage cables	5.864	171.662	-25.059	1316.162						
Final Relaxation loss for 1st stage cables	17.592		-75.176							
Creep and shrinkage loss between 10 days & 21 days	28.879	200.541	-123.410	1192.751					1098.000	
Weight of Deck slab, cast-in-situ diaphragms	344.959	545.501	-374.432	818.319					792.266	
Creep & Shrinkage loss between 21 days & 49	12.640	558.140	-54.013	764.306					744.618	
2nd stage prestress	-26.464	519.036	898.303	1716.622	-104.085	-104.085	-26.464	-26.464		
Loss due to elastic shortenings during 1st stage cables	8.216	527.252	-35.110	1681.513						
Loss due to elastic shortenings during 2nd stage cables	0.204	527.456	-6.916	1674.596	0.801	-103.283	0.204	-26.261		1546.447
Relaxation loss of 2nd stage cables	0.353	527.809	-11.975	1662.622	1.388	-101.896	0.353	-26.613		
Final Relaxation loss for 2nd stage cables	1.058		-35.925		4.163		1.058			
Creep and shrinkage loss between 49 days & 60 days	4.774	532.583	-27.601	1635.021	0.954	-100.942	0.243	-26.371		1511.866
Self weight of hand rail and wearing course	176.890	709.473	-192.003	1443.018	71.581	-29.360	55.267	28.896		1361.072
Creep and shrinkage loss from 60 days to infinity	66.780	776.253	-386.067	1056.951	13.348	-16.012	3.394	32.290		1056.951
Additional loss due to full relaxation	12.434	788.687	-74.067	982.884	2.775	-13.237	0.706	32.995		
Carriage way live load and footpath live load	209.803	998.489	-528.073	454.811	271.736	258.499	209.803	242.798		
20 % higher time dependent loss	26.3440	1024.833	-140.4352	314.375	3.6930	262.192	0.9390	243.737		

Compressive Stress on 14 days	=	1390.043	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	764.306	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1716.622	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-104.085	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	998.489	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1024.833	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iii). SECTION AT 1/4TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-488.094	-488.094	2096.302	2096.302						
Self weight of Precast girder	529.091	40.998	-574.297	1522.005					1380.576	
Loss due to elastic shortenings during 1st stage cables	11.348	52.346	-48.739	1473.266					1337.575	
Relaxation loss of 1st stage cables	5.810	46.535	-24.955	1448.311						
Final Relaxation loss for 1st stage cables	17.431		-74.865							
Creep and shrinkage loss between 10 days & 21 days	28.685	75.221	-123.200	1325.111					1205.752	
Weight of Deck slab, cast-in-situ diaphragms	276.664	351.884	-300.301	1024.810					960.548	
Creep & Shrinkage loss between 21 days & 49	12.551	364.435	-53.905	970.905					912.989	
2nd stage prestress	-26.657	325.228	906.803	1931.613	-105.007	-105.007	-26.657	-26.657		
Loss due to elastic shortenings during 1st stage cables	8.161	333.389	-35.050	1896.563						
Loss due to elastic shortenings during 2nd stage cables	0.203	333.592	-6.915	1889.648	0.801	-104.206	0.203	-26.453		1715.819
Relaxation loss of 2nd stage cables	0.355	333.947	-12.088	1877.560	1.400	-102.806	0.355	-26.809		
Final Relaxation loss for 2nd stage cables	1.066		-36.265		4.199		1.066			
Creep and shrinkage loss between 49 days & 60 days	4.742	338.689	-27.561	1849.999	0.953	-101.853	0.242	-26.567		1681.168
Self weight of hand rail and wearing course	141.394	480.084	-153.475	1696.524	57.217	-44.635	44.177	17.610		1560.633
Creep and shrinkage loss from 60 days to infinity	66.345	546.429	-385.584	1310.940	13.338	-31.297	3.386	20.996		1225.535
Additional loss due to full relaxation	12.331	558.761	-74.086	1236.854	2.800	-28.498	0.711	21.707		
Carriage way live load and footpath live load	165.113	723.873	-415.588	821.265	213.854	185.356	165.113	186.819		
20 % higher time dependent loss	26.1644	750.038	-140.2767	680.989	3.6981	189.054	0.9388	187.758		

Compressive Stress on 14 days	=	1522.005	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	970.905	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1931.613	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-105.007	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	821.265	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	750.038	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). SECTION AT 1/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-455.862	-455.862	2010.927	2010.927						
Self weight of Precast girder	305.466	-150.396	-331.565	1679.362					1491.418	
Loss due to elastic shortenings during 1st stage cables	10.951	-139.445	-48.308	1631.054					1449.197	
Relaxation loss of 1st stage cables	5.427	-144.872	-23.939	1607.116						
Final Relaxation loss for 1st stage cables	16.280		-71.816							
Creep and shrinkage loss between 10 days & 28 days	27.682	-117.189	-122.113	1485.003					1320.433	
Weight of Deck slab, cast-in-situ diaphragms	159.790	42.600	-173.442	1311.561					1181.219	
Creep & Shrinkage loss between 21 days & 49	12.116	54.716	-53.445	1258.116					1134.508	
2nd stage prestress	-3.330	39.270	808.022	2119.582	-71.431	-71.431	-3.330	-3.330		
Loss due to elastic shortenings during 1st stage cables	7.875	47.146	-34.740	2084.843						
Loss due to elastic shortenings during 2nd stage cables	0.027	47.172	-6.470	2078.372	0.572	-70.859	0.027	-3.303		1807.790
Relaxation loss of 2nd stage cables	0.044	47.217	-10.771	2067.601	0.952	-69.906	0.044	-3.348		
Final Relaxation loss for 2nd stage cables	0.133		-32.314		2.857		0.133			
Creep and shrinkage loss between 49 days & 60 days	4.375	51.592	-26.863	2040.738	0.681	-69.225	0.032	-3.316		1775.757
Self weight of hand rail and wearing course	82.236	133.827	-89.262	1951.476	33.278	-35.947	25.693	22.378		1709.341
Creep and shrinkage loss from 60 days to infinity	61.200	195.028	-375.788	1575.688	9.528	-26.420	0.444	22.822		1391.766
Additional loss due to full relaxation	10.942	205.970	-69.420	1506.268	1.904	-24.515	0.089	22.910		
Carriage way live load and footpath live load	105.362	311.332	-265.196	1241.072	136.465	111.949	105.362	128.272		
20 % higher time dependent loss	24.3564	335.688	-136.4644	1104.607	2.6131	114.563	0.1218	128.394		

Compressive Stress on 14 days	=	1679.362	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-150.396	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1258.116	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	2119.582	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-71.431	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1241.072	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1104.607	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). SECTION AT WEB THICKENING

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-304.489	-304.489	1824.527	1824.527						
Self weight of Precast girder	143.257	-161.233	-155.496	1669.030					1386.531	
Loss due to elastic shortenings during 1st stage cables	7.423	-153.810	-44.478	1624.552					1350.064	
Relaxation loss of 1st stage cables	3.625	-157.435	-21.720	1602.832						
Final Relaxation loss for 1st stage cables	10.874		-65.159							
Creep and shrinkage loss between 10 days & 21 days	18.763	-138.671	-112.432	1490.400					1238.954	
Weight of Deck slab, cast-in-situ diaphragms	74.646	-64.025	-81.024	1409.376					1181.958	
Creep & Shrinkage loss between 21 days & 49 days	8.207	-55.818	-49.179	1360.197					1141.637	
2nd stage prestress	16.659	-47.367	746.145	2155.521	-44.571	-44.571	16.659	16.659		
Loss due to elastic shortenings during 1st stage cables	5.338	-42.029	-31.986	2123.536						
Loss due to elastic shortenings during 2nd stage cables	-0.135	-42.164	-6.063	2117.473	0.362	-44.209	-0.135	16.523		1722.594
Relaxation loss of 2nd stage cables	-0.222	-42.386	-9.947	2107.527	0.594	-43.615	-0.222	16.745		
Final Relaxation loss for 2nd stage cables	-0.666		-29.840		1.782		-0.666			
Creep and shrinkage loss between 49 days & 60 days	2.783	-39.603	-24.859	2082.668	0.431	-43.183	-0.161	16.584		1694.621
Self weight of hand rail and wearing course	38.527	-1.076	-41.819	2040.849	15.591	-27.593	12.037	28.621		1667.493
Creep and shrinkage loss from 60 days to infinity	38.927	37.851	-347.748	1693.100	6.032	-21.560	-2.255	26.367		1390.446
Additional loss due to full relaxation	6.805	44.656	-63.332	1629.768	1.188	-20.372	-0.444	25.923		
Carriage way live load and footpath live load	88.968	133.624	-223.932	1405.836	115.231	94.859	88.968	114.890		
20 % higher time dependent loss	15.7781	149.402	-125.8460	1279.990	1.6492	96.508	-0.6164	114.274		

Compressive Stress on 14 days = 1669.030 T/m^2 **Hence OK**
Tensile Stress on 14 days = -161.233 T/m^2 **Hence OK**
Compressive Stress on 21 days = 1360.197 T/m^2 **Hence OK**
Tensile Stress on 21 days = -55.818 T/m^2 **Hence OK**
Compressive Stress on 49 days = 2155.521 T/m^2 **Hence OK**
Tensile Stress on 49 days = -47.367 T/m^2 **Hence OK**
Compressive Stress on 60 days = 1405.836 T/m^2 **Hence OK**
Tensile Stress on 60 days = 0.000 T/m^2 **Hence OK**
Compressive Stress on infinity = 1279.990 T/m^2 **Hence OK**
Tensile stress on infinity = 0.000 T/m^2 **Hence OK**

I. **B) STRESS TABLES (Servicibility checking with 0.9 times prestressing force)** **cl-7.9.5(6), IRC-112:2011**

1. END GIRDER:

i). **SECTION AT 0.5 L**

Description of load and prestress	Stress at top Of Girder (T/m²)		Stress at bottom of Girder(T/m²)		Stress at top of Deck Slab (T/m²)		Stress at bottom of Deck Slab(T/m²)		Stress at C.G of 1st stage cables (T/m²)	Stress at C.G of all stage cables (T/m²)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-406.724	-406.724	1729.428	1729.428						
Self weight of Precast girder	705.709	298.985	-766.004	963.423					899.972	
Loss due to elastic shortenings during 1st stage cables	9.410	308.395	-40.014	923.410					864.678	
Relaxation loss of 1st stage cables	4.842	303.553	-20.588	902.822						
Final Relaxation loss for 1st stage cables	14.525		-61.763							
Creep and shrinkage loss between 10 days & 21 days	23.787	327.340	-101.144	801.678					756.381	
Weight of Deck slab, cast-in-situ diaphragms	369.248	696.589	-400.797	400.881					429.120	
Creep & Shrinkage loss between 21 days & 49	10.408	706.996	-44.254	356.627					390.085	
2nd stage prestress	-21.495	675.093	728.021	1128.902	-84.406	-84.406	-21.495	-21.495		
Loss due to elastic shortenings during 1st stage cables	6.767	681.861	-28.775	1100.127						
Loss due to elastic shortenings during 2nd stage cables	0.167	682.028	-5.660	1094.468	0.656	-83.750	0.167	-21.328		1048.393
Relaxation loss of 2nd stage cables	0.287	682.314	-9.705	1084.763	1.125	-82.625	0.287	-21.615		
Final Relaxation loss for 2nd stage cables	0.860		-29.115		3.376		0.860			
Creep and shrinkage loss between 49 days & 60 days	3.932	686.246	-22.611	1062.152	0.781	-81.843	0.199	-21.416		1020.159
Self weight of hand rail and wearing course	188.722	874.967	-204.846	857.306	76.369	-5.474	58.963	37.548		859.279
Creep and shrinkage loss from 60 days to infinity	54.992	929.959	-316.266	541.040	10.930	5.456	2.783	40.331		584.487
Additional loss due to full relaxation	10.257	940.216	-60.585	480.455	2.250	7.706	0.573	40.904		
Carriage way live load and footpath live load	218.116	1158.331	-548.997	-68.543	282.503	290.209	218.116	259.020		
20 % higher time dependent loss	21.7006	1180.032	-115.0305	-183.573	3.0173	293.227	0.7684	259.788		

Compressive Stress on 14 days	=	963.423	T/m²	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m²	**Hence OK**
Compressive Stress on 21 days	=	706.996	T/m²	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m²	**Hence OK**
Compressive Stress on 49 days	=	1128.902	T/m²	**Hence OK**
Tensile Stress on 49 days	=	-84.406	T/m²	**Hence OK**
Compressive Stress on 60 days	=	1158.331	T/m²	**Hence OK**
Tensile Stress on 60 days	=	-68.543	T/m²	**Value is less than 300 T/sqm, OK**
Compressive Stress on infinity	=	1180.032	T/m²	**Hence OK**
Tensile stress on infinity	=	-183.573	T/m²	**Value is less than 300 T/sqm, OK**

ii). SECTION AT 3/8TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-403.037	-403.037	1722.292	1722.292						
Self weight of Precast girder	658.702	255.665	-714.981	1007.311					935.532	
Loss due to elastic shortenings during 1st stage cables	9.348	265.012	-39.946	967.366					900.294	
Relaxation loss of 1st stage cables	4.798	260.214	-20.503	946.863						
Final Relaxation loss for 1st stage cables	14.394		-61.508							
Creep and shrinkage loss between 10 days & 21 days	23.629	283.843	-100.972	845.891					792.218	
Weight of Deck slab, cast-in-situ diaphragms	344.959	628.803	-374.432	471.458					486.484	
Creep & Shrinkage loss between 21 days & 49	10.342	639.144	-44.193	427.266					447.499	
2nd stage prestress	-21.653	607.150	734.976	1206.434	-85.160	-85.160	-21.653	-21.653		
Loss due to elastic shortenings during 1st stage cables	6.722	613.872	-28.726	1177.708						
Loss due to elastic shortenings during 2nd stage cables	0.167	614.039	-5.659	1172.049	0.656	-84.505	0.167	-21.486		1109.713
Relaxation loss of 2nd stage cables	0.289	614.328	-9.798	1162.252	1.135	-83.369	0.289	-21.775		
Final Relaxation loss for 2nd stage cables	0.866		-29.393		3.406		0.866			
Creep and shrinkage loss between 49 days & 60 days	3.906	618.234	-22.582	1139.669	0.781	-82.589	0.199	-21.576		1081.419
Self weight of hand rail and wearing course	176.890	795.124	-192.003	947.666	71.581	-11.007	55.267	33.691		930.625
Creep and shrinkage loss from 60 days to infinity	54.638	849.762	-315.873	631.793	10.921	-0.086	2.777	36.467		631.793
Additional loss due to full relaxation	10.173	859.935	-60.600	571.193	2.270	2.184	0.577	37.045		
Carriage way live load and footpath live load	209.803	1069.737	-528.073	43.120	271.736	273.920	209.803	246.847		
20 % higher time dependent loss	21.5542	1091.291	-114.9015	-71.782	3.0215	276.942	0.7682	247.615		

Compressive Stress on 14 days	=	1007.311	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	639.144	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1206.434	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-85.160	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1069.737	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	1091.291	T/m^2	**Hence OK**
Tensile stress on infinity	=	-71.782	T/m^2	**Value is less than 300 T/sqm, OK**

iii). SECTION AT 1/4TH L

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-399.350	-399.350	1715.156	1715.156						
Self weight of Precast girder	529.091	129.742	-574.297	1140.860					1044.302	
Loss due to elastic shortenings during 1st stage cables	9.285	139.027	-39.877	1100.982					1009.120	
Relaxation loss of 1st stage cables	4.754	134.273	-20.418	1080.564						
Final Relaxation loss for 1st stage cables	14.262		-61.253							
Creep and shrinkage loss between 10 days & 21 days	23.470	157.743	-100.800	979.764					901.265	
Weight of Deck slab, cast-in-situ diaphragms	276.664	434.406	-300.301	679.463					656.061	
Creep & Shrinkage loss between 21 days & 49	10.269	444.675	-44.104	635.359					617.149	
2nd stage prestress	-21.810	412.596	741.930	1421.393	-85.914	-85.914	-21.810	-21.810		
Loss due to elastic shortenings during 1st stage cables	6.677	419.273	-28.677	1392.716						
Loss due to elastic shortenings during 2nd stage cables	0.166	419.440	-5.658	1387.058	0.655	-85.259	0.166	-21.644		1278.964
Relaxation loss of 2nd stage cables	0.291	419.730	-9.890	1377.168	1.145	-84.114	0.291	-21.934		
Final Relaxation loss for 2nd stage cables	0.872		-29.671		3.436		0.872			
Creep and shrinkage loss between 49 days & 60 days	3.880	423.610	-22.550	1354.618	0.780	-83.334	0.198	-21.736		1250.613
Self weight of hand rail and wearing course	141.394	565.005	-153.475	1201.143	57.217	-26.116	44.177	22.440		1130.079
Creep and shrinkage loss from 60 days to infinity	54.283	619.287	-315.478	885.665	10.913	-15.204	2.770	25.211		855.907
Additional loss due to full relaxation	10.089	629.377	-60.616	825.049	2.291	-12.913	0.581	25.792		
Carriage way live load and footpath live load	165.113	794.489	-415.588	409.460	213.854	200.941	165.113	190.905		
20 % higher time dependent loss	21.4072	815.897	-114.7719	294.689	3.0257	203.966	0.7681	191.673		

Compressive Stress on 14 days	=	1140.860	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	635.359	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1421.393	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-85.914	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	794.489	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	815.897	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

iv). <u>SECTION AT 1/8TH L</u>

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-372.978	-372.978	1645.304	1645.304						
Self weight of Precast girder	305.466	-67.512	-331.565	1313.739					1171.863	
Loss due to elastic shortenings during 1st stage cables	8.960	-58.552	-39.525	1274.215					1137.319	
Relaxation loss of 1st stage cables	4.440	-62.992	-19.586	1254.628						
Final Relaxation loss for 1st stage cables	13.320		-58.758							
Creep and shrinkage loss between 10 days & 28 days	22.649	-40.343	-99.911	1154.718					1031.966	
Weight of Deck slab, cast-in-situ diaphragms	159.790	119.447	-173.442	981.276					892.753	
Creep & Shrinkage loss between 21 days & 49	9.913	129.359	-43.728	937.548					854.535	
2nd stage prestress	-2.724	116.722	661.109	1642.384	-58.443	-58.443	-2.724	-2.724		
Loss due to elastic shortenings during 1st stage cables	6.443	123.166	-28.423	1613.961						
Loss due to elastic shortenings during 2nd stage cables	0.022	123.188	-5.294	1608.667	0.468	-57.975	0.022	-2.703		1410.782
Relaxation loss of 2nd stage cables	0.036	123.224	-8.813	1599.854	0.779	-57.196	0.036	-2.739		
Final Relaxation loss for 2nd stage cables	0.109		-26.439		2.337		0.109			
Creep and shrinkage loss between 49 days & 60 days	3.579	126.803	-21.979	1577.875	0.557	-56.639	0.026	-2.713		1384.573
Self weight of hand rail and wearing course	82.236	209.039	-89.262	1488.613	33.278	-23.361	25.693	22.980		1318.157
Creep and shrinkage loss from 60 days to infinity	50.073	259.112	-307.463	1181.150	7.795	-15.566	0.363	23.344		1058.323
Additional loss due to full relaxation	8.953	268.065	-56.798	1124.352	1.558	-14.007	0.073	23.416		
Carriage way live load and footpath live load	105.362	373.427	-265.196	859.156	136.465	122.457	105.362	128.778		
20 % higher time dependent loss	19.9280	393.355	-111.6527	747.503	2.1380	124.595	0.0997	128.878		

Compressive Stress on 14 days	=	1313.739	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-67.512	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	937.548	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1642.384	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-58.443	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	859.156	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	747.503	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

v). SECTION AT WEB THICKENING

Description of load and prestress	Stress at top Of Girder (T/m^2)		Stress at bottom of Girder(T/m^2)		Stress at top of Deck Slab (T/m^2)		Stress at bottom of Deck Slab(T/m^2)		Stress at C.G of 1st stage cables (T/m^2)	Stress at C.G of all stage cables (T/m^2)
	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative	Due to load described	Cumulative		
1st stage prestress	-249.128	-249.128	1492.795	1492.795						
Self weight of Precast girder	143.257	-105.871	-155.496	1337.298					1114.547	
Loss due to elastic shortenings during 1st stage cables	6.073	-99.798	-36.391	1300.907					1084.710	
Relaxation loss of 1st stage cables	2.966	-102.763	-17.771	1283.136						
Final Relaxation loss for 1st stage cables	8.897		-53.312							
Creep and shrinkage loss between 10 days & 21 days	15.352	-87.411	-91.990	1191.146					993.802	
Weight of Deck slab, cast-in-situ diaphragms	74.646	-12.766	-81.024	1110.122					936.806	
Creep & Shrinkage loss between 21 days & 49 days	6.715	-6.050	-40.237	1069.885					903.816	
2nd stage prestress	13.630	0.864	610.482	1720.605	-36.467	-36.467	13.630	13.630		
Loss due to elastic shortenings during 1st stage cables	4.367	5.232	-26.170	1694.435						
Loss due to elastic shortenings during 2nd stage cables	-0.111	5.121	-4.960	1689.474	0.296	-36.171	-0.111	13.519		1381.498
Relaxation loss of 2nd stage cables	-0.182	4.939	-8.138	1681.336	0.486	-35.685	-0.182	13.701		
Final Relaxation loss for 2nd stage cables	-0.545		-24.414		1.458		-0.545			
Creep and shrinkage loss between 49 days & 60 days	2.277	7.216	-20.339	1660.997	0.353	-35.332	-0.132	13.569		1358.611
Self weight of hand rail and wearing course	38.527	45.743	-41.819	1619.178	15.591	-19.741	12.037	25.606		1331.483
Creep and shrinkage loss from 60 days to infinity	31.849	77.593	-284.521	1334.657	4.935	-14.806	-1.845	23.761		1104.809
Additional loss due to full relaxation	5.568	83.161	-51.817	1282.839	0.972	-13.833	-0.363	23.398		
Carriage way live load and footpath live load	88.968	172.128	-223.932	1058.907	115.231	101.398	88.968	112.366		
20 % higher time dependent loss	12.9094	185.038	-102.9649	955.942	1.3493	102.747	-0.5043	111.862		

Compressive Stress on 14 days	=	1337.298	T/m^2	**Hence OK**
Tensile Stress on 14 days	=	-105.871	T/m^2	**Hence OK**
Compressive Stress on 21 days	=	1069.885	T/m^2	**Hence OK**
Tensile Stress on 21 days	=	-6.050	T/m^2	**Hence OK**
Compressive Stress on 49 days	=	1720.605	T/m^2	**Hence OK**
Tensile Stress on 49 days	=	-36.467	T/m^2	**Hence OK**
Compressive Stress on 60 days	=	1058.907	T/m^2	**Hence OK**
Tensile Stress on 60 days	=	0.000	T/m^2	**Hence OK**
Compressive Stress on infinity	=	955.942	T/m^2	**Hence OK**
Tensile stress on infinity	=	0.000	T/m^2	**Hence OK**

J. CHECK FOR ULTIMATE STRENGTH:

Minimum Area of longitudinal reinforcement				0.18%
Area at mid section=				1.267 m^2
Required area of steel				2280.6 sqmm
Provide	**12 mm dia**	**35**	**no. bar.**	**OK**
Area at end section=				2.443 m^2
Required area of steel				4397.4 sqmm
Provide	**12 mm dia**	**59**	**no. bar.**	**OK**

Checking as Non-Prestressed high tensile reinforcement

Mu=M1+M2   1.35DL+1.75 SIDL+1.5LL+1.15FPLL

Mu=Design Moment=   2551.42 T-m

M1=.9*f_p*Asp*db1   **5632.86** T-m

f_p= Ultimate tensile strength of steel=   185915 t/m^2

No. of cables at mid section = 6

Total area of cable=Asp=   112.632 cm^2 = 1.13E-02 m^2

d_b = the depth of beam from the maximum compression edge to the centre of gravity of tendons

= 2.69 m for composite section

No extra reinforcement is required.

Checking as Crushing of Concrete

Mult= .176*b*d^2*f_{ck}+(2/3)*.8*(B_f-b)*(d_b-t/2)*t*f_{ck}

= **3808.6 T-m**

b=Width of web=	300 mm
d= Total depth=	3000 mm
f_{ck}=	45 Mpa
B_f=	1500 mm
t=	225 mm

No extra reinforcement is required.

N. DESIGN OF SHEAR :

Concrete strength = **45** Mpa

Strength of HYSD bar = **500** Mpa

i) **At Support Section**

Design shear force, V_{ED}=   = 1.35DL+1.75 SIDL+1.5LL+1.15FPLL

= 3283.78 KN

VRds=VNs = 3283.78 KN

f_{cp}= Stress at composite centroid due to prestress

= 0.6889X[651.279-(928.087X0.542/1.331)]+0.0807X203.705

= 204.75 t/m^2

= 2.048 Mpa   0.10 f_{cd}

f_{cd}= .67*fck/1.5=   = 20.1 Mpa   **PN-87,IRC-112:2011**

Maximum allowable shear force, taking,θ=45°   **Eq-10.8,IRC-112:2011**

V_{Rdmax}= α_{cw}*b_w*z*v_1*(f_{cd}/(cotθ+tanθ))   = **10166**   **The section is safe in shear**

α_{cw}=	=	1.10
b_w=	=	850 mm
z=lever arm	=	0.6 d
	=	1800 mm
v1= strength reduction factor for concrete cracked in shear		0.6
θ=		45 °

Allowable shear force without shear reinforcement — **Cl-10.3.2(2),IRC-112:2011**

V_{Rdc}=	$[0.12*K*(80*\rho_1*f_{ck})^{0.33}+0.15*\sigma_{cp}]*b_w*d$	=	**1724**	**Shear Reinforcement is required.**
$V_{rd.c\ min}$ =	$(V_{min}+0.15\sigma_{cp})b_w*d$	=	1513 KN	
V_{min} =	$0.031*K^{3/2}f_{ck}^{1/2}$	=	0.293	
K =		=	1.26	
ρ_1 =		=	0.0044	
σ_{cp} =		=	2.000 Mpa	

Calculation of Reinforcement — **Eq-10.7,IRC-112:2011**

$V_{Rds}=A_{sw}/s*z*f_{ywd}*\cot\theta$

Provide 4 L 16 mm dia stp@ 150 mm c/c.

A_{sw}=	=	804 sqmm	
$f_{ywd}=.8*f_{yk}/\gamma_m$=	=	347.83 Mpa	
θ=	=	21.8 °	
S=	=	383.17885 mm	**Hence OK**
Reinforcement Ratio=	=	0.0021015	**Hence OK**
Minimum shear reinforcement ratio	=	0.000966	**Eq-10.20,IRC-112:2011**

N. <u>DESIGN OF INTERFACE SHEAR :</u> (Cl-10.3.4, IRC-112:2011)

Section	Formula	Support	Web-Thickening	L/8	L/4	3L/8	Mid-Section
V_{EDi}=Interface Shear stress, Mpa	$\beta*V_{ED}/z*bi$	1.82	1.36	1.21	1.00	0.59	0.41
β=	Conservatively	1	1	1	1	1	1
V_{ED}=in KN		3284	2456	2184	1795	1059	736
z=in mm	=.6d for PSC	1800	1800	1800	1800	1800	1800
b_i=		1000	1000	1000	1000	1000	1000
V_{Rdi}=in KN	Resistance capacity	3.48	2.98	2.32	2.08	1.64	1.45
	$\mu*\sigma n+\rho*f_{yd}*[\mu*\sin\alpha+\cos\alpha]$=	3.477	2.981	2.315	2.082	1.640	1.447
	$0.5*v*f_{cd}$	6.03	6.03	6.03	6.03	6.03	6.03
μ=		0.6	0.6	0.6	0.6	0.6	0.6
σn=	$<.6*f_{cd}$	3.28	2.46	2.18	1.79	1.06	0.74
f_{yd}=in Mpa	$.8*f_{yd}$	400	400	400	400	400	400
α		90	90	90	90	90	90
No. of leg		**2**	**2**	**2**	**2**	**2**	**2**
Dia		**16**	**16**	**16**	**16**	**16**	**16**
Spacing		**100**	**100**	**150**	**150**	**150**	**150**
Area of steel		**4019**	**4019**	**2679**	**2679**	**2679**	**2679**
No. of leg		**2**	**2**	**2**	**2**	**2**	**2**
Dia		**12**	**12**	**12**	**12**	**12**	**12**
Spacing		**100**	**100**	**150**	**150**	**150**	**150**
Area of steel		**2261**	**2261**	**1507**	**1507**	**1507**	**1507**
As=		6280	6280	4187	4187	4187	4187
Asmin=	=.15% of Aj=	1500	1500	1500	1500	1500	1500
Check for minimum reinforcement		OK	OK	OK	OK	OK	OK
ρ=	As/Aj	0.0063	0.0063	0.0042	0.0042	0.0042	0.0042
v		0.6	0.6	0.6	0.6	0.6	0.6
Check for shear capacity		OK	OK	OK	OK	OK	OK

O. <u>DESIGN OF END BLOCK FOR BURSTING TENSILE FORCE:</u> (Cl-13.5.1, IRC-112:2011)

Prestressing force applied at cable-1=P_k= 317.59 T

Load Factor= 1.3 1025

Side of equivalent square of bearing plate,$2Y_{p0}$= 177 mm

Side of loaded area,$2Y_0$= 350 mm 350

Y_{p0}/Y_0= 0.5

F_{bst}/P_k= 0.16

Bursting tensile force=F_{bst}= 508.14 KN 350

Tensile strength for mild steel, Fe250= 217.5 Mpa 350

Reinforcement required= 2336 mm^2 2775

Provide 1 no 20 dia spiral @ 40 mm pitch. 350

Steel provided= 2747.5 mm^2

Hence Ok 350

245 245

360

K. DEFLECTION CHECK:

Deflection Calculation: Long term

Properties of composite girder(Edge)

Area = 2.024 m^2

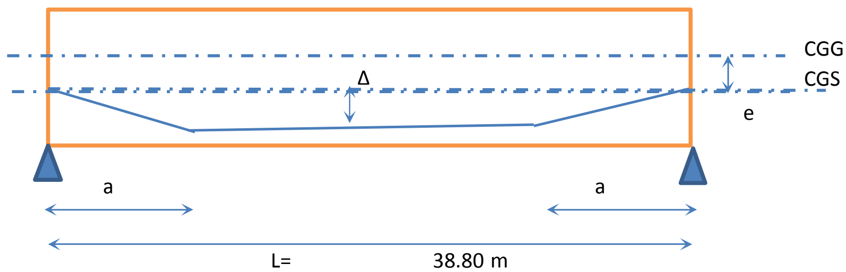
Y_{bg} = 1.986 m

Z_{tg} = 2.887 m^3

Z_{bg} = 1.147 m^3

Z_{ts} = 2.229 m^3

I_g = 2.278 m^4



Deflection due to prestress=δ_{ps}=$P.L^2/8EI\ [e+\Delta-4\Delta a^2/3L^2]$

E= 3481769.63 T/m^2

Location	Prestressing force after anchorage slip(T)	% loss at service	Effective prestressing force (T)	a (m)	Δ (in m)	e (m)	Upward Deflection (in m)
Cable-1	219.25	29.71	154.1	4.525	0.220	1.636	6.77
Cable-2	219.25	29.71	154.1	4.525	0.220	1.636	6.77
Cable-3	216.4	29.71	152.1	7.325	0.570	1.286	6.60
Cable-4	216.4	29.71	152.1	8.22	0.740	0.936	5.89
Cable-5	214.5	18.65	174.5	9.22	0.910	0.586	5.91
Cable-6	214.5	18.65	174.5	9.72	1.080	0.236	5.07

Total upward deflection= 37.02 mm

Downward deflection:

Dead load deflection		2.53 mm	From SAP
SIDL		5.97 mm	2000
Live load deflection		15.5 mm	output
	Total	**24 mm**	

Net deflection= -13.02 mm **Upward**

Allowable deflection=L/600 **64.67 mm**

(Cl-12.4.1, IRC-112:2011) **Hence OK**

SUBSTRUCTURE DESIGN OF IRANG BRIDGE CH._95.500 KM

ABUTMENT DESIGN
CHAINAGE -95.500 KM
3X41.0M PSC T-GIRDER
IRANG RIVER

ANALYSIS OF ABUTMENT

Basic design data:-

a) Superstructure:-		
Formation level =	240.000 m	
R.L. of carriageway at end long girder =	239.762 m	
Overall length of Bridge=	123.840 m	
Span (c/c of expansion joint) =	41.000 m	
Distance between C/L of bearing to C/L of exp. joint =	1.100 m	
Effective span (C/C of bearing) =	38.800 m	
Clear carraige way width =	9.500 m	
Depth of girder+deck slab at CL of carraigeway =	3.000 m	
Thickness of Bituminus concrete Wearing Coat =	0.065 m	
Thickness of cement concrete Wearing Coat =	0.075 m	
b) Hydraulic and survey data :-		
Lowest Bed Level, LBL	217.847 m	
Max. scour level =	0.000 m (Non seismic case)	
b) Sub-structure :-		
Bearing and pedestal :		
Level of bearings (near to median)=	236.894 m.	
Thickness of bearing =	0.300 m	
Thickness of pedestal =	0.440 m	
Maximum ht of bearing + pedestal =	0.740 m	
c) Abutment cap :		
Top of abutment Cap =	236.154 m	
Bottom of Abutment cap =	235.154 m	
Length of abutment Cap =	12.750 m	
Thickness of abutment cap=	1 m	
Width of abutment cap at top (including dirt wall part) =	2.070 m	
Width of abutment cap at bottom =	2.070 m	
abutment shaft :		
Total length of abutment shaft =	12.50 m	
Thickness of abutment wall =	1.200 m	
Height of abutment wall =	1.700 m	
Height of frame at abutment location =	3.440 m	
Dirt wall :		
Thikcness of Dirt wall =	0.400 m	
Height of Dirt wall =	3.780 m	
Length of dirt wall =	12.750 m	
Return wall :		
Width of return wall = (avg.)	0.500 m	
Length of return wall =	4.500 m	
Height of return wall =	4.000 m	
Foundation and foundation Slab :		
Thickness of foundation slab at abutment face=	1.500 m	
Thickness of foundation slab at edge=	1.000 m	
Length of foundation slab =	12.8 m	
Width of foundation slab =	7.4 m	
Bottom of foundation slab =	231.954 m	
Top of foundation slab =	233.454 m	
Approach Slab :		
Length of approach slab =	3.500 m	
Thickness of approach slab =	0.300 m	(At mid-section)
Thickness of approach slab =	0.300 m	(At edge)
d) Material Properties :		
Grade of steel =	Fe 500	
Grade of Concrete =	M 30	
Unit Weight of concrete =	25 KN/m^3	
Unit Weight of Cement concrete Wearing Coat =	25 KN/m^3	
Unit Weight of Bituminus concrete Wearing Coat =	22 KN/m^3	

e) Soil property:-

Density of soil = 18.00 KN/m^3

Angle of Shearing resistance of backfill soil = 30^o

Property of Backfill material behind Abutment and Return wall

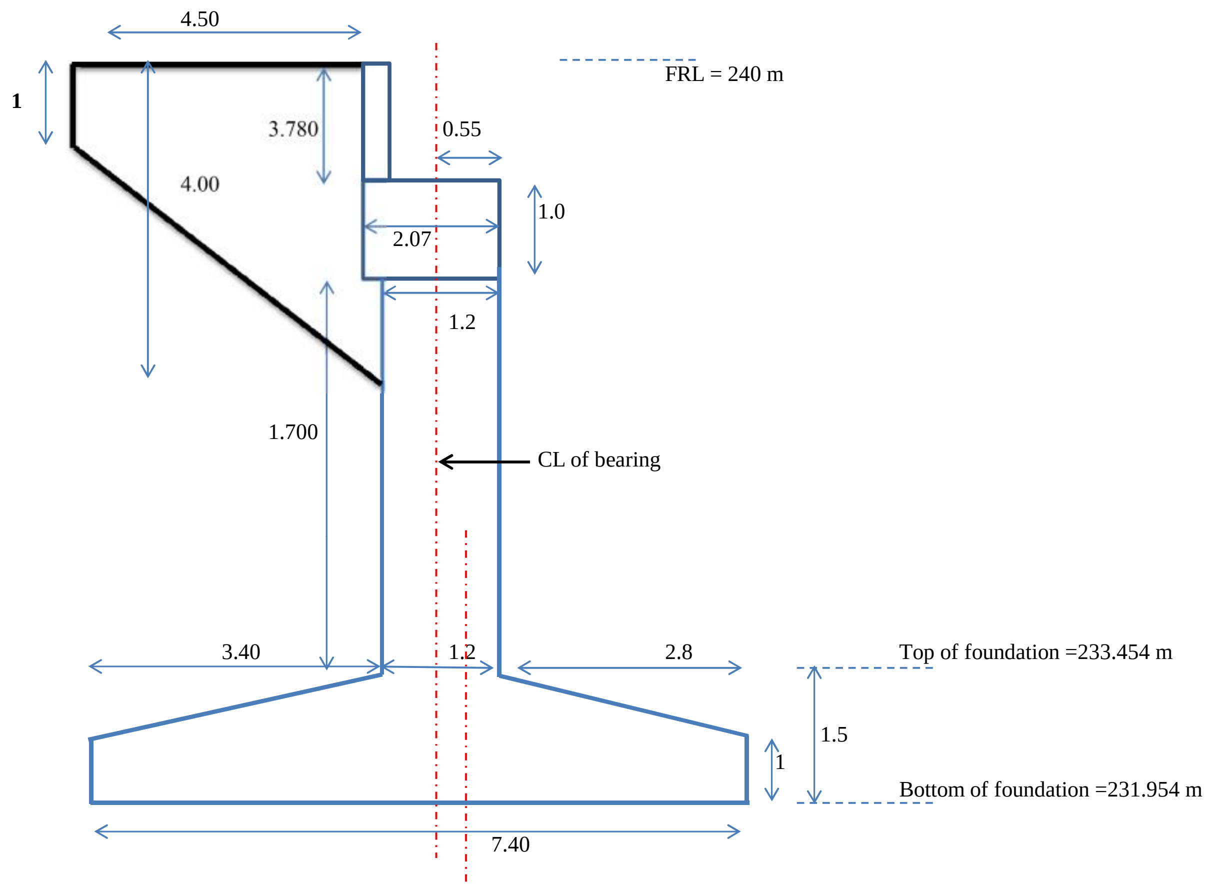
ϕ = 30.00 ^{o}

δ = 20.00 ^{o}

α = 0.00 ^{o}

ι = 0.00 ^{o}

γ= 18.00 KN/m^3



Details of abutment

LOAD CALCULATIONS

Permanent Load :

Self Weight/Dead Load

Dead Load from super-structure =		10850 KN
So, dead load in one abutment =		5425 KN
Line of action of load from center of abutment =		-0.05 m
Line of action of load from CG of foundation slab =		0.325 m
Self weight of dirt wall =	(12.75 m x 3.78 m x 0.4 m x 25)=	481.95 KN
Line of action of load from center of Abutment =		-1.27 m
Line of action of load from center of foundation slab =		-0.995 m
Self weight of Abutment Cap =	(12.75 m x 2.07 m x 1 m x 25)=	659.813 KN
Line of action of load from center of abutment =		-0.435 m
Line of action of load from center of foundation slab =		-0.16 m
Self weight of abutment shaft =	{ 12.5 m x 1.7 m x 0.5 x (1.2 + 1.2) m x 25 }=	637.500 KN
Line of action of load from center of foundation slab =		0.375 m
Self weight of foundation slab =	12.8 m x [{ 0.5 x (7.4 + 1.2) m x (1.5 - 1) m } + (7.4 m x 1 m)] x 25	

= 3056 KN

Self weight of Fin wall = { 2 x 4.5m x 0.5m x 0.5 x (4 +)m x 25}= 450 KN

Line of action of load from center of abutment = -2.200 m

Line of action of load from center of foundation slab = -2.595 m

Backfill Weight

Height of backfill= 6.468 m

Weight of backfill above the hill side of the abutment foundation slab=

(12.8 m x 3.4 m x 3.55 m x 18)-(0.87 m x 4.78 m x 12.8 m x 18)= 4304.471 KN

Distance of center of load from abutment center = 2.3 m

Line of action of load from center of foundation slab = -1.70 m

Earth Pressure

Calculation of Active Earth Pressure Coefficient (Static)

$\cos^2(\phi-\alpha)$ = 0.75

$\cos^2\alpha$ = 1.00

$\cos(\delta+\alpha)$ = 0.94

$\cos(\alpha-\iota)$ = 1.00

$\sin(\phi+\delta)$ = 0.77

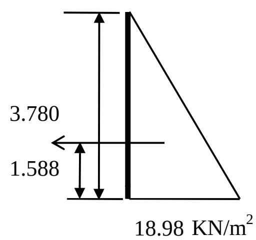
$\sin(\phi-\iota)$ = 0.50

K_a = 0.279

Considering dirt wall

Height of Dirt wall = H = 3.780 m

Length of dirt wall = 12.750 m



Earth pressure at base due to backfill soil =0.279x18x3.78 = 18.98 KN/m^2

Total load on dirt wall = 457.37 KN

This load is located 0.42 of the height of wall above base,

Lever arm = 1.588 m…..(Clause 214.1,IRC:6-2014,page-41)

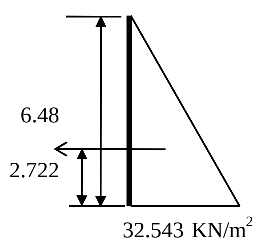
Moment at dirt wall base = 726.121 KN-m

Considering abutment wall :

Height of the abutment=Height of abutment wall+Dirt wall= 6.480 m

Length of abutment wall = 12.50 m

Height from foundation slab bottom to deck level = 7.980 m



Calculation of moment due to earth pressure :-

Earth pressure at base due to backfill soil =0.279x18x6.48 = 32.543 KN/m^2
Horizontal force due to backfill earth = 1317.97 KN (Considering abutment bottom)
This load is located 0.42 of the height of wall above base,
Lever arm = 2.722 m…..(Clause 214.1,IRC:6-2014,page-41)
Acting at a ht of = 2.722 m (from base of abutment)
Earth pressure at base due to backfill soil =0.279x18x7.98 = 40.076 KN/m^2
Horizontal force due to backfill earth = 1998.77 KN (Considering foundation slab bottom)
Acting at a ht of = 0.42 m of total height = 3.35 m (from bottom of foundation slab level)
Total moment at the base of abutment due to earth pressure = 3587.00 KN-m
Total moment at the base of foundation slab due to earth pressure = 6699.0727 KN-m

Variable gravity loads treated as permanent loads

Super Imposed Dead Load (SIDL) (except surfacing)
Super Imposed Dead Load acting on abutment = 703 KN

Surfacing and Wearing Coat
Surfacing or loading on abutment due to cement concrete wearing coat = (2 x 40.96m x 1.5m x 0.075m x 25KN/cum)KN /2
(For footpath) = 115.20 KN

Surfacing or loading on abutment due to cement concrete wearing coat = (40.96m x 9.5m x 0.065m x 22KN/cum)KN /2
(For carriageway) = 278.22 KN

So,on one abutment total surfacing load or load due to wearing coat = 393.42 KN

Vehicular Live Load

Carriageway Live load

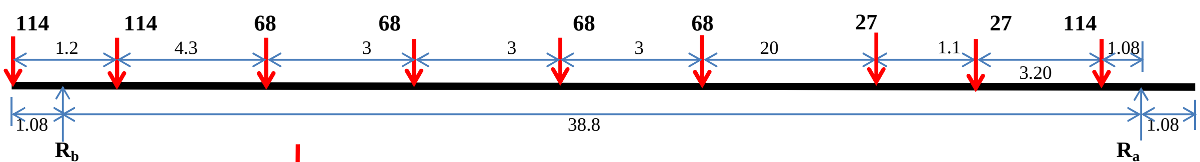
(I) 2 lanes of Class-A
(II) 1 lane of 70R
(III) 1 lane of 70R Wheele
(IV) 1 lane of Special Vehicle(385 T)

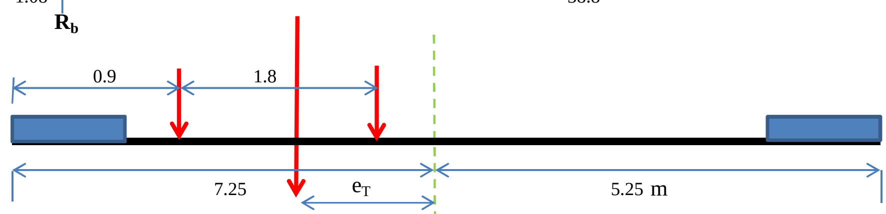
1 **Type of Loading =** **Class A train of vehicle.**

A) One Span Loaded

Span, L_e =		38.80	m
L_c =		1.08	m
Expansion gap =		0.04	m
Impact Factor =		1.101	

Case - 1: One Lane / one span loaded.					
Minimum Clearence			=	150	mm
Width of ground contact (In transverse direction)			=	500	mm
Width of Footpath with crush barrier & kerb			=	2500	mm
Width of carriageway			=	9.50	m
Width of Crash Barrier (only)			=	500	mm

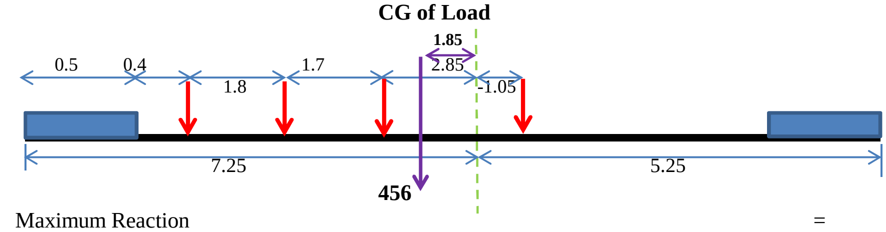




Maximum Reaction = R_b = 495.653 kN
And transverse eccentricity, wrt deck, e_T = 3.45 m
And longitudinal eccentricity, wrt abutment, e_L = -0.05 m

Case - 2: Two Lane / One span loaded.

Minimum clearence = 1200 mm between two outer edges of vehicle.



Maximum Reaction = 991.3 kN

And transverse eccentricity, wrt deck, e_T = 1.85 m

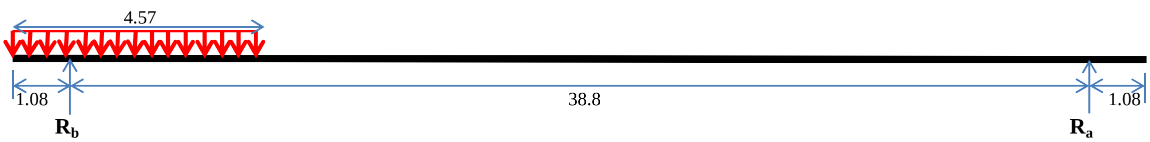
And longitudinal eccentricity, wrt abutment, e_L = -0.05 m

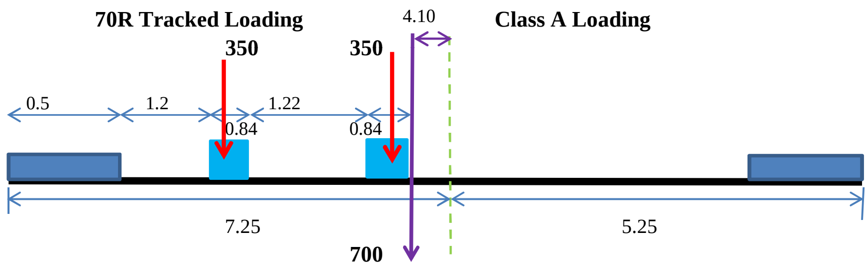
2 Type of Loading = IRC class 70R Tracked

A) One Span Loaded

				Case - 1: 70R Tracked + Class A					
Span, L_e =		38.8	m	Minimum Clearence			=	1200	mm
L_c =		1.08	m	Width of ground contact			=	840	mm
Expansion gap =		0.04	m	Width of footpath with kerb & crash barrier			=	2500	mm
Impact factor =		1.100		Width of carriageway			=	9.50	m

700/4.57 = **153.17** kN/m





Maximum Reaction for 70R Tracked = Rb = 725.8 kN

Hence,Total Reaction Rb = 725.8 kN

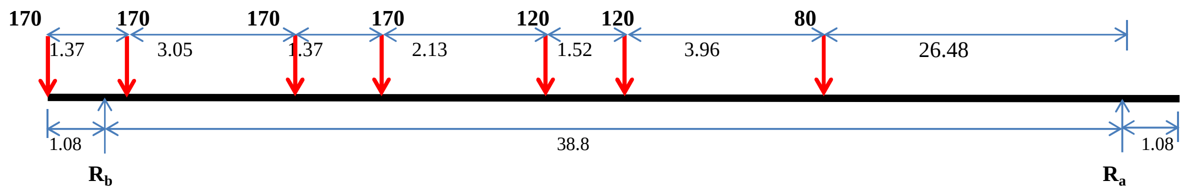
And transverse eccentricity, wrt deck, e_T = 4.10 m

And longitudinal eccentricity, wrt abutment, e_L = -0.05 m

3 Type of Loading = **IRC 70 R Wheel + single lane Class A**

A) One Span Loaded

				Case - 1: 70 R Wheel + Class A					
Span, L_e =		38.8	m	Minimum Clearence			=	1200	mm
L_c =		1.08	m	Width of ground contact			=	860	mm
Expansion gap =		0.04	m	Width of footpath with kerb & crash barrier			=	2500	mm
Impact factor =		1.101		Width of carriageway			=	9.5	m

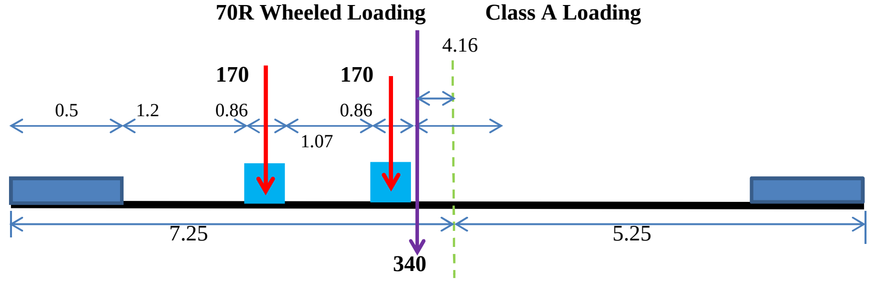


Maximum reaction = Rb = 986.25 kN

Hence,Total Reaction Rb = 986.25 kN

max longitudinal eccentricity = eL = -0.050 m

70R Wheeled Loading **Class A Loading**



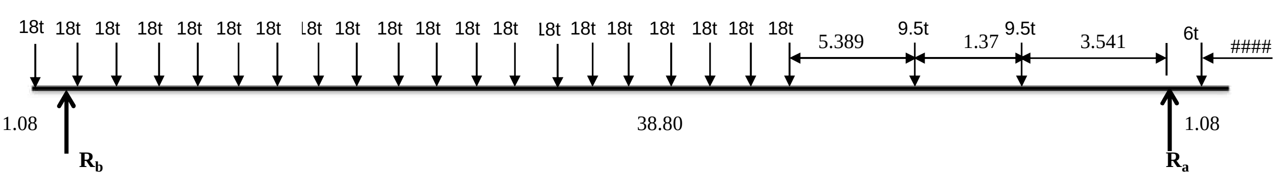
max transverse eccentricity = eT = 4.16 m

4 Type of Loading = **IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle**

(AMENDMENT TO IRC:6-2014, AMENDMENT NO.1_CLAUSE 204.5)

A) One Span Loaded

				Case - 1: IRC Class SV Loading					
Span, L_e =		38.8	m	Minimum Clearence			=	-	
L_c =		1.08	m	Width of ground contact			=	860	mm
Expansion gap =		0.04	m	Width of crash barrier			=	2500	mm
Impact factor =		1		Width of carriageway			=	9.5	m



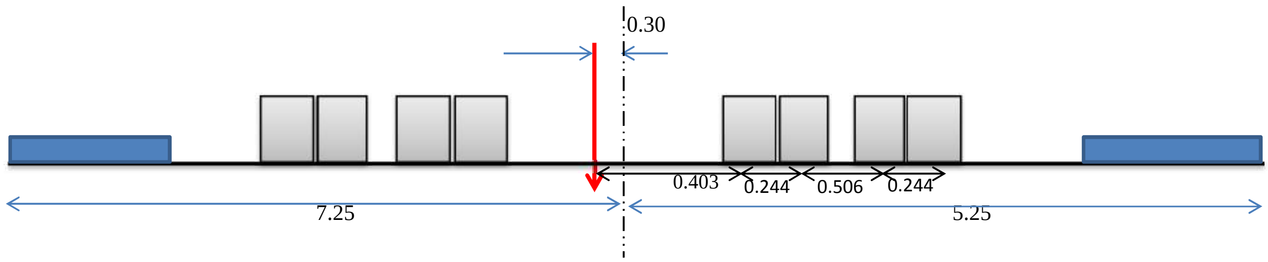
Loading= 20 nos. of wheels each 180 KN @ c/c 1.5 m for 28.5 M Span.

so,(28.5/1.5+1)= 20

hence for, 39.88 m Sapn = 20

Left side of Ra 39.88 m Sapn = 20

Maximum reaction =		Rb	=	2299.06 kN
	Hence,Total Reaction Rb		=	2299.06 kN
max longitudinal eccentricity =		eL	=	-0.050 m



max transverse eccentricity = eT = 0.30 m (AMENDMENT TO IRC:6-2014, AMENDMENT NO.1_CLAUSE 204.5.3)

Vehicular live load

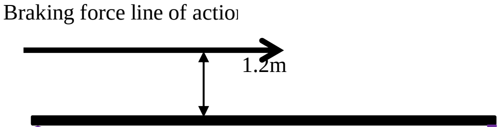
Detail analysis of vehicular live loads are done in previous pier analysis sheets

Load cases considered for abutment are tabulated bellow :

CASE	Load due to main wheel	Load due to additional wheel	R_A (kN)	eL (m)	eT (m)	M_L (kN-m)	M_T (kN-m)
CLASS A TWO LANE SINGLE SPAN	668	668	991.31	-0.050	1.850	-50	1833.915
70R TRACKED ONE LANE	700	0	725.817	-0.050	4.100	-36	2975.848
70R WHEELED ONE LANE & CLASS A ONE LANE SINGLE SPAN	1000	0	986.252	-0.050	4.155	-49	4097.877
IRC CLASS SV LOADING : SPECIAL MULTI AXEL HYDRAULIC TRAILER VEHICLE	3850	0	2299.057	-0.050	0.300	-115	689.717

Longitudinal Forces

Calculation of Braking Forces ……. (Ref. cl. 211 of IRC 6-2014, page-37)

				Case - I	Case - II	Case - III	Case - IV
	Braking force line of actioi 			**Class A - Two lane / one span loaded**	**70R Tr. , one span loaded**	**70R Wh. Load , one span loaded**	**IRC CLASS SV LOADING : SPECIAL MULTI AXEL HYDRAULI**
a	Total Load	kN	=	1336	700	1000	3850
b	Braking force Fh	kN	=	267.20	140	200	0
c	Braking force at Abutment side Fh	kN	=	267.20	140.00	200.00	0.00
d	Friction forces at bearing level μ(Rg + Rq) =	kN	=	192.49	184.52	192.34	231.72
e	ThickNess of wearing coat	m	=	0.065	0.065	0.065	0.065
f	Ht. of Braking force act above bearing	m	=	4.275	4.275	4.275	4.275
g	Moment at bearing level	kN-m	=	1142.28	598.50	855.00	0.00
h	Reaction as push/pull (+/-)	kN	=	29.44	15.43	22.04	0.00
i	For moment at Abt. base, lever arm	m	=	3.440	3.440	3.440	3.440
j	Longitudinal moment at Abt. base due to friction force	kN-m	=	662.16	634.76	661.64	797.12

Type of bearing at support = Rocker-Roller

At abutment side = one side is fixed(Rocker Bearing).

As per IRC 6-2014 Clause-211.5.1 μ= 0.03

Longitudinal forces at Bearing level =	Case-I	F_h - μ(Rg + Rq) =	74.71	-44.52	7.66	-231.72
	Case-II	F_h /2 + μ(Rg + Rq) =	326.09	254.52	292.34	231.72

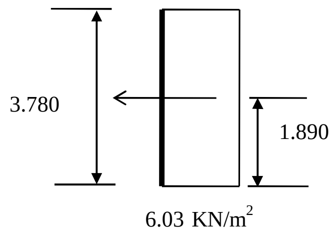
Live load surcharge pressure is to be considered

Equivalent to 1.2 m height of soil = 6.03 KN/m^2 …..(Clause 214.1,IRC:6-2010,page-37)

Considering dirt wall :

Height of Dirt wall = H = 3.780 m
Length of dirt wall = 12.750 m
Total horizontal force in longitudinal direction = 290.442 KN

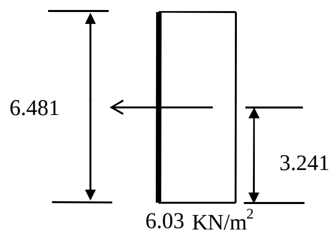
Moment at base of dirt wall due to live load surcharge = 548.936 KN-m



Considering Abutment Shaft

Height of the abutment=Height of abutment wall+Dirt wall= 6.481 m
Length of abutment wall = 12.50 m
Height from foundation slab bottom to deck level = 7.521 m
Total horizontal force in longitudinal direction = 488.214 KN (For abutment)
Total horizontal force in longitudinal direction = 566.557 KN (For foundation slab base)

Moment at base of abutment wall due to live load surcharge = 1582.06 KN-m
Moment at base of foundation slab due to live load surcharge = 2181.670 KN-m



Wind load calculation:

(As per IRC:6-2014, clause 209, page-27)

p_z = 463.7 N/m^2 (From IRC:6-2014, table 5, for 33m/s basic wind speed)
Basic wind speed at bridge location = 50 m/s
Actual hourly mean wind pressure = 1.06 KN/m^2

Wind force = Area x p_z x G x (C_D or C_L)
= Area x pz'
= Area x 4.152 KN/m^2 for super-structure

Where G = 2.00
C_D = 1.95 for super-structure (from table 5, IRC:6-2014, page-33)
1.3 for abutment shaft
C_L = 0.75 (Lift co-efficient)

Wind load as per clause 209, IRC:6-2014

a Wind Force on superstructure:

i. Transverse wind force (F_T) :

Solid area (A1) = Exposed area in Transverse direction = 159.900 m2
F_T for abutment = 332 kN

ii. Longitudinal wind force (F_L) :

F_L = 25% of transverse wind force = 82.98 kN

iii. Vertical wind load (F_V) :

Plan area (A3) for abutment = 261.4 m2
Lift Coefficient (C_L) = 0.75
F_V = 417 kN

b Wind force on live load:

as per clause 209.3.7 of IRC: 6, 2014, bridge shall not be considered to carry any live load if the basic wind velocity exceeds 36m/sec.

c Wind force on Substructure:

Pier Cap

i. Transverse wind force:

Exposed area = 2.07 m2

Transverse wind force = 5.7 kN

ii. Longitudinal Wind Force:

Exposed area = 12.8 m2

Velocity Of Wind			
Position	**Direction**	**vert. comp.**	**Hortz. Comp.**
		kN	**kN**
Super structure	Transverse	**417.35**	**331.92**
	Longitudinal		**82.98**

ACCIDENTAL ACTIONS

Seismic Hazards

Seismic Zone of bridge location = V

Zone factor, Z = 0.36 (Table 7, IRC:6-2014, page-51)

Seismic importance factor of the structure(I)= 1.2 (Table 8, IRC:6-2014, page-55)

Average response acceleration co-efficient (S_a/g) 2.5 (Clause 219.5.1, IRC:6-2014, page-54)

Horizontal seismic co-efficient A_h

= (Z/2)X (I)X(Sa/g)/R = 0.540 /R

Where R is response reduction factor to be considered

Is ductile detailing to be done ? yes

Value of R for sub-structure = 3

Hence horizontal seismic co-efficient (A_h) for sub-structure = 0.180

Horizontal seismic force, F_{eq} = A_h.(Dead Load+Appropriate Live Load)

Seismic force due to dead load

(Inertia loads due to self-mass generated in bridge structure by ground acceleration)

A. ***Seismic on Superstructure:***

Seismic forces in super-structure and SIDL without surfacing is done in previous pier calculation sheets

Dead Load from super-structure and SIDL without surfacing =	=	12255	KN	
C.G. of Deck from girder bottom	=	1.688	m	
Design Horizontal Seismic coefficient A_h	=	0.180		
Seismic force in longitudinal direction F_h	A_h x (Total Dead Load)=	2205.98	KN	
Seismic force in longitudinal direction taken by one support	F_h =	2205.98	KN	………r_1
Acting at RL	=	238.375	m	
Lever arm for moment at bearing level	=	1.688	m	
Longitudinal moment at bearing level	=	3723.69	KN-m	
Vertical pull-push effect due to Horizontal seismic force	=	95.971	KN	
Lever arm for moment at abutment base	=	3.440	m	
Longitudinal moment at abutment base	=	7588.568	KN-m	……..M_z
Horizontal seismic force in transeverse direction = Fh/2	=	1102.99	KN	………..r_2
Acting at RL	=	238.375	m	
Lever arm for moment at abutment base	=	3.440	m	
Transeverse moment at abutment base	=	3794.28	KN-m	……M_x
Vertical component of seismic force	=	735.326	KN	…r_3 (Clause 219.3, IRC:6-2014, page-51)
Combination of force components	…….(Clause 219.4, IRC:6-2014, page-51)			
Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	2757.474	KN	
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	1985.381	KN	
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	1728.017	KN	
Design longitudinal moment at Abutment base= $M_z + 0.3M_x$	=	8726.852	KN-m	
Design transeverse moment at Abutment base = $0.3M_z + M_x$	=	6070.85	KN-m	
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	12532.17	KN-m	
Design transverse moment at bottom of foundation slab=0.3 Mz + Mx	=	8718.03	KN-m	

B. ***Seismic on Sufacing:***

Surfacing or load due to wearing coat taken by one abutment	=	393.421	KN
C.G. of wearing coat from girder bottom	=	3.038	m
Design Horizontal Seismic coefficient A_h	=	0.18	kN
Seismic force in longitudinal direction F_h	A_h x (Total Dead Load) =	70.816	KN
Seismic force in longitudinal direction taken by one support F_h	=	70.816	KN ………r_1
Acting at RL	=	239.96	m
Lever arm for moment at bearing level	=	3.038	m
Longitudinal moment at bearing level	=	215.10	KN-m
Vertical pull-push effect due to Horizontal seismic force	=	5.54	KN
Lever arm for moment at abutment base	=	3.440	m
Longitudinal moment at abutment base	=	243.61	KN-m ……..M_z
Horizontal seismic force in transeverse direction = Fh/2	=	35.408	KN ………..r_2
Acting at RL	=	239.96	m
Lever arm for moment at abutment base	=	3.440	m
Transeverse moment at abutment base	=	121.80	KN-m ……M_x
Vertical component of seismic force	=	23.605	KN ……r_3 (Clause 219.3, IRC:6-2014, page-47)
Combination of force components	…….(Clause 219.4, IRC:6-2010, page-47)		
Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	88.52	KN
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	63.73	KN
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	55.47	KN
Design longitudinal moment = $M_z + 0.3M_x$	=	280.15	KN-m
Design transeverse moment = $0.3M_z + M_x$	=	194.88	KN-m
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	402.30	KN-m
Design transverse moment at bottom of foundation slab= 0.3Mz + Mx	=	279.86	KN-m

C. ***Seismic on Dirt wall :***

CG of the dirt wall from top of foundation slab =			3.590	m
Longitudinal seismic force =	A_h x W_dirt wall	=	86.75	KN ……….r_1
Acting at RL		=	237.044	m RL
Longitudinal moment at Abutment base		=	311.432	KN-m ……..M_z
Transverse seismic =	A_h x W_dirt wall	=	86.75	KN ……….r_2
Acting at RL		=	237.044	m RL
Transeverse moment at Abutment base		=	311.432	KN-m ……..M_x
Vertical component of seismic force	=(2/3)x Horizontal Force Component	(Clause 219.3, IRC:6-2014, page-51)		
		=	57.833	KN ………r_3

Combination of force components …….(Clause 219.4, IRC:6-2014, page-52)

Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	130.125	KN
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	130.125	KN
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	109.883	KN
Design longitudinal moment = $M_z + 0.3M_x$	=	404.862	KN-m
Design transeverse moment = $0.3M_z + M_x$	=	404.862	KN-m
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	574.025	KN-m
Design transverse moment at bottom of foundation slab= 0.3Mz + Mx	=	574.025	KN-m

D. ***Seismic on Abutment Cap:***

CG of the abutment cap from top of foundation slab		=	2.200	m
Longitudinal seismic force =	A_h x W_cap	=	118.77	KN ……….r_1
	Acting at RL	=	235.654	m RL
Longitudinal moment =		=	261.294	KN-m ……..M_z
Transverse seismic =	A_h x A_cap	=	118.77	KN ……….r_2
	Acting at RL	=	235.654	m RL
Transeverse moment		=	261.294	KN-m ……..M_x
Vertical component of seismic force	=(2/3)x Horizontal Force Component			
	(Clause 219.3, IRC:6-2014, page-51)			
		=	79.180	KN ………r_3

Combination of force components …….(Clause 219.4, IRC:6-2014, page-52)

Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	178.155	KN
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	178.155	KN
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	150.442	KN
Design longitudinal moment = $M_z + 0.3M_x$	=	339.682	KN-m
Design transeverse moment = $0.3M_z + M_x$	=	339.682	KN-m
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	571.284	KN-m
Design transverse moment at bottom of foundation slab= 0.3Mz + Mx	=	571.284	KN-m

E. *Seismic on Abutment Shaft:*

CG of the abutment shaft from top of foundation slab		=	0.850	m	
Longitudinal seismic force =	A_h x A_shaft	=	114.75	KN	……….r_1
	Acting at RL	=	234.304	m RL	
Longitudinal moment at Abutment base		=	97.54	KN-m	……..M_z
Transverse seismic =	A_h x A_cap	=	114.75	KN	……….r_2
	Acting at RL	=	234.304	m RL	
Transeverse moment at Abutment base		=	97.54	KN-m	……..M_x

Vertical component of seismic force =(2/3)x Horizontal Force Component (Clause 219.3, IRC:6-2014, page-51)

= 76.500 KN ………r_3

Combination of force components …….(Clause 219.4, IRC:6-2014, page-52)

Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	172.125	KN
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	172.125	KN
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	145.350	KN
Design longitudinal moment = $M_z + 0.3M_x$	=	126.799	KN-m
Design transeverse moment = $0.3M_z + M_x$	=	126.799	KN-m
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	350.561	KN-m
Design transverse moment at bottom of foundation slab= 0.3Mz + Mx	=	350.561	KN-m

F. *Seismic on Return Walls:*

CG of the return walls from top of foundation slab		5.147	m	
Longitudinal seismic force =	A_h x R_walls	81.00	KN	……….r_1
	Acting at RL	238.626	m RL	
Longitudinal moment at Abutment base		418.905	KN-m	……..M_z
Transverse seismic =	A_h x R_walls	81.00	KN	……….r_2
	Acting at RL	238.626	m RL	
Transeverse moment at Abutment base		418.905	KN-m	……..M_x

Vertical component of seismic force =(2/3)x Horizontal Force Component (Clause 219.3, IRC:6-2014, page-51)

= 54.000 KN ………r_3

Combination of force components …….(Clause 219.4, IRC:6-2014, page-52)

Design force in longitudinal direction = $\pm r_1 \pm 0.3r_2 \pm 0.3r_3$	=	121.500	KN
Design force in transeverse direction = $\pm 0.3r_1 \pm r_2 \pm 0.3r_3$	=	121.500	KN
Design force in vertical direction = $\pm 0.3r_1 \pm 0.3r_2 \pm r_3$	=	102.600	KN
Design longitudinal moment = $M_z + 0.3M_x$	=	544.576	KN-m
Design transeverse moment = $0.3M_z + M_x$	=	544.576	KN-m
Design longitudinal moment at bottom of foundation slab= Mz + 0.3Mx	=	702.53	KN-m
Design transverse moment at bottom of foundation slab= 0.3Mz + Mx	=	702.53	KN-m

G. Seismic on carriageway live load …….(Clause 219.5.2, IRC:6-2014, page-55)

(Inertia loads due to mass of vehicular live load)

sl. No.	Live Load Case	20% Reaction (KN)		Ah	Transverse seismic force	Acting RL at (+1.20)	Lever arm at Abutment base	Transverse moment at Abutment base	vertical force
	For Class-A	**Class A**	**70R**						
2	Three lane class A/ one span loaded	**180.07**		**0.180**	**32.41**	**241.20**	**7.75**	**251.07**	**21.61**
	For IRC class 70R Tracked								
5	One lane 70R TR & one lane class A / one span loaded		**131.97**	**0.180**	**23.75**	**241.20**	**7.75**	**184.00**	**15.84**
	For IRC class 70R Wheeled								
7	One lane 70R Wh. & one lane class A/ one span loaded		**179.16**	**0.180**	**32.25**	**241.20**	**7.75**	**249.79**	**21.50**

I. *Increased Earth Pressure due to Seismic*

Calculation of Dynamic Earth Pressure Coefficient …..(Clause 8, IS:1893-1984, page-46)

Horizontal seismic force co-efficient = A_h = 0.18

Horizontal seismic force co-efficient = A_v = 0.12

Calculation of Active Earth Pressure Coefficient (Dynamic)

For $+A_v$:		For $-A_v$:	
$1+A_v$ =	1.120	$1-A_v$ =	0.880

$\lambda = \tan^{-1}\{A_h/(1+A_v)\}$= 9.1302 ^{o}

$\cos^2(\phi-\alpha-\lambda)$ = 0.8731
$\cos(\delta+\alpha+\lambda)$ = 0.8735
$\cos(\alpha-\iota)$ = 1.0000
$\cos\lambda$ = 0.9873
$\cos^2\alpha$ = 1.0000
$\sin(\phi+\delta)$ = 0.7660
$\sin(\phi-\iota-\lambda)$ = 0.3562
C_a = 0.467

$\lambda = \tan^{-1}\{A_h/(1-A_v)\}$ = 11.5601 ^{o}

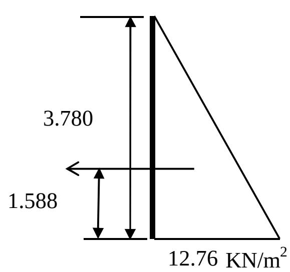
$\cos^2(\phi-\alpha-\lambda)$ = 0.8999
$\cos(\delta+\alpha+\lambda)$ = 0.8521
$\cos(\alpha-\iota)$ = 1.0000
$\cos\lambda$ = 0.9797
$\cos^2\alpha$ = 1.0000
$\sin(\phi+\delta)$ = 0.7660
$\sin(\phi-\iota-\lambda)$ = 0.3163
C_a = 0.404

Maximum of these two = 0.467 to be considered

Considering dirt wall portion

Height of Dirt wall = H = 3.780 m

Length of dirt wall = 12.750 m



Dynamic increment :

Earth pressure at base due to backfill = 12.76 KN/m²

Horizontal force due to backfill soil = 307.48 KN

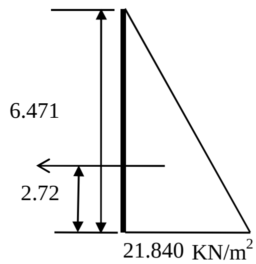
Acting at =3.78 x 0.42= 1.588 m

(Clause-8.1.1.2,IS:1893-1984,Page-47)

Bending moment increment for dynamic condition= 488.16 KNm

Considering Abutment Shaft

Height of the abutment=Height of abutment wall+Dirt wall= 6.471 m
Length of abutment wall = 12.50 m
Height from foundation slab bottom to deck level = 7.971 m



Dynamic increment

Earth pressure at abutment base due to backfill = 21.840 KN/m^2
Horizontal force increment due to backfill soil = 883.29 KN (For abutment shaft)
Acting at a ht. of = 2.72 m (from base of abutment)
Horizontal force increment due to backfill soil = 1340.47 KN (from bottom of foundation slab)
Acting at a ht. of = 3.35 m (from bottom of foundation slab level)
Total moment increment at the base of abutment due to Earth pressure = 2400.63 KN-m
Total moment increment at the base of foundation slab due to Earth pressure = 4487.67 KN-m

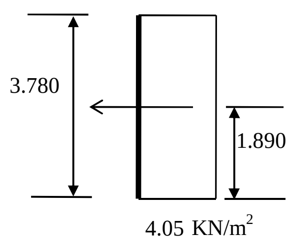
Earthpressure surcharge effect due to live load :

Live load surcharge pressure is to be considered
Equivalent to 1.2 m height of soil = 4.05 KN/m^2 …..(Clause 214.1,IRC:6-2014,page-41)

Considering dirt wall :

Height of Dirt wall = H = 3.780 m
Length of dirt wall = 12.750 m
Total horizontal force in longitudinal direction = 195.190 KN

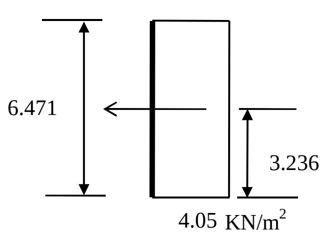
Moment at base of dirt wall due to live load surcharge = 368.909 KN-m



Considering Abutment Shaft

Height of the abutment=Height of abutment wall+Dirt wall= 6.471 m
Length of abutment wall = 12.75 m
Height from foundation slab bottom to deck level = 7.971 m
Total horizontal force in longitudinal direction = 334.146 KN (For abutment)
Total horizontal force in longitudinal direction = 411.603 KN (For foundation slab bnase)

Moment at base of abutment wall due to live load surcharge = 1081.130 KN-m
Moment at base of foundation slab due to live load surcharge = 1640.442 KN-m



Load combination at abutment base:-

SL NO	LOAD DESCRIPTION	V	HL	HT	LA	ML	MT
A	**Permanent Action**						
i	Dead load from super-structure	5425				-271.25	
ii	Self weight of dirt wall	481.95				-612.077	
iii	Self weight of abutment cap	659.81				-287.02	
iv	Self weight of abutment wall	637.50					
v	Self weight of return wall	450.0				-990.00	
vi	Earth Pressure due to back-Fill		1317.97		2.72	3587.00	
B	**Variable gravity treated as Permanent**						
i	Super Imposed Dead Load	703				-35.14	
ii	Surfacing	393.421				-19.67	
D	**Variable Actions**						
	Vehicular Live Load						
i	CLASS A-3 LANE 1SPAN	991.3				-49.57	1833.91
ii	70R TRACKED -1LANE+ CL-A 1L 1 SPAN	725.8				-36.29	2975.85
iii	70R WHEELED-1 LANE + CL-A 1L 1 SPAN	986.3				-49.31	4097.88
iv	IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle-1 LANE 1 SPAN	2299.1				-114.95	689.72
i	**Longitudinal Forces (Braking)**						
i	CLASS A-3 LANE 1SPAN	29.4	326.09		3.440	1121.75	
iii	70R TRACKED -1LANE+ CL-A 1L 1 SPAN	15.4	254.52		3.440	875.56	
v	70R WHEELED-1 LANE + CL-A 1L 1 SPAN	22.0	292.34		3.440	1005.64	
iv	IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle-1 LANE 1 SPAN	0.0	231.72		3.440	797.12	
	Earth Surcharge due to Live Load		488.21		3.24	1582.06	
E	**Seismic Forces**						
	Seismic forces for dead load						
	a) on superstructure	1728.02	2757.47	1985.38		8726.85	6070.85
	b) Surfacing	55.47	88.52	63.73		280.15	194.88
	c) on dirt wall	109.88	130.13	130.13		404.86	404.86
	d) on abutment cap	150.44	178.16	178.16		339.68	339.68
	e) on abutment wall	145.35	172.12	172.12		126.80	126.80
	f) on return wall	102.60	121.50	121.50		544.58	544.58
	vertical push-pull due to seismic at sup.	101.52					
	Seismic forces for Live Load						
i	CLASS A-3 LANE 1SPAN	21.6		32.41	7.75		251.07
iii	70R TRACKED -1LANE+ CL-A 1L 1 SPAN	15.8		23.75	7.75		184.00
v	70R WHEELED-1 LANE + CL-A 1L 1 SPAN	21.5		32.25	7.75		249.79
	Wind Load						
	Wind Load on super-structure	417.35	82.98	331.92	3.27	271.59	1086.371
	Increased Earth Pressure due to seismic		883.29		2.72	2400.63	
	Increased surcharge earth pressure due to seismic		334.146		3.236	1081.13	

Load combination at Bottom of foundation level (For open foundation) :-

SL NO	LOAD DESCRIPTION	V	HL	HT	LA	ML	MT
A	**Permanent Action**						
i	Dead load from super-structure	5425.00				1763.125	
ii	Self weight of dirt wall	481.95				-479.540	
iii	Self weight of abutment cap	659.81				-105.57	
iv	Self weight of abutment wall	637.50				239.06	
v	Self weight of return wall	450.0				-1167.75	
vi	Self weight of foundation slab	3056.0					
vii	Back Fill weight on foundation slab	4304.5				-7317.60	
viii	Earth Pressure due to back-Fill		1998.77		3.35	6699.07	
B	**Variable gravity treated as Permanent**						
i	Super Imposed Dead Load	702.720				228.38	
ii	Surfacing	393.421				127.86	
D	**Variable Actions**						
	Vehicular Live Load						
i	**CLASS A-3 LANE 1SPAN**	900.37				292.62	1665.68
ii	**70R TRACKED -1LANE+ CL-A 1L 1 SPAN**	659.83				214.45	2705.32
iii	**70R WHEELED-1 LANE + CL-A 1L 1 SPAN**	895.78				291.13	3721.96
iv	**IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle-1 LANE 1 SPAN**	2299.06				747.19	689.72
	Longitudinal Forces (Braking)						
i	**CLASS A-3 LANE 1SPAN**	29.44	326.09		4.94	1610.88	
ii	**70R TRACKED -1LANE+ CL-A 1L 1 SPAN**	15.43	254.52		4.94	1257.35	
iii	**70R WHEELED-1 LANE + CL-A 1L 1 SPAN**	22.04	292.34		4.94	1444.15	
iv	**IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle-1 LANE 1 SPAN**	0.00	231.72		4.94	1144.71	
	Earth Surcharge due to Live Load		566.56		3.76	2130.54	
E	**Seismic Forces**						
	Seismic forces for dead load						
	a) on superstructure	2160.02	3446.84	2481.73	1.50	15665.21	10897.54
	b) Surfacing	69.34	110.65	79.67	1.50	502.88	349.83
	c) on dirt wall	137.35	162.66	162.66	1.50	717.53	717.53
	d) on abutment cap	188.05	222.69	222.69	1.50	714.10	714.10
	e) on abutment wall	181.69	215.16	215.16	1.50	438.20	438.20
	f) on return wall	128.25	151.88	151.88	1.50	878.16	878.16
	vertical push-pull due to seismic at sup.	126.89					
	Seismic forces for Live Load						
i	**CLASS A-3 LANE 1SPAN**	27.01		40.52	9.25		374.62
iii	**70R TRACKED -1LANE+ CL-A 1L 1 SPAN**	19.80		29.69	9.25		274.54
v	**70R WHEELED-1 LANE + CL-A 1L 1 SPAN**	26.87		40.31	9.25		372.71
	Wind Load						
	Wind Load on super-structure	417.35	82.98	331.92	4.77	396.06	1584.25
	Increased Earth Pressure due to seismic		1675.592		3.35	5609.58	
	Increased surcharge earth pressure seismic		514.503		3.99	2050.55	

LOAD COMBINATION FOR PIER SHAFT BASE (For Ultimate Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	7654.26	-2160.34	0.00	0.00	0.00
SIDL	702.72	-35.14	0.00	0.00	0.00
Surfacing	393.42	-19.67	0.00	0.00	0.00
Class A(3L/1S) LL1	991.31	-49.57	1833.91	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL2	725.82	-36.29	2975.85	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL3	986.25	-49.31	4097.88	0.00	0.00
Class SV LL4	2299.06	-114.95	689.72	0.00	0.00
BrakingClass A(3L/1S) LL1	29.44	1121.75	0.00	326.09	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL2	15.43	875.56	0.00	254.52	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL3	22.04	1005.64	0.00	292.34	0.00
Friction Class SV LL4	0.00	797.12	0.00	231.72	0.00
Earth Pressure	0.00	3587.00	0.00	1317.97	0.00
LL surcharge on Earth Pr.	0.00	1582.06	0.00	488.21	0.00
Dead Load Seismic	2393.28	10422.92	7681.66	3447.90	2651.02
Seismic Class A(3L/1S) LL1	21.61	0.00	251.07	0.00	32.41
Seismic 70R Tr.1L+CL-A 1L(1S) LL2	15.84	0.00	184.00	0.00	23.75
Seismic 70R Wh.1L+CL-A 1L(1S) LL3	21.50	0.00	249.79	0.00	32.25
Wind load	417.35	271.59	1086.37	82.98	331.92
Increased Earth Pr. due to Sis.	0.00	2400.63	0.00	883.29	0.00
Increased EP surch. due to Sis.	0.00	1081.13	0.00	334.15	0.00

NON-SEISMIC CASE

A DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	1.50
Braking LL	1.15
EP	1.50
EP LL surcharge	1.20

B WIND

DL+SIDL+Surfacing+LL+Br. LL+EP+EP DL+/-WL

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	1.50
Braking LL	1.15
EP	1.50
EP LL surcharge	1.20
WL	1.50

SEISMIC CASE

A DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL+DL S+LL S+EP S+EP LL S

Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	0.2
Braking LL	0.2
EP	1
EP LL surcharge	0.2
DL sis	1.5
LL sis	1.5
EP LL sis	1.5
EP sis	1.5

C DL+SIDL+Surfacing+EP+DL seis+EP seis

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
EP	1.00
DL sis	1.50
EP seis	1.50

			V_u	ML_u	MT_u	HL_u	Ht_u
NON SEISMIC	1	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge	10271.22	6279.47	2750.87	2937.82	0.00
	2	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge	9856.87	6016.27	4463.77	2855.52	0.00
	3	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge	10255.12	6146.33	6146.82	2899.01	0.00
	4	DL+SIDL+Surfacing+LL4+Br. LL4+EP+EP LL Surcharge	12198.99	5808.07	1034.58	2829.30	0.00
	5	DL+SIDL+Surfacing+LL4+EP+EP LL Surcharge	12198.99	4891.38	1034.58	2562.82	0.00
	6	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge+WL	10897.25	6686.86	4380.43	3062.29	497.88
	7	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge+WL	10482.90	6423.66	6093.33	2979.99	497.88
	8	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge+WL	10881.15	6553.72	7776.37	3023.47	497.88
	9	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge-WL	9645.19	5872.08	1121.31	2813.35	-497.88
	10	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge-WL	9230.84	5608.89	2834.22	2731.05	-497.88
	11	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge-WL	9629.09	5738.94	4517.26	2774.54	-497.88
SEISMIC	12	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge+DL S+LL1 S+EP S+EP LL Surcharge	12544.47	22759.71	366.78	8478.84	0.00
	13	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge+DL S+LL2 S+EP S+EP LL Surcharge	12488.57	22713.13	595.17	8464.53	0.00
	14	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge+DL S+LL3 S+EP S+EP LL Surcharge	12541.98	22736.54	819.58	8472.09	0.00
	15	DL+SIDL+Surfacing+EP+DL seis+EP seis	12340.32	20607.16	0.00	7814.76	0.00

LOAD COMBINATION FOR ABUTMENT SHAFT BASE (For Servicibility Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	7654.26	-2160.34	0.00	0.00	0.00
SIDL	702.72	-35.14	0.00	0.00	0.00
Surfacing	393.42	-19.67	0.00	0.00	0.00
Class A(3L/1S) LL1	991.31	-49.57	1833.91	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL2	725.82	-36.29	2975.85	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL3	986.25	-49.31	4097.88	0.00	0.00
Class SV LL4	2299.06	-114.95	689.72	0.00	0.00
BrakingClass A(3L/1S) LL1	29.44	1121.75	0.00	326.09	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL2	15.43	875.56	0.00	254.52	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL3	22.04	1005.64	0.00	292.34	0.00
Friction Class SV LL4	0.00	797.12	0.00	231.72	0.00
Earth Pressure	0.00	3587.00	0.00	1317.97	0.00
LL surcharge on Earth Pr.	0.00	1582.06	0.00	488.21	0.00
Dead Load Seismic	2393.28	10422.92	7681.66	3447.90	2651.02
Seismic Class A(3L/1S) LL1	21.61	0.00	251.07	0.00	32.41
Seismic 70R Tr.1L+CL-A 1L(1S) LL2	15.84	0.00	184.00	0.00	23.75
Seismic 70R Wh.1L+CL-A 1L(1S) LL3	21.50	0.00	249.79	0.00	32.25
Wind load	417.35	271.59	1086.37	82.98	331.92
Increased Earth Pr. due to Sis.	0.00	2400.63	0.00	883.29	0.00
Increased EP surch. due to	0.00	1081.13	0.00	334.15	0.00

NON-SEISMIC CASE

A DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	1.00
Braking LL	1.00
EP	1.00
EP LL surcharge	0.80

B WIND

DL+SIDL+Surfacing+LL+Br. LL+EP+EP DL+/-WL

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	1.00
Braking LL	1.00
EP	1.00
EP LL surcharge	0.80
WL	1.00

			V_u	ML_u	MT_u	HL_u	Ht_u
NON SEISMIC	1	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL	9771.15	3709.67	1833.91	2034.63	0.00
	2	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL	9491.65	3476.76	2975.85	1963.07	0.00
	3	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL	9758.69	3593.82	4097.88	2000.88	0.00
	4	DL+SIDL+Surfacing+LL4+Br. LL4+EP+EP LL	11049.46	3319.66	689.72	1940.27	0.00
	5	DL+SIDL+Surfacing+LL4+EP+EP LL	11049.46	2522.54	689.72	1708.54	0.00
	6	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL+WL	10188.50	3981.26	2920.29	2117.61	331.92
	7	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL+WL	9909.00	3748.36	4062.22	2046.05	331.92
	8	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL+WL	10176.05	3865.41	5184.25	2083.86	331.92
	9	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL-WL	9353.79	3438.08	747.54	1951.65	-331.92
	10	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL-WL	9074.29	3205.17	1889.48	1880.09	-331.92
	11	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL-WL	9341.34	3322.23	3011.51	1917.90	-331.92

ABUTMENT SHAFT DESIGN

Total Ultimate Loads (Loads in KN, moments in KN-m)

Load Case		V_u	ML_u	MT_u
12	Maximum longitudinal Moment case	12544.47	22759.7	366.78
12	Maximum vertical load case	12544.47	22759.7	366.78
10	Minimum vertical load case	9230.84	5608.9	2834.22
15	Unloaded case	12340.32	20607.2	0.00

Section is checked at abutment base

Length of abutment = 12.50 m
Abutment stem thickness at bottom = 1.2 m
Area of section = 15.00 m^2

Grade of concrete : M 30
Grade of steel = Fe 500
E_{cm} of concrete = 31000 N/mm^2 (From table 6.5, IRC:112-2011, page no. 38)
E_s of steel = 200000 N/mm^2 (From clause6.3.5, IRC:112-2011, page no. 32)

Design compressive strength of concrete = $\sigma_c = f_{cd} = \alpha f_{ck}/\gamma_m$ = 13.400 N/mm^2
(From clause6.3.5, IRC:112-2011, page no. 49)
Design peak strength of steel = f_y/γ_s = 434.783 N/mm^2

Checking as wall:
Ref. Cl-7.6.4.1, IRC-112:2011
$0.1.f_{cd}.A_c$= 20100 kN
Maximum Design verical Load= 12544.47 kN
The section is to be designed for pure bending element also.
Checking for pure Bending:
Design moment = **22759.71** KN-m
Width of section = 12.50 m
Depth of section = 1.2 m
Concrete failure strain = ε_{cu1} = 0.0035 (Table 6.5, IRC:112-2011, page-38)
Concrete limiting strain = ε_{c2} = 0.002 (Table 6.5, IRC:112-2011, page-38)
Yield strain of steel = $0.87f_y/E_s$ = 0.00218
Limiting strain of steel = $(0.87f_y/E_s+0.002)$ = 0.00418

Total reinforcement provided = 82425 mm^2
Clear cover = 50 mm
Effective depth "d" = 1137.5 mm
Actual Neutral Axis depth x_u = $(0.87f_yA_{st})/(0.36f_{ck}b)$= 265.59 mm
Actual strain in steel = 0.015 mm
Stress in steel = =0.87*fy= 434.783 Mpa
Balanced Neutral Axis depth $x_{u,max}$ = 518.7 mm
So, **Section is under reinforced, ok, proceed**
CG of compressive force = 110.486 mm from most compressed surface
Moment of resistance, Mu =0.87*fy*Ast*(d-0.416*xu) =
36805.05 kN-m **OK**

Slenderness criteria check:

Clear height of Abutment shaft = 1.700 m (upto abutment cap top)

Effective length, $l_e = 1.3 l_0$ = 2.21 m (Table 11.1, case-4, IRC:112-2011, page-114)

Now thickness of the wall, t = 1.2 m

Ratio of effective length to its thickness, l_e/t = 1.842

As the ratio does not exceed 12, it is short and no secondary effect to be considered

(clause 7.6.4, IRC:112-2011, page-57)

1 Analysis of section longitudinal direction : (Check for load combination case 12)

Provide 25 mm dia. Bar @ 150 mm c/c

Provide	25 mm dia. Bar	84 nos	41212.5 mm^2
+	25 mm dia. Bar	84 nos	41212.5 mm^2
and	25 mm dia. Bar	9 nos	4415.625 mm^2
+	25 mm dia. Bar	9 nos	4415.625 mm^2

Effective cover = 62.5 mm

The section is divided into 9 segments. Depth of each = 133 mm

Area of reinforcement in segment 1 = 82425 mm^2

Area of steel in segment 2 to 8 8831.25 mm^2

Area of steel in segment 9 82425 mm^2

total area of steel = 82425 mm^2

Interaction check

(M_{Edx}/M_{Rdx})^a+(M_{Edy}/M_{Rdy})^a<= 1 (Eq. 8.3, IRC:112-2011,page-75)

					Load Case =	L/C - 12	L/C - 12	L/C - 10	L/C - 15
P_u=	Design Load =				KN	12544.47	12544.47	12340.32	9230.84
M_{Edx} =	Design moment in longitudinal direction =				KN-m	22759.71	22759.7	20607.2	5608.9
M_{Edy} =	Design moment in transeverse direction =				KN-m	366.78	366.78	0.00	2834.22
M_{Rdx} =	Resisting moment in longitudinal direction(from PM curve -ML) =				KN-m	26000.00	26000.00	25300.00	24000.00
M_{Rdy} =	Resisting moment in transeverse direction(from PM curve-MT) =				KN-m	90000.0	90000.0	88000.0	70000.0
N_{Ed} =	Design axial force =				KN	12544.47	12544.47	12340.32	9230.84
N_{Rd} =	Design axial resistance =				KN	235732.46	235732.46	235732.46	235732.46
N_{Ed}/N_{Rd} =						0.1	0.1	0.1	0.1
Type of cross section of abutment =						Rectangular	Rectangular	Rectangular	Rectangular
a =						1.00	1.00	1.00	1.00
(M_{Edx}/M_{Rdx})^a+(M_{Edy}/M_{Rdy})^a =						0.88	0.88	0.81	0.27
Check =						**OK**	**OK**	**OK**	**OK**

ABUTMENT SHAFT DESIGN (SLS)

Load Case				V_u	ML_u	MT_u
6	Maximum longitudinal Moment case			10188.50	3981.3	2920.29
4	Maximum vertical load case			11049.46	3319.7	689.72
10	Minimum vertical load case			9074.29	3205.2	1889.48

Stress level check:

Grade of concrete = M 30
Grade of steel = Fe 500
Width of section considered = 1 m

Section is checked for SLS
Design moment = 318.50 KN-m (for 1m width)
Width of section = 1 m
Depth of section = 1.2 m
"E" value of steel = 200000 Mpa
"E" value of concrete = 31000 Mpa
Modular ratio in tension = 9.3
Concrete failure strain = 0.0035
Maximum allowable stress in concrete = $0.48f_{ck}$ = 14.4 Mpa
(Clause 12.2.1(1), IRC:112-2011, page-120)
Maximum allowable stress in steel = $0.8f_{yk}$ = 400 Mpa
(Clause 12.2.2, IRC:112-2011, page-120)

Total reinforcement provided = 3291 mm^2
Effective depth "d" = 1137.5 mm
Netral axis depth = x = 132.54 mm
CG of compressive force = 55.140 mm from most compressed surface
Moment , Mu =σst*Ast*(d-0.416*xu) = 1549.3333 OK

So, stress in steel = 89.4 Mpa OK, within permissible limit
Total force = 294.3 KN
Stress in concrete = 4.4 Mpa OK, within permissible limit

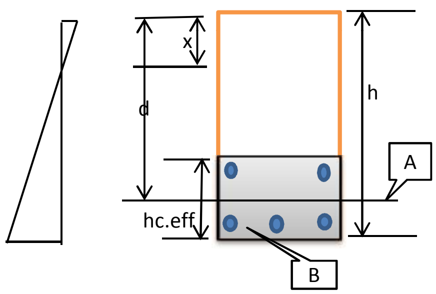
Crack width check:

Crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) Where, $S_{r.max}$ = Maximum crack spacing
™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads
™$_{cm}$ = mean strain in the concrete between cracks.

Now,

(Eq. 12.6, IRC:112-2011, page-125)

Where, σ_{sc} = stress in the tension reinforcement = 89.42 Mpa
$\alpha_e = E_s/E_{cm}$ = 6.4516
$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.5 Mpa
$\rho_{p.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$
Where, $h_{c.eff}$ = lesser of the followings
2.5(h-d);(h-x/3);or h/2



Where, A = level of steel centroid
B = Effective tension area, $A_{c.eff}$
™$_1$,™$_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 156.25 mm
$A_{c.eff}$ = 156250 mm^2
Now. $\rho_{p,eff} = A_s/A_{c.eff}$ = 0.0210602
k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+ϕ/2)), the maximum crack spacing,

Where, ϕ = diameter of bar = 25 mm c = clear cover = 50 mm
k1 = co-efficient taking acount of bond properties of reinforcement = 0.8
k2 = co- efficient taking account of distribution of strain = 0.5
So,$S_{r.max}$ = 371.802 mm
And, ™$_{sm}$-™$_{cm}$ = 0.00011
Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.0002683
So, governing value of ™$_{sm}$ - ™$_{cm}$ = 0.000268273

So, crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.100 mm
Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)
crack width within permissible limit

p

D. LOAD COMBINATION FOR ABUTMENT FOUNDATION BASE (Ultimate Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	10710.26	-7068.27	0.00	0.00	0.00
Backfill weight	4304.47	0.00	0.00	0.00	0.00
SIDL	702.72	228.38	0.00	0.00	0.00
Surfacing	393.4208	127.86	0.00	0.00	0.00
Class A(3L/1S) LL1	900.37	292.62	1665.68	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL2	659.83	214.45	2705.32	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL3	895.78	291.13	3721.96	0.00	0.00
Class SV LL4	2299.06	747.19	689.72	0.00	0.00
BrakingClass A(3L/1S) LL1	29.44	1610.88	0.00	326.09	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL2	15.43	1257.35	0.00	254.52	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL3	22.04	1444.15	0.00	292.34	0.00
Friction Class SV LL4	0.00	1144.71	0.00	231.72	0.00
Earth Pressure	0.00	6699.07	0.00	1998.77	0.00
LL surcharge on Earth Pr.	0.00	2130.54	0.00	566.56	0.00
Dead Load Seismic	2991.60	18916.09	13995.36	4309.87	3313.78
Seismic Class A(3L/1S) LL1	27.01	0.00	374.62	0.00	40.52
Seismic 70R Tr.1L+CL-A 1L(1S) LL2	19.80	0.00	274.54	0.00	29.69
Seismic 70R Wh.1L+CL-A 1L(1S) LL3	26.87	0.00	372.71	0.00	40.31
Wind load	417.35	396.06	1584.25	82.98	331.92
Increased Earth Pr. due to Sis.	0.00	5609.58	0.00	1675.59	0.00
Increased EP surch. due to Sis.	0.00	2050.55	0.00	514.50	0.00

NON-SEISMIC CASE

A DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL

Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.5
Braking LL	1.15
EP	1.5
EP LL surcharge	1.2
Thermal loads	0.90
Bakfill weight	1.50

B DL+SIDL+Surfacing+LL+Br. LL+EP+EP DL+/-WL

Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.5
Braking LL	1.15
EP	1.5
EP LL surcharge	1.2
WL	1.5

SEISMIC CASE

A DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL+DL S+LL S+EP S+EP LL S

Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	0
Braking LL	0.2
EP	1
EP LL surcharge	0.2
DL sis	1
LL sis	1
EP LL sis	1
EP sis	1
Backfill weight	1

C DL+SIDL+Surfacing+EP+DL seis+EP seis

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
EP	1.00
DL sis	1.00
EP seis	1.00

	COMB		V	ML	MT	HL	HT
NON SEISMIC	1	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge	23937.13	5886.60	2498.52	4053.02	0.00
	2	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge	23560.21	5362.78	4057.98	3970.72	0.00
	3	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge	23921.73	5692.62	5582.94	4014.21	0.00
	4	DL+SIDL+Surfacing+LL4+EP+EP LL Surcharge	26001.30	4715.95	1034.58	3678.02	0.00
	5	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge+WL	24563.16	6480.70	4874.90	4177.49	497.88
	6	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge+WL	24186.24	5956.88	6434.35	4095.19	497.88
	7	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge+WL	24547.76	6286.72	7959.31	4138.68	497.88
	8	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge-WL	23311.10	5292.51	122.15	3928.55	-497.88
	9	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge-WL	22934.18	4768.69	1681.60	3846.25	-497.88
	10	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge-WL	23295.70	5098.53	3206.56	3889.74	-497.88
SEISMIC	11	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge+DL S+LL1 S+EP S+EP LL Surcharge	19123.60	26667.20	0.00	8546.8309	0.00
	12	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge+DL S+LL2 S+EP S+EP LL Surcharge	19119.18	26737.90	0.00	8561.14	0.00
	13	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge+DL S+LL3 S+EP S+EP LL Surcharge	19124.94	26701	0.00	8553.58	0.00
	14	DL+SIDL+Surfacing+EP+DL seis+EP seis	19102.47	24512.71	0.00	7984.23	0.00

H. LOAD COMBINATION FOR ABUTMENT FOUNDATION BASE (Servicebility Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	10710.26	-7068.27	0.00	0.00	0.00
Backfill weight	4304.47	0.00	0.00	0.00	0.00
SIDL	702.72	228.38	0.00	0.00	0.00
Surfacing	393.42	127.86	0.00	0.00	0.00
Class A(3L/1S) LL1	900.37	292.62	1665.68	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL2	659.83	214.45	2705.32	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL3	895.78	291.13	3721.96	0.00	0.00
Class SV LL4	2299.06	747.19	689.72	0.00	0.00
BrakingClass A(3L/1S) LL1	29.44	1610.88	0.00	326.09	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL2	15.43	1257.35	0.00	254.52	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL3	22.04	1444.15	0.00	292.34	0.00
Friction Class SV LL4	0.00	1144.71	0.00	231.72	0.00
Earth Pressure	0.00	6699.07	0.00	1998.77	0.00
LL surcharge on Earth Pr.	0.00	2130.54	0.00	566.56	0.00
Dead Load Seismic	2991.60	18916.09	13995.36	4309.87	3313.78
Seismic Class A(3L/1S) LL1	27.01	0.00	374.62	0.00	40.52
Seismic 70R Tr.1L+CL-A 1L(1S) LL2	19.80	0.00	274.54	0.00	29.69
Seismic 70R Wh.1L+CL-A 1L(1S) LL3	26.87	0.00	372.71	0.00	40.31
Wind load	417.35	396.06	1584.25	82.98	331.92
Increased Earth Pr. due to Sis.	0.00	5609.58	0.00	1675.59	0.00
Increased EP surch. due to Sis.	0.00	2050.55	0.00	514.50	0.00

NON-SEISMIC CASE

A

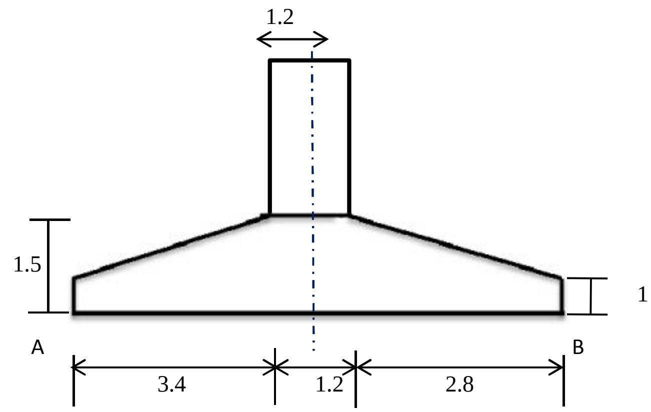
DL+SIDL+Surfacing+LL+Br. LL+EP+EP LL	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	1
Braking LL	0.75
EP	1
EP LL surcharge	1
Thermal loads	0.90
Bakfill weight	1.00

(II)

DL+SIDL+Surfacing+LL+Br. LL+EP+EP DL+/-WL	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	1
Braking LL	0.75
EP	1
EP LL surcharge	1
WL	1
Thermal loads	0.90

	COMB		V	ML	MT	HL	HT
NON SEISMIC	1	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge	17033.32	3618.36	1665.68	2809.89	0.00
	2	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge	16782.28	3275.04	2705.32	2756.22	0.00
	3	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge	17023.18	3491.82	3721.96	2784.58	0.00
	4	DL+SIDL+Surfacing+LL4+EP+EP LL Surcharge	14105.46	3723.30	689.72	2739.12	0.00
	5	DL+SIDL+Surfacing+LL4+EP+EP LL Surcharge	18409.93	2864.78	689.72	2565.33	0.00
	5	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge+WL	17450.68	4014.42	3249.93	2892.87	331.92
	6	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge+WL	17199.63	3671.10	4289.57	2839.20	331.92
	7	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge+WL	17440.53	3887.88	5306.21	2867.56	331.92
	8	DL+SIDL+Surfacing+LL1+Br. LL1+EP+EP LL Surcharge-WL	16615.97	3222.30	81.43	2726.91	-331.92
	9	DL+SIDL+Surfacing+LL2+Br. LL2+EP+EP LL Surcharge-WL	16364.92	2878.98	1121.07	2673.24	-331.92
	10	DL+SIDL+Surfacing+LL3+Br. LL3+EP+EP LL Surcharge-WL	16605.83	3095.76	2137.71	2701.60	-331.92

CHECK FOR STABILITY



Distance of CL of abutment from toe = 3.4

OVERTURNING CHECK
Non-Seismic

Loads	V	ML	Overturning moment	LA	Restoring moment
Dead Load	10710.26	-7068.27	0.00	3.4	36414.893
Earth weight	4304.47	0.00	0.00	5.7	24535.485
SIDL	702.72	228.38	0.00	3.4	2389.248
Surfacing	393.42	127.86	0.00	3.4	1337.631
Class A(3L/1S) LL1	900.37	292.62	0.00	3.4	3061.251
Braking Class A(3L/1S) LL1	29.44	1610.88	1610.88	3.4	100.097
Earth pressure	0.00	6699.07	6699.07	0	0.000
Earth pressure LL Surcharge	0.00	2130.54	2130.54	0	0.000
		Total	**10440.49**		**67838.60**

FOS against overturning moment= 67838.604 / 10440.49 = 6.50 **OK**

Seismic case [Combination III , as per IRC:78-2014, Cl. 706.1]

Loads	V	ML	Overturning moment	LA	Restoring moment
Dead Load	10710.26	-7068.27	0.00	3.4	36414.89
Earth weight	4304.47	0.00	0.00	5.7	24535.48
SIDL	702.72	228.38	0.00	3.4	2389.25
Surfacing	393.42	127.86	0.00	3.4	1337.63
Class A(3L/1S) LL1	180.07	292.62	292.62	3.4	612.25
Braking Class A(3L/1S) LL1	5.89	322.18	322.18	3.4	20.02
Earth Pressure	0.00	6699.07	6699.07	0	0.00
LL Surcharge earth pressure	0.00	2130.54	2130.54	0	0.00
Dead Load Seismic	2991.60	18916.09	18916.09	3.4	10171.44
Increased earth due to seismic	0.00	7660.13	7660.13	0	0.00
		Total	**36020.63**		**75480.965**

FOS against overturning moment= 75480.965 / 36020.625 = 2.10 **OK**

SLIDING CHECK
Considering frictional co-efficient; m = 0.70
Non-Seismic

Loads	V	HL	Sliding force	Restoring force
Dead Load	10710.26	0.00	0.000	7497.184
Earth weight	4304.47	0.00	0.000	3013.130
SIDL	702.72	0.00	0.000	491.904
Surfacing	393.42	0.00	0.000	275.395
Class A(3L/1S) LL1	900.37	0.00	0.000	630.258
Braking Class A(3L/1S) LL1	29.44	326.09	326.089	20.608
Earth pressure	0.00	1998.77	1998.769	0.000
Earth pressure LL Surcharge	0.00	566.56	566.557	0.000
		Total	**2891.415**	**11928.478**

FOS against sliding= 11928.478 / 2891.415 = 4.13 **OK**

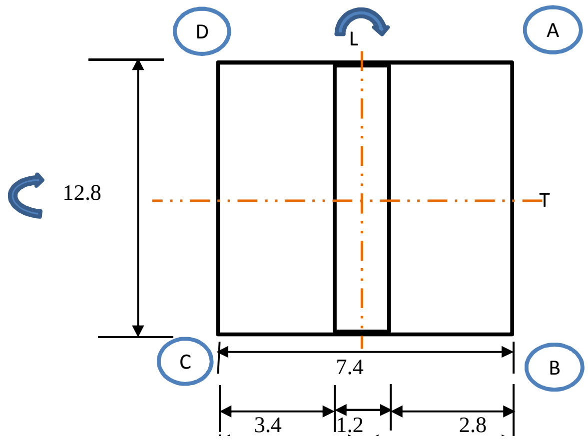
Seismic

Loads	V	HL	Sliding force	Restoring force
Dead Load	10710.26	0.00	0.00	7497.184
Earth weight	4304.47	0.00	0.00	3013.130
SIDL	702.72	0.00	0.00	491.904
Surfacing	393.42	0.00	0.00	275.395
Class A(3L/1S) LL1	180.07	0.00	0.00	126.052
Braking Class A(3L/1S) LL1	5.89	65.22	65.22	4.122
Earth pressure	0.00	1998.77	1998.77	0.000
Earth pressure LL Surcharge	0.00	566.56	566.56	0.000
Dead Load Seismic	2991.60	4309.87	4309.87	2094.120
Increased earth due to seismic	0.00	2190.10	2190.10	0.000
		Total	**9130.512**	**13501.905**

FOS against sliding= 13501.905 / 9130.512 = 1.48 **OK**

CHECK FOR BASE PRESSURE

Length of foundation =	12.8	m
Width along heel side =	3.4	m
Width along toe side =	2.8	m
Thickness of foundation at abutment face =	1.5	m
Thickness of foundation at end =	1	m
Abutment shaft thickness =	1.2	m
Total width =	7.4	m
Area of foundation base =	94.72	m^2
Section modulus longitudinal direction; Z_L =	116.821	m^3
Section modulus transverse direction; Z_T =	202.069	m^3



Foundation CG from B =	3.675	m
Load center line from B =	3.35	m
Unfactored load only due to foundation wt. =	3056.00	KN

Calculation with unfactored load:

	Load Case	V	Moment for V, wrt point B (M_R)	Overturning Moment ML (M_O)	Net Moment (M_R-M_O)	Resultant reaction from B; *x* = (M_R-M_O)/V	Eccentricity (*e*) = b/2-x	Status wrt tension development	Remarks
		KN	KN-m	KN-m	KN-m	m	m		
MLmax	1	17458.037	69593.129	4417.14	65175.985	3.73	-0.0333	***e<b/6, OK***	Considering Wind
Vmax	2	18827.285	74180.112	4405.54	69774.568	3.71	-0.0060	***e<b/6, OK***	
MLmax	1	20909.098	81154.186	29074.50	52079.685	2.49	1.2092	***e<b/6, OK***	Considering Seismic
Vmax	2	20962.091	81331.711	29074.50	52257.210	2.49	1.2071	***e<b/6, OK***	
MLmax	3	17040.683	68194.994	4021.08	64173.911	3.77	-0.0659	***e<b/6, OK***	Normal Case
Vmax	4	18409.931	72781.976	5154.19	67627.790	3.67	0.0266	***e<b/6, OK***	
MLmax	5	16110.874	65080.136	2117.582	62962.554	3.91	-0.2081	***e<b/6, OK***	Unloaded Non seismic

Base pressure calculation

	Load Case	V	Net ML	MT	Base pressure (Corner A)	Base pressure (Corner B)	Base pressure (Corner C)	Base pressure (Corner D)	Remarks
		KN	KN-m	KN-m	KN/m^2	KN/m^2	KN/m^2	KN/m^2	
MLmax	1	17458.04	65175.985	3249.93	195.420	163.253	173.204	205.371	Considering Wind
Vmax	2	18827.28	69774.568	2273.97	209.049	186.542	188.487	210.994	
MLmax	3	17040.68	64173.911	1665.68	178.533	162.046	181.279	197.765	Normal Case
Vmax	4	18409.93	67627.790	689.72	201.960	195.134	186.763	193.589	
MLmax	5	16110.87	62962.554	0.000	141.393	141.393	198.786	198.786	Unloaded (Non-seismic)

Maximum base pressure = 210.994 KN/m^2

Allowable base pressure = 230 KN/m^2 **OK**

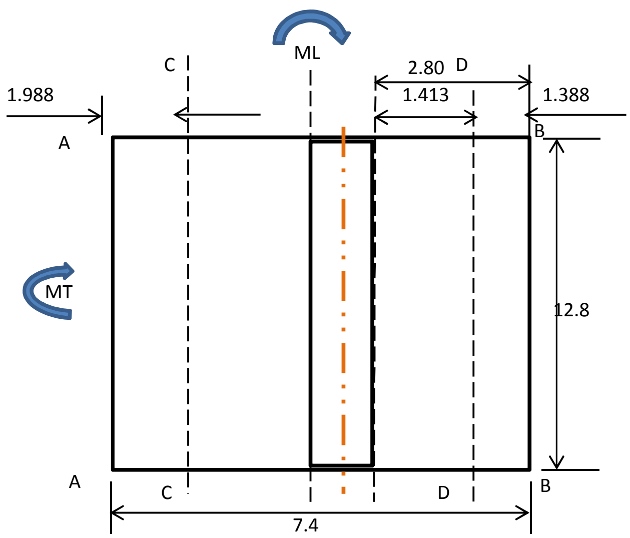
Minimum base pressure = 141.393 KN/m^2 **OK, No tension developed**

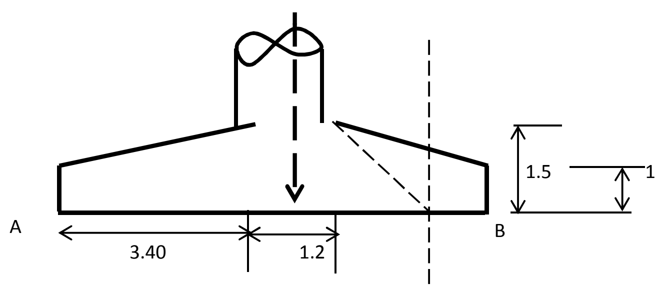
G. DESIGN OF FOUNDATION BY BENDING ANALOGY (ULS)

COMB	LOAD CASE	V_u	ML_u	MT_u	HL_u	HT_u
12	MLmax	19119.184	26737.901	0.000	8561.144	0.000
4	Vmax	26001.304	4715.951	1034.576	3678.021	0.000
14	Vmin	19102.474	24512.712	0.000	7984.234	0.000

Area of foundation base = 94.720 m^2

Height of earth above heel slab = 7.470 m





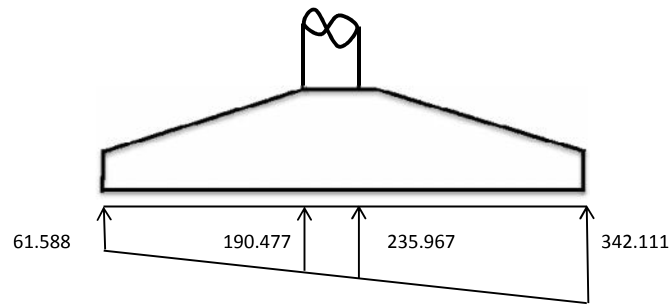
Foundation center line from B = 3.7 m

Load center line from B = 3.4 m

Factored load only due to backfill weight = 6456.71 KN

Factored load only due to foundation wt. = 4125.60 KN

COMB	V	Resistive Moment WRT point B (M_R)	Overturning Moment ML (M_O)	Net Moment (M_R-M_O)	Resultant reaction from B; x = (M_R-M_O)/V	Eccentricity (e) = b/2-x	Base pressure (Toe side)	Base pressure (Heel side)
	KN	KN-m	KN-m	KN-m	m	m	KN/m^2	KN/m^2
12	19119.18	81093.331	26737.90	54355.430	2.843	0.857	342.111	61.588
4	26001.30	104492.540	4715.951	99776.589	3.837	-0.137	243.932	305.082
14	19102.47	81036.517	24512.712	56523.806	2.959	0.741	322.844	80.502



DESIGN OF TOE SLAB:

Moment calculation at abutment face;

Load Case	Total Load (KN/m)	Lever arm (m)	Moment (KN-m)		
12	809.310	1.486	1202.381		
Wt. of slab	-87.500	0.925	-80.905		
		Total	**1121.476**		

<u>Design of foundation in flexure</u> (toe side)

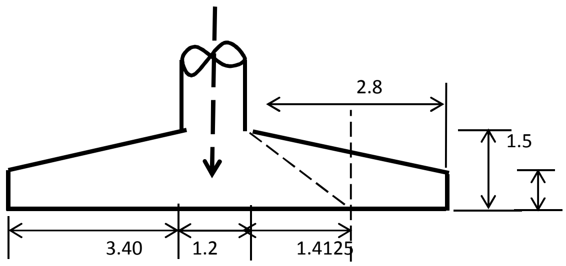
Grade of concrete = M 30
Grade of steel = Fe 500
Width of section considered = 1 m

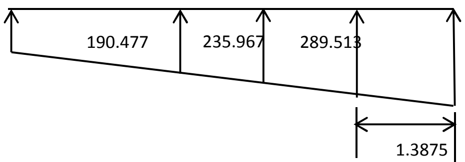
Section is checked for ULS
Design moment = 1121.48 KN-m (for 1m width)
Width of section = 1 m
Depth of section = (at face) 1.5 m
Depth of section = (at edge) 1 m
"E" value of steel = 200000 Mpa
"E" value of concrete = 31000 Mpa
Design compressive strength of concrete =
$f_{cd}=\alpha f_{ck}/\gamma_m$ = 13.40 Mpa Where, α = 0.67
γ_m = 1.5
Design peak strength of steel = f_y/γ_s = 434.783 Mpa Where, γ_s = 1.15
Concrete failure strain = ™$_{cu1}$ = 0.0035 (Table 6.5, IRC:112-2011, page-38)
Concrete limiting strain = ™$_{c2}$ = 0.002 (Table 6.5, IRC:112-2011, page-38)
Yield strain of steel = $0.87f_y/E_s$ = 0.00218
Limiting strain of steel = $(0.87f_y/E_s+0.002)$ = 0.00418

Reinforcement provided: **25** mm dia.
125 mm c/c distance
Total reinforcement provided = 4416 mm^2
Clear cover = 75 mm
Effective depth "d" = 1412.5 mm
Actual Neutral Axis depth x_u = $(0.87f_yA_{st})/(0.36f_{ck}b)$= 177.85 mm
Actual strain in steel = 0.028 mm
Stress in steel = 434.783 Mpa
Balanced Neutral Axis depth $x_{u,max}$ = 644.1 mm
So, Section is under reinforced, ok
CG of compressive force = 73.986 mm from most compressed surface
Moment of resistance, Mu = (Stress in steel)x(area of steel)x(d-CG of compressive force) =
or, Mu =σst*Ast*(d-0.416*xu) = 2569.728 **OK**

CHECK FOR SHEAR IN FOUNDATION TOE SIDE (Clause 10.3.2, IRC:112-2011, page-88)

Shear force should be checked at a distance d from abutment face





Moment calculation at 'd' distance fron abutment face;

Load Case	Total Load (KN/m)	Lever arm (m)	Moment (KN-m)	Shear force at "d" distance from face (KN)
12	438.189	0.713	312.432	438.189
Wt. of slab	-38.989	0.925	-36.050	-38.989
		Total	**276.382**	**399.200**

Total depth at a distance 'd' from abutment face = 1248 mm
Effective depth at a distance 'd' from abutment face = 1160 mm

Design Shear Force = 399.200 KN

The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

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

Where, $K = 1+\sqrt{(200/d)}<=2.0$
So, K = 1.376
$\rho_1 = A_{sl}/b_w.d$
Where A_{sl} = Area of steel provided = 4416 mm^2
b_w = Width of section = 1000 mm
d = 1160 mm
ρ_1 = 0.0038
$\sigma_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0
A_c = Cross Sectional area of concrete
σ_{cp} = 0
So,$V_{Rd.c}$ = 397.55 KN

Now, $V_{Rd.c}$ minimum = $(v_{min}+0.15\sigma_{cp})b_w.d$
where v_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.274
So, $V_{Rd.c}$ minimum = 318.013 KN
So, governing shear resistance = 397.55 KN **Shear Reinforcement Required**

Calculation of shear reinforcement:

α_{cw}=	***1 for σ_{cp}=0, Ref: Eq-10.9,***	1.00
b(mm)=		1000
z(mm)=	0.9*d for RCC	900.00
v_1=	for f_{ck}<80MPa	0.60
f_{cd}= Design value of concrete	$0.67*f_{ck}/\gamma_m$	13.40
Value of θ° =		45.0
tanθ=		1.00
cotθ=		1.00
$V_{rd.min}$=		3618.00

SHEAR REINFORCEMENT DETAILS

Stirrup Dia (mm),ϕ=			12
No. of Leg,			2
Spacing of the stirrup (mm), S=			150
$A_{sw,}$ Provided (mm^2)=			226.19
$A_{sw,}$ Required (mm^2)=	$VRd.s=A_{sw}/S*z*fywd*cot\theta$		191.28
f_{ywd} (Mpa) =	$0.8*f_{yk}/\gamma_m$, γ_m =1.15		347.83
Check	**Ref.: Eqn-10.7, IRC-**		**Ok**
$A_{st,}$ Provided (mm^2)=			1507.9645
Reinforcement ratio for shear	$A_{sw}/(b_w*d)$		0.00130
Min. Permissible Reinforcement ratio for shear	$0.072*f_{ck}^{0.5}/f_{yk}$		0.00079
Check	**Ref.: cl-10.3.3.5, IRC-**		**Ok**

DESIGN OF HEEL SLAB:

Moment calculation at abutment face;

Load Case	Total Load (KN/m)	Lever arm (m)	Moment (KN-m)	Shear force at face (KN)
12	428.510	1.410	604.304	428.510
Wt. of slab	-106.250	0.925	-98.242	-106.250
Earth Wt. above slab	-507.960	1.700	-863.532	-507.960
		Total	**-357.470**	**-185.700**

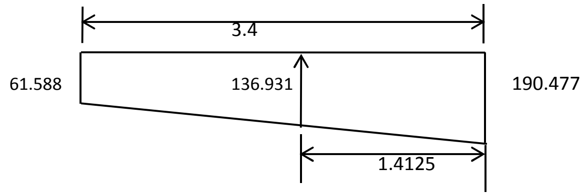
<u>Design of foundation in flexure</u> (heel side)

Section is checked for ULS			
Design moment =		-357.47	KN-m (for 1m width)
		357.470	KN-m Moment downword
Reinforcement provided:		**25**	mm dia.
		125	mm c/c distance
Total reinforcement provided =		3925	mm^2
Clear cover =		75	mm
Effective depth "d" =		1412.5	mm
Actual Neutral Axis depth x_u =	$(0.87f_yA_{st})/(0.36f_{ck}b)$=	158.09	mm
Actual strain in steel =		0.031	mm
Stress in steel =		434.783	Mpa
Balanced Neutral Axis depth $x_{u,max}$ =		644.1	mm
So, Section is under reinforced, ok			
CG of compressive force =		65.766	mm from most compressed surface

Moment of resistance, Mu = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

or, Mu =σst*Ast*(d-0.416*xu) = 2298.232 **OK**

CHECK FOR SHEAR IN FOUNDATION HEEL SIDE (Clause 10.3.2, IRC:112-2011, page-88)



Moment calculation at "d" distance from abutment face;

Load Case	Total Load (KN/m)	Lever arm (m)		Shear force at "d" distance from face (KN)
12	197.278	0.868		197.278
Wt. of slab	-56.942	0.663		-56.942
Earth Wt. above slab	-296.933	0.994		-296.933
		Total		**-156.596**

Total depth at a distance 'd' from abutment face = 1292 mm
Effective depth at a distance 'd' from abutment face = 1205 mm

Design Shear Force = -156.596 KN
156.596 KN Downword direction

The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

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

Where, $K = 1+\sqrt{(200/d)}<=2.0$

So, K = 1.407

$\rho_1 = A_{sl}/b_w.d$

Where A_{sl} = Area of steel provided = 3925 mm^2

b_w = Width of section = 1000 mm

d = 1204.5 mm

ρ_1 = 0.0033

$\sigma_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0

A_c = Cross Sectional area of concrete

σ_{cp} = 0

So,$V_{Rd.c}$ = 401.05 KN

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

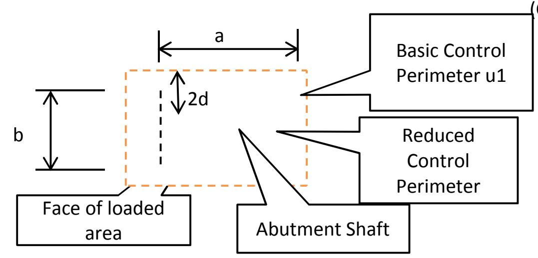
where v_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.2835228

So, $V_{Rd.c}$ minimum = 341.503 KN

So, governing shear resistance = 401.05 KN **OK, Safe in shear**

CHECK FOR PUNCHING SHEAR FOR ABUTMENT SHAFT

(Clause 10.4, IRC:112-2011, page-98)



Punching shear is to be checked at distance "2d" from abutment shaft face

Now from abutment shaft, Maximum Vertical design force, V_{Ed} = 12544.47 KN

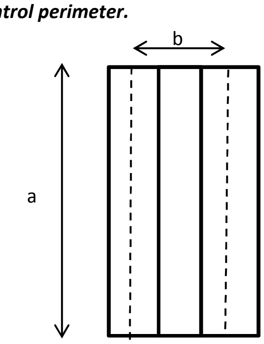
Corresponding longitudinal design moment, ML_u = 22759.71 KN-m

Longitudinal eccentricity, el = 1.814 m

Corresponding transeverse design moment, MT_u = 366.78 KN-m

Transeverse eccentricity, et = 0.029 m

As the basic control perimeter is exceeding the foundation dimension, we will check the ***punching shear at reduced control perimeter.***



a = 12.8 m

b = 6.85 m   OK, Shear check at basic control perimeter

Now, $\beta = 1+1.8\sqrt{((el/a)^2+(et/b)^2)}$ (β = Factors accounting for effects of bending moment and axial load acting on loaded area)

= 1.255

Now, reduced control perimeter, u = 48.2 m

So, total load for punching shear = Load from abutment shaft+ Wt. within reduced control perimeter

V_{Ed} = 15284.4723 KN

Shear stress, $\tau_{Ed} = \beta V_{Ed}/ud$ = 0.343 Mpa

Punching shear check at face of abutment shaft

a = 12.80 m

b = 1.2 m

Now, $\beta = 1+1.8\sqrt{((el/a)^2+(et/b)^2)}$ (β = Factors accounting for effects of bending moment and axial load acting on loaded area)

= 1.259

Now, reduced control perimeter, u = 28.000 m

So, total load for punching shear = Load from abutment shaft

V_{Ed} = 12544.47 KN

Shear stress, $\tau_{Ed} = \beta V_{Ed}/ud$ = 0.486 Mpa

Calculation for punching shear resistance at reduced control perimeter

Maximum punching shear resistance, $v_{Rd.max} = (1/2).v.f_{cd}$

where, v = 0.6[1-fck/310]

= 0.542 (Eq. 10.6, IRC:112-2011. page-90)

So, $v_{Rd.max}$ = 3.631 Mpa

Again, Minimum punching shear resistance, v_{min} .2d/a=

where v_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.274

a = distance from the face of column to control perimeter

= 2.32 m

Minimum punching shear resistance = 0.274 Mpa

Now punching shear resistance, $v_{Rd} = 0.12K(80\rho f_{ck})^{1/3}.2d/a$

where, $\rho l = \sqrt{(\rho_{ll}.\rho_{lt})}$ = 0.235

So, v_{Rd} = 1.364 Mpa

So, governing v_{Rd} = 1.364 Mpa **OK**

Punching shear resistance at face

Punching shear resistance at face, $v_{Rd} = 0.18/\gamma_c K(80\rho f_{ck})^{1/3}+(0.1\sigma_{cp})>=v_{min}+0.1\sigma_{cp}$

Here, $\sigma_{cp} = 0$

So, v_{Rd} = 1.364

v_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.274

So, governing shear resistance, v_{Rd} = 1.364 Mpa **OK**

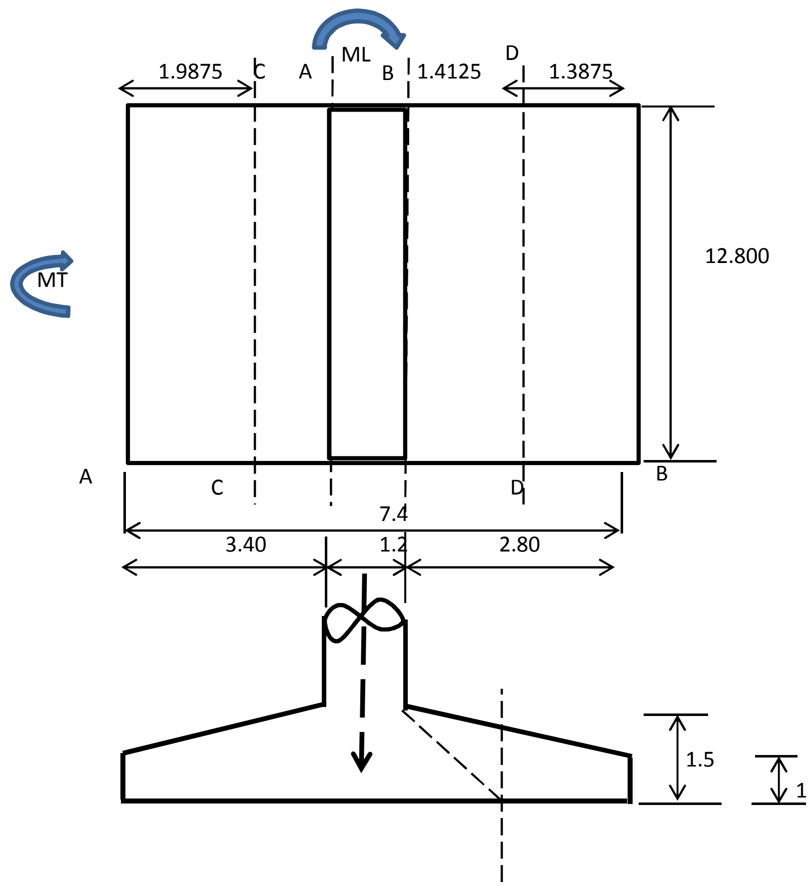
I. DESIGN OF FOUNDATION (SLS)

COMB	LOAD CASE	V_u (KN)	ML_u (KN-m)	MT_u (KN-m)	HL_u (KN)	HT_u(KN)
12	MLmax	17450.676	4014.425	3249.931	2892.872	331.919
4	Vmax	18409.931	2864.776	689.717	2565.325	0.000
14	Vmin	14105.460	3723.305	689.717	2739.117	0.000

Area of foundation base = 94.72 m^2

Section modulus longitudinal direction; Z_L = 116.821 m^3

Section modulus longitudinal direction; Z_T = 202.069 m^3



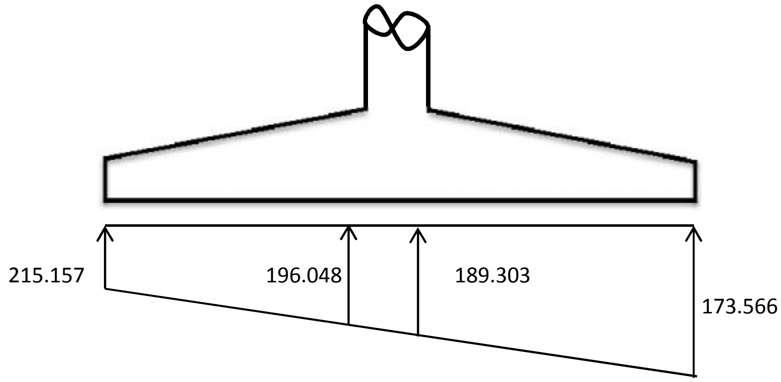
Foundation center line from B = 3.7 m

Load center line from B = 3.4 m

Load center line from base center = 0.3 m

Factored load only due to foundation wt. = 3056.00 KN

COMB	V	Resistive Moment WRT point B (M_R)	Overturning Moment ML (M_O)	Net Moment (M_R-M_O)	Resultant reaction from B; x = (M_R-M_O)/V	Eccentricity (e) = b/2-x	Base pressure (Toe side)	Base pressure (Heel side)
	KN	KN-m	KN-m	KN-m	m	m	KN/m^2	KN/m^2
12	17450.68	70149.383	4014.42	66134.959	3.790	-0.0898	170.817	197.652
4	18409.93	73410.849	2864.78	70546.073	3.832	-0.1320	173.566	215.157
14	14105.46	58775.647	3723.30	55052.343	3.903	-0.2029	124.417	173.418



CHECK FOR TOE SLAB:

Moment calculation at abutment face (considering 1m width);

Load Case	Total Load (KN/m)	Lever arm (m)	Moment (KN-m)	Shear force at face (KN)	Shear force at "d" distance from face (KN)
4	508.018	1.380	700.943	508.018	251.741
Wt. of slab	-87.500	1.400	-122.500	-87.500	-43.359
		Total	**578.443**	**420.518**	**208.381**

Stress level check:

Grade of concrete = M 30

Grade of steel = Fe 500

Width of section considered = 1 m

Section is checked for SLS

Design moment = 578.44 KN-m (for 1m width)

Width of section = 1 m

Depth of section = (at face) 1.5 m

Depth of section = (at edge) 1 m

"E" value of steel = 200000 Mpa

"E" value of concrete = 31000 Mpa

Modular ratio in tension = 9.333

Concrete failure strain = 0.0035

Maximum allowable stress in concrete = $0.48f_{ck}$ = 14.4 Mpa

(Clause 12.2.1(1), IRC:112-2011, page-120)

Maximum allowable stress in steel = $0.8f_{yk}$ = 400 Mpa

(Clause 12.2.2, IRC:112-2011, page-120)

Total reinforcement provided = 4416 mm^2

Effective depth "d" = 1412.5 mm

Netral axis depth = x = 302.48

CG of compressive force = 100.826 mm from most compressed surface

Now moment, M = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

So, stress in steel =	99.9 Mpa	OK, within permissible limit
Total force =	441.0 KN	
Stress in concrete =	2.9 Mpa	OK, within permissible limit

Crack width check:

Crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$)   Where, $S_{r.max}$ = Maximum crack spacing

™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

™$_{cm}$ = mean strain in the concrete between cracks.

Now, $$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct.eff}}{\rho_{peff}}(1 + \alpha_e \rho_{p.eff})}{E_s} => 0.6\frac{\sigma_{sc}}{E_s}$$

(Eq. 12.6, IRC:112-2011, page-125)

Where, σ_{sc} = stress in the tension reinforcement = 99.87 Mpa

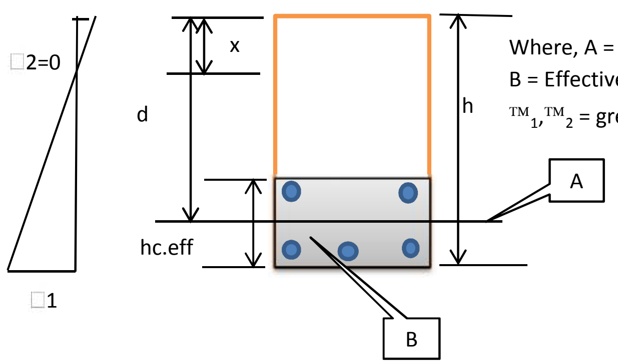
$\alpha_e = E_s/E_{cm}$ = 6.4516129

$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.5 Mpa

$\rho_{p.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$

Where, $h_{c.eff}$ = lesser of the followings

2.5(h-d);(h-x/3);or h/2



Where, A = level of steel centroid

B = Effective tension area, $A_{c.eff}$

™$_1$,™$_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 218.75 mm

$A_{c.eff}$ = 218750 mm^2

Now. $\rho_{p.eff} = A_s/A_{c.eff}$ = 0.020185714

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+ϕ/2)), the maximum crack spacing,

$$Sr.\max - 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{p.eff}}$$

Where, ϕ = diameter of bar = 25 mm c = clear cover = 50

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 380.545 mm

And, ™$_{sm}$-™$_{cm}$ = 0.000149411

Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.000299615

So, governing value of ™$_{sm}$ - ™$_{cm}$ = 0.00029962

So, crack width, W_k = $S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.114 mm

Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)

crack width within permissible limit

CHECK FOR HEEL SLAB:

Moment calculation at abutment face (considering 1m width);

Load Case	Total Load (KN/m)	Lever arm (m)	Moment (KN-m)	Shear force at face (KN)	Shear force at "d" distance from face (KN)
4	699.048	1.726	1206.790	699.048	408.635
Wt. of slab	-106.250	1.700	-180.625	-106.250	-62.109
Earth Wt. above slab	-457.164	1.700	-777.179	-457.164	-267.239
		Total	**248.986**	**135.634**	**79.286**

Stress level check:

Width of section considered = 1 m

Section is checked for SLS

Moment = 248.99 KN-m (for 1m width)

= 248.99 KN-m upword direction

Total reinforcement provided = 3925 mm^2

Effective depth "d" = 1412.5 mm

Netral axis depth = x = 287.14 mm

CG of compressive force = 95.714 mm from most compressed surface

Now moment, M = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

So, stress in steel = 48.2 Mpa OK, within permissible limit

Total force = 189.1 KN

Stress in concrete = 1.3 Mpa OK, within permissible limit

Crack width check:

Crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) Where, $S_{r.max}$ = Maximum crack spacing

™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

™$_{cm}$ = mean strain in the concrete between cracks.

Now, $$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct.eff}}{\rho_{\rho eff}}(1 + \alpha_e \rho_{\rho.eff})}{E_s} => 0.6\frac{\sigma_{sc}}{E_s}$$

(Eq. 12.6, IRC:112-2011, page-125)

Where, σ_{sc} = stress in the tension reinforcement = 48.17 Mpa

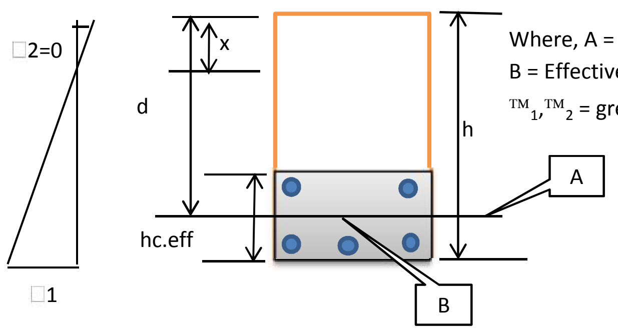
$\alpha_e = E_s/E_{cm}$ = 6.4516129

$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.5 Mpa

$\rho_{\rho.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$

Where, $h_{c.eff}$ = lesser of the followings

2.5(h-d);(h-x/3);or h/2



Where, A = level of steel centroid

B = Effective tension area, $A_{c.eff}$

™$_1$,™$_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 218.75 mm

$A_{c.eff}$ = 218750 mm^2

Now. $\rho_{\rho.eff} = A_s/A_{c.eff}$ = 0.017942857

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+ϕ/2)), the maximum crack spacing,

$$Sr.max = 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{\rho.eff}}$$

Where, ϕ = diameter of bar = 25 mm c = clear cover = 50

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 406.863 mm

And, ™$_{sm}$-™$_{cm}$ = -1.478E-04

Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.000144524

So, governing value of ™$_{sm}$ - ™$_{cm}$ = 1.45E-04

So, crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.059 mm

Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)

crack width within permissible limit

mm

mm

Design of dirt wall

Grade of concrete = M 30
Grade of steel = Fe 500
Width of section considered = 1 m

Section is checked for ULS
Design moment (Non-Seismic) = 150.007 KN-m (for 1m width)
Design moment (Seismic) = 132.783 KN-m (for 1m width)
Width of section = 1 m
Depth of section = 0.4 m
"E" value of steel = 200000 Mpa
"E" value of concrete = 31000 Mpa
Design compressive strength of concrete =
f_{cd}=af_{ck}/g_m = 13.40 Mpa Where, a = 0.67
g_m = 1.5
Design peak strength of steel = f_y/g_s = 434.783 Mpa Where, g_s = 1.15
Concrete failure strain = $Ô_{cu1}$ = 0.0035 (Table 6.5, IRC:112-2011, page-38)
Concrete limiting strain = $Ô_{c2}$ = 0.002 (Table 6.5, IRC:112-2011, page-38)
Yield strain of steel = $0.87f_y/E_s$ = 0.00218
Limiting strain of steel = $(0.87f_y/E_s+0.002)$ = 0.00418

Reinforcement provided: **20** mm dia.
125 mm c/c distance
Total reinforcement provided = 2512 mm^2
Clear cover = 50 mm
Effective depth "d" = 340 mm
Actual Neutral Axis depth x_u = $(0.87f_yA_{st})/(0.36f_{ck}b)$= 101.18 mm
Actual strain in steel = 0.012 mm
Stress in steel = 434.783 Mpa
Balanced Neutral Axis depth $x_{u,max}$ = 155.04 mm
So, Section is under reinforced, ok
CG of compressive force = 42.090 mm from most compressed surface
Moment of resistance, Mu = (Stress in steel)x(area of steel)x(d-CG of compressive force) =
325.370 kN-m **OK**

CHECK FOR SHEAR IN RETURN WALL (Clause 10.3.2, IRC:112-2011, page-88)

Design Shear Force (Non-Seismic) = 87.978 KN (For 1m strip)
Design Shear Force (Seismic) = 79.854 KN (For 1m strip)
The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

$$=[0.12K(80r_1.f_{ck})^{0.33}+0.15s_{cp}]b_w.d$$

Where, K = 1+√(200/d)<=2.0
So, K = 1.767
$r_1 = A_{sl}/b_w.d$

Where A_{sl} = Area of steel provided = 2512 mm^2

b_w = Width of section = 1000 mm

d = 340 mm

r_1 = 0.0074

$s_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0

A_c = Cross Sectional area of concrete

s_{cp} = 0

So, $V_{Rd.c}$ = 186.20 KN

Now, $V_{Rd.c}$ minimum = $(n_{min}+0.15s_{cp})b_w.d$

where n_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.399

So, $V_{Rd.c}$ minimum = 135.595 KN

So, governing shear resistance = 186.20 KN **OK, Safe in shear**

<u>Serviceability Limit State check</u>

For serviceability limit state, design moment = 150.007 KN-m

Modular ratio in tension = 9.33

Concrete failure strain = 0.0035

Maximum allowable stress in concrete = $0.48f_{ck}$ = 14.4 Mpa

(Clause 12.2.1(1), IRC:112-2011, page-120)

Maximum allowable stress in steel = $0.8f_{yk}$ = 400 Mpa

(Clause 12.2.2, IRC:112-2011, page-120)

Total reinforcement provided = 2512 mm^2

Effective depth "d" = 340 mm

Netral axis depth = x = 104.98 mm

CG of compressive force = 34.993 mm from most compressed surface

Now moment, M = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

So, stress in steel = 195.79 Mpa OK, within permissible limit

Total force = 491.81 KN

Stress in concrete = 9.370 Mpa OK, within permissible limit

<u>Crack width check:</u>

Crack width, $W_k = S_{r.max}(\hat{O}_{sm}-\hat{O}_{cm})$ Where, $S_{r.max}$ = Maximum crack spacing

$\hat{O}_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

$\hat{O}_{cm}$ = mean strain in the concrete between cracks.

Now,

$$\varepsilon_{sm} \quad \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct.eff}}{\rho_{p.eff}}(1 + \alpha_e \rho_{p.eff})}{E_s} => 0.6\frac{\sigma_{sc}}{E_S}$$

(Eq. 12.6, IRC:112-2011, page-125)

Where, s_{sc} = stress in the tension reinforcement = 195.79 Mpa

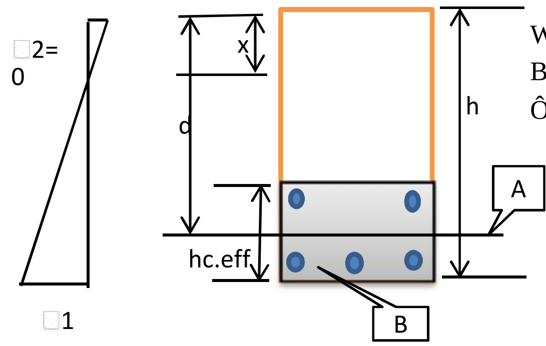
$a_e = E_s/E_{cm}$ = 6.451613

$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.5 Mpa

$r_{r.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$

Where, $h_{c.eff}$ = lesser of the followings

2.5(h-d);(h-x/3);or h/2



Where, A = level of steel centroid

B = Effective tension area, $A_{c.eff}$

$\hat{O}_1$,$\hat{O}_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 150 mm

$A_{c.eff}$ = 150000 mm^2

Now. $\rho_{\rho.eff} = A_s/A_{c.eff}$ = 0.016747

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+ϕ/2)), the maximum crack spacing,

$$Sr.max - 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{\rho.eff}}$$

Where, f = diameter of bar = 20 mm c = clear cover = 50 mm

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 373.025 mm

And, $\hat{O}_{sm}$-$\hat{O}_{cm}$ = 0.000565

Minimum value of $\hat{O}_{sm}$ - $\hat{O}_{cm}$ = 0.000587

So, governing value of $\hat{O}_{sm}$ - $\hat{O}_{cm}$ = 0.0005874

So, crack width, $W_k = S_{r.max}(\hat{O}_{sm}-\hat{O}_{cm})$ = 0.219 mm

Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)

crack width within permissible limit

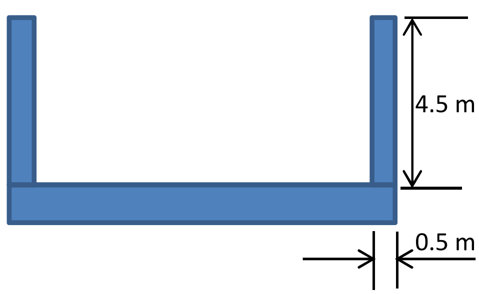
DESIGN OF FINN

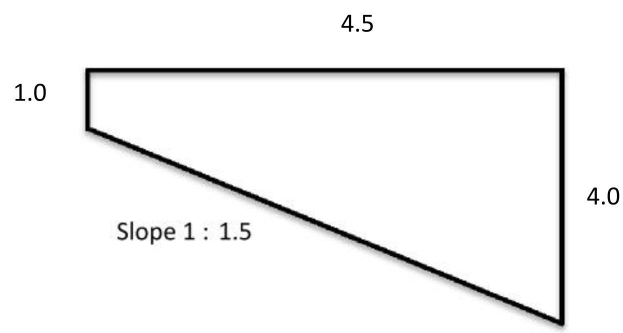
Height of finn wall=	4	m
Length of finn wall=	4.5	m
Coefficient of horizontal active earth pressure =	0.279	
For soil unit weight =	18.00	KN/m^3
Earth pressure at bottom of wall =	22.320	KN/m^2
Eart pressure at 1m above wall bottom =	16.740	KN/m^2
Average eatrh pressure in 1m strip =	19.530	KN/m
Length of the finn at this strip =	1.50	m
Surcharge earth pressure equivalent to earth pressure of 1.2 m height =	6.026	KN/m^2
Bending moment at wall junction due to earth pressure, M_1 =	197.741	KN-m
Bending moment at wall junction due to surcharge earth pressure, M_2 =	61.017	KN-m
Design moment at junction, $M_u = 1.5M_1+1.2M_2$ =	369.833	KN-m
Design shear force at junction, $F_u = 1.5F_1+1.2F_2$ =	54.790	KN

<u>Moment Calculation of Fly wing wall taking 1m strip 2m below the top</u>

Earth pressure at 1 m below the wall from top =	5.022	KN/m^2
Surcharge earth pressure equivalent to earth pressure of 1.2 m height =	6.026	KN/m^2
Bending moment at wall junction due to earth pressure, M_1 =	50.848	KN-m
Bending moment at wall junction due to surcharge earth pressure, M_2 =	61.017	KN-m
Design moment at junction, $M_u = 1.5M_1+1.2M_2$ =	149.492	KN-m
Design shear force at junction, $F_u = 1.5F_1+1.2F_2$ =	66.441	KN

<u>Design of Finn in flexure</u>





Grade of concrete =	M	30
Grade of steel =	Fe	500
Width of section considered =	1	m

Section is checked for ULS

Design moment =	369.833	KN-m (for 1m width)
Width of section =	1	m
Depth of section =	0.5	m
"E" value of steel =	200000	Mpa
"E" value of concrete =	32000	Mpa
Design compressive strength of concrete =		
$f_{cd}=\alpha f_{ck}/\gamma_m$ =	13.40	Mpa Where, α = 0.67, γ_m = 1.5
Design peak strength of steel = f_y/γ_s =	434.783	Mpa Where, γ_s = 1.15
Concrete failure strain = ™$_{cu1}$ =	0.0035	(Table 6.5, IRC:112-2011, page-38)
Concrete limiting strain = ™$_{c2}$ =	0.002	(Table 6.5, IRC:112-2011, page-38)
Yield strain of steel = $0.87f_y/E_s$ =	0.00218	
Limiting strain of steel = $(0.87f_y/E_s+0.002)$ =	0.00418	
Reinforcement provided:	**20**	mm dia.
	125	mm c/c distance
Total reinforcement provided =	2826	mm^2
Clear cover =	50	mm
Effective depth "d" =	440	mm
Actual Neutral Axis depth x_u = $(0.87f_yA_{st})/(0.36f_{ck}b)$=	113.83	mm
Actual strain in steel =	0.014	mm
Stress in steel =	434.783	Mpa
Balanced Neutral Axis depth $x_{u,max}$ =	200.64	mm

So, Section is under reinforced, ok

CG of compressive force = 47.351 mm from most compressed surface

Moment of resistance, Mu = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

482.446 **OK**

CHECK FOR SHEAR IN FINN (Clause 10.3.2, IRC:112-2011, page-88)

Design Shear Force = 66.441 KN

The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

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

Where, $K = 1+\sqrt{(200/d)}<=2.0$

So, K = 1.674

$\rho_1 = A_{sl}/b_w.d$

Where A_{sl} = Area of steel provided = 2826 mm^2

b_w = Width of section = 1000 mm

d = 440 mm

ρ_1 = 0.0064

$\sigma_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0

A_c = Cross Sectional area of concrete

σ_{cp} = 0

So,$V_{Rd.c}$ = 218.00 KN

Now, $V_{Rd.c}$ minimum = $(\nu_{min}+0.15\sigma_{cp})b_w.d$

where ν_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.368

So, $V_{Rd.c}$ minimum = 161.840 KN

So, governing shear resistance = 218.00 KN **OK, Safe in shear**

Serviceability Limit State check

For serviceability limit state, design momeny = $M_1+0.8M_2$ = 246.555 KN-m

Modular ratio in tension = 9.333333

Concrete failure strain = 0.0035

Maximum allowable stress in concrete = $0.48f_{ck}$ = 14.4 Mpa

(Clause 12.2.1(1), IRC:112-2011, page-120)

Maximum allowable stress in steel = $0.8f_{yk}$ = 400 Mpa

(Clause 12.2.2, IRC:112-2011, page-120)

Total reinforcement provided = 2826 mm^2
Effective depth "d" = 440 mm
Netral axis depth = x = 128.24 mm
CG of compressive force = 42.747 mm from most compressed surface
Now moment, M = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

So, stress in steel = 219.62 Mpa OK, within permissible limit
Total force = 620.65 KN
Stress in concrete = 9.679 Mpa OK, within permissible limit

Crack width check:

Crack width, $W_k = S_{r.max}$(™$_{sm}$-™$_{cm}$) Where, $S_{r.max}$ = Maximum crack spacing

™$_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

™$_{cm}$ = mean strain in the concrete between cracks.

Now, $\varepsilon_{sm} \quad \varepsilon_{cm} = \dfrac{\sigma_{sc} - k_t \frac{f_{ct.eff}}{\rho_{\rho.eff}}(1+\alpha_e \rho_{p.eff})}{E_s} => 0.6\dfrac{\sigma_{sc}}{E_s}$

(Eq. 12.6, IRC:112-2011, page-125)

Where, σ_{sc} = stress in the tension reinforcement = 219.62 Mpa

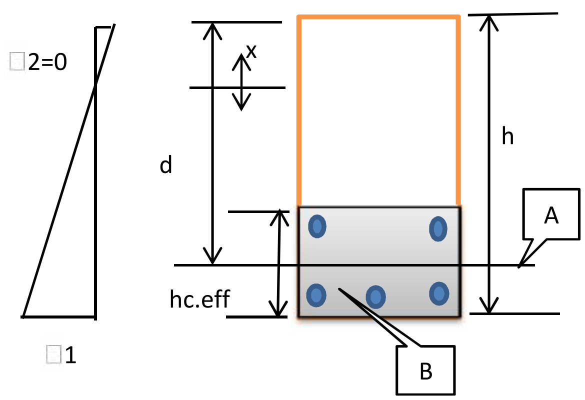
$\alpha_e = E_s/E_{cm}$ = 6.25

$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.5 Mpa

$\rho_{\rho.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$

Where, $h_{c.eff}$ = lesser of the followings

2.5(h-d);(h-x/3);or h/2



Where, A = level of steel centroid
B = Effective tension area, $A_{c.eff}$
™$_1$,™$_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 150 mm

$A_{c.eff}$ = 150000 mm^2

Now. $\rho_{\rho.eff} = A_s/A_{c.eff}$ = 0.01884

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+ϕ/2)), the maximum crack spacing,

$$S_{r.max} = 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{p.eff}}$$

Where, ϕ = diameter of bar = 20 mm c = clear cover = 50 mm

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 350.467 mm

And, ™$_{sm}$-™$_{cm}$ = 0.000727

Minimum value of ™$_{sm}$ - ™$_{cm}$ = 0.000659

So, governing value of ™$_{sm}$ - ™$_{cm}$ = 0.000727

So, crack width, W_k = $S_{r.max}$(™$_{sm}$-™$_{cm}$) = 0.255 mm

Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)

crack width within permissible limit

Summarized Reinforcement Detailing
(According to chapter 16, IRC:112-2011, page-171)
<u>ABUTMENT SHAFT</u>

Total vertical reinforcement provided =			7301	mm^2
Concrete area, A_c =			1200000	mm^2
Now, $0.0024A_c$ =	2880 mm^2	**< steel provided, OK**		
and $0.04A_c$ =	48000 mm^2	**>steel provided, OK**		

Horizontal Reinforcement provide **16** mm dia. @ **150** mm c/c.

Total horizontal reinforcement provided =			1340 mm^2
Now 25% of vertical steel =	913 mm^2		**< steel provided, OK**
(For one face)			
and $0.001A_c$ =	1200 mm^2	**OK**	**< steel provided, OK**

Transverse reinforcement provided =
Now, $0.02A_c$ = 24000 mm^2
> steel provided,No transeverse reinforcement required

Vertical reinforcement provided =
25 mm dia @ **150** mm c/c.
Horizontal reinforcement provided =
16 mm dia @ **150** mm c/c.

FOUNDATION SLAB

Reinforcement of toe slab

Main reinforcement provided	**25** mm dia.		**125** mm c/c	
Steel in 1m strip =	3925 mm^2			
$A_{s.min}$ =($0.26f_{ctm}/f_{yk})b_td$=		1836.25 mm^2		**OK**
or, $0.0013b_td$ =		1836.25 mm^2		**OK**

Reinforcement of heel slab

Main reinforcement provided	**25** mm dia.		**125** mm c/c	
Steel in 1m strip =	3925 mm^2			
$A_{s.min}$ =($0.26f_{ctm}/f_{yk})b_td$=		1836.25 mm^2		**OK**
or, $0.0013b_td$ =		1836.25 mm^2		**OK**

Calculation of distrubution reinforcement

Top bar at toe slab		**16** mm dia.	**150** mm c/c	
Steel in 1m strip =		1339.73333 mm^2	**OK**	
25% of main steel =	981.25			
Bottom bar at heel slab		**16** mm dia.	**150** mm c/c	
Steel in 1m strip =		1339.73333 mm^2	**OK**	
25% of main steel =	981.25			

Transverse reinforcement provided at bottom and at top

	16 mm dia.	**150** mm c/c
Steel in 1m strip =	1339.73333 mm^2	
	OK	
Provide surface reinforcement	**16** mm dia.	**4** Nos. each face

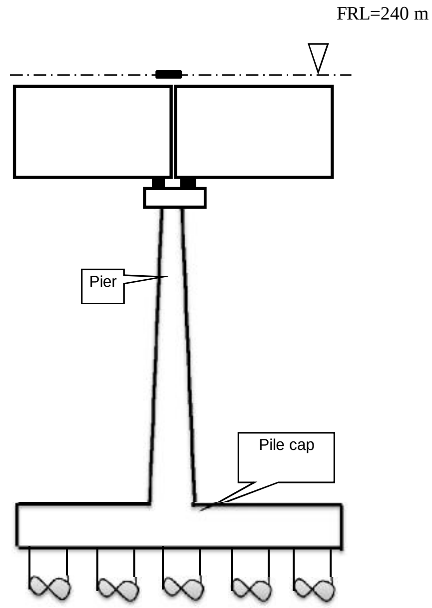
FINN WALL

Main reinforcement provided	**20** mm dia.		**125** mm c/c
Distributor reinforcement	**12** mm dia.		**125** mm c/c
		OK	

DIRT WALL

Provide main reinforcement	**20** mm dia.		**125** mm c/c
Distributor reinforcement	**12** mm dia.		**125** mm c/c
		OK	

PIER DESIGN
CHAINAGE -95.500 KM
3X41.0 PSC T-GIRDER
IRANG RIVER



a) Superstructure:-

At the proposed Bridge Site, the following data are available;

a) High Flood Level, HFL	=	223.358	m
b) Lowest Bed Level, LBL	=	218.500	m
At Pier Locations			
Max. Scour Level	=	207.948	m
Formation level =	=	240.000	m
R.L. of carriageway at end of carriageway	=	239.763	m
Depth of girder+deck slab at CL of carraigeway	=	3.016	m
Thickness of cement concrete Wearing Coat	=	0.065	m

b) Substructure:-

Level of bearings (near to median)=	=	236.894	m
Height fo bearing=	=	0.300	m
Height of pedestal=	=	0.440	m
Top of Pier cap level	=	236.154	m
Hence, 236.894 - 217.904	=	18.990	m
Height of frame at Pier locations	=	18.990	m
Height of Pier with cap	=	18.250	m
Height of Pier shaft	=	16.250	m

c) Foundation:-

E.G.L. at Pier	=	220.298	m
The lowest E.G.L. at Pier	=	220.298	m
Bottom pile cap at	=	215.904	m
Height of pile cap	=	2.000	m
Top of pile cap level	=	217.904	m

LOAD CALCULATIONS

PERMANENT ACTIONS

Self Weight/Dead Load

Wt. of deck with girder	=	10806.525	kN
Dead Load from super-structure =		10806.525	kN

Weight of pier cap =		=	1761.50	kN
Weight of pier (Circular part) =		=	1544.29	kN
Weight of pier (Straight part) =		=	7150.00	kN
Total weight of pier		=	8694.29	kN
Total weight of pile cap =		=	10269.00	kN

VARIABLE GRAVITY LOAD TREATED AS PERMANENT LO

Super Imposed Dead Load (SIDL) (except surfacing)

Super Imposed Dead Load acting on pier =

Live load on Footpath	=	492	kN
Wt. of Kerb	=	230	kN
Wt. of Railing	=	123	kN
Wt. of crash barrier	=	676	kN
TOTAL SIDL LOAD	=	1521	kN

Surfacing and Wearing Coat

Surfacing or loading due to cement concrete wearing coat on footpath	=	230.40	kN
Surfacing or loading due to bituminous concrete wearing coat on carriageway	=	556.44	kN
So, Total load of surfacing	=	786.84	kN

VARIABLE ACTIONS

Vehicular Live Load

Carriageway Live load

(I) 2 lanes of Class-A

(II) 1 lane of 70R Tracked

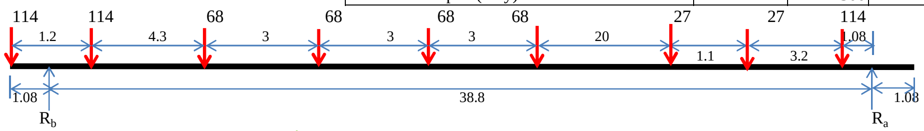
(III) 1 lane of 70R Wheeled

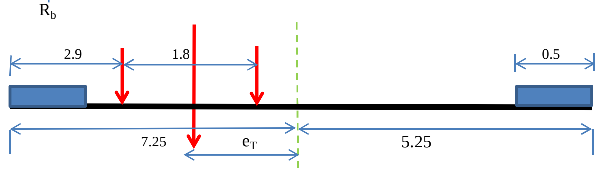
(IV) 1 lane of Special Vehicle(385 T)

1 Type of Loading = Class A train of vehicle.

A) *One Span Loaded*

Span, L_e =	38.80	m	**Case - 1: One Lane / one span loaded.**				
L_c =	1.08	m	Minimum Clearence		=	150	mm
Expansion gap =	0.04	m	Width of ground contact (In transverse direction)		=	500	mm
Impact Factor =	1.101		Width of Footpath with crush barrier & kerb		=	2500	mm
			Width of carriageway		=	9.50	m
			Width of Footpath(only)		=	500	mm





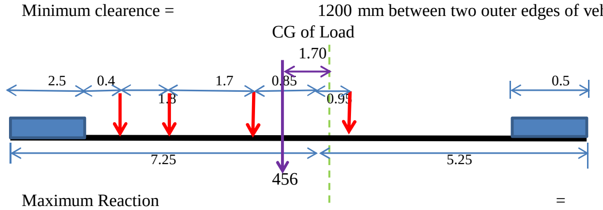
Maximum Reaction = R_b = 495.7 kN

And transverse eccentricity, wrt deck, e_T = 3.45 m

And longitudinal eccentricity, wrt abutment, e_L = 1.1 m

Case - 2: Two Lane / One span loaded.

Minimum clearence = 1200 mm between two outer edges of vehicle.



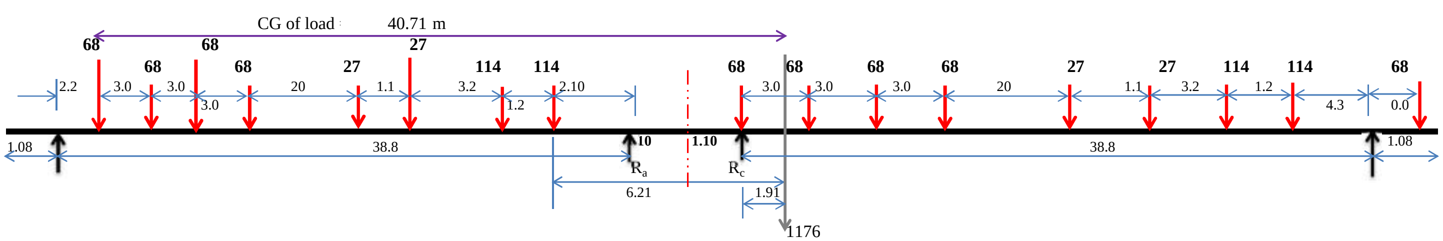
Maximum Reaction = 991.3 kN

And transverse eccentricity, wrt deck, e_T = 1.70 m

And longitudinal eccentricity, wrt abutment, e_L = 1.1 m

B) *Both Span Loaded*

Case - 3: One Lane / both span loaded.



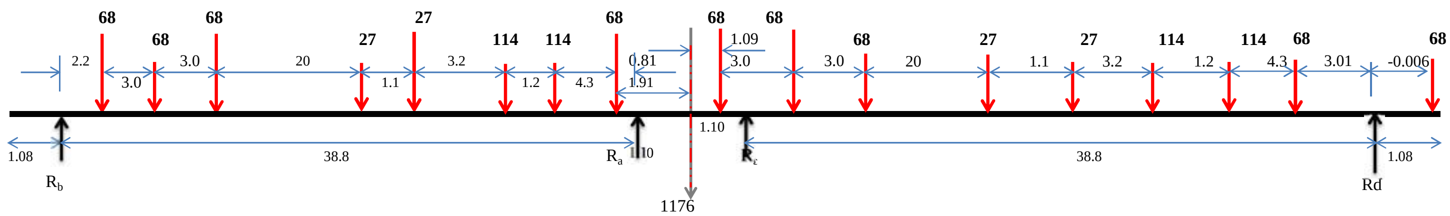
CG calculation of load:

Taking moment with respect to the left most load, CG of load = 40.71 m

With above consideration, x = 6.21 m

Similarly, we have y = 1.91 m

In order to get the maximum pier reaction, we have place the loads in such a manner so that the CG of the load passes through the centre line of the pier.



Maximum reaction, R_A = 361.5 kN

Similarly, max. reaction, R_C = 283.0 kN

R = Total Pier reaction = R_A+R_C = 644.5 kN

And transverse eccentricity, wrt deck, e_T = 3.45 m

And longitudinal eccentricity, wrt pier, e_L = 0.13 m

Case - 4: Two Lane / both span loaded.

R = Total Pier reaction = R_A+R_C = 1289.1 kN

And transverse eccentricity, e_T = 1.70 m

And longitudinal eccentricity, e_L = 0.13 m

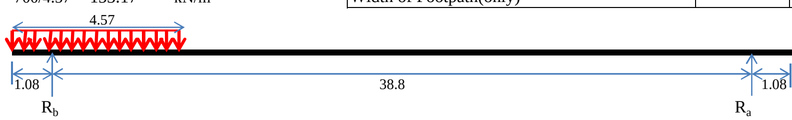
II Type of Loading = IRC class 70R Tracked

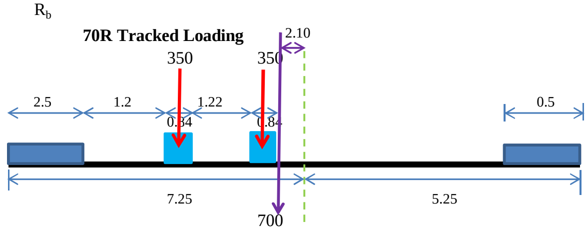
A) One Span Loaded

Span, L_e =		38.8	m
L_c =		1.08	m
Expansion gap =		0.04	m
Impact factor =		1.1	

700/4.57 = 153.17 kN/m

Case - 1: 70R Tracked			
Minimum Clearence	=	1200	mm
Width of ground contact	=	840	mm
Width of footpath with kerb & crash barrier	=	2500	mm
Width of carriageway	=	9.50	m
Width of Footpath(only)	=	500	mm





Maximum Reaction for 70R Tracked = Rb = 725.8 kN

Hence,Total Reaction Rb = 725.8 kN

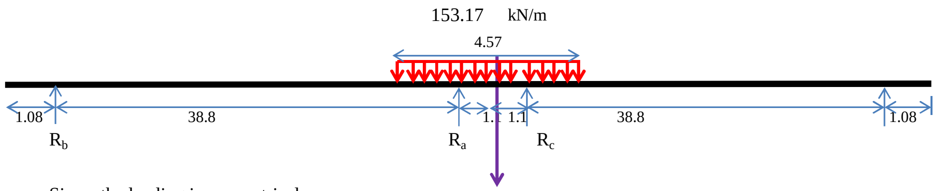
And transverse eccentricity, wrt deck, e_T = 2.10 m

And longitudinal eccentricity, wrt abutment, e_L = 1.1 m

B) Both Span Loaded

Case - 2: 70R Tracked

In order to get the maximum pier reaction, we have place the loads in such a manner so that the CG of the load passes through the centre line of the pier.



Since, the loading is symmetrical

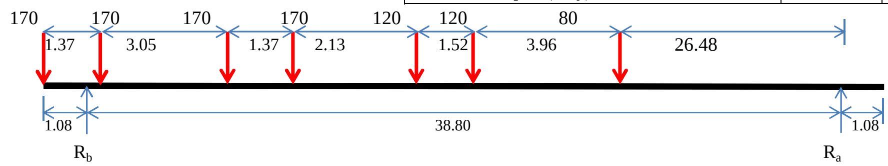
Maximum reaction, R_A	=	384.58	kN
Similarly, max. reaction, R_C	=	384.58	kN
R = Total Pier reaction = R_A+R_C	=	769.16	kN
Hence,Total Reaction R	=	769.2	kN
And transverse eccentricity, wrt deck, e_T	=	2.10	m
And longitudinal eccentricity, wrt pier, e_L	=	0.000	m

III Type of Loading = IRC 70 R Wheel

A) One Span Loaded

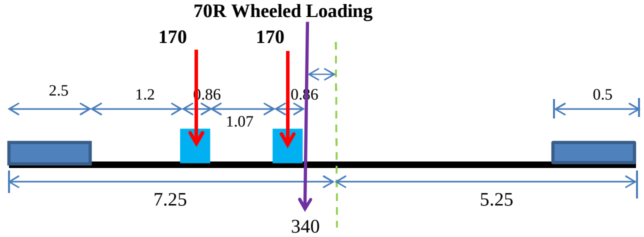
Span, L_e =	38.8	m
L_c =	1.08	m
Expansion gap =	0.04	m
Impact factor =	1.101	

Case - 1: 70 R Wheel			
Minimum Clearence	=	1200	mm
Width of ground contact	=	860	mm
Width of footpath with kerb & crash barrier	=	2500	mm
Width of carriageway	=	9.5	m
Width of Footpath(only)	=	500	mm



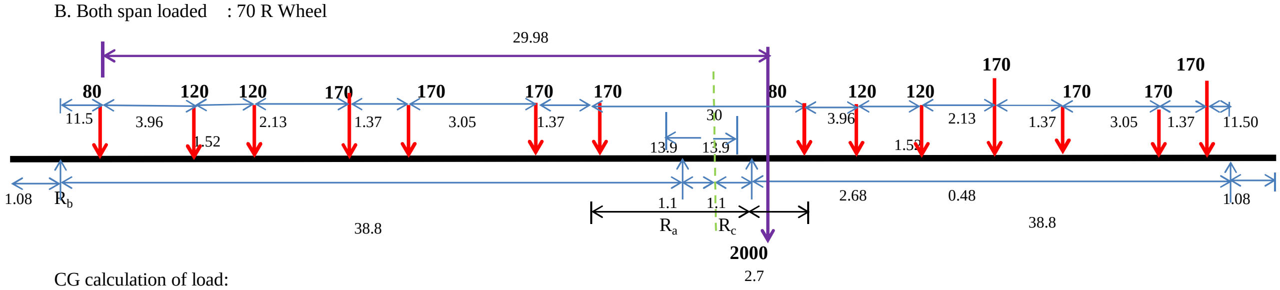
Maximum reaction =	Rb	=	986.25	kN
Hence,Total Reaction Rb		=	986.25	kN
max longitudinal eccentricity =	eL	=	1.10	m

70R Wheeled Loading



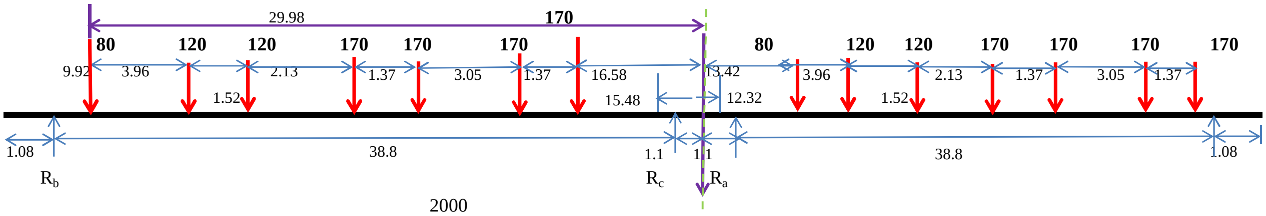
max transverse eccentricity = eT = 2.155 m

B. Both span loaded : 70 R Wheel



CG calculation of load:

Taking moment with respect to the left most load, CG of load	=	29.98	m
With above consideration, x	=	2.68	m
Similarly, we have y	=	0.48	m

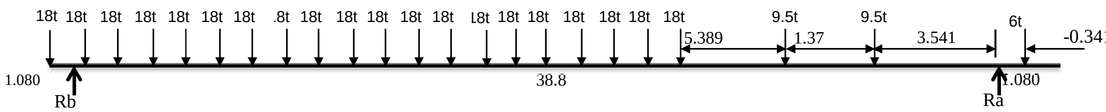


Maximum reaction, R_A		584.55	kN
Similarly, max. reaction, R_C	=	516.45	kN
R = Total Pier reaction = R_A+R_C	=	1101.00	kN
Hence,Total Reaction R	=	1101.00	kN
And transverse eccentricity, wrt deck, e_T	=	2.16	m
And longitudinal eccentricity, wrt pier, e_L	=	0.068	m

4 Type of Loading = IRC Class SV Loading : Special Multi Axel Hydraulic Trailer Vehicle
(AMENDMENT TO IRC:6-2014, AMENDMENT NO.1_CLAUSE 204.5)

A) One Span Loaded

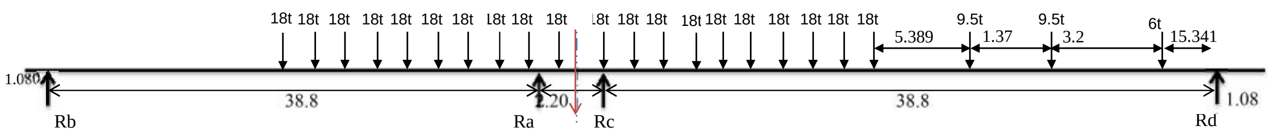
			Case - 1: IRC Class SV Loading				
Span, L_e =	38.8	m	Minimum Clearence		=	-	
L_c =	1.08	m	Width of ground contact		=	156	mm
Expansion gap =	0.04	m	Width of crash barrier		=	2500	mm
Impact factor =	1		Width of carriageway		=	10	m



Loading= 20 nos. of wheels each 180 KN @ c/c 1.5 m for 28.5 M Span.
so,(28.5/1.5+1)= 20
hence for, 40.96 m Sapn = 20 no. of wheels

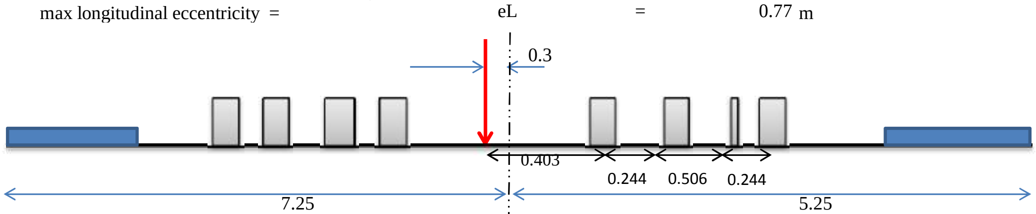
Maximum reaction =	Rb	=	2299	kN
	Hence,Total Reaction Rb	=	2299	kN
max longitudinal eccentricity =	eL	=	1.10	m

B) Both Span Loaded



Loading= 20 nos. of wheels each 180 KN @ c/c 1.5 m for 28.5 M Span.
so,(28.5/1.5+1)= 20
Total loading = 3600 KN
Spacing of loads= 1.5 m centre to centre
No. of wheels per span = 10 (for maximum reaction)

Maximum reaction =	Rc	=	1604.7	kN
	Ra	=	280.7	kN
	Total	=	1885.4	kN
Hence,Total Reaction Rb		=	1885.4	kN
max longitudinal eccentricity =	eL	=	0.77	m



max transverse eccentricity = eT = 0.30 m (AMENDMENT TO IRC:6-2014, AMENDMENT NO.1_CLAUSE 204.5.3)

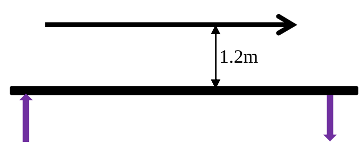
CWLL Load on Pier

sl. No	Live Load Case	Load due to main wheel	Load due to additional wheel	Reaction (KN)			e_L (m)	e_L (m)	e_L (m)	e_T (m)	M_L (KN-m)	M_T (KN-m)
	1	2	3	4	5	6	7	8	9	10		
	For Class-A			Class A	70R	**SV**	Class A	70R	**SV**		(col 4 x col 6) + (col 5 x col 7)	(col 4 x col 8) + (col 5 x col 8)
1	One lane / one span loaded	668	0	495.65			1.10			3.450	545.22	1710.00
2	Two lane / one span loaded	1336	0	991.31			1.10			1.700	1090.44	1685.22
3	One lane / Both span loaded	1176	0	644.55			0.13			3.450	86.39	2223.70
4	Two lane / Both span loaded	2352	0	1289.10			0.13			1.700	172.78	2191.47
	For IRC class 70R Tracked											
5	One span loaded	700	0		725.82			1.100		2.100	798.40	1524.22
6	Both span loaded	700	0		769.16			0.000		2.100	0.00	1615.23
	For IRC class 70R Wheeled											
7	One span loaded	1000	0		986.25			1.100		2.155	1084.88	2125.37
8	Both span loaded	1000	0		1101.00			0.068		2.155	74.91	2372.66
	For IRC SV											
9	One lane / one span loaded	3850	0			2299.06			1.10	0.300	2528.96	689.72
10	One lane / Both span loaded	3850	0			1885.40			0.77	0.300	1456.46	565.62

Longitudinal forces

1 *Calculation of Braking Forces*

Caused by braking of vehicles ……. (Ref. cl. 211 of IRC 6-2014, page-37)

				Case - I	Case - II	Case - III	Case - IV	Case - V	Case - VI	Case - VII	Case - VIII
	Braking force line of action 			Class A - Two lane / one span loaded	Class A - two Lane both span loaded	70R Tr. - Two Lane , one span loaded	70R Tr.- two lane / Both span loaded	70R Wh. Load - Two Lane , one span loaded	70R Wh. - Two lane / Both span loaded	SV One lane / one span loaded	SV One lane / Both span loaded
a	Total Load	kN	=	1336	2352	700	700	1000	1000	3850	3850
b	Braking force F_h	kN	=	267.2	470.4	140.0	140.0	200.0	200.0	0	0
c	Each side F_h	kN	=	267.2	470.4	140.0	140.0	200.0	200.0	0.0	0.0
d	Bearing forces at bearing level μ(Rg + Rq) =	kN	=	353.9	362.9	324.2	324.2	324.2	324.2	393.2	380.8
e	ThickNess of wearing coat	m	=	0.065	0.065	0.065	0.065	0.065	0.065	0.065	0.065
f	Ht. of Braking force act above bearing	m	=	4.490	4.490	4.490	4.490	4.490	4.490	4.490	4.490
g	Moment at bearing level	kN-m	=	1199.73	2112.10	628.60	628.60	898.00	898.00	0.00	0.00
h	Reaction as push/pull (+/-)	kN	=	30.92	54.44	16.20	16.20	23.14	23.14	0.00	0.00
i	For moment at pier base, lever arm	m	=	18.990	18.990	18.990	18.990	18.990	18.990	18.990	18.990
j	Longitudinal moment at pier base	kN-m	=	6721.22	6890.88	6156.48	6156.48	6156.48	6156.48	7466.25	7230.59

Type of bearing at support = Pot cum PTFE

At pier = Pot Bearing

As per IRC 6-2014 Clause-211.5.1 μ= 0.03

Longitudinal forces at Bearing level = Maximum of { F_h - μ(Rg + Rq) =	-86.73	107.53	-184.20	-184.20	-124.20	-124.20	-393.17	-380.76
F_h /2 + μ(Rg + Rq) =	487.53	598.07	394.20	394.20	424.20	424.20	393.17	380.76
Hence longitudinal force =	487.53	598.07	394.20	394.20	424.20	424.20	393.17	380.76

Calculation of WIND LOAD

a ***Wind Force on superstructure:***

i. Transverse wind force (F_T) :

Solid area (A1) = Exposed area in Transverse direction = 169.125 m2

F_T = Area x Pz x G x C_D = 517 kN

ii. Longitudinal wind force (F_L) :

F_L = 25% of transverse wind force = 129.27 kN

iii. Vertical wind load (F_V) :

Plan area (A3) = 512.5 m2

Lift Coefficient (C_L) = 0.75

F_V = Area x pz x G x C_L = 603 kN

b ***Wind force on live load:***

as per clause 209.3.7 of IRC: 6, 2014, bridge shall not be considered to carry any live load if the basic wind velocity exceeds 36m/sec

c ***Wind force on Substructure:***

Pier Cap

i. Transverse wind force:

Exposed area = 6.60 m2

Transverse wind force = 8.3 kN

ii. Longitudinal Wind Force:

Exposed area = 22.8 m2

Transverse wind force = 28.5 kN

Pier Shaft

i. Transverse wind force:

Exposed area = 42.5 m2

Transverse wind force = 53.3 kN

ii. Longitudinal Wind Force:

Exposed area = 113.3 m2

Transverse wind force = 142.2 kN

Velocity Of Wind			
Position	Direction	vert. comp.	Hortz. Comp.
		kN	kN
Super structure	Transverse	602.67	517.09
	Longitudinal		129.27
Live load	Transverse	0.00	0.00
	Longitudinal		0.00
Sub structure (pile cap)	Transverse	0.00	4.14
	Longitudinal		14.27
Sub structure (pier shaft)	Transverse	0.00	26.65
	Longitudinal		71.08

ACCIDENTAL ACTIONS

Seismic Hazards

Seismic Zone of bridge location = V

Zone factor, Z = 0.36 (Table 6, IRC:6-2014, page-51)

Seismic importance factor of the structure I = 1.20 (Table 8, IRC:6-2014, page-56)

Average response acceleration co-efficient (S_a/g) = 2.50 (Clause 219.5.1, IRC:6-2014, page-55)

Horizontal seismic co-efficient A_h (Z/2)X (I)X(Sa/g)/R = 0.54 /R — Where R is response reduction factor to be considered

Is ductile detailing to be done ? Yes

Value of R for sub-structure = 3

Hence horizontal seismic co-efficient (A_h) for sub-structure = 0.180

Horizontal seismic force, F_{eq} = A_h.(Dead Load+Appropriate Live Load)

Seismic force due to dead load

(Inertia loads due to self-mass generated in bridge structure by ground acceleration)

A. *Seismic on Superstructure:*

Dead Load from super-structure and SIDL without surfacing = = 12327.53 KN

C.G. of Deck from girder bottom = 1.688 m

Design Horizontal Seismic coefficient A_h = 0.180

Seismic force in longitudinal direction F_h= $A_{h\,x}$ (Total Dead Load) 2218.95 KN

Seismic force in longitudinal direction taken by one support F_h = 2218.95 KN ………r_1

Acting at RL = 238.58 m

Lever arm for moment at bearing level = 1.688 m

Longitudinal moment at bearing level = 3745.60 KN-m

Vertical pull-push effect due to Horizontal seismic force = 48.27 KN

Lever arm for moment at pier base = 18.990 m

Longitudinal moment at pier base = 42137.95 KN-m ……..M_z

Horizontal seismic force in transeverse direction = F_h	=	2218.955	KN	………..r_2
Acting at RL	=	238.582	m	
Lever arm for moment at pier base	=	18.990	m	
Transeverse moment at pier base	=	42137.946	KN-m	……M_x
Vertical component of seismic force	=	1479.303	KN	………r_3 (Clause 219.3, IRC:6-2014, page-51)
Combination of force components				…….(Clause 219.4, IRC:6-2014, page-51)
Design force in longitudinal direction = $r_1 + 0.3r_2 + 0.3r_3$	=	3328.432	KN	
Design force in transeverse direction = $0.3r_1 + r_2 + 0.3r_3$	=	3328.432	KN	
Design force in vertical direction = $0.3r_1 + 0.3r_2 + r_3$	=	2810.676	KN	
Design longitudinal moment at Pier-base = $M_z + 0.3M_x$	=	54779.330	KN-m	
Design transeverse moment at Pier-base = $0.3M_z + M_x$	=	54779.330	KN-m	
Design longitudinal moment at pile cap-base = Mz + 0.3Mx	=	60548.611	KN-m	
Design transeverse moment at pile cap-base = 0.3Mz + Mx	=	60548.611	KN-m	

B. *Seismic on Sufacing:*

Surfacing or load due to wearing coat	=	786.842	KN	
C.G. of wearing coat from girder bottom	=	3.258	m	
Design Horizontal Seismic coefficient A_h	=	0.180		
Seismic force in longitudinal direction $F_{h=}$	A_h x (Total Dead Load) =	141.631	KN	
Seismic force in longitudinal direction taken by one support F_h	=	141.631	KN	………r_1
Acting at RL	=	240.152	m	
Lever arm for moment at bearing level	=	3.258	m	
Longitudinal moment at bearing level	=	461.360	KN-m	
Vertical pull-push effect due to Horizontal seismic force	=	5.946	KN	
Lever arm for moment at pier base	=	18.990	m	
Longitudinal moment at pier base	=	2689.582	KN-m	……..M_z
Horizontal seismic force in transeverse direction = Fh	=	141.631	KN	………..r_2
Acting at RL	=	240.152	m	
Lever arm for moment at pier base	=	18.990	m	
Transeverse moment at pier base	=	2689.582	KN-m	……M_x
Vertical component of seismic force	=	94.421	KN	………r_3 (Clause 219.3, IRC:6-2014, page-51)
Combination of force components	…….(Clause 219.4, IRC:6-2014, page-51)			
Design force in longitudinal direction = $r_1 + 0.3r_2 + 0.3r_3$	=	212.447	KN	
Design force in transeverse direction = $0.3r_1 + r_2 + 0.3r_3$	=	212.447	KN	
Design force in vertical direction = $0.3r_1 + 0.3r_2 + r_3$	=	179.400	KN	
Design longitudinal moment = $M_z + 0.3M_x$	=	3496.457	KN-m	
Design transeverse moment = $0.3M_z + M_x$	=	3496.457	KN-m	
Design longitudinal moment at pile cap base = Mz + 0.3Mx	=	3864.698	KN-m	
Design transeverse moment at pile cap base = 0.3Mz + Mx	=	3864.698	KN-m	

C. *Seismic on Pier Cap:*

CG of the pier cap from top of pile cap	=	17.260	m	
Longitudinal seismic force = A_h x W_cap	=	317.070	KN	……….r_1
Acting at RL	=	235.164	m RL	
Longitudinal moment	=	5472.628	KN-m	……..M_z
Transverse seismic = A_h x W_cap	=	317.070	KN	……….r_2
Acting at RL	=	235.164	m RL	
Transeverse moment	=	5472.628	KN-m	……..M_x
Vertical component of seismic force	=	211.380	KN	………r_3 (Clause 219.3, IRC:6-2014, page-51)
Combination of force components				…….(Clause 219.4, IRC:6-2014, page-51)
Design force in longitudinal direction = $r_1 + 0.3r_2 + 0.3r_3$	=	475.605	KN	
Design force in transeverse direction = $0.3r_1 + r_2 + 0.3r_3$	=	475.605	KN	
Design force in vertical direction = $0.3r_1 + 0.3r_2 + r_3$	=	401.622	KN	
Design longitudinal moment = $M_z + 0.3M_x$	=	7114.417	KN-m	
Design transeverse moment = $0.3M_z + M_x$	=	7114.417	KN-m	

D. *Seismic on Pier :*

CG of the pier from top of pile cap = 8.125 m

Longitudinal seismic force = A_h x W_Pier = 1564.970 KN ……….r_1

Acting at RL = 226.029 m RL

Longitudinal moment = 12715.381 KN-m ……..M_z

Transverse seismic = A_h x W_Pier = 1564.970 KN ……….r_2

Acting at RL = 226.029 m RL

Transeverse moment = 12715.381 KN-m ……..M_x

Vertical component of seismic force = 1043.313 KN ………r_3 (Clause 219.3, IRC:6-2014, page-51)

Combination of force components …….(Clause 219.4, IRC:6-2010, page-51)

Design force in longitudinal direction = $r_1 + 0.3r_2 + 0.3r_3$ = 2347.455 KN

Design force in transeverse direction = $0.3r_1 + r_2 + 0.3r_3$ = 2347.455 KN

Design force in vertical direction = $0.3r_1 + 0.3r_2 + r_3$ = 1982.295 KN

Design longitudinal moment = $M_z + 0.3M_x$ = 16529.996 KN-m

Design transeverse moment = $0.3M_z + M_x$ = 16529.996 KN-m

E. Seismic on pile cap :

CG of the pile cap from bottom of pile cap = 1.000 m

Longitudinal seismic force = A_h x W_pile cap = 1848.420 KN ……….r_1

Acting at RL = 216.904 m RL

Longitudinal moment = 1848.420 KN-m ……..M_z

Transverse seismic = A_h x W_pile cap = 1848.420 KN ……….r_2

Acting at RL = 216.904 m RL

Transeverse moment = 1848.420 KN-m ……..M_x

Vertical component of seismic force = 1232.280 KN ………r_3 (Clause 219.3, IRC:6-2014, page-51)

Combination of force components …….(Clause 219.4, IRC:6-2014, page-51)

Design force in longitudinal direction = $r_1 + 0.3r_2 + 0.3r_3$ = 2772.630 KN

Design force in transeverse direction = $0.3r_1 + r_2 + 0.3r_3$ = 2772.630 KN

Design force in vertical direction = $0.3r_1 + 0.3r_2 + r_3$ = 2341.332 KN

Design longitudinal moment = $M_z + 0.3M_x$ = 2402.946 KN-m

Design transeverse moment = $0.3M_z + M_x$ = 2402.946 KN-m

F. *Seismic on carriageway live load* …….(Clause 219.5.2, IRC:6-2014, page-55)

(Inertia loads due to mass of vehicular live load)

sl. No.	Live Load Case	20% Reaction (KN)			Ah	Transverse seismic force,	Acting RL at (+1.20)	Lever arm at Pier base	Transverse moment at pier vase	vertical force component
	For Class-A	Class A	70R							
1	One lane / one span loaded	90.04			0.180	16.21	241.20	23.30	377.55	251.70
2	Three lane / one span loaded	180.07			0.180	32.41	241.20	23.30	755.10	503.40
3	One lane / Both span loaded	117.08			0.180	21.08	241.20	23.30	490.97	327.31
4	Three lane / Both span loaded	234.17			0.180	42.15	241.20	23.30	981.94	654.62
	For IRC class 70R Tracked									
5	One span loaded		131.97		0.180	23.75	241.20	23.30	553.37	368.92
6	Both span loaded		139.85		0.180	25.17	241.20	23.30	586.42	390.94
	For IRC class 70R Wheeled									
7	One span loaded		179.16		0.180	32.25	241.20	23.30	751.25	500.83
8	Both span loaded		200.00		0.180	36.00	241.20	23.30	838.66	559.10
	IRC CLASS SV LOADING									
9	Two lane / one span loaded			0	0.180	0	241.20	23.30	0.00	0.00
10	Two lane / Both span loaded			0	0.180	0	241.20	23.30	0.00	0.00

All the actions at pier base analised above are summarised bellow, loads are in KN

Sl.NO	LOAD DESCRIPITION	V	HL	HT	LA	ML	MT
A	**PERMANENT LOADS**						
i	Dead Load from super-structure	10806.53					
ii	Self weight of Pier Cap	1761.50					
iii	Self weight of pier	8694.29					
	VARIABLE GRAVITY TREATED AS PERMANEN						
i	SIDL except surfacing	1521.00					
ii	Surfacing	786.84					
B	**VARIABLE LOAD**						
	Carriageway Live Load						
a	***For Class-A***						
i	One lane / one span loaded	495.65				545.22	1710.00
ii	Two lane / one span loaded	991.31				1090.44	1685.22
iii	One lane / Both span loaded	644.55				86.39	2223.70
iv	Two lane / Both span loaded	1289.10				172.78	2191.47
b	***For IRC class 70R Tracked***						
i	One span loaded	725.82				798.40	1524.22
ii	Both span loaded	769.16				0.00	1615.23
c	***For IRC class 70R Wheeled***						
i	One span loaded	986.25				1084.88	2125.37
ii	Both span loaded	1101.00				74.91	2372.66
d	***For IRC SV***						
i	One lane / one span loaded	2299.06				2528.96	689.72
ii	One lane / Both span loaded	1885.40				1456.46	565.62
	Braking /Friction Force						
a	***For Class-A***						
i	One lane / one span loaded	30.92	357.60		18.990	6790.74	
ii	Two lane / one span loaded	54.44	487.53		18.990	9258.29	
iii	One lane / Both span loaded	30.92	383.00		18.990	7273.09	
iv	Two lane / Both span loaded	54.44	598.07		18.990	11357.33	
b	***For IRC class 70R Tracked***						
i	One span loaded	16.20	394.20		18.990	7485.78	
ii	Both span loaded	16.20	394.20		18.990	7485.78	
c	***For IRC class 70R Wheeled***						
i	One span loaded	23.14	424.20		18.990	8055.48	
ii	Both span loaded	23.14	424.20		18.990	8055.48	
d	***For IRC SV***						
i	One lane / one span loaded	0.00	393.17		18.990	7466.25	
ii	One lane / Both span loaded	0.00	380.76		18.990	7230.59	
	WIND LOAD						
	Wind load from super-structure	602.67	129.27	517.09	20.17	2607.83	10431.32
	Wind load from sub-structure	0.00	30.79	85.34	9.13	280.98	778.77
	SEISMIC EFFECTS						
	on superstructure except surfacing	2810.68	3328.43	3328.43		54779.33	54779.33
	on pier cap	401.62	475.61	475.61		7114.42	7114.42
	on pier	1982.30	2347.46	2347.46		16530.00	16530.00
	On surfacing/ wearing coat	179.40	212.45	212.45		3496.46	3496.46
	Vertical push-pull	54.21					
	for Live load						
	For Class-A						
	One lane / one span loaded	251.70		16.21	23.30		377.55
	Three lane / one span loaded	503.40		32.41	23.30		755.10
	One lane / Both span loaded	327.31		21.08	23.30		490.97
	Three lane / Both span loaded	654.62		42.15	23.30		981.94
	For IRC class 70R Tracked						
	One span loaded	368.92		23.75	23.30		553.37
	Both span loaded	390.94		25.17	23.30		586.42
	For IRC class 70R Wheeled						
	One span loaded	500.83		32.25	23.30		751.25
	Both span loaded	559.10		36.00	23.30		838.66
	HYDRAULIC LOAD						
	Water current force on shaft		0.00	40.87		0.00	0.00
	Buoyant force on shaft	1693.04					

Load combination at bottom of pile cap level

Sl.NO	LOAD DESCRIPITION	V	HL	HT	LA	ML	MT
A	**PERMANENT LOADS**						
i	Dead Load from super-structure	10806.53					
ii	Self weight of Pier Cap	1761.50					
iii	Self weight of pier	8694.29					
iv	Self weight of pile cap	10269.00					
	VARIABLE GRAVITY TREATED AS PERMANENT						
i	SIDL except surfacing	1521.00					
ii	Surfacing	786.84					
B	**VARIABLE LOAD**						
	Carriageway Live Load						
a	***Class-A***						
i	One lane / one span loaded	450.18				495.20	1553.13
ii	Two lane / one span loaded	900.37				990.40	1530.63
iii	One lane / Both span loaded	585.42				78.46	2019.71
iv	Two lane / Both span loaded	1170.84				156.93	1990.44
b	***For IRC class 70R Tracked***						
i	One span loaded	659.83				725.82	1385.65
ii	Both span loaded	699.23				0.00	1468.39
c	***For IRC class 70R Wheeled***						
i	One span loaded	895.78				985.36	1930.40
ii	Both span loaded	1000.00				68.04	2155.00
d	For IRC SV						
i	One lane / one span loaded	2299.06				2528.96	689.72
ii	One lane / Both span loaded	1885.40				1456.46	565.62
	Braking / Friction Force						
a	***Class-A***						
i	One lane / one span loaded	30.92	357.60		20.99	7505.93	
ii	Two lane / one span loaded	54.44	487.53		20.99	10233.36	
iii	One lane / Both span loaded	30.92	383.00		20.99	8039.08	
iv	Two lane / Both span loaded	54.44	598.07		20.99	12553.46	
b	***For IRC class 70R Tracked***						
i	One span loaded	16.20	394.20		20.99	8274.17	
ii	Both span loaded	16.20	394.20		20.99	8274.17	
c	***For IRC class 70R Wheeled***						
i	One span loaded	23.14	424.20		20.99	8903.87	
ii	Both span loaded	23.14	424.20		20.99	8903.87	
d	***For IRC SV***						
i	One lane / one span loaded	0.00	393.17		20.99	8252.58	
ii	One lane / Both span loaded	0.00	380.76		20.99	7992.10	
	THERMAL LOAD						
i	Temperature variation effect		0.00		2.00	0.00	
	WIND LOAD						
	Wind load from super-structure	602.67	129.27	517.09	22.17	2866.38	11465.51
	Wind load from sub-structure		30.79	85.34	11.13	342.57	949.45
	SEISMIC EFFECTS						
	on superstructure except surfacing	3513.34	4160.54	4160.54		75685.76	75685.76
	on pier cap	502.03	594.51	594.51		9923.50	9923.50
	on pier	2477.87	2934.32	2934.32		25748.65	25748.65
	On surfacing/ wearing coat	224.25	265.56	265.56		4830.87	4830.87
	on pile cap	2926.67	3465.79	3465.79		3003.68	3003.68
	Vertical Push-Pull	67.77					
	for Live load						
	For Class-A						
	One lane / one span loaded	314.62		20.26	25.30		512.45
	Three lane / one span loaded	629.25		40.52	25.30		1024.91
	One lane / Both span loaded	409.14		26.34	25.30		666.40
	Three lane / Both span loaded	818.28		52.69	25.30		1332.80
	For IRC class 70R Tracked						
	One span loaded	461.14		29.69	25.30		751.10
	Both span loaded	488.68		31.47	25.30		795.95
	For IRC class 70R Wheeled						
	One span loaded	626.04		40.31	25.30		1019.68
	Both span loaded	698.88		45.00	25.30		1138.32
	HYDRAULIC LOAD						
	Water current force on shaft		0.00	40.87	4.73	0.00	193.20
	Buoyant force on pile cap	4107.60					

LOAD COMBINATION FOR PIER SHAFT BASE (For Ultimate Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	21262.31	0.00	0.00	0.00	0.00
SIDL	1521.00	0.00	0.00	0.00	0.00
Surfacing	786.84	0.00	0.00	0.00	0.00
Class A(3L/1S) LL1	991.31	1090.44	1685.22	0.00	0.00
Class A(3L/BS) LL2	1289.10	172.78	2191.47	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL3	725.82	798.40	1524.22	0.00	0.00
70R Tr.1L+CL-A 1L(BS). LL4	769.16	0.00	1615.23	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL5	986.25	1084.88	2125.37	0.00	0.00
70R Wh.1L+CL-A 1L(BS) LL6	1101.00	74.91	2372.66	0.00	0.00
Class SV(1S) LL7	2299.06	2528.96	689.72	0.00	0.00
Class SV(BS) LL8	1885.40	1456.46	565.62	0.00	0.00
BrakingClass A(3L/1S) LL1	54.44	9258.29	0.00	487.53	0.00
BrakingClass A(3L/BS) LL2	54.44	11357.33	0.00	598.07	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL3	16.20	7485.78	0.00	394.20	0.00
Braking70R Tr.1L+CL-A 1L(BS). LL4	16.20	7485.78	0.00	394.20	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL5	23.14	8055.48	0.00	424.20	0.00
Braking70R Wh.1L+CL-A 1L(BS) LL6	23.14	8055.48	0.00	424.20	0.00
Friction Class SV(1S) LL7	0.00	7466.25	0.00	393.17	0.00
Friction Class SV(BS) LL8	0.00	7230.59	0.00	380.76	0.00
Dead Load Seismic	5428.21	81920.20	0.00	6363.94	0.00
Seismic Class A(3L/1S) LL1	503.40	0.00	755.10	0.00	32.41
Seismic Class A(3L/BS) LL2	654.62	0.00	981.94	0.00	42.15
Seismic 70R Tr.1L+CL-A 1L(1S) LL3	368.92	0.00	553.37	0.00	23.75
Seismic 70R Tr.1L+CL-A 1L(BS). LL4	390.94	0.00	586.42	0.00	25.17
Seismic 70R Wh.1L+CL-A 1L(1S) LL5	500.83	0.00	751.25	0.00	32.25
Seismic 70R Wh.1L+CL-A 1L(BS) LL6	559.10	0.00	838.66	0.00	36.00
Wind load	602.67	2888.82	11210.09	160.07	602.44
Water Current force	-1693.04	0.00	0.00	0.00	40.87

NON-SEISMIC CASE

A

HFL / DRY CONDITION	
DL+SIDL+Surfacing+LL+Br. LL+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.50
Braking LL	1.15
Water Current force	1.00
Special vehicle	1.0

B

HFL / DRY CONDITION	WIND
DL+SIDL+Surfacing+/-WL+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.50
Wind load	1.50
Water Current force	1.00
Special vehicle	1.0

C

ONE SPAN DISLODGED CASE	
DL+SIDL+Surfacing+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
Water Current force	1

SEISMIC CASE

A

HFL / DRY CONDITION	
DL+SIDL+Surfacing+LL+Br. LL+Sis. LL+WCF+DL SEISMIC	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	0.2
Braking LL	0.2
Seismic LL	1.5
DL Seismic	1.5
WCF	1

`

B

ONE SPAN DISLODGED CASE	
DL+SIDL+Surfacing+WCF (Non-seismic/Seismic)	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
Water Current force	1
DL Seismic	1.5

			V_u	ML_u	MT_u	HL_u	Ht_u
HFL Condition Non Seismic	1	DL+SIDL+Surfacing+LL1+Braking LL1+WCF	31990.96	12282.68	2527.83	560.67	40.87
	2	DL+SIDL+Surfacing+LL2+Braking LL2+WCF	32437.65	13320.09	3287.20	687.78	40.87
	3	DL+SIDL+Surfacing+LL3+Braking LL3+WCF	31548.76	9806.24	2286.32	453.33	40.87
	4	DL+SIDL+Surfacing+LL4+Braking LL4+WCF	31613.77	8608.64	2422.84	453.33	40.87
	5	DL+SIDL+Surfacing+LL5+Braking LL5+WCF	31947.40	10891.11	3188.06	487.83	40.87
	6	DL+SIDL+Surfacing+LL6+Braking LL6+WCF	32119.52	9376.17	3558.98	487.83	40.87
	7	DL+SIDL+Surfacing+LL7+Braking LL7+WCF	32740.46	2528.96	689.72	0.00	40.87
	8	DL+SIDL+Surfacing+LL8+Braking LL8+WCF	32326.80	1456.46	565.62	0.00	40.87
	9	DL+SIDL+Surfacing+LL1+Braking LL1+WCF+WL(UP)	31086.95	16615.91	19342.96	800.76	944.53
	10	DL+SIDL+Surfacing+LL2+Braking LL2+WCF+WL(UP)	31533.65	17653.31	20102.33	927.88	944.53
	11	DL+SIDL+Surfacing+LL3+Braking LL3+WCF+WL(UP)	30644.75	14139.46	19101.45	693.42	944.53
	12	DL+SIDL+Surfacing+LL4+Braking LL4+WCF+WL(UP)	30709.76	12941.87	19237.97	693.42	944.53
	13	DL+SIDL+Surfacing+LL5+Braking LL5+WCF+WL(UP)	31043.39	15224.34	20003.19	727.92	944.53
	14	DL+SIDL+Surfacing+LL6+Braking LL6+WCF+WL(UP)	31215.51	13709.39	20374.11	727.92	944.53
DRY Condition Non Seismic	15	DL+SIDL+Surfacing+LL1+Braking LL1	33684.01	12282.68	2527.83	560.67	0.00
	16	DL+SIDL+Surfacing+LL2+Braking LL2	34130.70	13320.09	3287.20	687.78	0.00
	17	DL+SIDL+Surfacing+LL3+Braking LL3	33241.80	9806.24	2286.32	453.33	0.00
	18	DL+SIDL+Surfacing+LL4+Braking LL4	33306.81	8608.64	2422.84	453.33	0.00
	19	DL+SIDL+Surfacing+LL5+Braking LL5	33640.44	10891.11	3188.06	487.83	0.00
	20	DL+SIDL+Surfacing+LL6+Braking LL6	33812.56	9376.17	3558.98	487.83	0.00
	21	DL+SIDL+Surfacing+LL7	34433.50	2528.96	689.72	0.00	0.00
	22	DL+SIDL+Surfacing+LL8	34019.84	1456.46	565.62	0.00	0.00
	23	DL+SIDL+Surfacing+LL1+Braking LL1+WL(UP)	32780.00	16615.91	19342.96	800.76	903.66
	24	DL+SIDL+Surfacing+LL2+Braking LL2+WL(UP)	33226.69	17653.31	20102.33	927.88	903.66
	25	DL+SIDL+Surfacing+LL3+Braking LL3+WL(UP)	32337.79	14139.46	19101.45	693.42	903.66
	26	DL+SIDL+Surfacing+LL4+Braking LL4+WL(UP)	32402.80	12941.87	19237.97	693.42	903.66
	27	DL+SIDL+Surfacing+LL5+Braking LL5+WL(UP)	32736.43	15224.34	20003.19	727.92	903.66
	28	DL+SIDL+Surfacing+LL6+Braking LL6+WL(UP)	32908.55	13709.39	20374.11	727.92	903.66
HFL Condition Seismic	29	DL+SIDL+Surfacing+LL1+Br. LL1+Sis. LL1+WCF+DL SEISMIC	30228.57	124950.04	337.04	9643.42	40.87
	30	DL+SIDL+Surfacing+LL2+Br. LL2+Sis. LL2+WCF+DL SEISMIC	30288.13	125186.32	438.29	9665.52	40.87
	31	DL+SIDL+Surfacing+LL3+Br. LL3+Sis. LL3+WCF+DL SEISMIC	30167.83	124537.13	304.84	9624.75	40.87
	32	DL+SIDL+Surfacing+LL4+Br. LL4+Sis. LL4+WCF+DL SEISMIC	30176.49	124377.45	323.05	9624.75	40.87
	33	DL+SIDL+Surfacing+LL5+Br. LL5+Sis. LL5+WCF+DL SEISMIC	30221.30	124708.37	425.07	9630.75	40.87
	34	DL+SIDL+Surfacing+LL6+Br. LL6+Sis. LL6+WCF+DL SEISMIC	30244.25	124506.38	474.53	9630.75	40.87
DRY Condition Seismic	35	DL+SIDL+Surfacing+LL1+Br. LL1+Sis. LL1+DL SEISMIC	31921.61	124950.04	337.04	9643.42	0.00
	36	DL+SIDL+Surfacing+LL2+Br. LL2+Sis. LL2+DL SEISMIC	31981.17	125186.32	438.29	9665.52	0.00
	37	DL+SIDL+Surfacing+LL3+Br. LL3+Sis. LL3+DL SEISMIC	31860.87	124537.13	304.84	9624.75	0.00
	38	DL+SIDL+Surfacing+LL4+Br. LL4+Sis. LL4+DL SEISMIC	31869.54	124377.45	323.05	9624.75	0.00
	39	DL+SIDL+Surfacing+LL5+Br. LL5+Sis. LL5+DL SEISMIC	31914.34	124708.37	425.07	9630.75	0.00
	40	DL+SIDL+Surfacing+LL6+Br. LL6+Sis. LL6+DL SEISMIC	31937.29	124506.38	474.53	9630.75	0.00
No LL	41	DL+SIDL+Surfacing+DL SEISMIC	31712.47	122880.30	0.00	9545.91	0.00
One Span dislodged	42	DL+SIDL+Surfacing	14374.18	17673.95	0.00	0.00	40.87
	43	DL+SIDL+Surfacing+DL Seismic	23363.01	87256.40	0.00	9545.91	40.87

LOAD COMBINATION FOR PIER SHAFT BASE (For Serviceability Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	21262.31	0.00	0.00	0.00	0.00
SIDL	1521.00	0.00	0.00	0.00	0.00
Surfacing	786.84	0.00	0.00	0.00	0.00
Class A(3L/1S) LL1	991.31	1090.44	1685.22	0.00	0.00
Class A(3L/BS) LL2	1289.10	172.78	2191.47	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL3	725.82	798.40	1524.22	0.00	0.00
70R Tr.1L+CL-A 1L(BS). LL4	769.16	0.00	1615.23	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL5	986.25	1084.88	2125.37	0.00	0.00
70R Wh.1L+CL-A 1L(BS) LL6	1101.00	74.91	2372.66	0.00	0.00
Class SV(1S) LL7	2299.06	2528.96	689.72	0.00	0.00
Class SV(BS) LL8	1885.40	1456.46	565.62	0.00	0.00
BrakingClass A(3L/1S) LL1	54.44	9258.29	0.00	487.53	0.00
BrakingClass A(3L/BS) LL2	54.44	11357.33	0.00	598.07	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL3	16.20	7485.78	0.00	394.20	0.00
Braking70R Tr.1L+CL-A 1L(BS). LL4	16.20	7485.78	0.00	394.20	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL5	23.14	8055.48	0.00	424.20	0.00
Braking70R Wh.1L+CL-A 1L(BS) LL6	23.14	8055.48	0.00	424.20	0.00
Dead Load Seismic	5428.21	81920.20	0.00	6363.94	0.00
Seismic Class A(3L/1S) LL1	503.40	0.00	755.10	0.00	32.41
Seismic Class A(3L/BS) LL2	654.62	0.00	981.94	0.00	42.15
Seismic 70R Tr.1L+CL-A 1L(1S) LL3	368.92	0.00	553.37	0.00	23.75
Seismic 70R Tr.1L+CL-A 1L(BS). LL4	390.94	0.00	586.42	0.00	25.17
Seismic 70R Wh.1L+CL-A 1L(1S) LL5	500.83	0.00	751.25	0.00	32.25
Seismic 70R Wh.1L+CL-A 1L(BS) LL6	559.10	0.00	838.66	0.00	36.00
Wind load	602.67	2888.82	11210.09	160.07	602.44
Water Current force	-1693.04	0.00	0.00	0.00	40.87

NON-SEISMIC CASE

A

HFL / DRY CONDITION

DL+SIDL+Surfacing+LL+Br. LL+WCF

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	1.00
Braking LL	0.75
Water Current force	1.00
Special vehicle	1.0

B

HFL / DRY CONDITION

DL+SIDL+Surfacing+/-WL+WCF

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
Wind load	1.00
Water Current force	1.00
Special vehicle	1.0

C

ONE SPAN DISLODGED CASE

DL+SIDL+Surfacing+WCF

Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
Water Current force	1.00
Special vehicle	1.0

			Vu	MLu	MTu	HLu	Htu
HFL Condition Non Seismic	1	DL+SIDL+Surfacing+LL1+Braking LL1+WCF	22909.25	8034.15	1685.22	365.65	40.87
	2	DL+SIDL+Surfacing+LL2+Braking LL2+WCF	23207.04	8690.77	2191.47	448.55	40.87
	3	DL+SIDL+Surfacing+LL3+Braking LL3+WCF	22615.08	6412.73	1524.22	295.65	40.87
	4	DL+SIDL+Surfacing+LL4+Braking LL4+WCF	22658.42	5614.33	1615.23	295.65	40.87
	5	DL+SIDL+Surfacing+LL5+Braking LL5+WCF	22880.72	7126.49	2125.37	318.15	40.87
	6	DL+SIDL+Surfacing+LL6+Braking LL6+WCF	22995.47	6116.52	2372.66	318.15	40.87
	7	DL+SIDL+Surfacing+LL7+WCF	24176.17	2528.96	689.72	0.00	40.87
	8	DL+SIDL+Surfacing+LL8+WCF	23762.51	1456.46	565.62	0.00	40.87
	9	DL+SIDL+Surfacing+LL1+Braking LL1+WCF+WL(UP)	22306.57	10922.97	12895.31	525.72	643.31
	10	DL+SIDL+Surfacing+LL2+Braking LL2+WCF+WL(UP)	22604.37	11579.58	13401.56	608.62	643.31
	11	DL+SIDL+Surfacing+LL3+Braking LL3+WCF+WL(UP)	22012.41	9301.55	12734.30	455.71	643.31
	12	DL+SIDL+Surfacing+LL4+Braking LL4+WCF+WL(UP)	22055.75	8503.15	12825.32	455.71	643.31
	13	DL+SIDL+Surfacing+LL5+Braking LL5+WCF+WL(UP)	22278.05	10015.30	13335.46	478.21	643.31
	14	DL+SIDL+Surfacing+LL6+Braking LL6+WCF+WL(UP)	22392.80	9005.34	13582.74	478.21	643.31
DRY Condition Non Seismic	15	DL+SIDL+Surfacing+LL1+Braking LL1	24602.29	8034.15	1685.22	365.65	0.00
	16	DL+SIDL+Surfacing+LL2+Braking LL2	24900.08	8690.77	2191.47	448.55	0.00
	17	DL+SIDL+Surfacing+LL3+Braking LL3	24308.12	6412.73	1524.22	295.65	0.00
	18	DL+SIDL+Surfacing+LL4+Braking LL4	24351.46	5614.33	1615.23	295.65	0.00
	19	DL+SIDL+Surfacing+LL5+Braking LL5	24573.77	7126.49	2125.37	318.15	0.00
	20	DL+SIDL+Surfacing+LL6+Braking LL6	24688.51	6116.52	2372.66	318.15	0.00
	21	DL+SIDL+Surfacing+LL7	25869.21	2528.96	689.72	0.00	0.00
	22	DL+SIDL+Surfacing+LL8	25455.55	1456.46	565.62	0.00	0.00
	23	DL+SIDL+Surfacing+LL1+Braking LL1+WL(UP)	23999.61	10922.97	12895.31	525.72	602.44
	24	DL+SIDL+Surfacing+LL2+Braking LL2+WL(UP)	24297.41	11579.58	13401.56	608.62	602.44
	25	DL+SIDL+Surfacing+LL3+Braking LL3+WL(UP)	23705.45	9301.55	12734.30	455.71	602.44
	26	DL+SIDL+Surfacing+LL4+Braking LL4+WL(UP)	23748.79	8503.15	12825.32	455.71	602.44
	27	DL+SIDL+Surfacing+LL5+Braking LL5+WL(UP)	23971.09	10015.30	13335.46	478.21	602.44
	28	DL+SIDL+Surfacing+LL6+Braking LL6+WL(UP)	24085.84	9005.34	13582.74	478.21	602.44
One Span dislodged	29	DL+SIDL+Surfacing+WCF (Non-seismic)	10092.04	0.00	0.00	0.00	40.87

PIER SHAFT DESIGN(ULS)

Total Ultimate Loads (Loads in KN, moments in KN-m)

Load Case		V_u	ML_u	MT_u
30	Maximum longitudinal Moment case	30288.13	125186.32	438.29
21	Maximum vertical load case	34433.50	2528.96	689.72
31	Minimum vertical load case	30167.83	124537.13	304.84
42	1 Span dislodged case	23363.01	87256.40	0.00

Section chech at pier base

Pier Type = Wall type with semicircular end
Depth of pier = 3.00 m
Length of pier in transverse direction = 8.000 m
Length of pier = (semi-circular portion) (on each end) 1.50 m
Pier steam thickness at bottom = 3.00 m
Area of section = 31.07 m^2

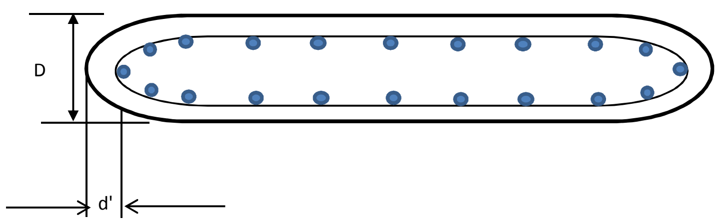
Grade of concrete : M 30
Grade of steel = Fe 500
E_{cm} of concrete = 31000 N/mm^2 (From table 6.5, IRC:112-2011, page no. 38)
E_s of steel = 200000 N/mm^2 (From clause6.3.5, IRC:112-2011, page no. 32)
Design compressive strength of concrete = $s_c = f_{cd} = af_{ck}/g_m$ = 13.400 N/mm^2
(From clause6.3.5, IRC:112-2011, page no. 32)
Design peak strength of steel = f_y/g_s = 434.783 N/mm^2

Slenderness criteria check:

Clear height of pier shaft = 16.25 m (upto pier cap top)
Effective length, $l_e = 1.3 l_0$ = 21.125 m (Table 11.1, case-4, IRC:112-2011, page-114)

Now thickness of the wall, t = 3 m
Ratio of effective length to its thickness, l_e/t = 7.04

As the ratio does not exceed 12, it is short and no secondary effect to be considered
(clause 7.6.4, IRC:112-2011, page-57)
Effective cover = 101 mm



1 **Analysis of section longitudinal direction :** (Check for load combination case 30)

Provide	32 mm dia bar @	150 mm c/c		
Provide	32 mm dia. Bar	53 nos	42603.52 mm^2	
+	32 mm dia. Bar	53 nos	42603.52 mm^2	
and	32 mm dia. Bar	20 nos	16076.8 mm^2	
+	32 mm dia. Bar	20 nos	16076.8 mm^2	

Effective cover = 101 mm
Total area of steel = 117360.64 mm^2
% of steel = 0.38

Interaction check

(M_{Edx}/M_{Rdx})^a+(M_{Edy}/M_{Rdy})^a<= 1 (Eq. 8.3, IRC:112-2011,page-75)

					Load Case =	30	21	31	42
P_u =	Design shear force =				KN	30288.13	34433.50	30167.83	23363.01
M_{Edx} =	Design moment in longitudinal direction =				KN-m	125186.32	2529.0	124537.1	87256.4
M_{Edy} =	Design moment in transeverse direction =				KN-m	438.29	689.72	304.84	0.00
M_{Rdx} =	Resisting moment in longitudinal direction (From M-L Curve) =				KN-m	130800.00	150000.00	130700.00	110400.00
M_{Rdy} =	Resisting moment in transeverse direction (From M-T Curve) =				KN-m	70000.0	72000.0	69500.0	60000.0
N_{Ed} =	Design axial force =				KN	30288.13	34433.50	30167.83	23363.01
N_{Rd} =	Design axial resistance =				KN	452223.82	452223.82	452223.8	452223.82
N_{Ed}/N_{Rd} =						0.10	0.1	0.10	0.1
Type of cross section of abutment =						Rectangular	Rectangular	Rectangular	Rectangular
a =						1.00	1.00	1.00	1.00
(M_{Edx}/M_{Rdx})^a+(M_{Edy}/M_{Rdy})^a =						0.96	0.03	0.96	0.79
Check =						**OK**	**OK**	**OK**	**OK**

PIER SHAFT DESIGN(SLS)

Load Case		**Vsls**	**MLsls**	**MTsls**
10	Maximum longitudinal Moment case	22604.37	11579.6	13401.56
11	Minimum vertical load case	22012.41	9301.5	12734.30
29	1 Span dislodged case	10092.04	0.0	0.00

Stress level check:

Grade of concrete = M 30
Grade of steel = Fe 500
Width of section considered = 1 m

Section is checked for SLS
Design moment = 1447.45 KN-m (for 1m width)
Width of section = 1 m
Depth of section = 3.00 m
"E" value of steel = 200000 Mpa
"E" value of concrete = 31000 Mpa
Modular ratio in tension = 9.3
Concrete failure strain = 0.0035
Maximum allowable stress in concrete = $0.48f_{ck}$ = 14.4 Mpa
(Clause 12.2.1(1), IRC:112-2011, page-120)
Maximum allowable stress in steel = $0.8f_{yk}$ = 400 Mpa
(Clause 12.2.2, IRC:112-2011, page-120)

Total reinforcement provided in 1 m width = 5325 mm^2
Effective depth "d" = 2899 mm
Netral axis depth = x = 214.50 mm
CG of compressive force = 89.230 mm from most compressed surface
Moment , Mu =σst*Ast*(d-0.416*xu) = 6509 OK

So, stress in steel = 96.7 Mpa OK, within permissible limit
Total force = 515.1 KN
Stress in concrete = 4.8 Mpa OK, within permissible limit

Crack width check:

Total depth of effective rectangular section= 3000 mm
Effective depth= 2899 mm
Area of tensile reinforcement provided= 42604 sqmm
Crack width, $W_k = S_{r.max}(\hat{O}_{sm}-\hat{O}_{cm})$ Where, $S_{r.max}$ = Maximum crack spacing
$\hat{O}_{sm}$ = mean strain in the reinforcement under the relavant combination of loads
$\hat{O}_{cm}$ = mean strain in the concrete between cracks.

Now,

(Eq. 12.6, IRC:112-2011, page-125)

Where, s_{sc} = stress in the tension reinforcement = 96.73 Mpa
$a_e = E_s/E_{cm}$ = 6.45
$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.9 Mpa
$r_{r.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$
Where, $h_{c.eff}$ = lesser of the followings
2.5(h-d);(h-x/3);or h/2

Where, A = level of steel centroid
B = Effective tension area, $A_{c.eff}$
$\hat{O}_1,\hat{O}_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 253 mm
$A_{c.eff}$ = 252500 mm^2
Now. $r_{r.eff} = A_s/A_{c.eff}$ = 0.1687268
k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+f/2)), the maximum crack spacing,

$$Sr.\max = 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{p.eff}}$$

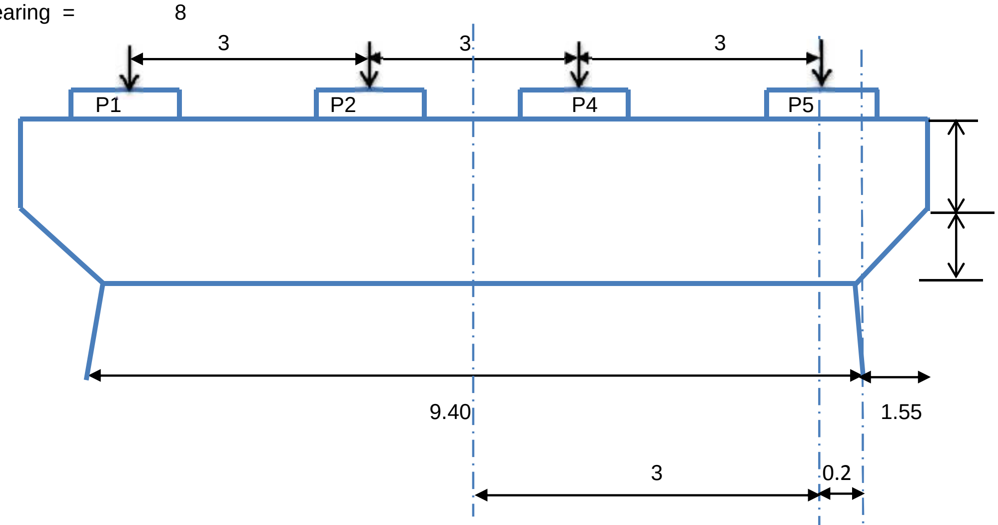
Where, f = diameter of bar = 32 mm c = clear cover = 75 mm
k1 = co-efficient taking acount of bond properties of reinforcement = 0.8
k2 = co- efficient taking account of distribution of strain = 0.5
So,$S_{r.max}$ = 287.241 mm

And, $\hat{O}_{sm}-\hat{O}_{cm}$ = 0.000393924
Minimum value of $\hat{O}_{sm} - \hat{O}_{cm}$ = 0.0002902
So, governing value of $\hat{O}_{sm} - \hat{O}_{cm}$ = 0.00039392

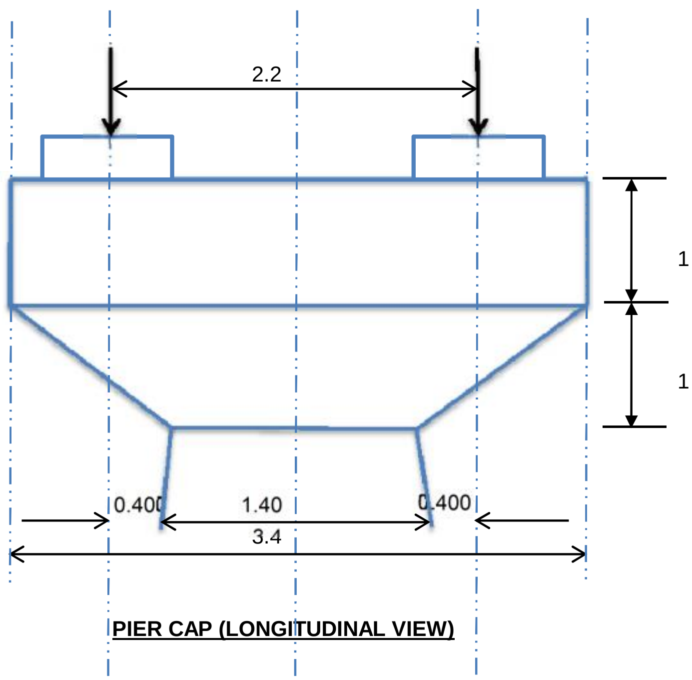
So, crack width, $W_k = S_{r.max}(\hat{O}_{sm}-\hat{O}_{cm})$ = 0.113 mm
Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)
crack width within permissible limit

Design of Pier Cap

No. of Bearing = 8



PIER CAP (TRANSVERSE VIEW)



PIER CAP (LONGITUDINAL VIEW)

Checking for transverse direction :

a_c =	0.200 m		
h =	2.00 m		
a_c / h =	0.1	< 1	**Hence pier cap is to be designed as corbel**

Checking for longitudinal direction :

a_c =	0.400 m		
h =	2.00 m		
a_c / h =	0.2	< 1	**Hence pier cap is to be designed as corbel**

CORBEL DESIGN:

Distance of central line of beraing from face of pier shaft = 0.400 m

Pier Cap thickness at face of pier shaft= 2 m

Which is less than the pier cap thickness at face of pier shaft .

hence the pier cap will be designed as a corbel.

Distance of load from face of pier support a = 400 mm

Considering dispersion of loads across span upto mid depth of uniform section of pier cap .

Taken width of corbel as =	3400	mm
Depth of corbel h =	2000	mm
Top uniform section s =	1000	mm
Considering clear cover to reinforcement as	50	mm
Dia of main bar =	25	mm
Calculated d' =	1937	mm

Grade of concrete used M30

Vertical reaction :

i) Dead load reaction on pier =	14875.87	KN
ii)Live load reaction on pier =	2299.06	KN
iii)Horrizontal Force=	598	KN
iv)Distribution factor for live load =	0.278	
v)Coefficient of friction at bearing =	0.03	
vi)Horizontal seismic coefficient =	0.180	

Vu =	Shear force=	3467.285	KN
Hu =	Horizental tensile force=	76.209	KN

Step I s/d' 0.5163 > 0.5 **ok**

Step II

d = 0.8 d' =	1549.6	mm			
Vu /bd =	0.658	N /mm2			
fc' =	24	N /mm2			
0.15 fc' =	3.6	N /mm2	>	0.658	**OK**

For monolithic construction μ = 1.4

fy = 500 N / mm^2

Design for non seismic case :

Step III Avf = Vu/0.85 fsy μ = 5827.369 mm^2 (μ = 1.4 for concrete placed monolithically across interface)

Step IV At = Hu/ 0.85 fsy = 179.315 mm^2

Step V Af = [Vu a + Hu (h - d')]/0.85 fsy d = 2113.206 mm^2

Step VI Total primary reinforcement will be max of the following :

i) Af + At =	2292.521	mm^2
ii)(2/3)Avf + At =	4064.2279	mm^2
iii)(0.04xfc'/fsy)x(bxd') =	12644.736	mm^2

Provide **25** mm tor bar @ **125** mm c/c

Ast provided (As)= 13836 mm^2 **OK**

Step VII

Ah =	1056.603	mm^2
Avf /3 =	1942.456	mm^2

Provide 6 legged 16 m dia stirrup in 4 layers

top (2/3 d') depth= 1.292 m

Ast provided = **4825.49** mm^2 **OK**

Design for seismic case :

i) Dead load reaction on pier =	2677.66	KN
ii) Live load reaction on pier =	234.17	KN
iii) Horrizontal Force=	42.15	KN

Vu =	381.541	KN
Hu =	11.601	KN

Step III	Avf =	641.245	mm^2
Step IV	At =	27.296	mm^2
Step V	Af =	2113.206	mm^2

Step VI Total primary reinforcement will be max of the following :

i) Af + At =	2140.502	mm^2
ii) (2/3)Avf + At =	454.79261	mm^2
iii) (0.04xfc'/fsu)x(bxd')	12644.736	mm^2

Provide **25** mm tor bar @ **125** mm c/c. in

Ast provided = **13345.000** mm^2 **OK**

Step VII	Ah =	1056.603	mm^2
	Avf /3 =	213.748	mm^2

Provide **25** mm tor bar @ **125** mm c/c

Ast provided = **4825.48632** mm^2 **OK**

Reinforcement provided in pier cap as follows :

At top along longitudinal direction provide **25** mm tor bar @ **125** mm c/c
At top along transverse direction provide **25** mm tor bar @ **125** mm c/c
At bottom along longitudinal direction provide **16** mm tor bar @ **125** mm c/c
At bottom along transverse direction provide **16** mm tor bar @**125** mm c/c
Provide **16**mm dia **6** L Stirrup **4** layers

D. LOAD COMBINATION FOR ABUTMENT FOUNDATION BASE (Ultimate Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	31531.31	0.00	0.00	0.00	0.00
SIDL	1521.00	0.00	0.00	0.00	0.00
Surfacing	786.84	0.00	0.00	0.00	0.00
Class A(3L/1S) LL1	900.37	990.40	1530.63	0.00	0.00
Class A(3L/BS) LL2	1170.84	156.93	1990.44	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL3	659.83	725.82	1385.65	0.00	0.00
70R Tr.1L+CL-A 1L(BS). LL4	769.16	0.00	1615.23	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL5	986.25	1084.88	2125.37	0.00	0.00
70R Wh.1L+CL-A 1L(BS) LL6	1101.00	74.91	2372.66	0.00	0.00
Class SV(1S) LL7	2299.06	2528.96	689.72	0.00	0.00
Class SV(BS) LL8	1885.40	1456.46	565.62	0.00	0.00
BrakingClass A(3L/1S) LL1	54.44	10233.36	0.00	487.53	0.00
BrakingClass A(2L/BS) LL2	54.44	12553.46	0.00	598.07	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL3	16.20	8274.17	0.00	394.20	0.00
Braking70R Tr.1L+CL-A 1L(BS). LL4	16.20	8274.17	0.00	394.20	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL5	23.14	8903.87	0.00	424.20	0.00
Braking70R Wh.1L+CL-A 1L(BS) LL6	23.14	8903.87	0.00	424.20	0.00
Friction Class SV(1S) LL7	0.00	8252.58	0.00	393.17	0.00
Friction Class SV(BS) LL8	0.00	7992.10	0.00	380.76	0.00
Dead Load Seismic	9711.92	119192.47	0.00	11420.71	0.00
Seismic Class A(2L/1S) LL1	629.25	0.00	1024.91	0.00	40.52
Seismic Class A(2L/BS) LL2	818.28	0.00	1332.80	0.00	52.69
Seismic 70R T(2L/1S) LL3	461.14	0.00	751.10	0.00	29.69
Seismic 70R T(2L/BS) LL4	488.68	0.00	795.95	0.00	31.47
Seismic 70R W(2L/1S) LL5	626.04	0.00	1019.68	0.00	40.31
Seismic 70R W(2L/BS) LL6	698.88	0.00	1138.32	0.00	45.00
Wind load	602.67	3208.95	12414.96	160.07	602.44
Water Current force	4107.60	0.00	193.20	0.00	40.87

NON-SEISMIC CASE

A

HFL / DRY CONDITION	
DL+SIDL+Surfacing+LL+Br. LL+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.50
Braking LL	1.15
Water Current force	1.00
Special vehicle	1.0

B

HFL / DRY CONDITION	
DL+SIDL+Surfacing+/-WL+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
LL	1.50
Wind load	1.50
Water Current force	1.00
Special vehicle	1.0

C

ONE SPAN DISLODGED CASE	
DL+SIDL+Surfacing+WCF	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
Water Current force	1
Special vehicle	1.0

SEISMIC CASE

A

HFL / DRY CONDITION	
DL+SIDL+Surfacing+LL+Br. LL+Sis. LL+WCF+DL SEISMIC	
Loads	FOS
Dead Load	1.00
SIDL	1.00
Surfacing	1.00
LL	0.20
Braking LL	0.20
Seismic LL	1.00
DL Seismic	1.00
WCF	1.00

B

ONE SPAN DISLODGED CASE	
DL+SIDL+Surfacing+WCF (Non-seismic/Seismic)	
Loads	FOS
Dead Load	1.35
SIDL	1.35
Surfacing	1.75
Water Current force	1.00
DL Seismic	1.50

	COMB		V	ML	MT	HL	HT
HFL Condition Non Seismic	1	DL+SIDL+Surfacing+LL1+Braking LL1+WCF	51518.35	13253.97	2489.14	560.67	40.87
	2	DL+SIDL+Surfacing+LL2+Braking LL2+WCF	51924.06	14671.87	3178.85	687.78	40.87
	3	DL+SIDL+Surfacing+LL3+Braking LL3+WCF	51113.58	10604.02	2271.67	453.33	40.87
	4	DL+SIDL+Surfacing+LL4+Braking LL4+WCF	51277.56	9515.29	2616.04	453.33	40.87
	5	DL+SIDL+Surfacing+LL5+Braking LL5+WCF	51611.19	11866.76	3381.26	487.83	40.87
	6	DL+SIDL+Surfacing+LL6+Braking LL6+WCF	51783.31	10351.82	3752.18	487.83	40.87
	7	DL+SIDL+Surfacing+LL7+WCF	52404.25	2528.96	882.92	0.00	40.87
	8	DL+SIDL+Surfacing+LL8+WCF	51990.59	1456.46	758.82	0.00	40.87
	9	DL+SIDL+Surfacing+LL1+Braking LL1+WCF+WL	52422.36	18067.39	21111.58	800.76	944.53
	10	DL+SIDL+Surfacing+LL2+Braking LL2+WCF+WL	52828.07	19485.29	21801.29	927.88	944.53
	11	DL+SIDL+Surfacing+LL3+Braking LL3+WCF+WL	52017.59	15417.44	20894.12	693.42	944.53
	12	DL+SIDL+Surfacing+LL4+Braking LL4+WCF+WL	52181.57	14328.72	21238.48	693.42	944.53
	13	DL+SIDL+Surfacing+LL5+Braking LL5+WCF+WL	52515.20	16680.19	22003.70	727.92	944.53
	14	DL+SIDL+Surfacing+LL6+Braking LL6+WCF+WL	52687.32	15165.24	22374.62	727.92	944.53
DRY Condition Non Seismic	15	DL+SIDL+Surfacing+LL1+Braking LL1	47410.75	13253.97	2295.94	560.67	0.00
	16	DL+SIDL+Surfacing+LL2+Braking LL2	47816.46	14671.87	2985.65	687.78	0.00
	17	DL+SIDL+Surfacing+LL3+Braking LL3	47005.98	10604.02	2078.48	453.33	0.00
	18	DL+SIDL+Surfacing+LL4+Braking LL4	47169.96	9515.29	2422.84	453.33	0.00
	19	DL+SIDL+Surfacing+LL5+Braking LL5	47503.59	11866.76	3188.06	487.83	0.00
	20	DL+SIDL+Surfacing+LL6+Braking LL6	47675.71	10351.82	3558.98	487.83	0.00
	21	DL+SIDL+Surfacing+ LL7	48296.65	2528.96	689.72	0.00	0.00
	22	DL+SIDL+Surfacing+ LL8	47882.99	1456.46	565.62	0.00	0.00
	23	DL+SIDL+Surfacing+LL1+Braking LL1+WL	48314.76	18067.39	20918.38	800.76	903.66
	24	DL+SIDL+Surfacing+LL2+Braking LL2+WL	48720.47	19485.29	21608.10	927.88	903.66
	25	DL+SIDL+Surfacing+LL3+Braking LL3+WL	47909.99	15417.44	20700.92	693.42	903.66
	26	DL+SIDL+Surfacing+LL4+Braking LL4+WL	48073.97	14328.72	21045.29	693.42	903.66
	27	DL+SIDL+Surfacing+LL5+Braking LL5+WL	48407.60	16680.19	21810.50	727.92	903.66
	28	DL+SIDL+Surfacing+LL6+Braking LL6+WL	48579.72	15165.24	22181.42	727.92	903.66
HFL Condition Seismic	29	DL+SIDL+Surfacing+LL1+Br. LL1+Sis. LL1+WCF+DL SEISMIC	47849.64	121437.22	499.32	11518.22	40.87
	30	DL+SIDL+Surfacing+LL2+Br. LL2+Sis. LL2+WCF+DL SEISMIC	47903.73	121734.54	591.29	11540.32	40.87
	31	DL+SIDL+Surfacing+LL3+Br. LL3+Sis. LL3+WCF+DL SEISMIC	47793.89	120992.46	470.33	11499.55	40.87
	32	DL+SIDL+Surfacing+LL4+Br. LL4+Sis. LL4+WCF+DL SEISMIC	47815.75	120847.30	516.24	11499.55	40.87
	33	DL+SIDL+Surfacing+LL5+Br. LL5+Sis. LL5+WCF+DL SEISMIC	47860.56	121190.21	618.27	11505.55	40.87
	34	DL+SIDL+Surfacing+LL6+Br. LL6+Sis. LL6+WCF+DL SEISMIC	47883.51	120988.22	667.73	11505.55	40.87
DRY Condition Seismic	35	DL+SIDL+Surfacing+LL1+Br. LL1+Sis. LL1+DL SEISMIC	43742.04	121437.22	306.13	11518.22	0.00
	36	DL+SIDL+Surfacing+LL2+Br. LL2+Sis. LL2+DL SEISMIC	43796.13	121734.54	398.09	11540.32	0.00
	37	DL+SIDL+Surfacing+LL3+Br. LL3+Sis. LL3+DL SEISMIC	43686.29	120992.46	277.13	11499.55	0.00
	38	DL+SIDL+Surfacing+LL4+Br. LL4+Sis. LL4+DL SEISMIC	43708.15	120847.30	323.05	11499.55	0.00
	39	DL+SIDL+Surfacing+LL5+Br. LL5+Sis. LL5+DL SEISMIC	43752.96	121190.21	425.07	11505.55	0.00
	40	DL+SIDL+Surfacing+LL6+Br. LL6+Sis. LL6+DL SEISMIC	43775.91	120988.22	474.53	11505.55	0.00
No LL	41	DL+SIDL+Surfacing+DL SEISMIC	43551.08	119192.47	0.00	0.00	0.00
One Span dislodged	42	DL+SIDL+Surfacing	27106.40	25298.68	193.20	0.00	40.87
	43	DL+SIDL+Surfacing+DL Seismic	34390.34	122705.36	193.20	0.00	40.87

H. LOAD COMBINATION FOR ABUTMENT FOUNDATION BASE (Servicebility Limit State)

Loads	V	ML	MT	HL	HT
Dead Load	31531.31	0.00	0.00	0.00	0.00
SIDL	1521.00	0.00	0.00	0.00	0.00
Surfacing	786.84	0.00	0.00	0.00	0.00
Class A(3L/1S) LL1	900.37	990.40	1530.63	0.00	0.00
Class A(3L/BS) LL2	1170.84	156.93	1990.44	0.00	0.00
70R Tr.1L+CL-A 1L(1S) LL3	659.83	725.82	1385.65	0.00	0.00
70R Tr.1L+CL-A 1L(BS). LL4	769.16	0.00	1615.23	0.00	0.00
70R Wh.1L+CL-A 1L(1S) LL5	986.25	1084.88	2125.37	0.00	0.00
70R Wh.1L+CL-A 1L(BS) LL6	1101.00	74.91	2372.66	0.00	0.00
Class SV(1S) LL7	2299.06	2528.96	689.72	0.00	0.00
Class SV(BS) LL8	1885.40	1456.46	565.62	0.00	0.00
BrakingClass A(3L/1S) LL1	54.44	10233.36	0.00	487.53	0.00
BrakingClass A(3L/BS) LL2	54.44	12553.46	0.00	598.07	0.00
Braking70R Tr.1L+CL-A 1L(1S) LL3	16.20	8274.17	0.00	394.20	0.00
Braking70R Tr.1L+CL-A 1L(BS). LL4	16.20	8274.17	0.00	394.20	0.00
Braking70R Wh.1L+CL-A 1L(1S) LL5	23.14	8903.87	0.00	424.20	0.00
Braking70R Wh.1L+CL-A 1L(BS) LL6	23.14	8903.87	0.00	424.20	0.00
Dead Load Seismic	9711.92	119192.47	0.00	11420.71	0.00
Seismic Class A(3L/1S) LL1	629.25	0.00	1024.91	0.00	40.52
Seismic Class A(3L/BS) LL2	818.28	0.00	1332.80	0.00	52.69
Seismic 70R Tr.1L+CL-A 1L(1S) LL3	461.14	0.00	751.10	0.00	29.69
Seismic 70R Tr.1L+CL-A 1L(BS). LL4	488.68	0.00	795.95	0.00	31.47
Seismic 70R Wh.1L+CL-A 1L(1S) LL5	626.04	0.00	1019.68	0.00	40.31
Seismic 70R Wh.1L+CL-A 1L(BS) LL6	698.88	0.00	1138.32	0.00	45.00
Wind load	602.67	3208.95	12414.96	160.07	602.44
Water Current force	4107.60	0.00	193.20	0.00	40.87

NON-SEISMIC CASE

A

HFL / DRY CONDITION	
DL+SIDL+Surfacing+LL+Br. LL+WCF	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
LL	1
Braking LL	0.75
Water Current force	1
Special vehicle	1.0

B

HFL / DRY CONDITION	
DL+SIDL+Surfacing+/-WL+WCF	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
Wind load	1
Water Current force	1

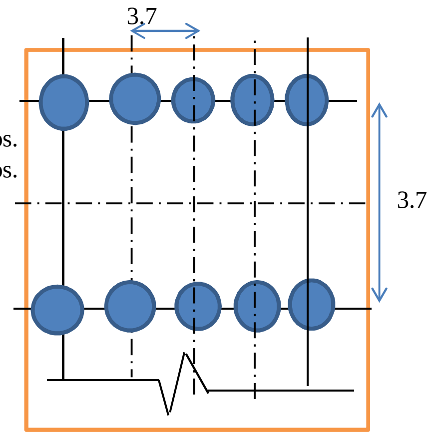
C

ONE SPAN DISLODGED CASE	
DL+SIDL+Surfacing+WCF	
Loads	FOS
Dead Load	1
SIDL	1
Surfacing	1
Water Current force	1

	COMB		V	ML	MT	HL	HT
HFL Condition Non Seismic	1	DL+SIDL+Surfacing+LL1+Braking LL1+WCF	38887.95	8665.42	1723.82	365.65	40.87
	2	DL+SIDL+Surfacing+LL2+Braking LL2+WCF	39158.43	9572.02	2183.63	448.55	40.87
	3	DL+SIDL+Surfacing+LL3+Braking LL3+WCF	38618.74	6931.44	1578.85	295.65	40.87
	4	DL+SIDL+Surfacing+LL4+Braking LL4+WCF	38728.06	6205.63	1808.43	295.65	40.87
	5	DL+SIDL+Surfacing+LL5+Braking LL5+WCF	38950.37	7762.78	2318.57	318.15	40.87
	6	DL+SIDL+Surfacing+LL6+Braking LL6+WCF	39065.11	6752.81	2565.85	318.15	40.87
	7	DL+SIDL+Surfacing+LL7+WCF	40245.81	2528.96	882.92	0.00	40.87
	8	DL+SIDL+Surfacing+LL8+WCF	39832.15	1456.46	758.82	0.00	40.87
	9	DL+SIDL+Surfacing+LL1+Braking LL1+WCF+WL	39490.62	11874.37	14138.79	525.72	643.31
	10	DL+SIDL+Surfacing+LL2+Braking LL2+WCF+WL	39761.10	12780.97	14598.59	608.62	643.31
	11	DL+SIDL+Surfacing+LL3+Braking LL3+WCF+WL	39221.41	10140.39	13993.81	455.71	643.31
	12	DL+SIDL+Surfacing+LL4+Braking LL4+WCF+WL	39330.74	9414.57	14223.39	455.71	643.31
	13	DL+SIDL+Surfacing+LL5+Braking LL5+WCF+WL	39553.04	10971.73	14733.53	478.21	643.31
	14	DL+SIDL+Surfacing+LL6+Braking LL6+WCF+WL	39667.79	9961.76	14980.81	478.21	643.31
DRY Condition Non Seismic	15	DL+SIDL+Surfacing+LL1+Braking LL1	34780.35	8665.42	1530.63	365.65	0.00
	16	DL+SIDL+Surfacing+LL2+Braking LL2	35050.83	9572.02	1990.44	448.55	0.00
	17	DL+SIDL+Surfacing+LL3+Braking LL3	34511.14	6931.44	1385.65	295.65	0.00
	18	DL+SIDL+Surfacing+LL4+Braking LL4	34620.46	6205.63	1615.23	295.65	0.00
	19	DL+SIDL+Surfacing+LL5+Braking LL5	34842.77	7762.78	2125.37	318.15	0.00
	20	DL+SIDL+Surfacing+LL6+Braking LL6	34957.51	6752.81	2372.66	318.15	0.00
	21	DL+SIDL+Surfacing+LL7	36138.21	2528.96	689.72	0.00	0.00
	22	DL+SIDL+Surfacing+LL8	35724.55	1456.46	565.62	0.00	0.00
	23	DL+SIDL+Surfacing+LL1+Braking LL1+WL	35383.02	11874.37	13945.59	525.72	602.44
	24	DL+SIDL+Surfacing+LL2+Braking LL2+WL	35653.50	12780.97	14405.40	608.62	602.44
	25	DL+SIDL+Surfacing+LL3+Braking LL3+WL	35113.81	10140.39	13800.61	455.71	602.44
	26	DL+SIDL+Surfacing+LL4+Braking LL4+WL	35223.14	9414.57	14030.19	455.71	602.44
	27	DL+SIDL+Surfacing+LL5+Braking LL5+WL	35445.44	10971.73	14540.33	478.21	602.44
	28	DL+SIDL+Surfacing+LL6+Braking LL6+WL	35560.19	9961.76	14787.62	478.21	602.44
One Span dislodged	29	DL+SIDL+Surfacing+WCF (Non-seismic)	37946.76	0.00	193.20	0.00	40.87

CHECK FOR PILE CAPACITY

Diameter of pile	=	1.2 m
C/C of pile along longitudinal direction	=	3.7 m
C/C of pile along transeverse direction	=	3.7 m
No. of piles along longitudinal direction	=	4 nos.
No. of piles along transeverse direction	=	5 nos.
Edge Clearance	=	0.15 m
Pile cap dimension in longitudinal direction	=	12.6 m
Pile cap dimension in transeverse direction	=	16.3 m
Pier dimension in longitudinal direction	=	3 m
Pier dimension in transeverse direction	=	8 m
Depth of pile cap	=	2 m
Height of earth above pile cap	=	3.555 m
Clear cover	=	0.075 m
Effective depth of pile cap	=	1.909 m

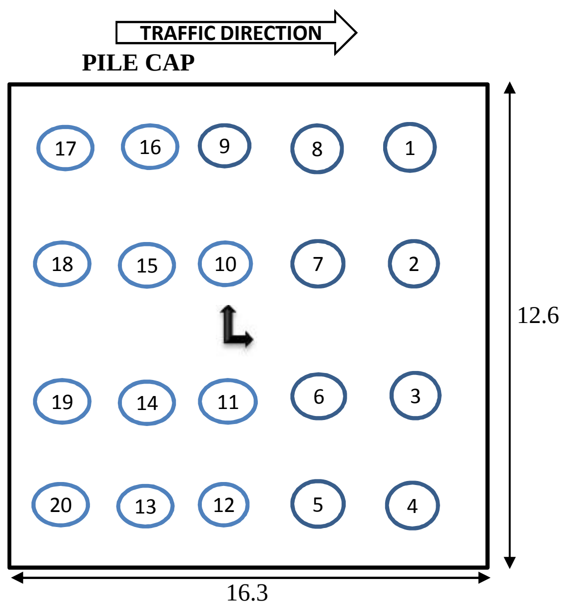


Load combination for pile load (with unfactored load)

	Load Case	V	ML	MT	HL	HT	
MLmax	1	36740.89	13990.49	13104.68	553.23	602.44	Considering Wind
Vmax	2	36740.89	13990.49	13104.68	553.23	602.44	
MLmax	3	45312.443	127033.50	13691.22	11777.4	647.44	Considering Seismic
Vmax	4	45312.443	127033.50	13691.22	11777.4	647.437	
MLmax	5	40245.812	10781.547	882.915	393.167	40.871	Normal Case
Vmax	6	40245.812	10781.547	882.915	393.167	40.871	
MLmax	7	21027.178	0.000	193.198	0.000	40.871	1 span dislodged (Seismic)

Check for pile capacity

Total no. of pile = 20



Pile No.	Trans.	Long.			
	x_i	y_i		x_i^2	y_i^2
1	7.40	5.55		54.76	30.80
2	7.40	1.85		54.76	3.42
3	7.40	-1.85		54.76	3.42
4	7.40	-5.55		54.76	30.80
5	3.70	-5.55		13.69	30.80
6	3.70	-1.85		13.69	3.42
7	3.70	1.85		13.69	3.42
8	3.70	5.55		13.69	30.80
9	0	5.55		0	30.80
10	0	1.85		0	3.42
11	0	-1.85		0	3.42
12	0	-5.55		0	30.80
13	-3.70	-5.55		13.69	30.80
14	-3.70	-1.85		13.69	3.42
15	-3.70	1.85		13.69	3.42
16	-3.70	5.55		13.69	30.80
17	-7.40	5.55		54.76	30.80
18	-7.40	1.85		54.76	3.42
19	-7.40	-1.85		54.76	3.42
20	-7.40	-5.55		54.76	30.80

Z_{L1} =	74	M^3	Z_{L2} =	148	M^3
Z_{T1}=	61.7	M^3	Z_{T2}=	185.0	M^3

Moment per unit load on pile at provided pile cap position = 3.97 KN-m/KN

Load Case	V/N	ML/Z_{L1}	ML/Z_{L2}	MT/Z_{T1}	MT/Z_{T2}	H/N	Moment in pile due to horizontal force
1	1837.04	189.06	94.530	212.51	70.84	40.90	162.153
2	1837.04	189.06	94.530	212.51	70.84	40.90	162.153
3	2265.62	1716.67	858.334	222.02	74.01	589.76	2338.387
4	2265.62	1716.67	858.334	222.02	74.01	589.76	2338.387
5	2012.29	145.70	72.848	14.32	4.77	19.76	78.365
6	2012.29	145.70	72.848	14.32	4.77	19.76	78.365
7	1051.36	0.00	0.000	3.13	1.04	2.04	8.103

Vertical Loads on piles

Load Case	Pile marks 1,4,20,17		Pile marks 2,3,19,18		Pile marks 8,5,13,16		Pile marks 7,6,14,15	
	Vmax	Vmin	Vmax	Vmin	Vmax	Vmin	Vmax	Vmin
1	2238.61	1435.48	2096.94	1577.15	2144.08	1530.01	2002.41	1671.68
2	2238.61	1435.48	2096.94	1577.15	2144.08	1530.01	2002.41	1671.68
3	4204.31	326.93	4056.30	474.95	3345.98	1185.27	3197.96	1333.28
4	4204.31	326.93	4056.30	474.95	3345.98	1185.27	3197.96	1333.28
5	2172.30	1852.28	2162.76	1861.82	2099.46	1925.12	2089.91	1934.67
6	2172.30	1852.28	2162.76	1861.82	2099.46	1925.12	2089.91	1934.67
7	1054.49	1048.23	1052.40	1050.31	1054.49	1048.23	1052.40	1050.31

Load Case	Pile marks 9,12		Pile marks 10,11	
	Vmax	Vmin	Vmax	Vmin
1	2049.55	1624.54	1907.88	1766.21
2	2049.55	1624.54	1907.88	1766.21
3	2487.64	2043.60	2339.63	2191.62
4	2487.64	2043.60	2339.63	2191.62
5	2026.61	1997.97	2017.06	2007.52
6	2026.61	1997.97	2017.06	2007.52
7	1054.49	1048.23	1052.40	1050.31

Maximum horizontal force = 40.896 KN (For non-seismic case)
Weight of pile = 36 KN

SEISMIC CASE		
V_{max} =	4204.311	KN
Load Case =	3	
Coresponding moment =	2338.387	KN-M
M_{max} =	2338.387	KN-M
Load Case =	3	
Coresponding min. vertical load =	326.933393	KN

WIND CASE		
V_{max} =	2238.613	KN
Load Case =	1	
Coresponding moment =	162.153	KN-M
M_{max} =	162.153	KN-M
Load Case =	1	
Coresponding min. vertical load =	1624.536	KN

NORMAL CASE		
V_{max} =	2172.305	KN
Load Case =	5	
Coresponding moment =	5.000	KN-M
M_{max} =	78.365	KN-M
Load Case =	5	
Coresponding min. vertical load =	1852.2765	KN

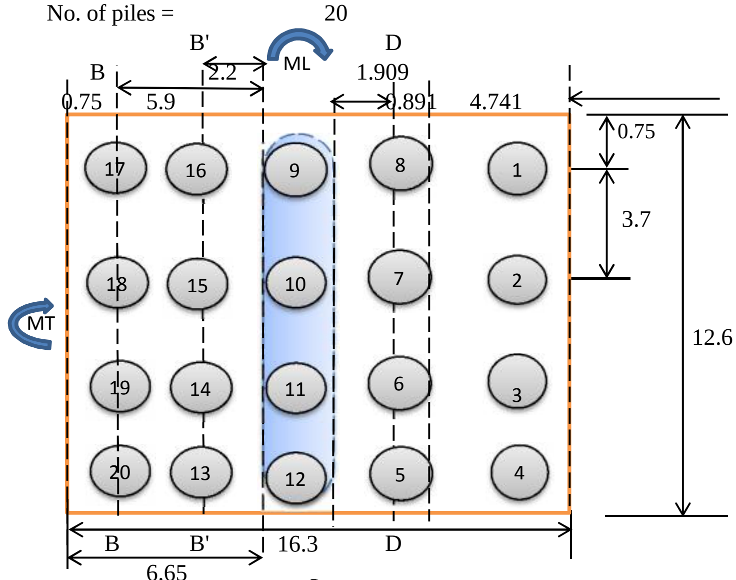
From geotechnical investigation report

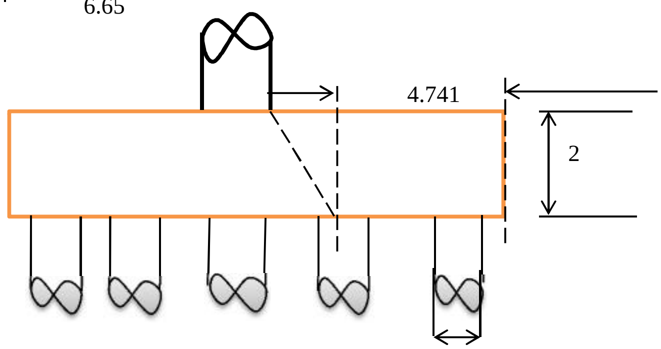
For pile length	15	m from Bottom of PileCap		
Vertical capacity =	5000	KN	**OK**	(Seismic)
Vertical capacity =	5000	KN	**OK**	(Wind)
Vertical capacity =	4000	KN	**OK**	(Normal)
Horizontal capacity =	400	KN	**OK**	

DESIGN OF PILE CAP BY BENDING ANALOGY (ULS)

load comb sl	LOAD CASE	V_u	ML_u	MT_u	HL_u	HT_u
30	MLmax	47903.73	121734.54	591.3	11540.32	40.87
10	Vmax	52828.07	19485.292	21801.29	927.878	944.527
41	Vmin	43551.08	119192.47	0.00	0.00	0.00
43	1 span dislodged	34390.34	122705.36	193.20	0.00	40.87

No. of piles = 20





Load Case	V/N	ML/Z_{L1}	ML/Z_{L2}	MT/Z_{T1}	MT/Z_{T2}
30	2395.19	1645.06	822.5307	9.588	3.196
10	2641.40	263.31	131.6574	353.534	117.845
41	2177.55	1610.71	805.3545	0.000	0.000
43	1719.52	1658.18	829.0903	3.133	1.044

Loads on piles

Load Case	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Moment in each pile
30	4049.84	4043.44	4037.052	4030.660	3208.1290	3214.52	3220.91	3227.31	2287.88
10	3258.25	3022.56	2786.874	2551.184	2419.5266	2655.22	2890.91	3126.60	262.49
41	3788.26	3788.26	3788.263	3788.263	2982.9084	2982.91	2982.91	2982.91	0.00
43	3380.83	3378.74	3376.653	3374.565	2545.4744	2547.56	2549.65	2551.74	8.10

Considering case 30

Bending Moment at section B-B & B'-B' (in KN-m)

Sl. No.	Force due to	Force	Moment arm	Moment
1	Pile 1	4049.837	5.9	23894.036
2	Pile 2	4043.444	5.9	23856.321
3	Pile 3	4037.052	5.9	23818.607
4	Pile 4	4030.660	5.9	23780.892
5	Pile 5	3208.129	2.2	7057.884
6	Pile 6	3214.521	2.2	7071.947
7	Pile 7	3220.914	2.2	7086.010
8	Pile 8	3227.306	2.2	7100.073
9	Self Wt.	-4189.500	3.325	-13930.09
10	Earth Wt.	-595.747	3.325	-1980.86
		Total		**107754.82**

Design moment positive means tension at bottom of pile

Shear force at section D-D ("d" distance away from B-B) in KN

Sl. No.	Force due to	Total Force	For Shear at D-D
1	Pile 1	4049.837	4049.837
2	Pile 2	4043.444	4043.444
3	Pile 3	4037.052	4037.052
4	Pile 4	4030.660	4030.660
5	Pile 5	3208.129	2382.036
6	Pile 6	3214.521	2386.782
7	Pile 7	3220.914	2391.528
8	Pile 8	3227.306	2396.275
9	Self Wt.	-4189.500	-2986.83
10	Earth Wt.	-595.747	-424.73
		Total	**22306.056**

Design of pile cap in flexure

Grade of concrete = M 30

Grade of steel = Fe 500

Width of section considered = 1 m

Section is checked for ULS

Design moment = 8551.97 KN-m (for 1m width)

Width of section = 1 m

Depth of section = 2 m

"E" value of steel = 200000 Mpa

"E" value of concrete = 31000 Mpa

Design compressive strength of concrete =

$f_{cd} = af_{ck}/g_m$ = 13.40 Mpa — Where, a = 0.67, g_m = 1.5

Design peak strength of steel = f_y/g_s = 434.783 Mpa — Where, g_s = 1.15

Concrete failure strain = $\hat{O}_{cu1}$ = 0.0035 (Table 6.5, IRC:112-2011, page-38)

Concrete limiting strain = $\hat{O}_{c2}$ = 0.002 (Table 6.5, IRC:112-2011, page-38)

Yield strain of steel = $0.87f_y/E_s$ = 0.00218

Limiting strain of steel = $(0.87f_y/E_s+0.002)$ = 0.00418

Reinforcement provided: **32** mm dia.

125 mm c/c distance in **2** layers

Total reinforcement provided = 12861 mm^2

Clear cover = 75 mm

Effective depth "d" = 1909 mm

Actual Neutral Axis depth $x_u (0.87f_yA_{st})/(0.36f_{ck}b)$= 518.03 mm

Actual strain in steel = 0.013 mm

Stress in steel = 434.783 Mpa

Balanced Neutral Axis depth $x_{u,max}$ = 870.504 mm

So, Section is under reinforced, ok

CG of compressive force = 215.501 mm from most compressed surface

Moment of resistance, Mu = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

9469.931 kN **OK**

CHECK FOR SHEAR IN PILE CAP (Clause 10.3.2, IRC:112-2011, page-88)

Design Shear Force = 1770.322 KN

The design shear resistance of the member without shear reinforcement, $V_{Rd.c}$ =

$=[0.12K(80r_1.f_{ck})^{0.33}+0.15s_{cp}]b_w.d$

Where, K = 1+√(200/d)<=2.0

So, K = 1.324

$r_1 = A_{sl}/b_w.d$

Where A_{sl} = Area of steel provided = 12861 mm^2

b_w = Width of section = 1000 mm

d = 1909 mm

r_1 = 0.0067

$s_{cp} = N_{Ed}/A_c < 0.2f_{cd}$, where, N_{Ed} = Axial compressive force = 0

A_c = Cross Sectional area of concrete

s_{cp} = 0 So, $V_{Rd.c}$ = 759.69 KN

Now, $V_{Rd.c}$ minimum = $(n_{min}+0.15s_{cp})b_w.d$

where n_{min} = $0.031K^{3/2}f_{ck}^{1/2}$ = 0.25858

So, $V_{Rd.c}$ minimum = 493.630 KN

So, governing shear resistance = 759.69 KN **Shear reinforcement required**

Calculation of shear reinforcement:

α_{cw}=	***1 for σ $_{cp}$ =0, Ref: Eq-10.9, IRC-112:2011***	1.000
b_w(mm)=		1000
z(mm)=	0.9*d for RCC	1718
v_1=	for f_{ck}<80MPa	0.6
f_{cd}= Design value of concrete compressive strength=	$0.67*f_{ck}/\gamma_m$	13.4
Value of θ° =		45.0
tanθ=		1.0
cotθ=		1.0
$V_{rd.min}$=		6906.8

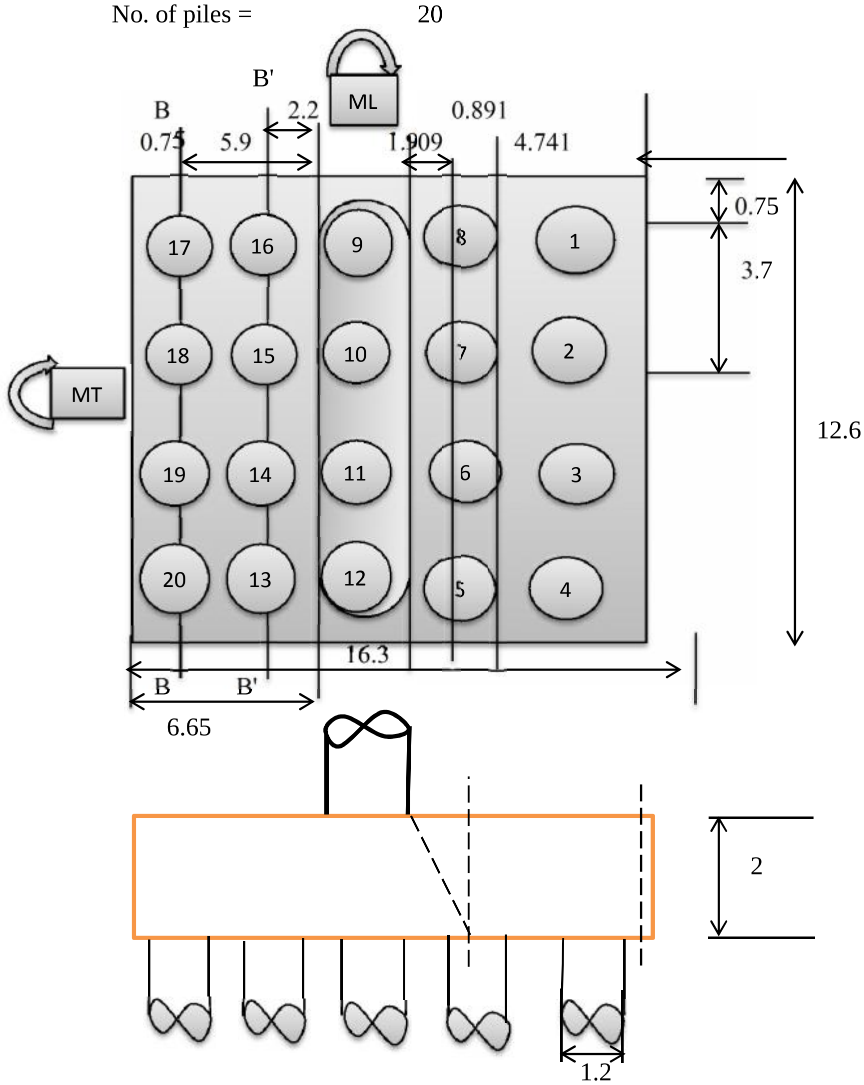
SHEAR REINFORCEMENT DETAILS

Stirrup Dia (mm),ϕ=			25
No. of Leg,			4
Spacing of the stirrup (mm), S=			175
$A_{sw,}$ Provided (mm^2)=			1962.5
$A_{sw,}$ Required (mm^2)=	VRd.s=(A_{sw}/S)*z*fywd*cotθ		518.4
f_{ywd} (Mpa) =	$0.8*f_{yk}/\gamma_m$, γ_m =1.15		347.8
Check	**Ref.: Eqn-10.7, IRC-112,2011**		**OK**
$A_{st,}$ Provided (mm^2)=			11214.3
Reinforcement ratio for shear	$A_{sw}/(b_w*d)$		0.0009
Min. Permissible Reinforcement ratio for shear	$0.072*f_{ck}^{0.5}/f_{yk}$		0.0008
Check	**Ref.: cl-10.3.3.5, IRC-112,2011**		**OK**

SERVICEABILITY LIMIT STATE CHECK

SL NO.	LOAD CASE	V_u	ML_u	MT_u	HL_u	HT_u
10	MLmax	39761.10	12780.97	14598.59	608.618	643.309
7	Vmax	40245.81	2528.96	882.92	0.00	40.87
17	Vmin	34511.140	6931.44	1385.65	295.647	40.871
29	1 span dislodged	37946.76	0.00	193.20	0.00	40.87

No. of piles = 20



Load Case	V/N	ML/Z_{L1}	ML/Z_{L2}	MT/Z_{T1}	MT/Z_{T2}
10	1988.05	172.72	86.357907	236.734	78.911
7	2012.29	34.18	17.087584	14.318	4.773
17	1725.56	93.67	46.834076	22.470	7.490
29	1897.34	0.00	0	3.133	1.044

Vertical Loads on piles

Load Case	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8
10	2397.50	2239.68	2081.86	1924.04	1837.68	1995.50	2153.32	2311.15
7	2060.78	2051.24	2041.69	2032.15	2015.06	2024.61	2034.15	2043.70
17	1841.70	1826.72	1811.74	1796.76	1749.92	1764.90	1779.88	1794.86
29	1900.47	1898.38	1896.29	1894.20	1894.20	1896.29	1898.38	1900.47

Considering case 10

Bending Moment at section B-B (in KN-m)

Sl. No.	Force due to	Force	Moment arm	Moment
1	Pile 1	2397.505	5.9	14145.28
2	Pile 2	2239.682	5.9	13214.12
3	Pile 3	2081.859	5.9	12282.97
4	Pile 4	2032.148	5.9	11989.67
5	Pile 5	2015.061	2.2	4433.13
6	Pile 6	2024.606	2.2	4454.13
7	Pile 7	2153.324	2.2	4737.31
8	Pile 8	2311.147	2.2	5084.52
4	Self Wt.	-4189.500	3.325	-13930.09
5	Earth Wt.	-595.747	3.325	-1980.86
		Total		**54430.20**

Design moment positive means tension at bottom of pile

Shear force at section D-D ("d" distance away from B-B) in KN

Sl. No.	Force due to	Total Force	For Shear at D-D
1	Pile 1	2397.505	2397.505
2	Pile 2	2239.682	2239.682
3	Pile 3	2081.859	2081.859
4	Pile 4	2032.148	2032.148
5	Pile 5	2015.061	1496.183
6	Pile 6	2024.606	1503.270
7	Pile 7	2153.324	1598.843
8	Pile 8	2311.147	1716.027
9	Self Wt.	-4189.500	-2986.830
10	Earth Wt.	-595.747	-424.727
		Total	**11653.959**

Stress level check:

Grade of concrete = M 30
Grade of steel = Fe 500
Width of section considered = 1.0 m

Section is checked for SLS

Design moment =	4319.86	KN-m (for 1m width)
Width of section =	1	m
Depth of section =	2	m
"E" value of steel =	200000	Mpa
"E" value of concrete =	31000	Mpa
Modular ratio in tension =	9.3333333	
Concrete failure strain =	0.0035	
Maximum allowable stress in concrete = $0.48f_{ck}$ =	14.4	Mpa
	(Clause 12.2.1(1), IRC:112-2011, page-120)	
Maximum allowable stress in steel = $0.8f_{yk}$ =	400	Mpa
	(Clause 12.2.2, IRC:112-2011, page-120)	

Total reinforcement provided =	12861	mm^2
Effective depth "d" =	1909	mm
Neutral axis depth = x =	567.51	
CG of compressive force =	189.169	mm from most compressed surface

Now moment, M = (Stress in steel)x(area of steel)x(d-CG of compressive force) =

So, stress in steel =	195.30	Mpa	OK, within permissible limit
Total force =	2511.79	KN	
Stress in concrete =	8.852	Mpa	OK, within permissible limit

Crack width check:

Crack width, $W_k = S_{r.max}(Ô_{sm}-Ô_{cm})$ Where, $S_{r.max}$ = Maximum crack spacing

$Ô_{sm}$ = mean strain in the reinforcement under the relavant combination of loads

$Ô_{cm}$ = mean strain in the concrete between cracks.

Now, $$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct.eff}}{\rho_{p.eff}}(1 + \alpha_e \rho_{p.eff})}{E_s} => 0.6\frac{\sigma_{sc}}{E_S}$$

(Eq. 12.6, IRC:112-2011, page-125)

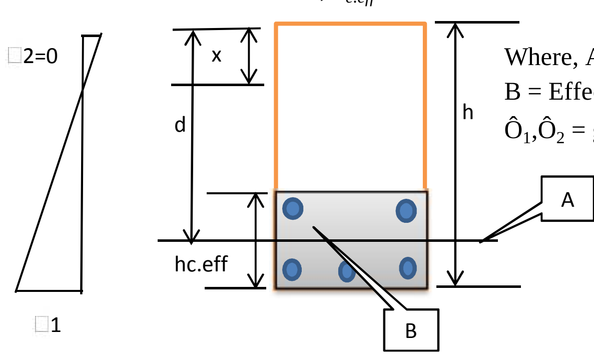
Where, s_{sc} = stress in the tension reinforcement = 195.30 Mpa

$a_e = E_s/E_{cm}$ = 6.4516129

$f_{ct.eff}$ = mean value of tensile strength of concrete = 2.9 Mpa

$r_{r.eff} = A_s/A_{c.eff}$ Where, $A_{c.eff}$ = Effective area of concrete in tension, surrounding the reinforcement of depth $h_{c.eff}$

Where, $h_{c.eff}$ = lesser of the followings



Where, A = level of steel centroid
B = Effective tension area, $A_{c.eff}$
$Ô_1,Ô_2$ = greater and lesser tensile strain

So, $h_{c.eff}$ = 227.5 mm

$A_{c.eff}$ = 227500 mm^2

Now. $r_{r.eff} = A_s/A_{c.eff}$ = 0.0565338

k_t = factor dependant on duration of the load may be taken as 0.5

Now in situations where spacing of bonded reinforcement within the tension zone is reasonably close (i.e <=5(c+f/2)), the maximum crack spacing,

$$Sr.max = 3.4c + \frac{0.425 k_1 k_2 \phi}{\rho_{p.eff}}$$

Where, f = diameter of bar = 32 mm c = clear cover = 75 mm

k1 = co-efficient taking acount of bond properties of reinforcement = 0.8

k2 = co- efficient taking account of distribution of strain = 0.5

So,$S_{r.max}$ = 351.226 mm

And, $\hat{O}_{sm}-\hat{O}_{cm}$ = 0.0008

Minimum value of $\hat{O}_{sm} - \hat{O}_{cm}$ = 0.0005859

So, governing value of $\hat{O}_{sm} - \hat{O}_{cm}$ = 0.0008015

So, crack width, $W_k = S_{r.max}(\hat{O}_{sm}-\hat{O}_{cm})$ = 0.281 mm

Maximum crack width = 0.3 mm (Table 12.1, IRC:112-2011, page-122)

Crack width within permissible limit

DESIGN OF PILES

Diameter of piles, D = 1200 mm

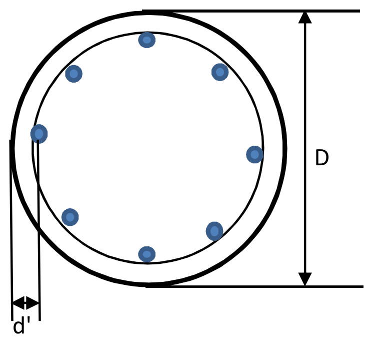
f_{ck} of concrete = 35 N/mm^2

f_y of steel = 500 N/mm^2

Clear cover to the reinforcement = 75 mm

Loads Loads on piles

Load Case	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Moment in each piles KN-m
30	4049.84	4043.44	4037.05	4030.66	3208.13	3214.521	3220.914	3227.31	2287.884
10	3258.25	3022.56	2786.87	2551.18	2419.53	2655.216	2890.906	3126.6	262.4915
41	3788.26	3788.26	3788.26	3788.26	2982.91	2982.908	2982.908	2982.91	0
43	3380.83	3378.74	3376.65	3374.56	2545.47	2547.563	2549.652	2551.74	8.102709



Diameter of bars provided = **32** mm

No. of bars provided = **48** Nos. in **2** **layers**

Percentage of stee, p = 3.413

So, p/f_{ck} = 0.098

d'/D = 0.076

Now for case 30, Pile No. 1

Pu = 4049.84 KN

$P_u/f_{ck}.D^2$ = 0.0804

Now for case 10,

Pile 5 Pu = 2419.53 KN

$P_u/f_{ck}.D^2$ = 0.0480

From SP 16, chart 60, we get,

$M_{u1}/f_{ck}D^3$ = 0.26

So, M_{u1} = 15724.8 KN-m

Now, M_u/M_{u1} = 0.017 **SAFE**

From SP 16, chart 60, we get,

$M_{u1}/f_{ck}D^3$ = 0.32

So, M_{u1} = 19353.6 KN-m

Now, M_u/M_{u1} = 0.118 **SAFE**

Continue all bars upto end of pile

Provide distribution reinforcement **10** mm dia.

@ **150** mm c/c

Summarised Reinforcement Detailing
(According to chapter 16, IRC:112-2011, page-171)
PIER SHAFT

Total vertical reinforcement provided = 10669.1 mm^2
Concrete area, Ac = 3883573 mm^2
Now, 0.0024Ac = 9320.575 mm^2 **< Steel provided, OK**
and 0.04Ac = 155342.9 mm^2 **> Steel provided, OK**

Tie bar provide **16** mm dia. @ **150** mm c/c.

Total horizontal reinforcement provided = 1608 mm^2
Now 25% of vertical steel = 1340 mm^2 **< Steel provided, OK**
and 0.001Ac = 239 mm^2 **< Steel provided, OK**

Vertical reinforcement provided =
32 mm dia @ **150** mm c/c.
Horizontal reinforcement provided =
16 mm dia @ **150** mm c/c.

PILE CAP

Main reinforcement provided		**32**	mm dia.	**125**	mm c/c
	in	**2**	layers		
Steel in 1m strip =		12861.44	mm^2		
$A_{s.min} = 0.26f_{ctm}/f_{yk}b_td=$		2878.77	mm^2	**OK**	
or, $0.0013b_td$ =		2481.7	mm^2	**OK**	

Distribution reinforcement provided at bottom and both direction at top

	25	mm dia.	**150**	mm c/c
		OK		
Provide surface reinforcement	**16**	mm dia.	**6**	Nos. each face
Provide shear reinforcement	**25**	mm dia	**4 legged stirrup @**	
	175	mm	spacing at toe side	

PIER CAP

At top along longitudinal direction provide **25** mm tor bar @ **125** mm c/c
At top along transverse direction provide **25** mm tor bar @ **125** mm c/c
At bottom along longitudinal direction provide **16** mm tor bar @ **125** mm c/c
At bottom along transverse direction provide **16** mm tor bar @**125** mm c/c
Provide **16** mm dia **6** L Stirrup **4** layers

PILE

Main reinforcement provided	**32**	mm dia.	**48**	**Nos. in 2 layers**
Continue all bars upto end of pile				
Distribution reinforcement provided	**10**	mm dia.	**150**	mm c/c

ESTIMATE OF IRANG BRIDGE
CH._95.500 KM

COST ABSTRACT

ABSTRACT

Sr No		Description of Bill Items	Amount (INR Crore)
A		Road Portion (Approach road)	
	I	Cutting , Earthfilling & Disposal	1.65
	II	Sub base	0.24
	III	Non-Bituminous Base Course	0.47
	IV	Bituminous Base Course	0.25
	V	Wearing Coat	0.14
		Sub Total A	2.75
B		Culvert (Sub Total B)	0.38
C		Bridge	
	I	Foundation	9.10
	II	Substructure	5.52
	III	Superstructure	10.28
	IV	Protection work	0.17
	V	Miscellaneous	0.03
		Sub Total C	25.1
D		Grand Total (A+B+C) (As per SOR 2016)	28.23
E		Inflation @ 2.93%	0.83
F		Add GST @ 6%	1.69
G		Civil work without Maintenance (D+E+F)	30.75

COST ESTIMATION

Summary Sheet of Major Bridge (Quantities & Amount)

ITEM NO.	Description	CHAINAGE 95.500 KM Span(m) x Height(m)= Unit	3X41M Twin Bridge of CW=9.5 BW=12.5	Rate (Rs.)	Amount (Rs.)
	A. Foundation				
Item no 1(a)	Excavation (upto 3 m depth)	cum	4191.910	423.50	17,75,274.00
Item no 1(b)	Excavation (3 m to 6 m depth) in rock	cum	2519.090	1,736.73	43,74,979.00
Item no 2	R.C.C M30 (Foundation)	cum	2132.000	11,269.97	2,40,27,572.00
Item no 3	P.C.C (M-15)	cum	209.510	8,873.64	18,59,116.00
Item no 4	Bored cast-in-situ M35 grade R.C.C. Piles	m	1200.000	19,561.41	2,34,73,692.00
Item no 5	Steel liner 6mm thick (1.2m DIA PILE)	ton	1.240	100,426.17	1,24,528.00
Item no 6	Steel (Foundation)	ton	469.160	75,930.20	3,56,23,412.00
	B. SubStructure				
Item no 1(a)	R.C.C M30 (Substructure) upto 5m	cum	784.810	10,581.08	83,04,136.00
Item no 1(b)	R.C.C M30 (Substructure) from 5m to 10m	cum	497.600	10,721.42	53,34,978.00
Item no 1(C)	R.C.C M30 (Substructure) above 10m	cum	637.90	10,861.77	69,28,722.00
Item no 2	R.C.C M35 (Substructure) upto 5m	cum	66.040	11,381.14	7,51,610.00
Item no 3	Steel (Substructure)	ton	289.812	75,930.20	2,20,05,483.00
Item no 4	Weep Holes	each	338.000	482.27	1,63,007.00
Item no 5	Backfilling - Granular Material	cum	667.860	2,152.78	14,37,756.00
Item no 6	Backfilling - Sandy Material	cum	1288.740	2,254.65	29,05,658.00
Item no 7	Filter Media	cum	339.280	2,168.88	7,35,858.00
Item no 8	Elastomeric Bearing	cc	1145609	3.19	36,54,493.00
Item no 9	Brick Masonary Wall at Median	cum	8.750	9,667.37	84,589.00
Item no 10	Pot & Pot cum PTFE	ton capacity	9600.000	504.74	48,45,504.00
	C. Super Structure				
Item no 1(a)	P.S.C M45 (Superstructure) upto 5m	cum	1553.840	18,764.75	2,91,57,419.00
Item no 1(b)	R.C.C M45 (Superstructure) upto 5m	cum	699.800	13,858.18	96,97,955.00
Item no 1(c)	RCC M30 Kerb	cum	52.210	12,774.37	6,66,950.00
Item no 2(a)	Steel (PSC)	ton	131.010	134,692.76	1,76,46,098.00
Item no 2(b)	Steel (Superstructure)	ton	458.320	76,311.35	3,49,75,017.00
Item no 3(a)	Bituminous Concrete Wearing Coat(40mm)	cum	101.040	14,039.14	14,18,514.00
Item no 3(b)	Mastic Asphalt (25mm)	sqm	2525.920	553.17	13,97,273.00
Item no 3(c)	Tack Coat	sqm	2525.920	15.63	39,470.00
Item no 3(d)	Cement concrete wearing course(75 mm)	cum	29.410	16,790.82	4,93,818.00
Item no 4	Railing	metre	261.440	2,361.87	6,17,486.00
Item no 5	Crash Barrier	metre	530.560	6,919.68	36,71,305.00
Item no 6	Drainage Spout	each	46.00	1,919.84	88,313.00
Item no 7	PCC below approach slab	cum	24.260	8,739.16	2,12,012.00
Item no 8	R.C.C. Approach Slab with steel	cum	50.400	16,567.57	8,35,005.00
Item no 9	Strip Seal Expansion Joint	metre	50.000	41863.77	20,93,189.00
Item no 10	Filler Joint				
	(i) copper plate	metre	50.000	6,379.01	3,18,950.00
	(ii) fibar board	metre	50.000	504.68	25,234.00
	(iii) 20mm thick premoulded joint filler	metre	50.000	602.12	30,106.00
	(iv) joint sealing compound	metre	50.000	32.00	1,600.00
	D. Protection Work				
Item no 1a	Boulder Pitching	cum	202.420	4,092.80	8,28,465.00
Item no 1b	Filter Blanket	cum	101.210	3,649.89	3,69,405.00
Item no 2	PCC(M15) Toe Wall	cum	48.440	9,950.67	4,82,011.00
Item no 5	750 mm thick Flexible appron	cum	46.238	2,971.45	1,37,394.00
Item no 6	Below Curtain Wall- PCC (M-15)	cum	10.105	9,950.67	1,00,552.00
Item no 7	Excavation	cum	318.444	423.50	1,34,861.00
Item no 9	PCC M20 curtain wall	cum	68.321	8926.714	6,09,882.00
	MISCELLANEOUS				
Item no 1a	Painting	sqm	2081.110	127.320	2,64,967.00
Item no 1b	Citizen information Board NH Project	no	2.000	25000.000	50,000.00
	ROAD PART				
Item no 1	**Earth cutting for Approach Road**	cum	15900.000	174.210	27,69,939.00
	Earth Filling under road				
Item no 1a	Granular Material	cum	2498.760	2152.780	53,79,280.00
Item no 1b	Sandy Material	cum	2498.760	2254.650	56,33,829.00
Item no 1b	**Disposal For Excavated Earth**	cum	10902.480	252.040	27,47,861.06
	Pavement Composition				
Item no 1a	BC	cum	99.000	14039.135	13,89,874.00
Item no 1b	DBM	cum	198.000	12478.705	24,70,784.00
Item no 1c	WBM	cum	990.000	4815.815	47,67,657.00
Item no 1d	GSB	cum	594.000	4013.178	23,83,828.00
				TOTAL =	**28,23,16,670.06**

LEAD DETAILS

Leads for Various Materials

Sl. No.	Name of Material	Name of Source	Distance from Source to BridgeLocation	Total Lead
1	Sand (Fine)	Noney	58 km by road to Irang Bridge Location	60 Km
2	Filling Material	Local	-	10km
3	Stone Metal	Barak	69 km by road to Irang Bridge Location	71 km
4	Stone Boulder	Barak	69 km by road to Irang Bridge Location	71 km
5	Stone Chips,	Noney	58 km by road to Irang Bridge Location	60 Km
6	Coarse Sand	Noney	58 km by road to Irang Bridge Location	60 Km
7	Cement	Imphal	105.5 km by road to Irang Bridge Location	107.5 Km
8	Steel	Imphal	105.5 km by road to Irang Bridge Location	107.5 Km
9	Bitumen	Numaligarh Refinery, Assam	415 km by road to Irang Bridge Location	417 Km
10	Structural Steel	Imphal	105.5 km by road to Irang Bridge Location	107.5 Km

Carriage Cost of Material (Including loading & unloading)

Rubbish

Name of Quarries **Local**
Lead Upto Site (KM)= **10**

Sl.No.	Lead (km)	Kilometer	Unit	Carriage (Km)	Rate (Rs)	Cost of Carriage (In Rs)
1	10.00	Upto 1	per m^3		156.01	
		Upto 2	per m^3		181.12	
		Upto 3	per m^3		205.75	
		Upto 4	per m^3		229.37	
		Upto 5	per m^3	5	252.04	252.04
		for Every km beyond 5 km up to 10 km	per m^3	5	24.59	122.95
					Total	**374.99**

Stone aggregate below 40mm nominal size

Name of Quarries **Noney**
Lead Upto Bridge Location (KM)= **58**
Lead Upto Quarry (KM)= **2**
Total Lead (KM)= **60**

Sl.No.	Lead in km	Kilometer	Unit	Carriage (Km)	Rate (Rs)	Cost of Carriage (In Rs)
2	60.17	Upto 1	per m^3		149.05	
		Upto 2	per m^3		173.04	
		Upto 3	per m^3		196.57	
		Upto 4	per m^3		219.13	
		Upto 5	per m^3	5	240.80	240.80
		for Every km beyond 5 km up to 10 km	per m^3	5	23.49	117.45
		for Every km beyond10 km up to 20 km	per m^3	10	18.79	187.90
		for Every km beyond 20 km	per m^3	40.17	15.05	604.48
					Total	**1150.63**

Sand

Name of Quarries **Noney**
Lead Upto Bridge Locationl (KM)= **58**
Lead on Quarry (KM)= **2.00**
Total Lead (KM)= **60**

Sl.No.	Lead in km	Kilometer	Unit	Carriage (Km)	Rate (Rs)	Cost of Carriage (In Rs)
3	60.17	Upto 1	per m^3		149.05	
		Upto 2	per m^3		173.04	
		Upto 3	per m^3		196.57	
		Upto 4	per m^3		219.13	
		Upto 5	per m^3	5	240.80	240.80
		for Every km beyond 5 km up to 10 km	per m^3	5	23.49	117.45
		for Every km beyond10 km up to 20 km	per m^3	10	18.79	187.90
		for Every km beyond 20 km	per m^3	40.17	15.05	604.48
					Total	**1150.63**

Boulder

Name of Quarries **Barak**
Lead Upto Bridge Location (KM)= **69**
Lead upto Quarry (KM)= **2.00**
Total Lead (KM)= **71**

Sl.No.	Lead in km	Kilometer	Unit	Carriage	Rate (Rs)	Cost of Carriage (In Rs)
4	71.49	Upto 1	per m^3		165.34	
		Upto 2	per m^3		191.94	
		Upto 3	per m^3		218.05	
		Upto 4	per m^3		243.08	
		Upto 5	per m^3	5	267.11	267.11
		for Every km beyond 5 km up to 10 km	per m^3	5.00	26.06	130.30
		for Every km beyond10 km up to 20 km	per m^3	10.00	20.85	208.50
		for Every km beyond 20 km	per m^3	51.49	16.7	859.95
					Total	**1465.86**

Cement, Steel

Name of Quarries **Imphal**
Lead Upto Bridge Location (KM)= **105.50**
Lead upto Quarry (KM)= **2.00**
Total Lead (KM)= **107.50**

Sl.No.	Lead in km	Kilometer	Unit	Carriage	Rate (Rs)	Carriage (In Rs)
5	107.50	Upto 1	per Tone		106.92	
		Upto 2	per Tone		124.12	
		Upto 3	per Tone		141.01	
		Upto 4	per Tone		157.19	
		Upto 5	per Tone	5	172.73	172.73
		for Every km beyond 5 km up to 10 km	per Tone	5	16.85	84.25
		for Every km beyond10 km up to 20 km	per Tone	10	13.48	134.80
		for Every km beyond 20 km	per Tone	87.50	10.80	945.00
					Total	**1336.78**

Bitumen

Name of Quarries **Numaligarh Refinery, Assam**
Lead Upto Imphal (KM)= **309.5**
Lead Upto Bridge Location form Imphal (KM)= **105.50**
Lead on Project Road (KM)= **2.00**
Total Lead (KM)= **417.00**

Sl.No.	Lead in km	Kilometer	Unit	Carriage	Rate (Rs)	Carriage (In Rs)
6	417.00	Upto 1	per Tone		106.92	
		Upto 2	per Tone		124.12	
		Upto 3	per Tone		141.01	
		Upto 4	per Tone		157.19	
		Upto 5	per Tone	5	172.73	172.73
		for Every km beyond 5 km up to 10 km	per Tone	5	16.85	84.25
		for Every km beyond10 km up to 20 km	per Tone	10	13.48	134.80
		for Every km beyond 20 km	per Tone	397.00	10.80	4287.60
					Total	**4679.38**

QUANTITY ESTIMATE OF BRIDGE & ROAD

DETAIL QUANTITY CALCULATION OF BRIDGE AT IRANG BRIDGE

3 No. X 41 M SPAN — CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos.	Length (m)	Breadth (m)	Height (m)	Quantity
FOUNDATION							
1.1	Excavation upto 3m depth						
	Abutment-1	cum	2	13.80	8.40	3.00	695.52
	Pier-1	cum	2	13.60	17.30	3.00	1411.68
	Pier-2	cum	2	13.60	17.30	3.00	1411.68
	Abutment-2	cum	2	13.80	8.40	2.20	510.05
						Total	4028.93
1.2	Excavation 3.0m to 6.0m depth(in rock)						
	Abutment-1	cum	2	13.80	8.40	1.862	431.69
	Pier-1	cum	2	13.60	17.30	2.859	1345.33
	Pier-2	cum	2	13.60	17.30	1.577	742.07
						Total	2519.09
2	Pile cap & Foundation Slab RCC M30						
Foundation Slab	Abutment (Rectangular Part)	cum	4	12.80	7.400	1.000	378.88
	Abutment (Trapezoidal Part)	cum	4	12.80	4.300	0.500	110.08
	Pile cap (Pier)	cum	4	12.60	16.300	2.00	1643.04
						Total=	2132.00
3	Bored Cast-in-situ Pile M35 (dia.=1.2m)						
	Pier-1	m	40	15.000			600.00
	Pier-2	m	40	15.000			600.00
						Total=	1200.00
4	PCC M-15 levelling course						
	Below Foundation Slab for Abutment	cum	4	13.10	7.70	0.15	60.52
	Below Pile-cap for Pier	cum	4	12.90	16.60	0.15	128.48
						Total=	189.00
5	Steel Liner(1.2m dia pile)						
	Pier-1	ton	1	3.77	0.006	3.50	0.62
	Pier-2	ton	1	3.77	0.006	3.50	0.62
						Total=	1.24
6	HYSD Bars						
	130kg/cum for pile cap & @ 160kg/m for pile	T				Total=	469.160
SUBSTRUCTURE							
7	RCC M-30 upto 5.0m height						
	Abutment wall (A1)	cum	2	12.50	1.200	1.700	51.00
	Abutment wall (A2)	cum	2	12.50	1.200	1.700	51.00
	Abutment cap	cum	4	12.75	2.070	1.000	105.57
	Pier shaft (P1)	cum	2	21.24		5.000	212.40
	Pier shaft (P2)	cum	2	21.24		5.000	212.40
	Dirt Wall	cum	4	12.50	0.400	2.300	46.00
	RCC Wall at median	cum	2	3.00	0.300	2.600	4.68
						Total=	683.05

DETAIL QUANTITY CALCULATION OF BRIDGE AT IRANG BRIDGE

3 No. X 41 M SPAN — CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos.	Length (m)	Breadth (m)	Height (m)	Quantity
8	RCC M-30 height above 5.0m upto 10.0m						
	Pier shaft (P1)	cum	2	21.24		5.000	212.40
	Pier shaft (P2)	cum	2	21.24		5.000	212.40
	Dirt Wall	cum	4	12.50	0.400	1.390	27.80
	Fin Wall	cum	4	4.50	0.500	4.000	36.00
	Bracket	cum	4	12.50	0.180		9.00
						Total=	497.600

9	RCC M-30 height above 10.0m						
	Pier shaft (P1)	cum	2	21.24		6.25	265.50
	Pier shaft (P2)	cum	2	21.24		6.25	265.50
	Pier cap(Trapizoidal portion)	cum	2	10.95	2.40	1.00	21.90
	Pier cap(Rectangular portion)	cum	2	12.50	3.40	1.00	85.00
						Total=	637.90

10	Brick Masonary Wall at Median						
		cum	2	3.00	0.30	4.86	8.75
						Total =	8.75

11	RCC M-35 for Pedestal & Seismic Arrestor Blocks height upto 5.0m						
	Pedestal at Abutment Pot bearing	cum	2	0.80	0.800	0.243	0.31
		cum	2	0.80	0.800	0.290	0.37
		cum	2	0.80	0.800	0.365	0.47
		cum	2	0.80	0.800	0.440	0.56
	Pedestal at Abutment Pot cum PTFE bearing	cum	2	0.80	0.800	0.243	0.31
		cum	2	0.80	0.800	0.290	0.37
		cum	2	0.80	0.800	0.365	0.47
		cum	2	0.80	0.800	0.440	0.56
	Pedestal at Pier for Pot bearing	cum	4	0.80	0.800	0.243	0.62
		cum	4	0.80	0.800	0.290	0.74
		cum	4	0.80	0.800	0.365	0.93
		cum	4	0.80	0.800	0.440	1.13
	Pedestal at Pier for Pot cum PTFE bearing	cum	4	0.80	0.800	0.243	0.62
		cum	4	0.80	0.800	0.290	0.74
		cum	4	0.80	0.800	0.365	0.93
		cum	4	0.80	0.800	0.440	1.13
	Block RB2	cum	16	1.000	0.550	1.425	12.54
	Block RB1	cum	24	0.800	1.321		25.36
	Block RB3	cum	16	1.060	0.740	1.425	17.88
						Total=	66.04

11	HYSD Bars						
	@ 150 kg/cum	T					282.689
						Total=	282.689

3 No. X 41 M SPAN CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos.	Length (m)	Breadth (m)	Height (m)	Quantity
12	Weep holes						
	Spacing for weep holes = 2 m in horizontal and 1 m in vertical direction						
	No of weep holes in horizontal direction per abutment = 11.7/2+1 =					7	
	No of weep holes in vertical direction per abutment = 1.7+1 =					3	
	No of weep holes in horizental direction per Fin wall = 4.5/2+1 =					4	
	No of weep holes in vertical direction per Fin wall = (4+1)/2/1+1 =					4	
	No of weep holes in horizental direction at median = (3)/2+1 =					3	
	No of weep holes in vertical direction at median= (7.44)/1+1 =					9	
	Total no of Weep holes per abutment = 7 x 3					21	
	Total no of Weep holes per Fin wall = 4 x 4					16	
	Total no of Weep holes at median= 9 x 3					27	
	Total no of weep holes = 21 x 4 + 16 x 4+27 x 2					Total	202.00

13.1	Backfilling - Granular Material						
	Behind Abutment	cum	8	21.20		1.000	169.60
	Behind Pier	cum	8	29.90		2.000	478.40
						Total	648.00

13.2	Backfilling - Sandy Material						
	Behind Abutment	cum	4	12.50	4.50	6.390	1437.75
	Front of Abutment	cum	4	12.50	2.800	0.750	105.00
	Deduct for filter media	cum					-254.01
						Total	1288.74

14	Filter media						
	Behind Abutment	cum	4	12.000	0.60	6.390	184.03
	Behind Fin wall	cum	4	4.500	0.60	4.000	43.20
	Behind RCC wall at median	cum	2	3.000	0.60	7.440	26.78
						Total	254.01

15	Pot & Pot cum PTFE Bearing						
		ton	48	200.00			9600.00
						Total	9600.00

16	Elastomeric Bearing						
	Bearing B1	cucm	24	37.00	54.50	9.80	474281
	Bearing B2	cucm	16	27.00	42.00	13.00	235872
	Bearing B3	cucm	32	27.00	42.00	12.00	435456
						Total	1145609

DETAIL QUANTITY CALCULATION OF BRIDGE AT IRANG BRIDGE

3 No. X 41 M SPAN CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos.	Length (m)	Breadth (m)	Height (m)	Quantity
SUPERSTRUCTURE							
17	PSC M-45 Girder portion						
	Long Girder middle portion	cum	6	32.400	5.028		977.44
	Long Girder straight end portion	cum	12	2.050	9.756		240.00
	Long Girder varying portion	cum	12	1.600	7.392		141.93
	End Cross Girder (portion in between the long girder)	cum	12	0.400	14.700		70.56
	End Cross Girder (triangular part)	cum	12	0.400	1.840		8.83
	Intermediate Cross Girder (portion in between the long girder)	cum	18	0.300	14.742		79.61
	Intermediate Cross Girder (rectangular part)	cum	18	0.300	6.568		35.47
						Total	1553.84
18	R.C.C. Deck slab (M40)						
	Deck Slab without Cantilever	cum	6	39.70	12.50	0.225	669.94
	Cantilever portion of Deck Slab	cum	12	0.63	12.50	0.316	29.86
						Total	699.80
19	RCC M30 Kerb						
		cum	2	130.720	0.500	0.225	29.41
						Total	29.41
19	PSC Steel						
	@550kg/m length of PSC girder	TON			131.01		131.01
						Total	131.01
20	Superstructure Steel (HYSD Bars)						
	@ 200 kg/cum	TON					456.610
						Total=	456.610
21	Railing						
		m	2	130.72			261.44
						Total=	261.44
22	RCC M40 Crash Barrier						
		m	4	130.72			522.88
						Total=	522.88
23	Drainage Spout	nos.	42			Total	42.00
24	M15 PCC below Approach Slab						
		cum	4	3.370	12.000	0.15	24.26
						Total	24.26
25	Approach Slab(M30)						
	Approach Slab	cum	4	3.500	12.000	0.300	50.40
						Total	50.40
26	Bituminus concrete wearing course						
		cum	2	130.72	9.50	0.040	99.35
						Total=	99.35

DETAIL QUANTITY CALCULATION OF BRIDGE AT IRANG BRIDGE

3 No. X 41 M SPAN — CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos.	Length (m)	Breadth (m)	Height (m)	Quantity
27	Mastic asphalt						
		sqm	2	130.72	9.50		2483.68
						Total=	2483.68
28	Tack coat						
		sqm	2	130.72	9.50		2483.68
						Total=	2483.68
29	Cement Concrete wearing coarse(75mm)						
		cum	2	130.72	1.50	0.075	29.41
						Total=	29.41
30	Filler joint						
	Providing & fixing 2 mm thick corrugated copper plate in expansion joint	m	4	12.50			50.00
	Providing & fixing 20 mm thick compressible fibre board in expansion joint	m	4	12.50			50.00
	Providing and fixing in position 20 mm thick premoulded joint filler	m	4	12.50			50.00
	Providing and filling joint sealing compound	m	4	12.50			50.00
31	Strip Seal Expansion Joint						
		m	4	12.500			50.00
						Total	50.00
PROTECTION WORK							
32	Pitching with Stone Blanket (A1 side)	cum	2	104.69		0.30	62.814
	Pitching with Stone Blanket (A2 side)	cum	2	232.67		0.30	139.602
						Total	202.42
33	Toe Wall						
	In A1 side	cum	2	26.992	0.353		19.056
	In A2 side	cum	2	41.616	0.353		29.381
						Total	48.44
34	Filter Blanket PCC(M15) below pitching						
	A1 side	cum	2	104.69		0.15	31.407
	A2 side	cum	2	232.67		0.15	69.801
						Total=	101.21
MISCELLANEOUS							
35	Painting						
	Railing(Post)	sqm	144	1.05		1.10	166.32
	Railing(Beam)	sqm	6	0.69		130.72	541.18
	Crash Barrier	sqm	4	2.627		130.72	1373.61
						Total	2081.110
36	Citizen information Board NH Project	no				Total	2.000

ESTIMATE OF QUANTITY OF BOX CULVERT

Box Size:- 1 cell of 3 m x 3 m

with Flexible Appron

Item No.	Description	Unit	nos	Length (m)	Breadth (m)	Height (m)	Quantity

A. FOUNDATION

1	Excavation(up to 3m)						
	Box Bridge	cum	1	6.640	13.000	0.870	75.098
	Shear Key	cum	2	6.640	1.680	0.780	17.402
	Return Wall-II	cum	4	4.710	4.300	0.870	70.480
		`				Total	162.980

2	PCC-M15						
	Box Bridge	cum	1	5.640	9.940	0.150	8.409
	Shear Key	cum	2	5.940	1.503	0.150	2.678
	Return Wall-II	cum	4	4.360	3.600	0.150	9.418
						Total	20.505

B. SUBSTRUCTURE

3	RCC-M30 (upto 5m)						
	Bottom Slab	cum	1	5.640	12.000	0.420	28.426
	Box Side Wall	cum	2	12.000	0.420	3.000	30.240
	Base slab of return wall II	cum	4	4.210	3.300	0.300	16.672
	Return wall I	cum	4	0.900	0.300	3.420	3.694
	Return wall II	cum	4	4.210	0.275	3.540	16.394
	Shear key	cum	2	5.640	0.538		6.069
	Haunch	cum	2	12.000	0.011		0.264
						Total=	101.759

4	Substructure Steel (HYSD Bars)						
	70 kg/cum of Concrete	ton	1		7.123		7.123
						Total	7.123

5	Weep holes						
	Spacing for weep holes = 2 m in horizontal and 1 m in vertical direction						
	No of weep holes in horizontal direction per abutment =11.4/2+1 =					7	
	No of weep holes in vertical direction per abutment =2.8/1+1 =					4	
	No of weep holes in horizontal direction per return wall =5.11/2+1 =					4	
	No of weep holes in vertical direction per return wall =3.54/1 +1=					5	
	Total no of Weep holes per abutment = 7 x 4					28	
	Total no of Weep holes per return wall = 4 x 5					16	
	Total no of weep holes = 28 x 2 + 16 x 4						136

6	Backfilling - Granular Material						
	Behind Side Wall	cum	2	0.900	11.400	3.420	70.178
	Behind Return wall II	cum	4	1.100	4.210	3.540	65.575
			2	7.600	4.210	2.670	170.859
			4	1.850	4.210	3.540	110.285
	Deduct for filter media	cum					85.273
	Box	cum	1	18.640		0.420	7.829
	Shear key	cum	2	2.820		0.780	4.399
	Return Wall-II	cum	4	6.360		0.300	7.632
						Total	19.860

7	Filter media						
	Behind Abutment	cum	2	11.400	0.600	3.420	46.786
	Behind Return Wall	cum	4	4.530	0.600	3.540	38.487
						Total	85.273

C. SUPERSTRUCTURE

8	RCC-M30(up to 5m)						
	Box Bridge	cum	1	3.840	12.000	0.489	22.533
	(+)Haunch	cum	2	12.000	0.011		0.264
						Total	22.797

9	Superstructure Steel (HYSD Bars)						
	75 kg/cum of concrete	ton	1	1.710			1.710
						Total	1.710

10	Bituminas Concrete Wearing Course						
		cum	1	3.840	11.000	0.040	1.690
						Total	1.690

11	Mastic Asphalt						
		sqm	1	3.840	11.000		42.240
						Total	42.240

12	Tack Coat						
		sqm	1	3.840	11.000		42.240
						Total	42.240

13	Crash Barrier R.C.C. M40	m	2	3.840			7.680
						Total	7.680

14	Drainage Spout	nos.	4				4

D. PROTECTION WORK

15	750 mm thick Flexible appron						
	Upstream	cum	1	13.700	1.500	0.750	15.413
	Downstream	cum	1	13.700	3.000	0.750	30.825
						Total	46.238

16	Curtain Wall- PCC (M-20)						
	Downstream side	cum	1	21.900	1.910		41.829
	Upstream side	cum	1	17.900	1.480		26.492
						Total	68.321

17	Excavation in Soil						
	Curtain Wall (downstream)	cum	1	22.900	2.850	2.650	172.952
	Curtain Wall (upstream)	cum	1	18.900	2.500	2.150	101.588
	Flexible appron(downstream)	cum	1	12.700	0.175	0.750	1.667
	Flexible appron(upstream)	cum	1	12.700	1.850	0.750	17.621
	Rigid appron(downstream)	cum	1	13.600	1.175	0.400	6.392
	Rigid appron(upstream)	cum	1	13.600	3.350	0.400	18.224
						Total	318.444

18	PCC (M-15)						
	Below Curtain Wall						
	Downstream side	cum	1	21.900	1.850	0.150	6.077
	Upstream side	cum	1	17.900	1.500	0.150	4.028
						Total	10.105

DETAIL QUANTITY CALCULATION OF ROAD PART

3 No. X 41 M SPAN CH._95.500 KM IRANG BRIDGE

Item Sl No.	Description	Unit	nos	Length (m)	Breadth (m)	Height (m)	Quantity
37	Earth cutting for Approach Road						
		cum	1	150.00	13.25	8.00	15900.00
38	EARTH FILLING UNDER ROAD						
	GRANULAR MATERIAL	cum	2	120	16.5	0.631	2498.76
	SANDY MATERIAL	cum	2	120	16.5	0.631	2498.76

39	Disposal For Excavated Earth						
		cum	1				10902.48

40	Pavement Composition						
2.1	BC	cum	2	120	16.5	0.025	99.000
2.2	DBM	cum	2	120	16.5	0.05	198.000
2.3	WBM	cum	2	120	16.5	0.25	990.000
2.4	GSB	cum	2	120	16.5	0.15	594.000

RESETTLEMENT REHABILITATION & SOCIAL IMPACT ASSESSMENT

INTRODUCTION AND BACKGROUND

1.1. The Project

Manipur is one of the Border States in the northeastern part of the country having an international boundary of about 352 kms. long stretch of land with Myanmar in the southeast. It is bounded by Nagaland in the north, Assam in the west and Mizoram in the south. It has a total area of 22327 sq. kms. It lies between 23.80° N to 25.70° N latitude and 93.50° E to 94.80° E longitude.

Geographically, the State of Manipur could be divided into two regions, viz. the hill and the valley. The valley lies in the central part of the State and the hills surround the valley. The average elevation of the valley is about 790 m above the sea level and that of the hills is between 1500 m and 1800m. The hill region comprises of ten districts viz. Senapati, Kangpokpi,Tamenglong, Noney, Churachandpur, Pherzawl, Tengnoupal, Kamjong, Chandel and Ukhrul and the valley region consists of six districts, viz. Imphal East, Imphal West, Thoubal, Jiribam, Kakching and Bishnupur. The hill districts occupy about 90 percent (20089 sq km) of the total area of the State and the valley occupies only about tenth (2238 sq km) of the total area of the State. Imphal is the capital city of Manipur. In the year 2009–10, the tertiary sector of the economy (service industries) was the largest contributor to the gross domestic product of the state, contributing 57.8% of the state domestic product compared to 24% from primary sector (agriculture, forestry, mining) and 18.2% from secondary sector (industrial and manufacturing). Agriculture is the leading occupation in Manipur. In terms net state domestic product (NSDP), Manipur has the sixth largest economy (2009–2010) in India, with an NSDP of 3663 billion Indian rupees.

The existing Bridge is located at Km 95.500 on NH-37 (NH-53) in Noney district of Manipur. The road is a part of economic corridor (EC NO. 44 North east North East Corridor) Silchar – Jiribam – Imphal. It provides direct connectivity to Silchar (Assam) via Jiribam with state capital Imphal. The road also provides connectivity to important major town / market areas of Tupul, Noney, Awangkhul, Khongsang, Nungba, Kaimai, Jiribam etc. The existing Bailey Bridge is unable to carry the current traffic load of the NH and also this narrow bridge causes congestion in that location. To avoid the congestion of traffic and considering the present condition of existing bridge NHIDCL has decided to provide a new 4 -lane bridge as per IRC standard.

The existing ROW width along the project road has been observed to be around 7m. However, the existing ROW does not cater to the codal provision of 24m ROW of Hill Road in open areas and hence land acquisition is required at proposed bridge approaches to accommodate the 4 lane bridge proposal.

The road approach of the bridge passes through mountainous terrain. The topography is mostly rural in nature. As per the reconnaissance survey it has been observed that there are 8 nos. of households are likely to be affected for the project

The existing Bridge is Single Lane Bailey Bridge. It is a single span bridge with span length of 43.7 m. The carriageway width of the existing bridge is 4 meter with outer to outer width

5.6m. The existing bridge condition along the road is poor. 3 x 41m PSC T- Girder 4-lane bridge is proposed just on the upstream side of the existing bridge.

Figure 1: Photograph of Existing Irang Bridge

Adequate attention has been given during the feasibility phases of the project preparation to minimize the adverse impacts on land acquisition and resettlement impacts. However, technical and engineering constraints were one of the major concerns during exploration of various alternative alignment option. With the available options proposed bridge alignment has been has finalised with best engineering solution as well as avoiding large scale land acquisition and involuntary resettlement impacts.

1.2. Scope of Land Acquisition and Resettlement Impacts

The existing ROW width along the project road has been observed to be around 7m in an average. However, the existing ROW is not sufficient to accommodate the 4 lane bridge proposal. Hence, adequate land is to be acquired near the bridge approaches.

1.3. Stakeholders Consultation and Participation

Focus Group Consultations with various stakeholders were carried out during various phases of project preparation. Key person and focus group consultations at section of the society were arranged at the stage of project preparation to ensure peoples' participation in the planning phase of this project and to treat public consultation and participation as a continuous two way process. Aiming at promotion of public understanding and fruitful solutions of developmental problems such as local needs and problem and prospects of resettlement, various sections of DPs and other stakeholders were consulted through focus group discussions and individual interviews.

To keep more transparency in planning and for further active involvement of APs and other stakeholders, the project information will be disseminated through disclosure of resettlement planning documents.

1.4. Legal and Policy Framework

The legal framework and principles adopted for addressing resettlement issues in the Project have been guided by the proposed legislation and policies of the Government of Manipur, Government of India and National Highway's guidelines. Prior to the preparation of the Resettlement Plan, a detailed analysis of the proposed national and state policies was undertaken and an entitlement matrix has been prepared for the entire program. The section below provides details of the various national and state level legislations studied and their applicability within this framework. This resettlement plan (RP) is prepared based on the review and analysis of all applicable legal and policy frameworks of the country and State policy requirements. Land acquisition for the project would be done as per State provisions in accordance with RTFCLARR 2013 and/or other prevailing acts and rules of Govt. Manipur.

All common property resources (CPR) lost due to the project will be replaced or compensated by the project.

The project will recognize two types of displaced persons like (i) persons with formal legal rights to land lost in its entirety or in part and (ii) persons who lost the land they occupy in its entirety or in part who have no formal legal rights to such land, but who have claims to such lands that are recognized or recognizable under national/state laws. The involuntary resettlement requirements apply to all types of displaced persons.

1.5. Entitlements, Assistance and Benefits

The project will have two types of displaced persons i.e., (i) persons with formal legal rights to land lost in its entirety or in part and (ii) persons who lost the land they occupy in its entirety or in part who have no formal legal rights to such land, but who have claims to such lands that are recognized or recognizable under national/state laws. The involuntary resettlement requirements apply to all types of displaced persons.

Compensation for the lost assets to all displaced persons will be paid on the basis of replacement cost. Resettlement assistance for lost income and livelihoods will be provided to title holders. Special resettlement and rehabilitation measures will be made available to the "Vulnerable Group" comprises of DPs living below poverty line (BPL), SC, ST, women headed households, the elderly and the disabled. The detail of the assistance and entitlements has been discussed in the following chapters.

1.6. Resettlement Budget

The resettlement cost estimate for this project includes eligible compensation, resettlement assistance and support cost for RP implementation. The support cost, which includes staffing requirement, monitoring and reporting, involvement of other stakeholders in project implementation and other administrative expenses are part of the overall project cost. The unit cost for land and other assets in this budget has been derived through field survey, consultation with affected families, relevant local authorities and reference from old practices. Contingency provisions have also been made to take into account variations from this estimate. All procedure is followed as per the provision of Section 26 to 30 and Section 41 of RTFCLARR Act 2013. The total R&R budget for the proposed project RP works out to **Rs. 3.81 Crore.** The detail of the same is depicted in the table below.

R&R Budget			
Road Name : IMPHAL TO JIRIBAM (NH-37)		**Location : Irang Bridge**	
Item	**Rate**	**Quantity Total Area (Ha) /Number**	**Cost**
	(in Rs. Per Ha)		**(in Rs.)**
I. Compensation for losss of Private Property			
1. Loss of Land (agricultural, homestead, commercial or otherwise)			
Effective Average Cost of Rural Land @ **Rs.14,400 per katta**	4,305,600.00	1.2369	5,325,596.64
		Sub Total (A)	**5,325,596.64**
2. Loss of Structure (house, shop, building or immovable property or assets attached to land			
Type of Structure	**Rs. Per Sqm**	**Area (Sqm)**	
Pucca	16218.00	49	794,682.00
Semi Pucca	12448.00	244	3,037,312.00
Kutchcha	3769.00	60	226,140.00
Boundary wall (in M)	6244.00	30	187,320.00
		Subtotal (B)	**4,245,454.00**
		100% Solatium for Land and Structure (C)	**19,142,101.28**
II. Rehabilitation and Resettlement (Land owners & families dependent on Land)			
3. Loss of Land			
Special Cash Assistance of Rs. 5 lakhs	500,000.00	0	-
Subsistence Allowance for 12 months	36,000.00	0	-
Additional Assistance to Vulnerable Groups	25,000.00	6	150,000.00
Transitional Allowance	50,000.00	6	300,000.00
One Time Resettlement Allowance	50,000.00	6	300,000.00
		Subtotal (D)	**750,000.00**
4. Loss of Residence			
Special Cash Assistance of Rs. 5 lakhs	500,000.00	0	-
Shifting Assistance to DPs	50,000.00	0	-
Subsistence Allowance for 12 months	36,000.00	0	-
Additional Assistance to Vulnerable Groups	25,000.00	5	125,000.00
Transitional Allowance	50,000.00	5	250,000.00
One Time Resettlement Allowance	50,000.00	5	250,000.00
		Subtotal (E)	**625,000.00**

5. Loss of Shop/trade/commercial structure			
Onetime financial assistance of Rs. 25,000 to families losing shop for reconstruction of shop	25,000.00	2	50,000.00
Special Cash Assistance of Rs. 5 lakhs	500,000.00	0	-
Subsistence Allowance for 12 months	36,000.00	2	72,000.00
Additional Assistance to Vulnerable Groups	25,000.00	2	50,000.00
Transitional Allowance	50,000.00	2	100,000.00
One Time Resettlement Allowance	50,000.00	2	100,000.00
		Subtotal (F)	**372,000.00**
IV. Impact to Standing Crops and Trees			
Average cost of the fruit bearing trees	300.00	42	12,600.00
		Subtotal (J)	**12,600.00**
III. Impact to Squatters/ Encroachers			
1. Loss of Residence			
House Construction Assistance of Rs. 50,000	50,000.00	0.00	-
Shifting Assistance to DPs	10,000.00	0.00	-
Subsistence Allowance for 3 months	18,000.00	0.00	-
		Subtotal (G)	**-**
2. Loss of Shop/trade/commercial structure			
Shop Construction Assistance of Rs. 20,000	20,000.00	0.00	-
Shifting Assistance to DPs	10,000.00	0.00	-
Subsistence Allowance for 3 months	18,000.00	0.00	-
		Subtotal (H)	**-**
3. Loss of commercial Kiosk/vendor			
Special one time Assistance of Rs. 18,000	18,000.00	0.00	-
Subsistence Allowance for 3months	9,000.00	0.00	-
		Subtotal (I)	**-**
IV. Impact to Vulnerable Household			
One time Assistance who have to relocate	25,000.00	0	-
		Subtotal (J)	**-**
V. Imapct to Tenant during Construction			
Subsistence Allowance for 3months	18,000.00	1	18,000.00
Rental Assistance of Rs. 9,000	9,000.00	1	9,000.00
		Subtotal (K)	**27,000.00**
VI. Common Property Resource			
Religious Structures	250,000.00	0	-

School/Community Property	100,000.00	0	-
Quasi Govt/ VC Buildings	500,000.00	1	500,000.00
Cost of structure in lieu of community Land	2,500,000.00	0	-
		Subtotal (L)	**500,000.00**
VIII. Unforeseen Impacts			
Contingency of 15%	Total of (A to L)	15%	4,649,962.79
		Subtotal (M)	**4,649,962.79**
		Total(O) = (A to M)	**35,649,714.71**
		Inflation (P) accounted for @ 7% of (O)	**2,495,480.03**
		Grand Total (Q) = (O+P))	**38,145,194.74**

1.7. Institutional Arrangements

For implementation of RP there will be a set of institutions involve at various levels and stages of the project. The Executing Agency (EA) for the Project is National Highways & Infrastructure Development Corporation Limited. They have already set up office headed by a General Manager (GM) with Technical Manager and Assistant General Managers (AGM) assisted by other staffs. This office will be functional for the whole Project duration. The EA, headed by GM will have overall responsibility for implementation of the project and will also be responsible for the overall coordination among Government of Manipur under Public Works Department and Project Implementing Unit (PIU) at the site. For resettlement activities, PIU will do the overall coordination, planning, implementation, and financing. Project Implementation Unit (PIU) will be established at project level for the implementation of project/sub-projects.

1.8. Implementation Schedule

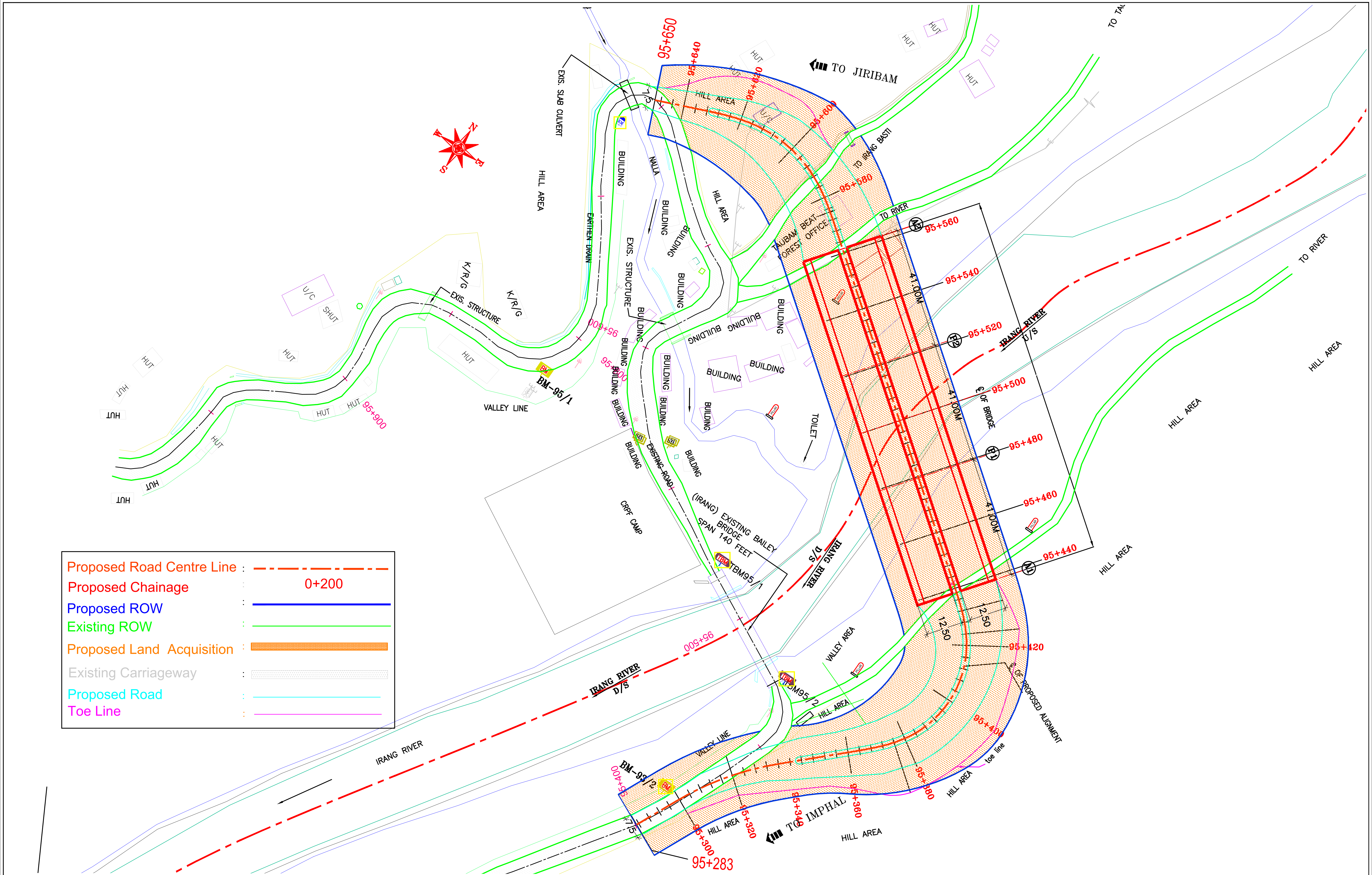
Implementation of RP mainly consists of compensation to be paid for affected structures and rehabilitation and resettlement activities. A composite implementation schedule for R&R activities in the project including various sub tasks and time line matching with civil work schedule is prepared. The cut-off date will be notified formally for titleholder as the date of LA notification. However, the sequence had change or delay had occurred due to circumstances beyond the control of the Project and accordingly the time can be adjusted for the implementation of the plan.

1.9. Monitoring and Reporting

Monitoring and reporting are critical activities in involuntary resettlement management in order to ameliorate problems faced by the DPs and develop solutions immediately. Monitoring is a periodic assessment of planned activities providing midway inputs. It facilitates change and gives necessary feedback of activities and the directions on which they are going. In other words, monitoring apparatus is crucial mechanism for measuring project performance and fulfilment of the project objectives.

PIU responsible for supervision and implementation of the RP will prepare monthly progress reports on resettlement activities and submit to NHIDCL. The Resettlement Officer of NHIDCL would be responsible for monitoring of the RP implementation will submit a quarterly review report to determine whether resettlement goals have been achieved, more importantly whether livelihoods and living standards have been restored/ enhanced and suggest suitable recommendations for improvement. All the resettlement monitoring reports will be disclosed to DPs as per procedure followed for disclosure of resettlement documents by the NHIDCL.

LAND ACQUISITION PLAN



LAND ACQUISITION PLAN

TOTAL AREA REQUIRED – 12369.407 SQ. M

302

MKD.	DATE	DESCRIPTION	CHKD.	APPRD.
REVISIONS				

SCALE: AS SHOWN

DATE: APRIL, 2018

CLIENT: NATIONAL HIGHWAYS AND INFRASTRUCTURE DEVELOPMENT CORPORATION LTD.
4, Parliament Street,
New Delhi - 110001

PROJECT: CONSULTANCY SERVICES FOR PREPARATION OF DETAILED PROJECT REPORT AND PRE-CONSTRUCTION SERVICES IN RESPECT OF 4 LANING WITH PAVED SHOULDER OF IMPHAL - JIRIBAM ROAD SECTION (LENGTH- 220KM) ON NH-37 (NH-53) FOR PROPOSED **BRIDGE OVER RIVER IRANG** IN THE STATE OF MANIPUR.

LAND ACQUISITION OF APPROACH ROAD

BRIDGE & APPROACH ROAD OVER IRANG RIVER

ROAD NAME:- IMPHAL - JIRIBAM (NH-53)

LAND ACQUISITION AREA									
BRIDGE OVER IRANG RIVER & DIVERSION ROAD (KM 95+283 TO KM 95+650)									
Sl. No.	Design			Land to be acquired (in sqm)			Land to be acquired (in Hect.)		
	Chainage From (Km)	Chainage To (Km)	Length in Km	Left	Right	Total	Left	Right	Total
1	95+283	95+650	0.367	5512.901	6856.506	12369.4070	0.5513	0.6857	1.2369

RATE ANALYSIS FOR BRIDGE WORKS

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	13.1	**Earth work in excavation of foundation ……**								
	(a)	**Ordinary soil**								
	(i)	**Depth upto 3 m**	Cum	423.50						423.50
	(ii)	**Depth 3 m to 6 m**	Cum	523.02						523.02
	(iii)	**Above 6 m depth**	Cum	677.49						677.49
	(b)	**Ordinary rock**								
	(i)	**If blasting is resorted to**	Cum	655.48						655.48
	(ii)	**If blasting is not resorted to**	Cum	1736.73						1736.73
	(c)	**Hard rock (requiring blasting)**	Cum	877.38						877.38
	(d)	**Hard rock (blasting prohibited)**	Cum	2662.10						2662.10
	(e)	**Marshy soil (upto 3 m depth)**	Cum	723.73						723.73
	13.2	**Filling in Foundation Trenches**								
	(i)	**Coarse sand**	Cum	2194.17	Sand	Cum	1.200	1150.63	1380.76	3574.93
	(ii)	**Sandy soil with PI value less than 6**	Cum	553.64						553.64
	13.3	**Backfilling abutment, wing wall and Return walls complete as per drawing and technical specification**								
	(a)	**Gravelly materials**	Cum	772.02	Aggregate/Stone Chips	Cum	1.2000	1150.63	1380.76	2152.78
	(b)	**Good Sandy Soil free from organic material**	Cum	873.89	Sand	Cum	1.2000	1150.63	1380.76	2254.65
	13.4	**Filter medium behind abutment,wing wall and return wall complete as per drawing and technical specification .**	Cum	788.12	Stone Aggregate	Cum	1.2000	1150.63	1380.76	2168.88
	14.1	Brick masonry work in cement mortar 1:3 in foundation ……	Cum	9445.33	Sand	Cum	0.260	1150.63	299.16	
					Cement	Tonne	0.123	1336.78	163.76	
					Brick	Nos.	480.00			9908.25
Foundation										
	14.2	**Stone masonry work in cement mortar 1:3 in foundation complete as drawing and Technical Specification**								
	(a)	Coursed rubble masonry(first sort)	Cum	5403.19	Stone	Cum	1.100	1465.86	1612.45	
					Sand	Cum	0.315	1150.63	362.45	
					Cement	Tonne	0.153	1336.78	204.53	7582.61
	(b)	Random Rubble Masonry (coursed/uncoursed)	Cum	5384.68	Stone	Cum	1.000	1465.86	1465.86	
					Sand	Cum	0.347	1150.63	399.27	
					Cement	Tonne	0.168	1336.78	224.58	7474.39
	14.3	**cement concrete for Plain/Reinforced concrete in open foundation ….**								
	A	PCC M15 Grade	Cum	6936.68	Aggt.	Cum	0.85	1150.63	978.04	
					Sand	Cum	0.45	1150.63	517.78	
					Cement	Tonne	0.33	1336.78	441.14	8873.64
	B	PCC M20 Grade	Cum	7542.48	Aggt.	Cum	0.90	1150.63	1035.57	
					Sand	Cum	0.45	1150.63	517.78	
					Cement	Tonne	0.38	1336.78	507.98	9603.81
	C	PCC M25 Grade	Cum	7736.69	Aggt.	Cum	0.980	1150.63	1127.62	
					Sand	Cum	0.365	1150.63	419.98	
					Cement	Tonne	0.400	1336.78	534.71	9819.00
	D	PCC M30 Grade	Cum	7949.99	Aggt.	Cum	0.980	1150.63	1127.62	
					Sand	Cum	0.365	1150.63	419.98	
					Cement	Tonne	0.420	1336.78	561.45	10059.04

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	E	RCC M20 Grade	Cum	7923.25	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.425	1150.63	489.02	
					Cement	Tonne	0.400	1336.78	534.71	9925.02
	F	RCC M25 Grade	Cum	8165.63	Aggt.	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.410	1336.78	548.08	10267.06
	G	RCC M30 Grade	Cum	8484.10	Aggt.	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	10569.97
	H	RCC M35 Grade	Cum	9263.66	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.475	1336.78	634.97	11336.92
	I	RCC M40 Grade	Cum	9530.57	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	11637.25
	14.4	**Providing and laying cutting edge of mild steel for well foundation …**	MT	128456.14						128456.14
	14.8	HYSD bar reinforcement in foundation …	MT	74526.58	HYSD bars	MT	1.050	1336.78	1403.62	75930.20
	14.10	**Providing and laying steel liner for cubs and steining for wells including fabrication and setting out as per detailed drawing**	MT	99022.55	Steel	MT	1.050	1336.78	1403.62	100426.17
	14.11	**Boring, Providing and installing bored cast-in-situ reinforcement cement concrete pile…..**								
	(a)	**1200 mm dia (M20 grade)**	Rm	15362.33						15362.33
	(b)	**1000 mm dia (M20 grade)**	Rm	10668.28						10668.28
	(C)	**750 mm dia (M20 grade)**	Rm	6000.91						6000.91
	Note :	**For load testing assume**								
	(a)	Initial & Routine test L.S. cost	Tonne	362.41						362.41
	(b)	For lateral testing test L.S. cost	Tonne	14641.26						14641.26
	14.12	**Boring, Providing and installing bored cast-in-situ reinforcement cement concrete pile of specified dia and length ……**								
	(a)	1200 mm dia (M35 grade)	Rm	17172.69	Aggt./Stone Chips	Cum	1.017	1150.63	1170.19	
					Sand	Cum	0.509	1150.63	585.10	
					Cement	Tonne	0.474	1336.78	633.43	19561.41
	(b)	1000 mm dia (M35 grade)	Rm	11925.48	Aggt./Stone Chips	Cum	0.707	1150.63	812.92	
					Sand	Cum	0.353	1150.63	406.46	
					Cement	Tonne	0.329	1336.78	440.04	13584.90
	©	750 mm dia (M35 grade)	Rm	6708.08	Aggt./Stone Chips	Cum	0.397	1150.63	457.03	
					Sand	Cum	0.199	1150.63	228.52	
					Cement	Tonne	0.185	1336.78	247.39	7641.02
	Note :	**For load testing assume**								
	(a)	Initial & Routine test L.S. cost	Tonne	503.12						503.12
	(b)	For lateral testing test L.S. cost	Tonne	15901.68						15901.68
	14.15	**Cement concrete for Reinforced concrete in pile cap**								
	(a)	**M-40 Grade**	Cum	10230.57	Aggt./Stone Chips	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	12337.25
	(b)	**M-35 Grade**	Cum	9963.66	Aggt./Stone Chips	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.475	1336.78	634.97	12036.92
	(c)	**M-30 Grade**	Cum	9184.10	Aggt./Stone Chips	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	11269.97
	(d)	**M-25 Grade**	Cum	8865.63	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.410	1336.78	548.08	10967.06

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
SUB-STRUCTURE										
	15.1	Brick masonry work in cement mortar 1:3 in Sub-structure complete excluding pointing and plastering, as per drawing and technical specifications	Cum	9667.37						9667.37
	15.2	**Stone masonry work in cement mortar 1:3 in Sub-structure complete as drawing and Technical Specification**								
	(a)	Coursed rubble masonry(first sort)	Cum	6377.48	Stone	Cum	1.100	1465.86	1612.45	
					Sand	Cum	0.315	1150.63	362.45	
					Cement	Tonne	0.153	1336.78	204.53	8556.90
	(b)	Random Rubble Masonry (coursed/uncoursed)	Cum	5658.74	Stone	Cum	1.0000	1465.86	1465.86	
					Sand	Cum	0.3470	1150.63	399.27	
					Cement	Tonne	0.1680	1336.78	224.58	7748.45
	15.3	**cement concrete for Plain/Reinforced concrete in open foundation**								
	(a)	**PCC M15 Grade**								
	(i)	**upto 5m height**	Cum	6936.68	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.275	1336.78	368.06	8858.09
	(ii)	**Between 5 to 10 m height**	Cum	7051.44	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.275	1336.78	368.06	8972.85
	(iii)	**Above 10 m**	Cum	7166.21	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.275	1336.78	368.06	9087.62
	(b)	**PCC M20 Grade**								
	(i)	**upto 5m height**	Cum	7542.48	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.344	1336.78	459.85	9555.69
	(ii)	**Between 5 to 10 m height**	Cum	7667.24	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.344	1336.78	459.85	9680.45
	(iii)	**Above 10 m**	Cum	7792.00	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.344	1336.78	459.85	9805.21
	(c)	**PCC M25 Grade**								
	(i)	**upto 5m height**	Cum	7736.69	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.399	1336.78	533.82	9823.87
	(ii)	**Between 5 to 10 m height**	Cum	7864.57	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.399	1336.78	533.82	9951.75
	(iii)	**Above 10 m**	Cum	7992.44	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.399	1336.78	533.82	10079.62

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	(d)	PCC M30 Grade								
	(i)	upto 5m height	Cum	7949.99	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.405	1336.78	541.84	10045.19
	(ii)	Between 5 to 10 m height	Cum	8081.39	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.405	1336.78	541.84	10176.59
	(iii)	Above 10 m	Cum	8212.80	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.405	1336.78	541.84	10308.00
	(e)	RCC M20 Grade								
	(i)	upto 5m height	Cum	7923.25	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.347	1336.78	464.31	9940.91
	(ii)	Between 5 to 10 m height	Cum	8054.33	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.347	1336.78	464.31	10071.99
	(iii)	Above 10 m	Cum	8185.40	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.347	1336.78	464.31	10203.06
	(f)	RCC M25 Grade								
	(i)	Upto 5m height	Cum	8165.63	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.403	1336.78	539.17	10258.15
	(ii)	Between 5 to 10 m height	Cum	8300.71	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.403	1336.78	539.17	10393.23
	(iii)	Above 10 m	Cum	8435.79	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.403	1336.78	539.17	10528.31
	(g)	RCC M30 Grade								
	(i)	upto 5m height	Cum	8484.10	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.407	1336.78	543.62	10581.08
	(ii)	Between 5 to 10 m height	Cum	8624.44	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.407	1336.78	543.62	10721.42
	(iii)	Above 10 m	Cum	8764.79	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.407	1336.78	543.62	10861.77
	(h)	RCC M35 Grade								
	(i)	upto 5m height	Cum	9263.66	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.422	1336.78	564.12	11381.14
	(ii)	Between 5 to 10 m height	Cum	9416.78	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.422	1336.78	564.12	11534.26
	(iii)	Above 10 m	Cum	9569.90	Aggt./Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.422	1336.78	564.12	11687.38

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	(i)	**RCC M40 Grade**								
	(i)	**upto 5m height**	Cum	9530.57	Aggt./Stone Chips	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	11637.25
	(ii)	**Between 5 to 10 m height**	Cum	9688.10	Aggt./Stone Chips	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	11794.78
	(iii)	**Above 10 m**	Cum	9845.63	Aggt./Stone Chips	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	11952.31
	15.5	**HYSD bar reinforcement in Sub-structure ….**	MT	74526.58	HYSD bars	MT	1.050	1336.78	1403.62	75930.20
	15.6	Supplying, fitting and fixing in position true to line and level cast steel rocker …..	Tonne Capacity	2.79						2.79
	15.7	Supplying, fitting and fixing in position true to line and level forged steel roller bearing ….	Tonne Capacity	2.79						2.79
	15.8	Supplying, fitting and fixing in position true to line and level sliding plate bearing with PTFE …..	Tonne Capacity	8.73						8.73
	15.9	Supplying, fitting and fixing in position true to line and level elastomeric bearing conforming to IRC: 83 (Part-II) …..	Cubic Centimetre	3.19						3.19
	15.10	Supplying, fitting and fixing in position true to line and level POT-PTFE bearing consisting of a metal piston supported by a disc or unreinforced …..	Tonne Capacity	504.74						504.74
	15.11	Supplying, fitting and fixing in position true to line and level sliding plate bearing with stainless steel plate sliding …….	Tonne Capacity	6.36						6.36
	15.12	Providing weep holes ….	Rm	479.12	Sand	Cum	0.0018	1150.63	2.01	
					Cement	Tonne	0.0009	1336.78	1.14	482.27
SUPER-STRUCTURE										
	16.1	**cement concrete Reinforced concrete in super-structure……**								
	(a)	**RCC Grade M25**								
	(i)	**For solid slab super-structure**	Cum	10042.08	Aggt.	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.410	1336.78	548.08	12143.51
	(ii)	**For T-beam & slab**	Cum	10667.57	Aggt.	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.410	1336.78	548.08	12769.00
	(iii)	**For box girder abnd balance cantelever**	Cum	11918.54	Aggt.	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.410	1336.78	548.08	14019.97
	(a)	**RCC Grade M30**								
	(i)	**For solid slab super-structure**	Cum	10688.50	Aggt.	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	12774.37
	(ii)	**For T-beam & slab**	Cum	11520.73	Aggt.	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	13606.60
	(iii)	**For box girder abnd balance cantelever**	Cum	13185.19	Aggt.	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	15271.06

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	(a)	RCC Grade M35								
	(i)	For solid slab super-structure	Cum	10715.54	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.480	1336.78	641.65	12795.49
	(ii)	For T-beam & slab	Cum	11420.33	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.475	1336.78	634.97	13493.59
	(iii)	For box girder abnd balance cantelever	Cum	12829.92	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.475	1336.78	634.97	14903.18
	(a)	RCC Grade M40								
	(i)	For solid slab super-structure	Cum	11025.71	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	13132.39
	(ii)	For T-beam & slab	Cum	11751.50	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	13858.18
	(iii)	For box girder and balance cantelever	Cum	13203.09	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	15309.77
	16.2	cement concrete for Prestressed concrete in super-structure as per drawing and Technical Specification…								
	(a)	M-35 grade								
	(i)	For girder and slab superstructure	Cum	15174.64	Aggt.	Cum	0.85	1150.63	978.04	
					Sand	Cum	0.40	1150.63	460.25	
					Cement	Bags	0.48	1336.78	641.65	17254.59
	(ii)	For box girder abnd balance cantelever	Cum	17352.53						17352.53
	(a)	M-40 grade								
	(i)	For girder and slab superstructure	Cum	16026.75	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.500	1336.78	668.39	18133.43
	(ii)	For box girder abnd balance	Cum	18335.41						18335.41
	(a)	M-45 grade								
	(i)	For girder and slab superstructure	Cum	16608.32	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.420	1150.63	483.27	
					Cement	Tonne	0.520	1336.78	695.13	18764.75
	(ii)	For box girder abnd balance cantelever	Cum	19003.13						19003.13
	(a)	M-50 grade								
	(i)	For girder and slab superstructure	Cum	17189.88	Aggt.	Cum	0.850	1150.63	978.04	
					Sand	Cum	0.400	1150.63	460.25	
					Cement	Tonne	0.540	1336.78	721.86	19350.03
	(ii)	For box girder abnd balance cantelever	Cum	19670.86						19670.86

HAULAGE CALCULATION SHEET FOR BRIDGE WORKS

Sl. No.	Cl No.	Item	Unit	SOR Rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
	16.3	**HYSD bar reinforcement in super-structure** complete ….	MT	74907.73	HYSD bars	MT	1.050	1336.78	1403.62	76311.35
	16.4	High tensile steel wires/strands including all accessories for stressing….	MT	166659.51	HYSD bars	MT	1.021	1336.78	1365.15	168024.66
	16.5	Cement concrete wearing coat M-30 grade including reinforcement complete …	Cum	14704.95	Aggt.	Cum	0.860	1150.63	989.54	
					Sand	Cum	0.430	1150.63	494.77	
					Cement	Tonne	0.450	1336.78	601.55	16790.82
	16.6	Asphaltic concrete wearing coat of 25mm compacted thickness complete …	Cum	14467.58						14467.58
	16.7	Bituminous Mastic wearing coat excluding tack coat complete ….	Sqm	658.49	Aggregate/Stone Chips	Cum	0.0135	1150.63	15.50	
					Bitumen	Tonne	0.0028	4679.38	13.24	
					Lime Stone	MT	0.0050	1150.63	5.72	692.94
	16.8	Reinforced concrete railing of M30 Gradecomplete ….	Rm	2158.29	Aggregate/Stone Chips	Cum	0.0767	1150.63	88.25	
					Sand	Cum	0.039	1150.63	44.87	
					Cement	Tonne	0.0347	1336.78	46.39	
					Steel	Tonne	0.0180	1336.78	24.06	2361.87
	16.9	Mild steel railling complete …	Rm	4486.97	Steel	Tonne	0.0429	1336.78	57.32	4544.29
	16.11	Drainage Spouts complete ….	Each	1914.49	Steel	Tonne	0.004	1336.78	5.35	1919.84
	16.12	Reinforced cement concrete approach slab M-25 including reinforcement….	Cum	14408.25	Aggregate/Stone Chips	Cum	0.9000	1150.63	1035.57	
					Sand	Cum	0.4500	1150.63	517.78	
					Cement	Tonne	0.4033	1336.78	539.12	
					Steel	Tonne	0.0500	1336.78	66.84	16567.57
	16.13	PCC M15 ordinary Grade leveling course below approach slab …	Cum	6817.79	Aggregate/Stone Chips	Cum	0.900	1150.63	1035.57	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.2753	1336.78	368.02	8739.16
	16.14	Painting in Kerb in black and yellow alternate bands complete …	Metre	127.32						127.32
	16.15	Providing Reinforced Elasomeric (neoprene) slab seal type of expansion joint complete ….								
	(i)	Expansion joint for movement upto 50mm	Rm	69792.80						69792.80
	16.16	Providing single gap(unitary) strip/seal type of expansion joint of movement capacity of 80 mm with fatigue tested structure section….	Rm	41863.77						41863.77
	16.17	Mastic asphalt (providing and laying 12mm thik mastic asphalt wearing coures on top of deck ...)	Sqm	518.72	Aggregate/Stone Chips	Cum	0.0135	1150.63	15.50	
					Bitumen	Tonne	0.0028	4679.38	13.24	
					Lime Stone	MT	0.0050	1150.63	5.72	553.17
	17.1	Laying apron complete as per drawing and Technical specification.								
	(a)	Boulder	Cum	2053.98	Stone Boulder	Cum	1.2000	1465.86	1759.03	3813.01
	(b)	**Boulder in wire crates.**	Cum	2014.46	Stone Boulder	Cum	1.2007	1465.86	1760.06	3774.52
	(c)	**Cement concrete block (M-15grade)**	Cum	7912.98	Aggregate/Stone Chips	Cum	0.275	1150.63	316.81	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.900	1336.78	1203.10	9950.67
	17.2	Filter material underneath pitching in slopes complete as per drawing and Technical specification	Cum	2269.13	Stone Aggregate	Cum	1.200	1150.63	1380.76	3649.89
	17.3	**Pitching on slopes**								
	(a)	**Stone**	Cum	2333.77	Stone Boulder	Cum	1.200	1465.86	1759.03	4092.80
	(b)	**Cement concrete block (M-15grade)**	Cum	7912.98	Aggregate/Stone Chips	Cum	0.275	1150.63	316.81	
					Sand	Cum	0.450	1150.63	517.78	
					Cement	Tonne	0.900	1336.78	1203.10	9950.67

RATE ANALYSIS FOR ROAD WORKS

HAULAGE CALCULATION SHEET FOR ROAD WORKS

Code No.	Item	Unit	SOR rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
2.1	**Cutting of Trees, including cutting of trunks.**	Each							
(i)	Girth from 300 mm to 600 mm	Each	272.60						272.60
(ii)	Girth above 600 mm to 900 mm	Each	539.76						539.76
(iii)	Girth above 900 mm to 1800 mm	Each	1363.01						1363.01
(iv)	Girth above 1800 mm to 2700 mm	Each	2270.78						2270.78
(v)	Girth above 2700 mm	Each	5438.49						5438.49
2.3	**Clearing and grubbing road land including uprooting rank vegetation…..**								
(a)	(by manual means)								
i)	In area of light jungle	Hectare	90145.00						90145.00
ii)	In area of thorny jungle	Hectare	135217.50						135217.50
(b)	(by mechanical means)	Hectare	6425.10						6425.10
2.4	**Dismantling upto 1.5m in foundation and/or 1.5m above ground level….**								
(i)	a) Lime concrete, cement concrete/lean mix concrete.	Cum	450.73						450.73
	b)Cement concrete 1:4:8 or 1:5:10 mix	Cum	901.45						901.45
	c)Cement concrete 1:3:6	Cum	1352.18						1352.18
	d)Cement concrete plain 1:2:4 mix and precast cement concrete blocks.	Cum	2264.46						2264.46
	e)Reinforced cement concrete with cleaning, straightening and cutting of bars and separating out from RCC.	Cum	5051.75						5051.75
(ii)	**Dismantling Brick / Tile work**								
	a)In lime	Cum	784.69						784.69
	b)In cement mortar	Cum	1574.82						1574.82
	c)In mud	Cum	450.73						450.73
	d)Dry brick pitching or brick saling	Cum	450.73						450.73
(iii)	**Dismantling stone masonry**								
	a) Rubble stone masonry in lime	Cum	790.30						790.30
	b) Rubble stone masonry in cement mortar	Cum	1574.82						1574.82
	c) Rubble stone masonry in mud	Cum	450.73						450.73
	d) Dry rubble masonry	Cum	450.73						450.73
	e) Dismantling stone pitching/dry stone spalls	Cum	450.73						450.73
	f) In wire crates including opening of crates and stacking crates materials.	Cum	766.23						766.23
(vii)	**Removing hume pipes class NP-3**								
	a) 300mm to 600mm dia	Rm	159.12						159.12
	b) Above 600mm to 900mm dia	Rm	191.18						191.18
	c) Above 900mm dia	Rm	270.44						270.44
(viii)	**Scarifying including picking up scarified material and stacking of old serviceable material within a lead of 100m**								
	a) Top bituminous surface dressing or premix carpet	Sqm	36.10						36.10
	c) Stone metal crust, 50mm to 100mm thick by road roller with scarifier along with 20mm,premix carpet/surface dre ssing	Sqm	68.76						68.76
	d) Kankar/Gravel metal crust upto 150mm thick with pickaxes.	Sqm	36.21						36.21
2.6	**Dismantling Guard Rails** ….	Rm	52.15						52.15
2.8	**Removal of Telephone / Electric Poles ….**	Each	217.18						217.18

Code No.	Item	Unit	SOR rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
3.12	**Construction of Embankment with Material Obtained from Borrow Pits**	Cum	126.81						126.81
3.13	**Construction of Embankment with Material Deposited from Roadway Cutting**	Cum	108.52						108.52
3.14	**Construction of Subgrade and Earthen Shoulders**	Cum	236.33						236.33
3.15	**Compacting Original Ground**								
	Compacting original ground supporting subgrade	Cum	59.93						59.93
3.16	**Compacting original ground supporting embankment**	Cum	30.27						30.27
3.17	**Stripping and Storing Top Soil**	Cum	257.42						257.42
3.19	**Turfing with Sods**	Sqm	43.07						43.07
	EARTH WORK ON HILL ROAD								
3.31	**Excavation in Hill Area in Soil by Mechanical Means**	Cum	174.21						174.21
3.32	**Excavation in Hilly Area in Ordinary Rock by Mechanical Means not Requiring Blasting.**	Cum	250.71						250.71
3.33	**Excavation in Hilly Areas in Hard Rock Requiring Blasting**	Cum	336.05						336.05
(C)	**EXCAVATION FOR STRUCTURE**								
3.1	Earth work in excavation of foundation of structures ……								
(i)	**Ordinary soil**								
	a) Manual Means (Depth upto 3m)	Cum	356.22						356.22
	b) Mechanical Means (Depth upto 3m)	Cum	104.14						104.14
(ii)	**Ordinary Rock (not requiring blasting)**								
	a) Manual Means (Depth upto 3m)	Cum	445.28						445.28
	b) Mechanical Means	Cum	140.51						140.51
(iii)	Hard Rock (requiring blasting)								
	a) Manual Means	Cum	802.37						802.37
	b) Hard Rock (blasting prohibited) Mechanical Means	Cum	794.49						794.49
(iv)	**Marshy soil**								
	a) Manual Means	Cum	602.74						602.74
	b) Mechanical Means	Cum	246.97						246.97
3.2	Earth work in excavation of foundation trenches etc. in drains and channels etc. …..								
(i)	**Ordinary Soil**	Cum	284.98						284.98
(ii)	**Blasting work**								
	a) Soft rock	Cum	641.89						641.89
	b) Hard rock	Cum	635.59						635.59
(iii)	**Chiselling/wedging out of rock (where blasting is prohibited).**								
	a) Soft rock	Cum	1559.87						1559.87
	b) Hard rock	Cum	2339.80						2339.80
3.3	Filling in foundation trenches as per drawing								
	a) Sandy Soil	Cum	350.30						350.30
	b) Sand Gravell	Cum	443.47						443.47
3.4	Earth filling with surplus soil excavated from								
(i)	**Ordinary Soil**	Cum	189.54						189.54

Code No.	Item	Unit	SOR rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
4.1	**Sub-base with Close Graded Material (Table:- 400-1)**								
	Plant Mix Method								
	For Grading- II Material	Cum	2693.92	Stone Metal	Cum	0.45	1465.86	656.71	
				Stone chips /agg	Cum	0.32	1150.63	368.20	
				Sand	Cum	0.51	1150.63	589.12	4307.95
	For Grading-III Material	Cum	2646.22	Stone chips /agg	Cum	0.45	1465.86	656.71	
				Stone chips /agg	Cum	0.16	1150.63	184.10	
				Sand	Cum	0.67	1150.63	773.23	4260.25
4.2	**By Mix in Place Method**								
	For Grading- II Material	Cum	2296.77	Stone Metal	Cum	0.45	1465.86	656.71	
				Stone chips /agg	Cum	0.32	1150.63	368.20	
				Sand	Cum	0.51	1150.63	589.12	3910.80
	For Grading-III Material	Cum	2249.07	Stone chips /agg	Cum	0.45	1150.63	515.48	
				Stone chips /agg	Cum	0.16	1150.63	184.10	
				Sand	Cum	0.67	1150.63	773.23	3721.88
4.3	**Granular Sub-Base with Coarse Graded Material** (Table:- 400- 2)								
	For Grading- II Material	Cum	2237.75	Stone Metal/Stone	Cum	0.96	1465.86	1407.23	
				Sand	Cum	0.32	1150.63	368.20	4013.18
	For Grading-III Material	Cum	2196.37	Stone chips /agg	Cum	0.85	1150.63	978.04	
				Sand	Cum	0.43	1150.63	494.77	3669.18
4.6	**Lime Stabilisation for Improving Subgrade**								
A	**By Mechanical Means**	Cum	1647.34	Lime	Tonne	0.053	374.99	19.69	1667.03
B	**By Manual Means**	Cum	1691.98	Lime	tonne	0.05	374.99	20.00	1711.98
5.1	**Water Bound Macdam(IRC Grade-II)**	Cum	2573.05	Stone Metal	Cum	1.21	1465.86	1773.69	
				Stone Metal/Stone	Cum	0.24	1465.86	351.81	
				Binding	Cum	0.08	1465.86	117.27	4815.82
5.2	**Wet Mix Macadam**	Cum	2902.46	Stone Metal	Cum	0.396	1465.86	580.48	
				Stone Metal/Stone	Cum	0.528	1465.86	773.97	
				Sand	Cum	0.396	1150.63	455.65	4712.57
6.1	Prime coat								
	A) On WBM/ WMM Surface @ 0.70-1.00 kg/sqm	Sqm	56.41	Bitumen Emulsion	Tonne	0.0008	4679.38	3.74	60.15
	B) Stabilised Soil Based / Crusher run macadam 0.9 - 1.2kg /sqm	Sqm	95.91	Bitumen Emulsion	Tonne	0.001	4679.38	4.68	100.59
6.2	Tack coat								
	Providing and applying tack coat with bitumen emulsion …..								
	i) On bituminious Surface @ 0.20 - 0.30 kg/sqm	Sqm	14.69	Bitumen Emulsion	Tonne	0.0002	4679.38	0.94	15.63
	ii) On granular Surface Pre treated with prime Coat @ 0.25 - 0.30 kg/sqm	Sqm	16.10	Bitumen Emulsion	Tonne	0.00025	4679.38	1.17	17.27
	iii) On cement concrete pavement @ 0.300 - 0.35 kg/sqm	Sqm	21.16	Bitumen Emulsion	Tonne	0.0003	4679.38	1.40	22.56
6.6	Dense Graded Bituminous Macadam								
(i)	**For Grading I (40 mm nominal size)**								
	Using bitumen 60/70	Cum	10320.84	Aggt.	Cum	1.44	1150.63	1656.91	
				Bitumen	Tonne	0.10	4679.38	485.95	
				Filler	Cum	0.04	374.99	15.00	12478.71
(ii)	**For GradingII(19 mm nominal size)**								
	Using bitumen 60/70	Cum	10368.53	Aggt.	Cum	1.44	1150.63	1656.91	
				Bitumen	Tonne	0.10	4679.38	485.95	
				Filler	Cum	0.04	374.99	15.00	12526.40

Code No.	Item	Unit	SOR rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
6.8	**Bituminous Concrete**								
(i)	**For Grading-I (19 mm nominal size)**								
	A) Using Bitumen 60/70	Cum	11743.61	Aggt.	Cum	1.455	1150.63	1674.17	
				Bitumen	Tonne	0.130	4679.38	606.35	
				Filler	Cum	0.040	374.99	15.00	14039.14
(ii)	**For Grading-II(13 mm nominal size)**								
	A) Using Bitumen 60/70	Cum	11631.09	Aggt.	Cum	1.455	1150.63	1674.17	
				Bitumen	Tonne	0.130	4679.38	606.35	
				Filler	Cum	0.040	374.99	15.00	13926.62
6.16	Mastic Asphalt	Sqm	1078.93	Bitumen	Tonne	0.006	4679.38	26.74	
				Fine Aggt.	Cum	0.011	1150.63	12.82	
				Lime Stone	MT	0.010	374.99	3.86	
				Coarse Aggt.	Cum	0.016	1150.63	18.08	
				Stone Chips	Cum	0.001	1150.63	0.66	
				Bitumen for Coating	Tonne	0.000014	4679.38	0.07	1141.15
8.1	**Precast Cement concrete M20 Kerb including fixing at site**	Rm	506.34	Crushed Stone Aggt.	Cum	0.061	1150.63	69.65	
				Coarse Sand	Cum	0.030	1150.63	34.84	
				Cement	Tonne	0.016	1336.78	21.17	631.99
8.2	Reinforced cement concrete M15 kilometer stone …..								
	a) 5th KM stone	Each	3607.40	Stone Chips	Cum	0.353	1150.63	405.60	
				Sand	Cum	0.176	1150.63	202.80	
				Cement	Tonne	0.108	1336.78	144.16	
				Steel	Tonne	0.004	1336.78	4.92	4364.87
	b) Ordinary kilometer stone	Each	2052.04	Stone Chips	Cum	0.242	1150.63	278.86	
				Sand	Cum	0.121	1150.63	139.43	
				Cement	Tonne	0.074	1336.78	99.11	
				Steel	Tonne	0.0019	1336.78	2.51	2571.96
8.6	Painting on Steel Surfaces	Sqm	74.28						74.28

Code No.	Item	Unit	SOR rate	Materials Required	Unit	Quantity	Rate	Cost for haulage (Rs)	Final Rates (Rs.)
8.11	**Retro- reflectorised Traffic signs**								
(i)	**90 cm equilateral triangle**	Each	4082.64	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	4897.76
(ii)	**60 cm equilateral triangle**	Each	3183.63	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	3998.75
(iii)	**60 cm circular**	Each	3656.60	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	4471.72
(iv)	**80 mm x 60 mm rectangular**	Each	7596.37	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	8411.49
(v)	**60 cm x 45 cm rectangular**	Each	3521.33	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	4336.45
(vi)	**60 cm x 60 cm square**	Each	3988.61	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	4803.73
(vii)	**90 cm high octagon**	Each	8092.74	Stone Chips	Cum	0.108	1150.63	124.27	
				Sand	Cum	0.540	1150.63	621.34	
				Cement	MT	0.033	1336.78	44.11	
				Steel	Tonne	0.019	1336.78	25.40	8907.86
8.12	Direction and Place Identification signs upto 0.9 sqm size board.	Sqm	11432.06	Stone Chips	Cum	0.120	1150.63	138.08	
				Sand	Cum	0.060	1150.63	69.04	
				Cement	MT	0.037	1336.78	49.46	
				Steel	Tonne	0.021	1336.78	28.07	11716.71
8.13	Direction and Place Identification signs with size more than 0.9 sqm size board.	Sqm	11978.04	Stone Chips	Cum	0.144	1150.63	165.69	
				Sand	Cum	0.072	1150.63	82.85	
				Cement	MT	0.044	1336.78	58.82	
				Steel	Tonne	0.038	1336.78	50.80	12336.19
8.14	Road Marking with Hot Applied Thermoplastic Compound with Reflectorising Glass Beads on Bituminous Surface	Sqm	1121.14						1121.14
8.15	Road Delineators 120x120 -Road Delineator	Piece	1061.01						1061.01
8.17	**RCC Crash Barrier**	m	4613.12						6919.68
8.18	**Metal Beam Crash Barrier**								
A	**Type - A, "W" : Metal Beam Crash Barrier**								
(a)	**For post Height of 1.2 m**	Rm	2639.41						2639.41
(b)	**For post Height of 1.5 m**	Rm	2571.65						2571.65
(c)	**For post Height of 1.8 m**	Rm	2680.55						2680.55
8.20	**Road Markers/Road stud with lense reflector**								
(i)	**Solar light emiiting Diodes**	Nos.	2470.87						2470.87
(ii)	**Light Reflecting Lense Type**	Nos.	364.55						364.55
8.21	Lighting on Bridges	Nos.	21114.20						21114.20