Doc No. 1718-081/TRB/FDPR/P2/S-6/REP-02

NATIONAL HIGHWAYS AND INFRASTRUCTURE DEVELOPMENT CORPORTION LIMITED
(MINISTRY OF ROAD TRANSPORT & HIGHWAYS)
GOVT. OF INDIA

Consultancy Services for Preparation of DPR for Development of Economic Corridors, Inter Corridors and Feeder Routes to Improve the Efficiency of Freight Movement in India under Bharatmala Pariyojana

Lot-1 : Package-II
(251.8 KM)

Section 6
Km 113+800 to Km 131+152

Final
Detailed Project Report

Volume - II
Design Report

June 2020
Revision : R0

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PART-I

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1. METHODOLOGY ADOPTED FOR DETAILED PROJECT STUDY

1.0 METHODOLOGY

The project involves a series of inter related activities, both in the field and in the design office. Methodology for carrying out these activities is described in the following paragraphs.

1.1 SITE INVESTIGATIONS AND SURVEYS

1.1.1 INVENTORY AND CONDITION SURVEY OF ROAD

Road Inventory

Inventory of the existing road shall cover all existing physical features such as terrain, landuse, roadway, carriageway, type of cross section (cut or fill), utility lines passing along or crossing the highway, roadside facilities and all other features that may have influence on the project preparation.

Road Condition Survey

Inventory of the existing road shall cover all existing physical features such as terrain, landuse, roadway, carriageway, type of cross section (cut or fill), utility lines passing along or crossing the highway, roadside facilities and all other features that may have influence on the project preparation.

Detailed field study shall be carried out for road and pavement surface conditions covering the following:

i. pavement condition (surface distress type and extent);
ii. shoulder condition;
iii. embankment condition; and
iv. drainage condition

The process ensures that complete information on condition of existing pavement and shoulder is collected so that design parameters related to pavement can be established.
The information collected shall consist of the details of cracking (narrow and wide), rut depth, raveling, potholing, patching in the form of percentage area as well as edge break in terms of length and rut depth in mm. affected of the existing pavement; and paved shoulder material loss, rut depth, corrugation, edge etc. in the case of unpaved shoulders.

The study shall identify defects and road section with similar characteristics i.e. homogeneous sections.

1.1.2 TOPO, DRAINAGE AND UTILITY SURVEY

According to the TOR, detailed topographic survey is required to be carried out for capturing the essential ground features along the alignment and for working out improvements, rehabilitation and upgrading costs.
Hence, the Consultants propose to carry out detailed topographic survey for the project road. Briefly the survey work would include:

- Topographic survey true to ground realities using high precision instruments like mobile Lidar, Total Stations and Auto Levels, and bringing out data in digital form (X,Y,Z format) for developing digital terrain model (DTM)
- Capturing all existing physical features, including utility poles, trees of girth greater than 0.3 m, oil and gas lines, hills, valley etc. within the survey corridor which should be compatible with the widening requirements subject to a minimum of 25 m on either side of the center line of the road or road land boundary, whichever is more.
- Additional surveys at bridge sites for hydraulic calculations and for all arms of crossing roads at intersections. Also, detailed topographic surveys along approved alignment of proposed bypasses approved by NHIDCL, if required.
- Where existing road crosses the alignments, the survey will extend a minimum of 100m on either side of the road center line to allow improvements, including at-grade intersections.
- Longitudinal sections shall be taken at 25 m interval and at the locations of curve points, small streams, and intersections and at change in elevation. Cross-sections, in general covering the full width of survey corridor at 50 m interval shall be taken and should show levels at every 2-5 m intervals also at all breaks in the profile. Cross sections shall be taken at closer interval (15-25 m depending on radius of curve) on curves.

  Fixing horizontal and vertical control points with concrete pillars. The Reference Pillars/BMs with levels drawn from GTS bench marks shall be of size 15cmX15cmX45cm, cast in RCC grade M15 with a nail fixed in the center of the top surface. The reference pillar shall be embedded in concrete up to a depth of 30 cm with CC M 10 (5 CM wide all around). The balance 15 cm above ground shall be painted yellow. The spacing shall be 250m apart.

1.1.3 STUDY OF ALTERNATIVES FOR REALIGNMENTS/POOR GEOMETRICS

Requirement of realignment to improve poor geometrics were explored at site and accordingly alignment was finalized.

1.1.4 JUNCTIONS/INTERSECTIONS AT GRADE

Improvement of the intersections has been thought off with minimum of land acquisition. However, merging lanes have been considered with proper traffic signage. In general, standard codal provisions have been followed for design of these intersections. 2nos major intersections have been proposed. Besides, there are 17 nos. of minor intersections along the project roadare studied at site for the purpose of improving them.Improvement proposals of the intersections are provided in **Chapter-6,Volume-I, Development Proposal.**

1.1.5 PAVEMENT STUDIES

Subgrade Characteristics and Strength

a) Division of project road into homogeneous sections with respect to pavement condition and structural strength. The delineation of segments homogeneous with respect to roughness and strength should be done using the cumulative difference approach (AASTHO, 1993).

b) For the widening of existing road within the ROW, sampling and testing of at least 3 subgrade soil samples for each homogeneous road sections or 3 samples for each soil type encountered, whichever is more.

c) In case of new alignments, the test pits for subgrade soil shall be @ 5 km interval or for each soil type, whichever is more. A minimum of three samples should be tested corresponding to each homogeneous segment.

d) The testing for subgrade soils shall include the following:
   - i) In situ density and moisture content at each test pit
   - ii) Field CBR using DCP at each test pit.
   - iii) Characterization (Grain size and Atterberg Limit test) for each test pit sample.
   - iv) Laboratory moisture density characteristics (modified AASHTO compaction).
   - v) Laboratory CBR (unsoak and 4-day soak compacted at three energy levels) and swell.

e) Apart from the above, permeability and consolidation test shall be carried out for problematic soils along project corridor. The frequency of sampling and testing of these soils shall be finalized in consultation with NHIDCL; officials.

1.1.6 MATERIAL INVESTIGATION

The activities included

i) Identification of potential sources (including use of fly-ash/slag), quarry sites and borrow areas.

ii) Collection of samples and conducting relevant laboratory tests.

iii) Evaluation of test results and assesses the suitability thereof for incorporation in various works and making recommendation on the use of the materials from different sources based on techno-economic principles.

iv) Assess adequacy of quality and quantities of various construction materials available

v) No material shall be used from the ROW except by way of leveling the ground as required from construction point of view or for landscaping and planting of trees. Environmental restrictions, if any and feasibility of availability of these sites to perspective civil works contractors should be duly taken into account.

vi) Preparation of mass haul diagram and quarry charts indicating the location of selected borrow areas, quarries and the respective estimated quantities.

vii) Recommend on how to make good this borrow and quarry areas after the exploitation of materials for construction of works.

viii) Preparation and testing of bituminous mixes for various layers and concrete mixes of different grades using suitable materials (binders, aggregates, sand fillers etc.) as identified during material investigation to conform with latest MORT&H specifications.

1.1.7 TRAFFIC STUDIES

Traffic survey stations have been selected by the Consultant on the basis of understanding of the road network as well as consideration of the following aspects:

- to represent homogeneous traffic section
- to be outside urban and local influence area
- to be located at a level with good visibility

Schedule of primary surveys

To capture the traffic flow characteristics and the travel pattern of the users passing through the project road and other characteristics related to miscellaneous requirements as per TOR following primary surveys were conducted.

Sl No.	Type of Survey	Date of Survey	Period of Survey	Location
1	Mid Block Volume Count	27.07.18 to 02.08.18 (8:00 AM) (8:00 AM)	7 days x 24 hrs	Km 62 of NH-29
		27.07.18 to 02.08.18 (8:00 AM) (8:00 AM)	7 days x 24 hrs	Km 127 of NH-29
		27.07.18 to 02.08.18 (8:00 AM) (8:00 AM)	7 days x 24 hrs	Km 138.450 of NH-29
2	O-D Survey	29.07.18 to 30.07.18 (8:00 AM) (8:00 AM)	1 day x 24 hrs	Km 127 of NH-29
3	Turning Movement Count Survey at Intersections	31.07.18 to 01.08.18 (10:00 AM) (10:00 AM)	1 day x 24 hrs	Daboka junction (Km39+500 of NH-29)
		30.07.18 to 31.07.18 (10:00 AM) (10:00 AM)	1 day x 24 hrs	HowraghatTinali junction (Km 85+400 of NH-29)
		31.07.18 to 01.08.18	1 day x 24	Manja junction (Km

Sl No.	Type of Survey	Date of Survey	Period of Survey	Location
		(8:00 AM) (8:00 AM)	hrs	128+300 of NH-29)
4	Speed Delay Survey	30.07.18-31.07.18	-	project road stretch
5	Axle load Survey	29.07.18 to 30.07.18 (8:00 AM) (8:00 AM)	2 days x 24 hrs	Km 127 of NH 29

1.2 DELIVERABLES OF DETAILED PROJECT REPORT STAGE

Volume-I - Main Report
Volume-IA - Appendices to Main Report
Volume-II - Design Report
Volume-III - Material Report
Volume-IV - Environmental Assessment Report
Volume-V - Technical Specification
Volume-VI - Rate Analysis
Volume-VII - Cost Estimates
Volume-VIII - Bill of Quantities
Volume-IX - Drawings (Road Works and Structure Works)

2.0 Design of Road features

2.1 General

The salient proposals for up-gradation and improvement of the project road are classified into the following engineering aspects:

- In general, in the section of proposed stretches follows existing NH-29.
- Widening of the project road based on traffic capacity/requirement.
- Improving the horizontal geometry of the existing road based on the design standards as per IRC: SP: 84-2019.
- Design of new pavement for widening and realignment of the existing road.
- Provision of overlay at strengthening stretches.
- Improvement of all major and minor intersections.
- Rehabilitation and widening of the existing structures including bridges, culverts etc. and design of new ones as per requirement.
- Provision of comprehensive road furniture for complete road safety measures.

2.1.1 Design Basis, Standards and Specifications:

The design criteria / method applied for important components of the project are as follows:

Geometric Design	:	IRC: SP: 84-2019 Manual of Specification & Standards for Four Laning of Highways with Paved Shoulder IRC: 73-1980 Geometric design standard for rural highways IRC and other relevant IRC Codes and guidelines on geometric design.
Pavement Design	:	Overlay - IRC 115-2014 for designing and strengthening requirements of existing pavement New Pavement - IRC 37-2018 for design of flexible pavements - IRC 58-2015 for design of rigid pavements
Road Furniture & Roadside Facilities	:	Related standards of IRC Manual of Specification & MoRT&H publications

The basis of preliminary design of various components of the project road is provided in **Table 2.1.**

Table 2.1

Sl No.	Project Component	Basis	Outcome
1	Road alignment and profile	• Geometric design standards • Road Inventory • Type of area, rural or urban including available ROW and roadside developments • Suitability of location for new bridges	• Location of widening carriageway • Improvement to sub-standard curves and steep grade sections
2	Intersections/ Junctions	• Peak-hour traffic intensities and turning movement data	• Design of at-grade intersections • Installation of traffic control measures
3	Pavement design strengthening of existing pavement	• Traffic loading in terms of cumulative standard axles for design lane • Falling weight deflection data • CBR of existing subgrade • Laboratory soaked CBR of subgrade material • Thickness and composition of existing pavement layers	Strengthening overlays for applicable stretches
4	Pavement design new pavement	• Traffic loading in terms of cumulative standard axles for design lane • Soaked laboratory CBR of soil	• Thickness and composition of various pavement courses

Sl No.	Project Component	Basis	Outcome
		samples from prospective borrow areas • Initial design life and stage development strategy	• Design life determination
5	Road furniture and safety measures	• Road inventory • Alignment plans • Locations of intersections on urban areas	• Identification of different types of signs on linear plans • Identification of locations for installation of crash barriers and pedestrian guard rails • Pavement marking details
6	Roadside Drains	Results from drainage study	• Location, type and size of roadside drains to be provided
7	Wayside Amenities	Inventory of existing wayside amenities by location, type and quality grade	• Evaluation of need and additional amenities • Locations and laybye design for bus stops

2.2 Geometric Design Standards

This project is essentially involves widening the existing standard/sub-standard 2-lane road to 4-lane road with paved shoulder along with construction of a new standard 4-lane road with paved shoulder at realignment section. The geometric designs are as per recommendations of IRC: SP: 84-2019. The general design standards for improvement are enumerated in **Table 2.1a**

Table 2.1a: Geometric Design Standards

Sl No.	Attributes	Geometric Design Standards
1	Design Speed	
	Plain and Rolling Terrain (Cross slope of the ground upto 25per cent)	Ruling: 100 kmph Minimum: 80 kmph
2	Carriageway Width	For four lane: 2 x 7.0m with 0.5 m Kerb shyness at either side
3	Width of Shoulder	
	a) Paved Shoulder	2 x 2.5 m
	b) Earthen Shoulder	1.5 m
4	Footpath width at built-up areas	2 x 1.5 m drain cum footpath
5	Camber	
	a) Carriageway	2.5%
	b) Shoulder	3.0%

Sl No.	Attributes	Geometric Design Standards
6	Maximum and Minimum Super-elevation	Maximum limited to 7.0% (for Radius less than Desirable minimum) Minimum limited to 5% (for Radius more than Desirable minimum)
7	Minimum Radius of Horizontal Curves	
	a) Plain and rolling Terrain	Desirable Minimum: 400m Absolute Minimum: 250m
8	Sight Distances for Various Speeds	180m – 360m
9	Longitudinal Gradient	
	a) Plain and Rolling Terrain	Ruling: 2.5%, Limiting: 3.3%
10	Extra Width of Pavement	
	Radius of Curve	Extra Width
	75-100m	0.9m
	101-300m	0.6m

2.3 Design of horizontal and vertical alignment:

The existing design of both horizontal and vertical alignments are not sound with few sharp curves and insufficient sight distance. The proposed design is emphasized in adhering to the IRC codes and manual. The design is carried out with MX-road software on the basis of design standards as mentioned in **Table 2.1a**. The design methodology is also supported by various directives received from Project coordinating consultants and as well as NHIDCL at various occasions.

Horizontal and Vertical reports are attached as an**Annexure 2.2 and Annexure 2.3**

2.4 Design Philosophy:

The design proposed is mostly eccentric widening with overlay or reconstruction depending on the condition of the existing road. Efforts have been made to follow the existing road in most of the places in order to minimizing the land acquisition and project cost.

2.5 Proposed Right of Way (ROW)

Proposed ROW of 42m is considered for the road sections in rural area. In built-up area 47m PROW is considered. In forest area, to minimize the land acquisition, 35.5m PROW and at the approach of elephant underpasses, 42.5m PROW are considered. 60 m PROW is also considered for approaches of grade separator.

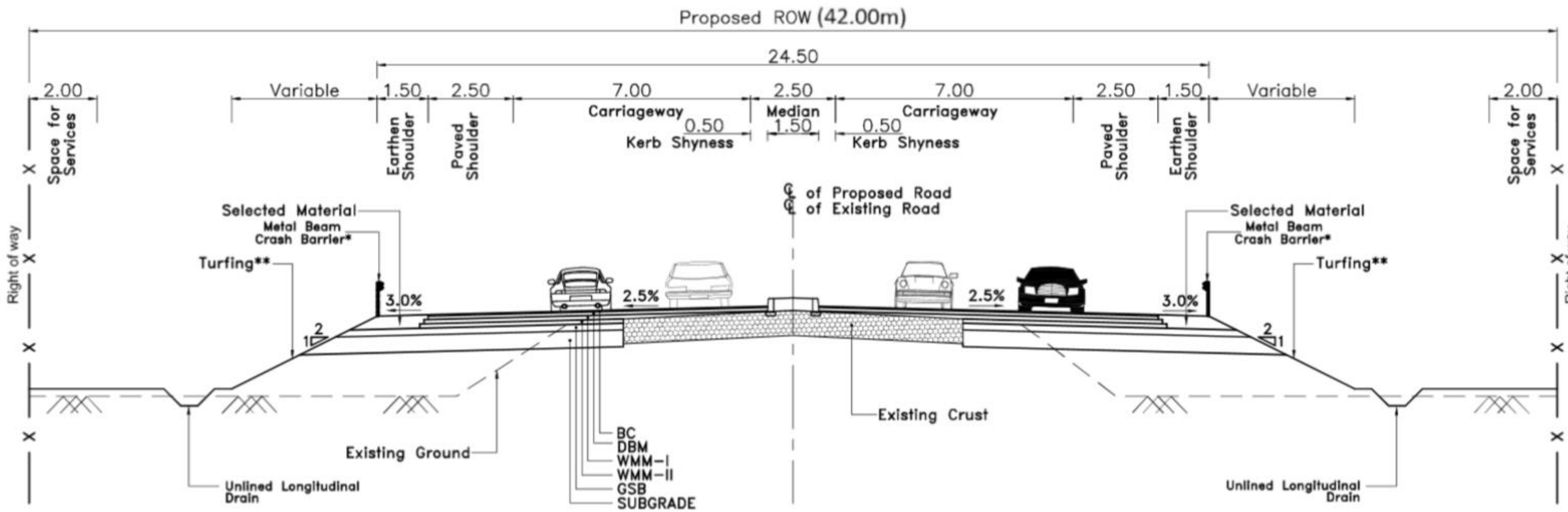
As per assessment at this stage tentative land acquisition is assessed as below:

- ***For Section 6: 57.49 Ha***

2.6 Typical Cross Sections

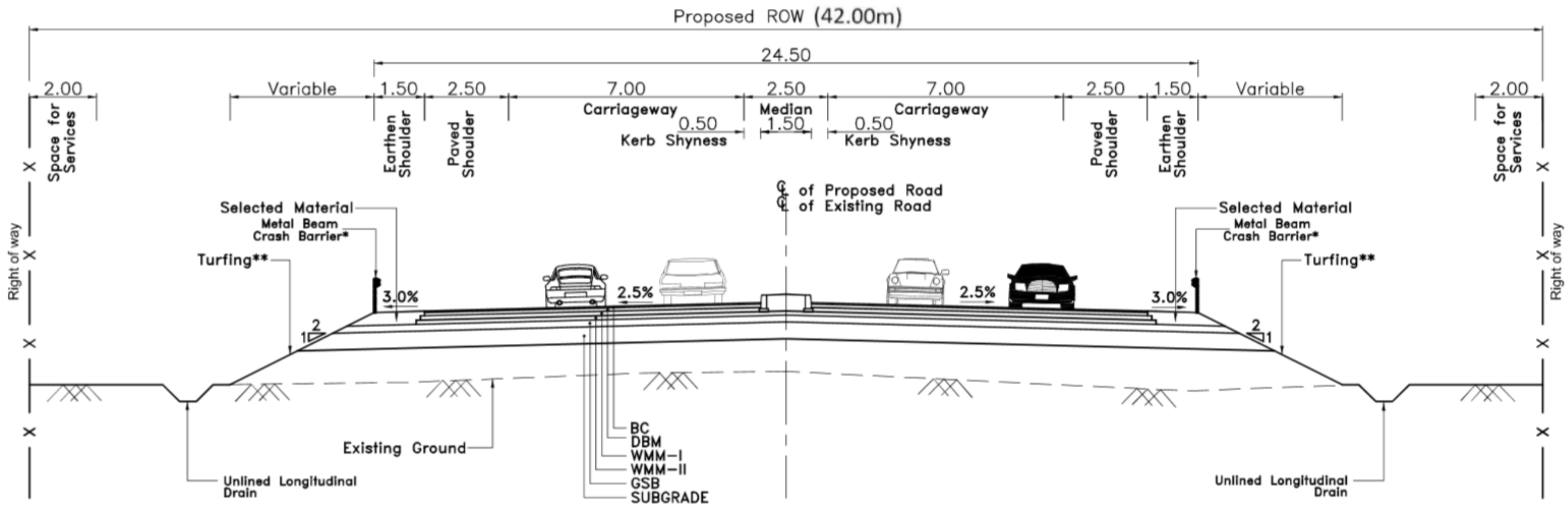
Selection of cross-section has been governed by the widening scheme adopted and by other considerations like land-use, drainage condition, traffic characteristics etc. The following typical cross sections are proposed for the widening/new construction of project highway. These depict generalized or critical/important features only within the related stretches.

Typical cross sections adopted for development of the project highway are attached as **Figure 2.1** to**2.9**.



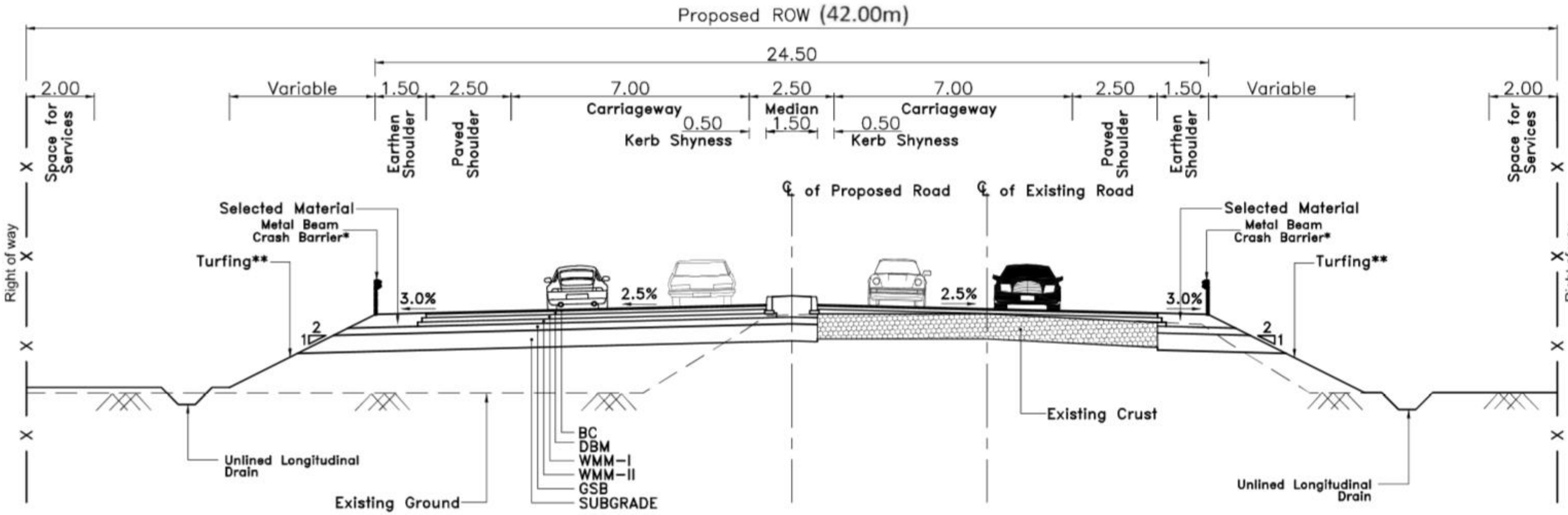
TYPE-1 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN RURAL AREA (CONCENTRIC WIDENING)

Fig 2.1



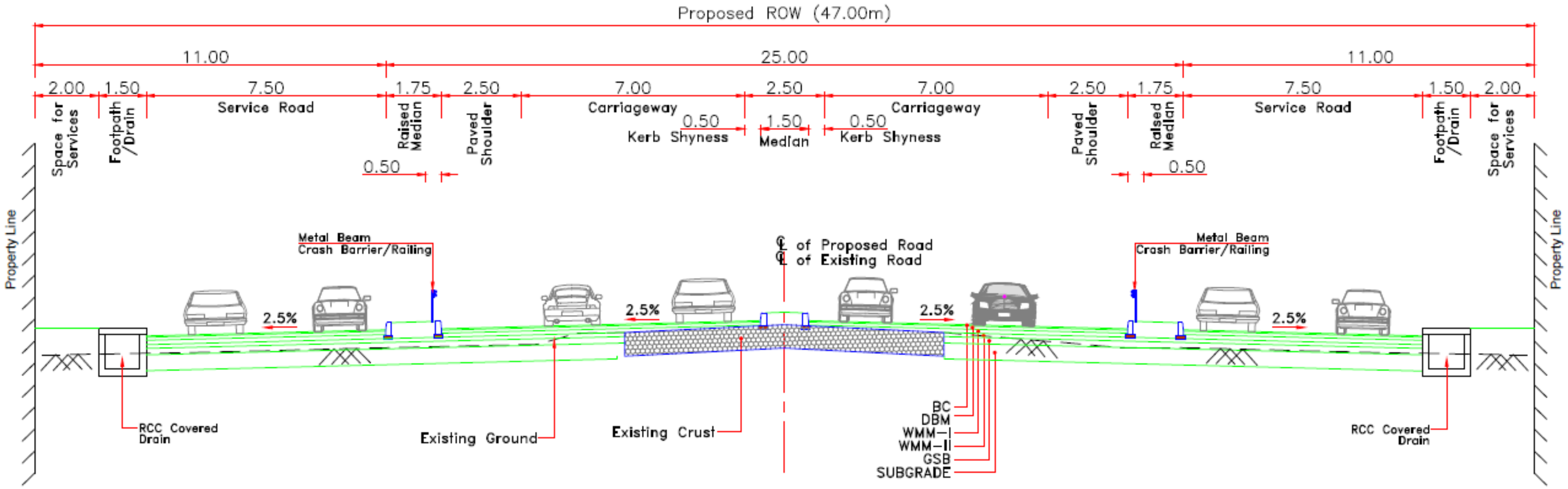
TYPE-1A : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN BYPASS/REALIGNMENT STRETCHES

Fig 2.2



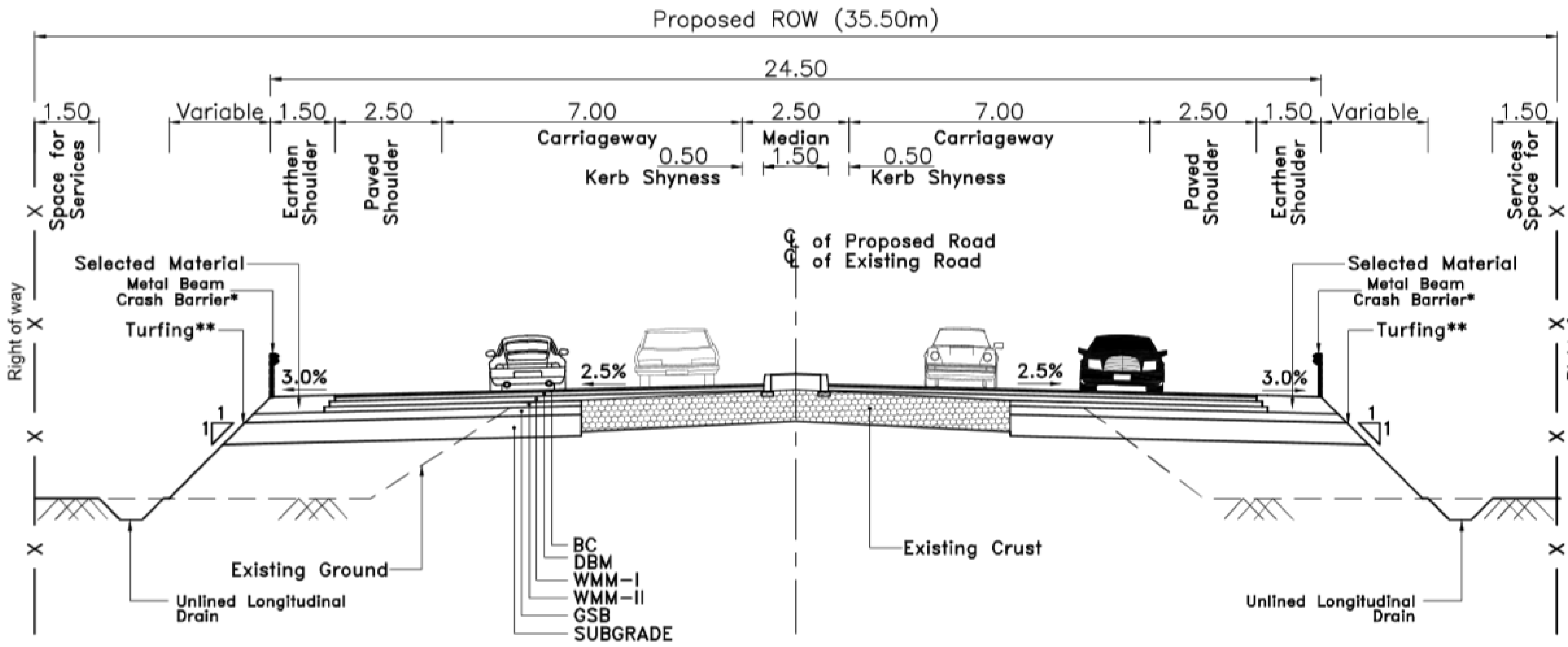
TYPE-2 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN RURAL AREA (ECCENTRIC WIDENING)

Fig 2.3



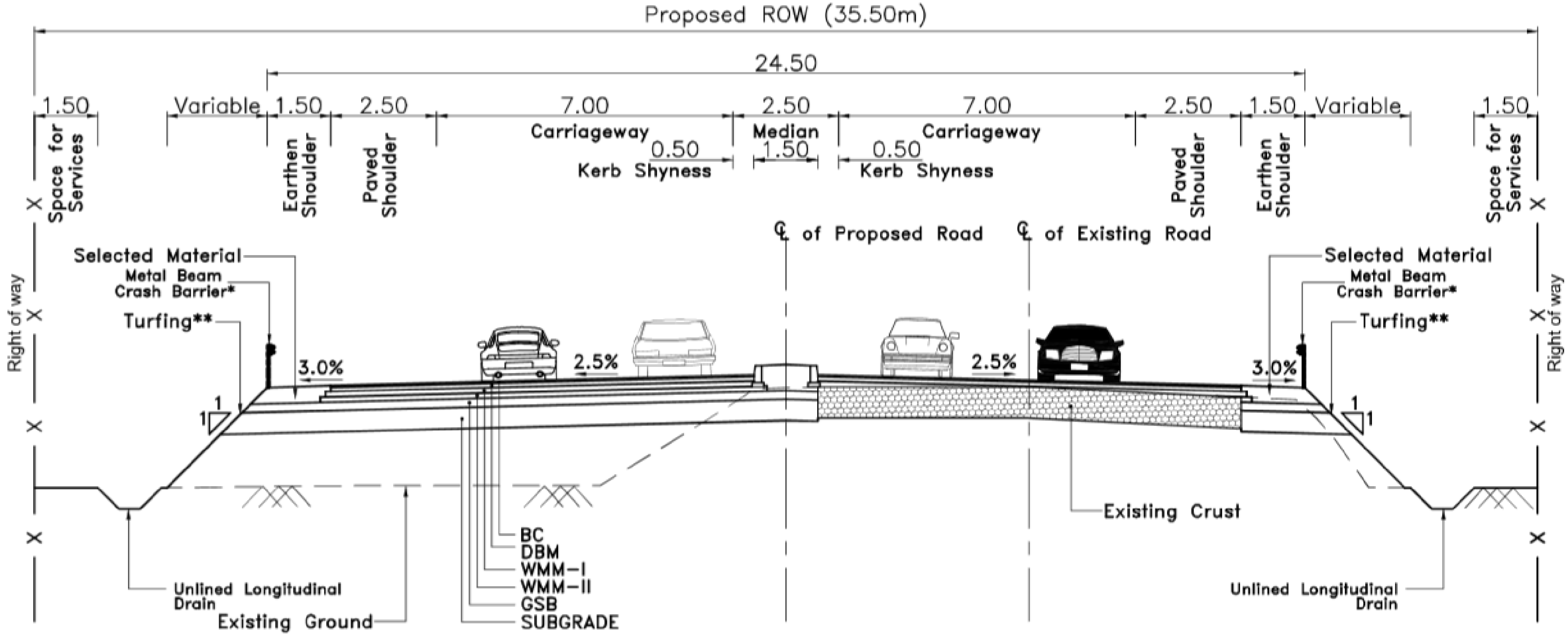
TYPE-3 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN AND WITH SERVICE ROAD ON BOTH SIDES IN BUILT UP AREA

Fig 2.4



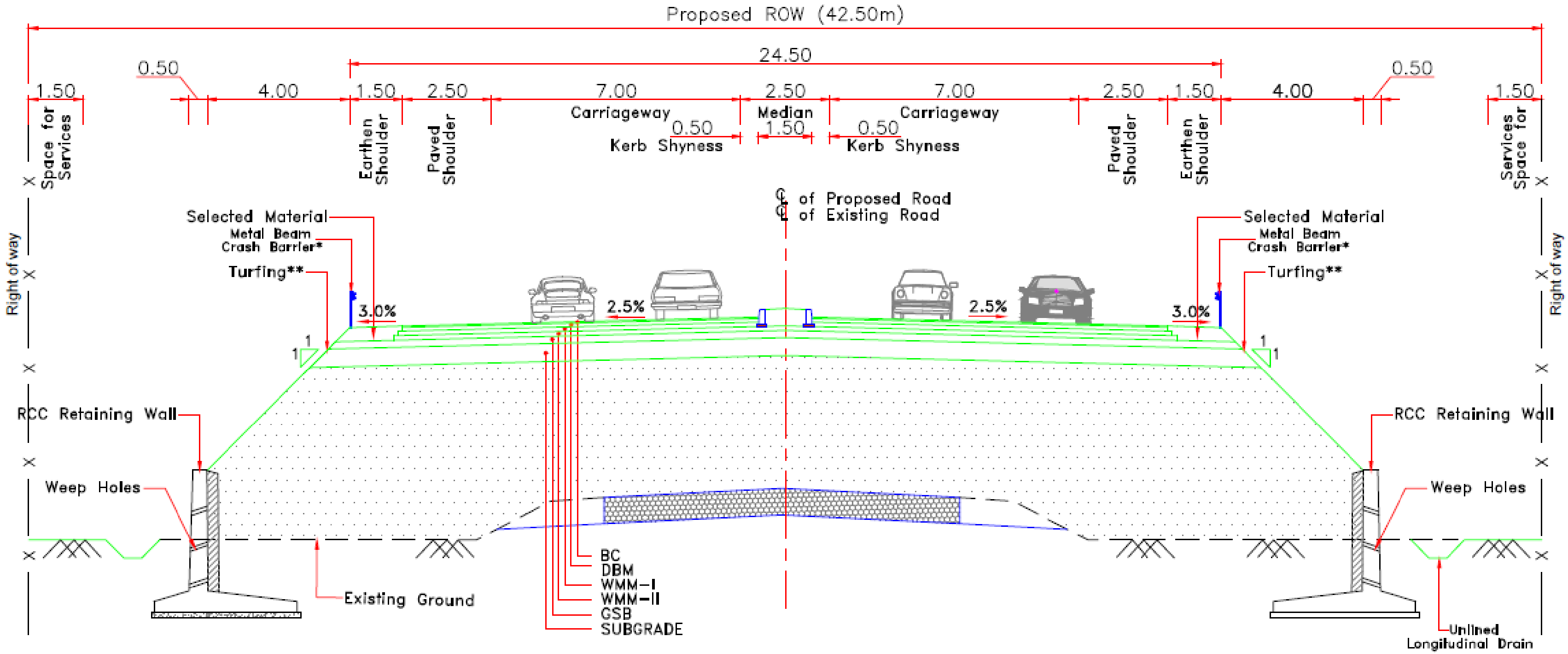
TYPE-4 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (CONCENTRIC WIDENING)

Fig 2.5



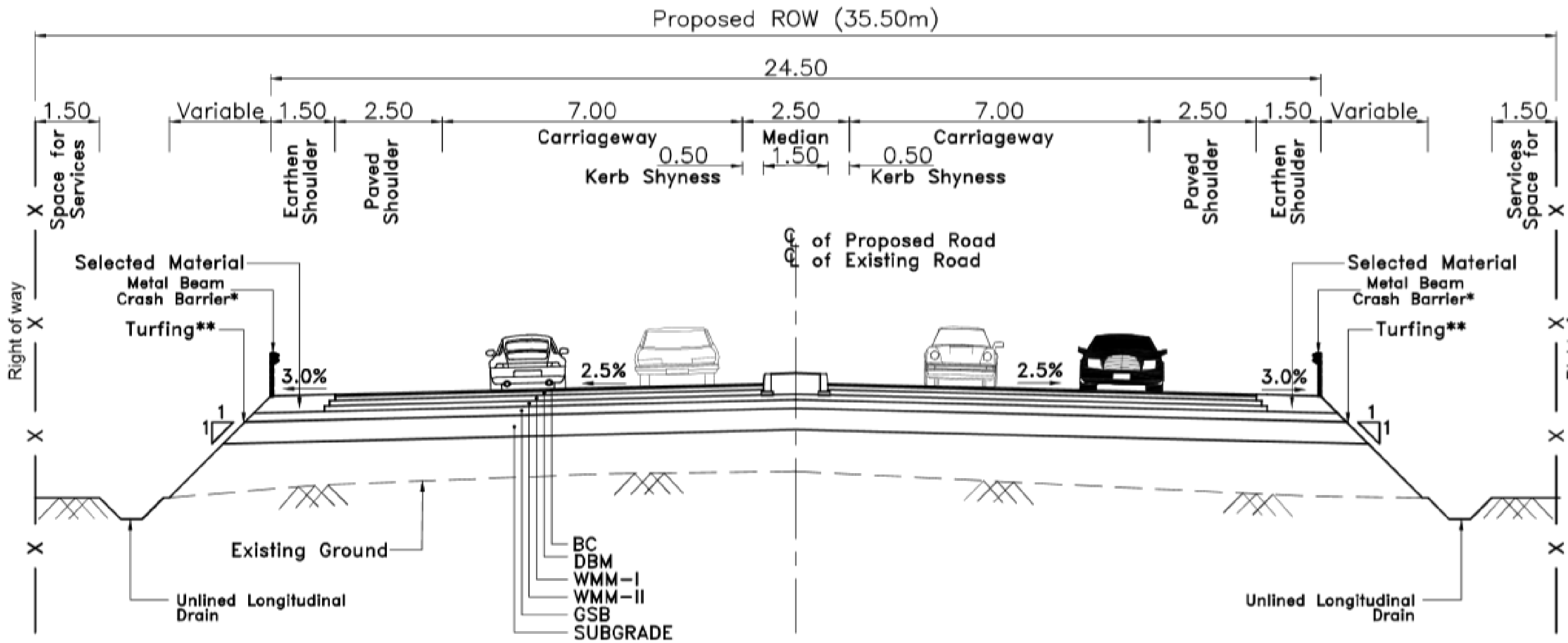
TYPE-4A : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (ECCENTRIC WIDENING)

Fig 2.6



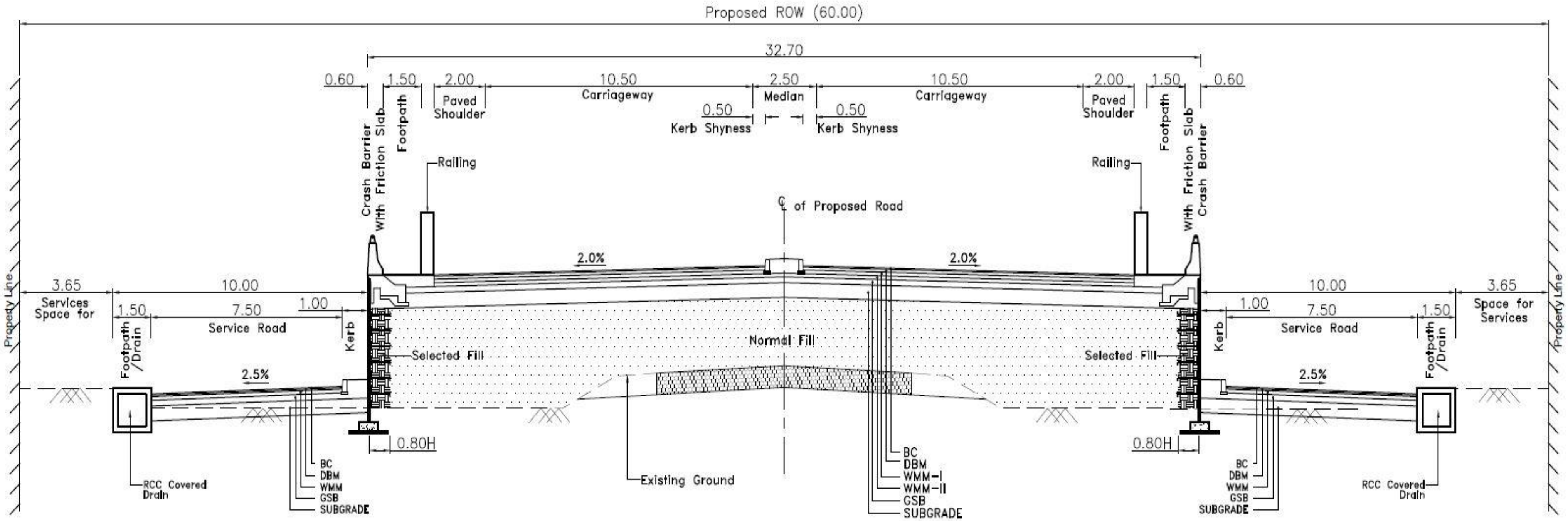
TYPE-5 : TYPICAL CROSS SECTION OF APPROACHES OF ELEPHANT UNDERPASSES (FOREST AREA)

Fig 2.7



TYPE-6 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (NEW CONSTRUCTION)

Fig 2.8



TYPE-07 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY AT GRADE SEPARATOR APPROACHES WITH SERVICE ROAD AND RE WALL ON BOTH SIDES

Fig 2.9

2.7 Widening and Strengthening of Carriageway

In all the cases the shoulders have to be rebuilt and all sections are required to be widened to four lane carriageway. The details of widening schedule are given in **Annexure-2.4.**

2.8 Extra Widening for curves with small radii

A Table for extra widening has been provided as **Annexure 2.1.**

2.9 Proposals for Realignments

Details of realignments proposed in the project road has been presented below.

Table 2.2: List of Realignments

Design Chainage (km)		Length (m)	TCS	Remarks
From	To			
Daboka – Lahorijan Road (NH 29)				
Section 6				
Newconstruction Bypass (Manja Bypass- 5190m and Realignment- 3824m)				

2.10 Road Side Drains

Due consideration has been given to drainage while preparing the design. The cross-sections incorporating roadside drains have been proposed at various stretches of the highway taking into account the existing and natural conditions as well as anticipated situation. In general, unlined trapezoidal drains have been considered on either side of road. Lined rectangular uncovered RCC drains have been considered for cut. Covered rectangular drain sections have been proposed in urban stretches. At super elevated sections with raised median, rectangular median drains have been considered. All the drains shall discharge into the nearest outfall. Summary of proposed drains is given in Table 2.3.

Table 2.3: Summary of Proposed Drains

Type of Drain	Side	Total Length including both side (m)	Applicable TCSs
Section-6			
Unlined Trapezoidal Drains	Both	34176	TCS – 1,1A,2,4,4A,5,6

2.11 Major and Minor Junctions

The proposed project road will form no major intersections with existing roads. Improvement of these intersections has been thought off with minimum of land acquisition. However, proper acceleration and deceleration lanes have been considered with proper traffic signage. In general, standard codal provisions have been followed for design of these intersections. Detail layouts are provided in Drawing Volume. There is no major intersections have been proposed. Besides, there are 18nos. of minor intersections along the project road which shall be operated as normal left-in and left-out principle. Improvement proposals of major and minor intersections are provided in**Table 2.4**.

Table 2.4A:List of Major Junctions

Sl No.	Design Chainage (Km)	Road Segment	Type of Intersection	Type	Side	Improvement Proposals	Remarks
1	125+500	Daboka - Lahorijan (NH 29)	Major	3 - legged	Right	At-grade Intersection	Start of Manja Bypass
2	130+500		Major	3 - legged	Right	At-grade Intersection	End of Manja Bypass

Table 2.4B: List of Minor Junctions

Sl. No.	Design Chainage (m)	Type of Intersection	Type	Side	Improvement Proposals
1	114+000	Minor	3 legged	Right	At Grade
2	114+530	Minor	3 legged	Right	At Grade
3	115+100	Minor	3 legged	Left	At Grade
4	116+500	Minor	3 legged	Left	At Grade
5	116+950	Minor	3 legged	Right	At Grade
6	117+420	Minor	3 legged	Left	At Grade
7	118+200	Minor	3 legged	Left	At Grade
8	119+860	Minor	3 legged	Left	At Grade
9	120+660	Minor	3 legged	Left	At Grade
10	121+300	Minor	3 legged	Right	At Grade
11	121+950	Minor	3 legged	Both	At Grade
12	123+800	Minor	3 legged	Left	At Grade
13	124+000	Minor	3 legged	Right	At Grade
14	124+600	Minor	3 legged	Left	At Grade
15	125+890	Minor	3 legged	Both	At Grade
16	126+850	Minor	3 legged	Both	At Grade
17	129+190	Minor	3 legged	Both	At Grade

2.12 Proposal for Congested Area

Where proposed alignment follows the built-up locations, both side service road has been proposed. And footpath cum covered drain is also proposed in both sides. Details of locations are mentioned below.

Table 2.5: List of Built-up Locations

Sl No.	Design Chainage (km)		Length (m)	Village
	From	To		
Daboka – Lahorijan Road (NH 29)				
Section 6				
NIL				

2.13 Bus Bays

Several towns, villages and settlements are abutting the project corridor and buses shall be one of the major mode of passenger traffic movement along the corridor. It is imperative to provide bus bays in order to eliminate the conflict between buses and other moving vehicles as well as to ensure safety of passengers boarding and alighting. Proposed bus bays have been kept sufficiently away from the intersections to avoid traffic congestion. Total 8 nos. of bus bays have been considered for this section.

2.14 Truck Lay Bye

2 location have been identified for proposed truck lay-byes.

2.15 Cattle Crossing

of these underpasses are provided in **Table 2.6**.

Table 2.6 : Details of Elephant Underpasses

Sl No.	Type of Underpasses	Design Chainage (km)	Span Arrangement (Nos. x Length in m)	Total Length (m)	Overall Width (m)	Structure Type
Section 6						
Nil						

2.16 Pedestrian Guard Rails

In bus bays and major junction, schools, RE wall section,service road.

2.17 Pedestrian Crossings

Has been provided at built up sections, major intersections, near school, bus bay location as per requirement.

2.18 Traffic Safety and Other Appurtenances

Following road furniture and miscellaneous items have been designed keeping safety aspect in mind.

- Road markings
- Road Signs & Delineators
- Crash Barriers
- Parapet Wall
- Noise Barriers
- Hard Topping

2.18.1 Road Markings

Road Markings on the carriageway and on the objects within and adjacent to the roadway are used as a means of guiding and controlling the traffic. They promote road safety and ensure smooth flow of traffic in the required paths of travel.

The location and type of marking lines, material and colour is followed using IRC: 35-2015 – "Code of Practice for Road Markings".

The road markings were carefully planned on carriageways, intersections, toll plazas and bridge locations.

2.18.2 Road Signs& Delineators

Road signs were planned to supply information, to regulate traffic by imparting messages to the drivers. The type, locations, sizes were planned using IRC: 67-2012 "Code of Practice for Road Sign".

The role of delineators is to provide visual assistance to driver about alignment of the road ahead, especially at night. Reflectors are used on the delineators for better night visibility. IRC: 79-1981 "Recommended Practice for Road Delineators" was followed to plan locations details. Two types of road delineators were planned i.e. hazard markers and object markers. Hazard markers are to define obstructions like guardrails, and abutments adjacent to the carriageway, for instance at culverts and bridges. Object markers are used to indicate hazards and obstructions within the vehicle flow path, at channeling islands close to intersections.

2.18.3 Crash Barrier

Metal Beam Crash Barriers are proposed/ provided for safety of the traffic on approaches of bridges, on the stretches where embankment height is more than 3m. It is also proposed on the curves for safety of traffic irrespective of embankment height as per NHAI Circular (NHAI/PH-II/NHDP/ADB/GM (NS)-I dated May 19, 2004).

3.0. PAVEMENT DESIGN

3.1.1 General

The pavement existing on the project stretch is flexible in nature. The project envisages new Two Lane with Paved Shoulder as well as Four Lanes with Paved Shoulder configuration. The general design Procedure for the flexible pavement for the proposed road as new construction of whole stretch as per the guidelines of IRC: 37-2018 – "Guidelines for the design of Flexible Pavements".

New pavement design is based on the design traffic (MSA) and the subgrade strength (soaked CBR).

3.1.2 Methodology of Pavement Design

Introduction

The flexible pavements are usually referred as a layered structure comprising generally bituminous surface like Bituminous Concrete (BC) and Dense Bituminous Macadam (DBM), Wet Mix Macadam (WMM) base and Granular Sub-Base (GSB) course of finite thickness, resting on subgrade of minimum thickness of 500 mm. The thickness design of these layers principally depends on the subgrade CBR and the traffic loads that the pavement has to carry during its design life. Ideally, the flexible pavement is built to such a depth that stresses on any given layer should not cause unwarranted rutting, fatigue, shoving, or other differential movements which may result in an uneven wearing surface. The chief function of the surfacing course is to provide a smooth wearing surface, resistant to traffic. However, the wearing course can provide some shearing resistance to the base structure and some added resistance to deformation.

Base courses are usually layers of aggregates that must possess high resistance to deformation in order to withstand the higher pressures imposed by wheel loads. High – quality processed aggregates are usually required, which also provide good internal drainage sub bases and generally made up of locally available aggregates, satisfying codal specification/requirements.

The design methodologies widely used for the flexible pavement design are Indian Road Congress (IRC) method, AASHTO methods and Asphalt Institute Method. For this project latest IRC (IRC:37-2018) method is used for designing the flexible pavement. The brief about the method is given below.

IRC: 37-2018 Method of New Flexible Pavement Design
It gives pavement design catalogue for subgrade CBR values ranging from 5% to 15 % and eight levels of design traffic ranging from 5 to 50 MSA. The pavement compositions given in the design catalogues are relevant to Indian conditions, materials and specifications. For higher traffic values, the pavement layer thicknesses are worked out using IITPAVE software.

IRC: 58-2015 Design of Rigid Pavement

IRC: 58-2015 “Guidelines for the design of plain jointed rigid pavements for highways” gives the design of rigid pavements and adopted for designing the rigid pavement for carriageway.

3.1.3 Design of New Flexible Pavement

IRC: 37-2018 method is adopted for the design which is based on the empirical – analytical approach, and provides catalogues for design of flexible pavements. The design inputs required for pavement design are explained as follows.

Design Theory

The pavement design method is based on elastic response of the pavement to traffic stresses (i.e. each of the materials in the pavement structure behaves in an elastic manner). The materials in the pavement are characterized by parameters whose values are determined from field and laboratory testing. The method assumes that failure will not occur as a result of permanent deformation of granular or bound materials (and this assumption will be valid as long as good construction procedures are followed, and the pavement is not subjected to very high wheel loads such as can be caused by a very heavily overloaded vehicle). The method also assumes that loss of pavement serviceability can occur due to:

- fatigue of bitumen bound or cemented layers due to repetitions of tensile strains at the bottom of such layers; and/or
- Permanent deformation of the sub-grade due to repeated vertical compressive strains induced in the sub-grade

The critical locations for pavement failure are therefore the bottom of bitumen bound layers (where tensile strains occur) and the top of the sub-grade (where compressive strains occur).

The base course and sub-grade are structural elements of the pavement. In conjunction with the overlying bituminous surface, their purpose is to distribute traffic wheel loads over the whole foundation. To perform this function, we build the base course and sub-grade with the necessary internal strength properties.

Bituminous pavement layers have both tensile and compressive strength to resist internal stresses. For example, Figure 3.1 shows how wheel load (W) slightly deflects the pavement structure, causing both tensile and compressive stresses within the pavement.

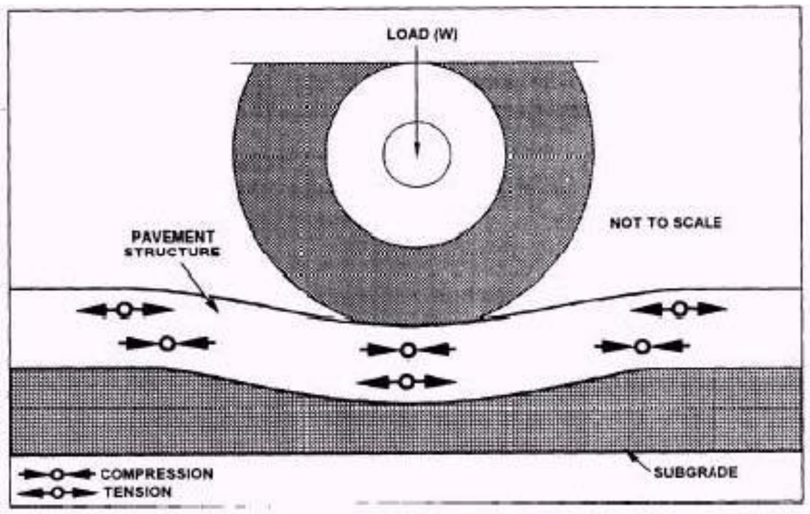


Figure 3.1 : Pavement Deflection Results in Tensile and Compressive Stresses in Pavement Structure

Required total thickness of the pavement layers is determined by engineering design procedure. Factors considered in the procedure are as follows:

- Traffic to be served initially and over the design service life of the pavement
- Strength and other pertinent properties of the prepared sub-grade
- Strength and other influencing characteristics of the materials available or chosen for the layers (or courses) in the total pavement structure
- Special factors such free swelling property of existing soil

Design of New Flexible Pavement

Design of new pavement has been carried out based on IRC 37-2018 "Guidelines for the Design of Flexible Pavements" for design life of 15 years. Procedure for the same is given below:

Step 1: To find out initial traffic in the year of completion of construction in terms of the number of the number of commercial vehicles per day (CVPD)
Step 2: To determine traffic growth rate factor by studying the past trends of traffic growth
Step 3: Design life of Pavement
Step 4: To find out Vehicle Damage Factor to convert the number of commercial vehicles of different axle loads and axle configuration to the number of standard axle load repetition. It may be obtained by conducting axle load survey at site.
Step 5: To find out lane distribution factor of traffic over the carriageway
Step6: To determine design traffic in cumulative number of standard axles (msa) by the following formula mentioned below:

N= [365 x {(1+r)n-1]/r}] x A x D x F

Where,

N = Cumulative number of standard axles to be catered for in the design in terms of msa
A = Initial traffic in the year of completion of construction in terms of number of commercial vehicles per day
D = Lane Distribution Factor
n = Design life in years
r = Annual growth rate of commercial vehicles
F = Vehicle damage factor

Step 7: To determine total pavement thickness and crust composition by charts/graphs with respect to CBR and cumulative number of standard axles.

Methodology flow chart for the design of new Flexible pavement has been shown in Figure 3.2 below.

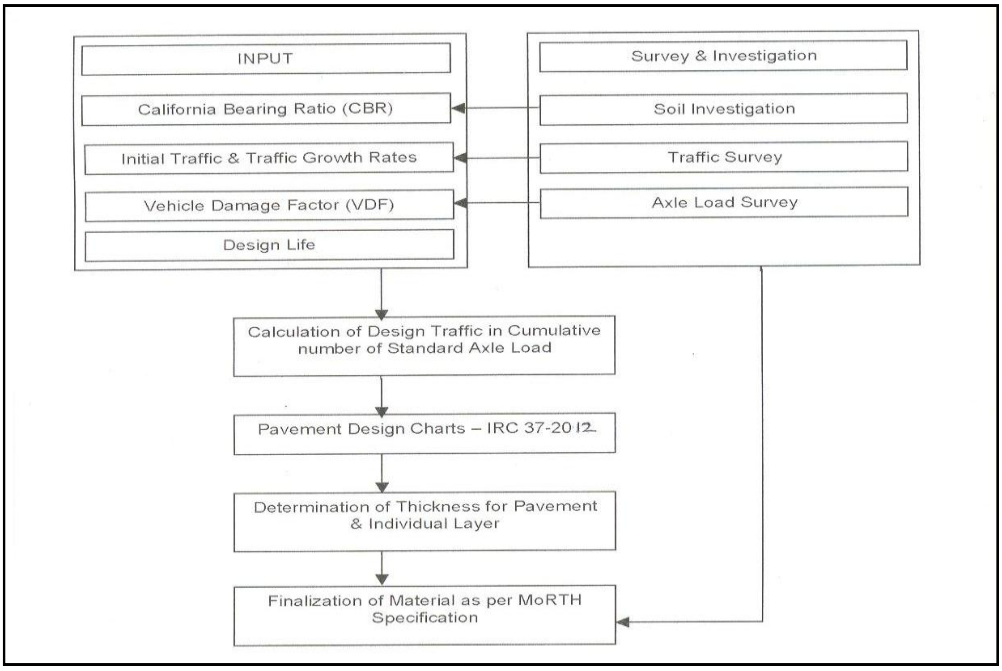


Figure 3.2 : Methodology Flow Chart for Design of New Flexible Pavement

MSA Calculation

MSA calculation has been presented in Annexure 6.4 of main report (Volume-I)

Adopted design life for pavement design has been considered for 15 years period.

Lane Distribution Factor

The lane distribution factor adopted for the project road is as given under:

- Two lane dual carriageway roads: 75 percent of number of Commercial vehicles in each direction as per IRC-37-2018.

Vehicle Damage Factor (VDF)

VDF summary is provided below.

Table 3.1: summary of VDF

Type of Vehicle	Daboka – Lahorijan Road (NH-29)	
	From Daboka to Lahorijan	From Lahorijan to Daboka
2-Axle Trucks	6.93	2.59
3-Axle Trucks	9.46	3.52
MAV	13.07	5.97
LCV	2.40	1.17
Bus	1.92	1.07

Design CBR

The subgrade CBR for design has been considered as 8.0%. Subgrade of 500 mm thickness is required as an integral part of the pavement structure. Details of msa calculated for flexible pavement design are provided in Table 3.2 for Daboka-Lahorijan Section.

For Daboka – Lahorijan stretch (NH-29)

Axle Load Survey conducted for the road stretch reveals that the Axle Load patterns differ significantly. Hence the pavement design is done considering different VDF values for the two carriageways as per Clause 4.5.2 of IRC:37-2018.

VDF values for LHS of the Daboka – Lahorijan stretch for 2-Axle Trucks, 3-Axle Trucks, MAVs, LCVs and Buses are 6.93, 9.46, 13.07, 2.40 and 1.92 respectively.
VDF values for RHS of the Daboka – Lahorijan stretch for 2-Axle Trucks, 3-Axle Trucks, MAVs, LCVs and Buses are 2.59, 3.52, 5.97, 1.17 and 1.07 respectively.

Traffic Surveys were conducted at Km. 62, Km. 127 and Km. 138.45 of the Daboka – Lahorijan road (NH-29) and the total stretch is to be developed as 4-Lane Dual Carriageway road. The details of the msa calculated are presented below:

Table 3.2A:msa for 4-Lane Dual Carriageway on LHS from Daboka to Lahorijan (NH-29)

Location	msa	Adopted msa
Km 62	**17.21**	**20 msa**
Km 127	**18.95**	

Km 138.45	**14.34**	

As all the calculated msas are below 20, the Design msa is considered as 20.

Table 3.2B: msa for 4-Lane Dual Carriageway on RHS from Daboka to Lahorijan (NH-29)

Location	msa	Adopted msa
Km 62	**7.59**	**20 msa**
Km 127	**8.34**	
Km 138.45	**6.32**	

As all the calculated msas are below 20, the Design msa is considered as 20.

Pavement layer thicknesses based on inputs mentioned above is given in Table 6.15. The GSB-II layer will be extended till earthen shoulder to facilitate of proper drainage in the pavement structure. The design has been carried out as per Plate-6 of IRC:37-2018.

Table 3.3 : Proposed Pavement Thickness

Pavement Layer Thickness in mm					
msa	**BC**	**DBM**	**WMM**	**GSB**	**Total Pavement Thickness**
20	30	90	250	200	570

3.1.4 Overlay Design

Condition of existing pavement is generally fair to poor.

Overlay design has been carried out based on the results of the FWD Tests. The Summary of the Analysis and selection of the 15th percentile moduli of in-service layers are presented in **Annexure 4.1 under Chapter 4 of Vol. I**.

The Analysis is carried out for the stretch. The 15th Percentile Moduli of the in-service layers and the average existing pavement thicknesses are presented in **Table 3.4**.

Table 3.4: 15th Percentile Moduli of the In-Service Layers and Average Existing Pavement Thicknesses

Road Sections	15^{th} Percentile Moduli of in-service Pavement Layers (MPa)			Existing Pavement Layers (mm)	
	Bituminous	Granluar	Subgrade	Bituminous	Granluar

Road Sections	15th Percentile Moduli of in-service Pavement Layers (MPa)			Existing Pavement Layers (mm)	
	Bituminous	**Granluar**	**Subgrade**	**Bituminous**	**Granluar**
Daboka to Lahorijan	918.5	215.6	98.0	53	311

Considering the above data, the remaining fatigue life and rutting life of the pavement are obtained from equation 16 and 17 respectively from IRC:115-2014 as presented in **Table 3.5.**

Table 3.5: Fatigue Life and Rutting Life of Pavement

Road Sections	Tensile Strain at the Bottom of the Bituminous Layer	Vertical Strain at the Top of the Subgrade	Fatigue Life	Rutting Life
Daboka to Lahorijan	480.6×10^{-6}	652.4×10^{-6}	1.70	3.90

From the above results it is evident that the existing pavement crust is not sufficient to carry the corresponding design traffic. Hence considering the bituminous overlay the results obtained are presented in **Table 3.6**.

Table 3.6:Results of FWD Data Analysis and Overlay Thickness

Road Sections	Considered Bituminous Overlay (mm)	Tensile Strain at the bottom of the Bituminous Layer	Vertical Strain at the top of the subgrade	Fatigue Life (msa)	Rutting Life (msa)	Design Traffic (msa)
Daboka to Lahorijan	*30mm BC + 70mm DBM*	250.2×10^{-6}	343.1×10^{-6}	21.48	71.88	20

Hence, the bituminous overlay as presented in **Table 3.6** above is recommended.

Annexures (Road Works)

Annexure 2.1 : Extra widening

Sl No.	Design Chainage (km)		Total Curve Length (m)	Spiral Curve Length (m)	Circular Curve (m)	Type of Horizontal Curve	Radius (m)	Extra Widths of Pavement (m)				Side of Extra Widening
								On Spiral Curve		On Circular Curve		
	From	To						Max	Min	Max	Min	
Section-3												
1	114+729.298	114+797.288	345	115	115	S-C-S	300	0.6	0	0.6	0.6	BOTH
2	115+135.652	115+283.236	225	75	75	S-C-S	280	0.6	0	0.6	0.6	BOTH
3	116+013.112	116+283.112	225	75	75	S-C-S	300	0.6	0	0.6	0.6	BOTH
4	117+030.759	117+065.180	165	55	55	S-C-S	300	0.6	0	0.6	0.6	BOTH
5	117+266.947	117+295.891	225	75	75	S-C-S	170	0.6	0	0.6	0.6	BOTH
6	117+445.421	117+576.309	210	70	70	S-C-S	170	0.6	0	0.6	0.6	BOTH
7	117+733.663	117+827.996	210	70	70	S-C-S	200	0.6	0	0.6	0.6	BOTH
8	118+205.926	118+352.617	180	60	60	S-C-S	300	0.6	0	0.6	0.6	BOTH
9	119+706.498	119+888.656	165	55	55	S-C-S	170	0.6	0	0.6	0.6	BOTH
10	119+954.198	120+055.636	120	40	40	S-C-S	260	0.6	0	0.6	0.6	BOTH
11	120+215.572	120+308.992	75	25	25	S-C-S	170	0.6	0	0.6	0.6	BOTH
12	120+412.470	120+462.294	120	40	40	S-C-S	170	0.6	0	0.6	0.6	BOTH
13	120+634.957	120+648.424	120	40	40	S-C-S	300	0.6	0	0.6	0.6	BOTH
14	120+816.339	120+860.614	225	75	75	S-C-S	200	0.6	0	0.6	0.6	BOTH
15	120+994.292	121+122.242	180	60	60	S-C-S	200	0.6	0	0.6	0.6	BOTH
16	121+274.528	121+361.581	180	60	60	S-C-S	200	0.6	0	0.6	0.6	BOTH
17	121+680.117	121+824.412	135	45	45	S-C-S	200	0.6	0	0.6	0.6	BOTH
18	122+829.890	122+913.966	75	25	25	S-C-S	170	0.6	0	0.6	0.6	BOTH
19	123+533.314	123+724.601	90	30	30	S-C-S	300	0.6	0	0.6	0.6	BOTH
20	130+288.753	130+467.037	345	115	115	S-C-S	300	0.6	0	0.6	0.6	BOTH
21	130+844.797	130+869.576	225	75	75	S-C-S	300	0.6	0	0.6	0.6	BOTH

Annexure-2.2: Horizontal Alignment Report

Element ID	ELEMENT DETAILS								Transition Details				Horizontal Intersection Point (HIP)			Deflectin Angle			Speed (Kmph)	Superelevtion	Extra Widening (m)
	Start Chainage	End Chainage	Start Easting	Start Northing	End Easting	End Northing	Radius (m)	Direction	Start Chainage	L1	L2	End Chainage	Chainage	Easting	Northing	Deg	Min	Sec			
Section-6																					
1	113+988.151	114+149.158	535179.082	2875089.674	535338.992	2875087.687	400	Left	113+873.151	55	55	114+264.158	114069.759	535258.835	2875072.368	23	3	45.082	65	4.7%	-
2	114+494.776	114+638.347	535667.243	2875195.247	535801.149	2875246.948	2000	Left					114566.592	535735.124	2875218.693	4	6	46.826	100	-	-
3	114+729.298	114+797.288	535885.886	2875279.816	535952.797	2875291.03	300	Right	114+654.298	115	115	114+872.288	114763.44	535918.704	2875289.231	12	59	6.432	80	5.0%	0.6
4	115+135.652	115+283.236	536290.494	2875273.065	536428.729	2875319.675	280	Left	115+060.652	75	75	115+358.236	115211.201	536365.9	2875277.721	30	11	59.633	65	5.0%	0.6
5	115+572.340	115+809.879	536645.844	2875510.241	536736.932	2875724.704	350	Left	115+497.340	75	75	115+884.879	115695.888	536729.24	2875601.396	38	53	7.987	80	5.0%	-
6	116+013.112	116+283.112	536733.621	2875927.79	536865.59	2876152.945	300	Right	115+938.112	75	75	116+358.112	116158.028	536745.224	2876072.242	51	33	58.592	80	5.0%	0.6
7	116+469.346	116+498.639	537029.9	2876240.414	537054.541	2876256.243	400	Left	116+414.346	75	75	116+553.639	116483.999	537042.511	2876247.877	4	11	45.535	80	5.0%	-
8	116+701.349	116+869.180	537213.393	2876382.135	537348.526	2876481.581	2000	Right					116785.314	537278.872	2876434.695	4	48	28.752	100	-	-
9	117+030.759	117+065.180	537480.727	2876574.331	537505.309	2876598.399	300	Left	116+955.759	55	55	117+140.180	117047.988	537493.709	2876585.659	6	34	25.951	65	5.0%	0.6
10	117+266.947	117+295.891	537627.719	2876758.467	537650.447	2876776.332	170	Right	117+196.947	75	75	117+365.891	117281.454	537638.321	2876768.369	9	45	18.176	65	5.0%	0.6
11	117+445.421	117+576.309	537785.515	2876839.821	537858.11	2876944.853	170	Left	117+375.421	70	70	117+646.309	117514.302	537843.091	2876877.63	44	6	49.164	65	5.0%	0.6
12	117+733.663	117+827.996	537868.08	2877101.652	537904.455	2877187.744	200	Right	117+673.663	70	70	117+887.996	117781.724	537875.923	2877149.069	27	1	27.701	65	5.0%	0.6
13	118+205.926	118+352.617	538167.241	2877459.077	538225.945	2877591.919	300	Left	118+130.926	60	60	118+427.617	118280.769	538213.164	2877518.175	28	0	57.689	65	5.0%	0.6
14	118+620.921	119+273.526	538242.819	2877859.584	538710.232	2878207.352	400	Right	118+565.921	75	75	119+328.526	119045.976	538291.749	2878281.813	93	28	43.73	80	5.0%	-
15	119+706.498	119+888.656	539130.138	2878102.122	539238.268	2877966.352	170	Right	119+666.498	55	55	119+928.656	119807.423	539224.505	2878066.334	61	23	35.901	65	5.0%	0.6
16	119+954.198	120+055.636	539241.489	2877900.921	539267.626	2877803.572	260	Left	119+929.198	40	40	120+080.636	120005.57	539244.94	2877849.664	22	21	14.063	65	5.0%	0.6
17	120+215.572	120+308.992	539343.316	2877662.736	539353.722	2877571.076	170	Right	120+175.572	25	25	120+348.992	120263.494	539361.438	2877618.373	31	29	7.726	65	5.0%	0.6
18	120+412.470	120+462.294	539328.231	2877470.852	539327.478	2877421.211	170	Left	120+372.470	40	40	120+502.294	120437.562	539324.191	2877446.087	16	47	33.041	65	5.0%	0.6
19	120+634.957	120+648.424	539365.443	2877252.886	539366.809	2877239.49	300	Right	120+559.957	40	40	120+723.424	120641.692	539366.276	2877246.203	2	34	18.857	65	5.0%	0.6
20	120+816.339	120+860.614	539365.23	2877071.723	539374.648	2877028.554	200	Left	120+756.339	75	75	120+920.614	120838.567	539367.54	2877049.615	12	41	1.622	65	5.0%	0.6
21	120+994.292	121+122.242	539430.394	2876907.203	539431.101	2876781.425	200	Right	120+934.292	60	60	121+182.242	121060.542	539451.58	2876844.431	36	39	18.373	65	5.0%	0.6
22	121+274.528	121+361.581	539368.392	2876642.817	539360.06	2876556.852	200	Left	121+214.528	60	60	121+421.581	121318.755	539354.721	2876600.756	24	56	20.19	65	5.0%	0.6
23	121+473.057	121+541.145	539386.328	2876448.601	539397.028	2876381.412	500	Right	121+428.057	60	60	121+586.145	121507.154	539393.969	2876415.372	7	48	8.421	80	5.0%	-
24	121+680.117	121+824.412	539406.916	2876242.889	539481.382	2876122.937	200	Left	121+620.117	45	45	121+884.412	121755.566	539421.524	2876168.867	41	20	15.104	65	5.0%	0.6
25	122+016.444	122+292.353	539647.462	2876026.744	539851.582	2875843.426	750	Right	121+951.444	60	60	122+357.353	122155.976	539766.574	2875954.073	21	4	40.235	100	5.0%	-
26	122+471.093	122+491.418	539956.013	2875698.397	539969.198	2875682.931	400	Left	122+416.093	65	65	122+546.418	122481.258	539962.409	2875690.497	2	54	41.019	80	5.0%	-
27	122+617.522	122+715.006	540058.623	2875594.047	540119.641	2875518.219	500	Right	122+592.522	55	55	122+740.006	122666.419	540092.84	2875559.117	11	10	15.081	80	5.0%	-
28	122+829.890	122+913.966	540184.28	2875423.473	540254.679	2875379.089	170	Left	122+759.890	25	25	122+983.966	122872.806	540213.877	2875392.395	28	20	11.792	65	5.0%	0.6
29	123+065.464	123+139.998	540404.472	2875357.993	540478.887	2875354.174	900	Left	123+035.464	70	70	123+169.998	123102.752	540441.6	2875354.542	4	44	41.961	100	4.9%	-
30	123+533.314	123+724.601	540872.098	2875353.558	541042.486	2875273.963	300	Right	123+458.314	30	30	123+799.601	123632.335	540970.428	2875341.88	36	31	59.25	65	5.0%	0.6
31	124+057.592	124+183.052	541260.865	2875022.901	541365.362	2874954.401	400	Left	123+942.592	75	75	124+298.052	124120.841	541307.698	2874980.389	17	58	15.206	80	5.0%	-
32	124+681.736	124+761.269	541842.01	2874808.653	541913.403	2874773.795	500	Right	124+586.736	95	95	124+856.269	124721.587	541879.096	2874794.069	9	6	49.494	100	5.0%	-
33	124+924.292	124+961.628	542047.67	2874681.433	542080.182	2874663.105	400	Left	124+869.292	115	115	125+016.628	124942.973	542063.498	2874671.51	5	20	52.409	100	5.0%	-
34	125+393.859	125+449.814	542478.79	2874496.122	542532.474	2874480.417	600	Left	125+313.859	80	80	125+529.814	125421.857	542505.265	2874487.017	5	20	35.738	100	5.0%	-
35	126+151.860	126+344.915	543222.734	2874353.609	543385.084	2874252.641	400	Right	126+036.860	115	115	126+459.915	126250.306	543316.334	2874323.104	27	39	10.717	100	5.0%	-
36	127+109.669	127+412.769	543843.85	2873641.267	544114.175	2873520.929	400	Left	126+994.669	115	115	127+527.769	127268.913	543955.059	2873527.289	43	24	57.253	100	5.0%	-
37	128+459.980	128+696.116	545156.417	2873618.444	545376.193	2873541.907	400	Right	128+344.980	115	115	128+811.116	128581.601	545277.941	2873613.587	33	49	26.205	100	5.0%	-
38	129+265.483	129+413.517	545783.006	2873144.236	545849.4	2873012.87	400	Right	129+150.483	115	115	129+528.517	129340.357	545828.497	2873084.767	21	12	15.691	100	5.0%	-
39	130+288.753	130+467.037	545979.309	2872147.645	546072.819	2871998.928	300	Left	130+213.753	115	115	130+542.037	130380.614	546003.295	2872058.97	34	2	59.103	100	5.0%	0.6
40	130+844.797	130+869.576	546387.078	2871789.722	546409.686	2871779.598	300	Left	130+769.797	75	75	130+944.576	130857.194	546398.173	2871784.193	4	43	56.393	80	5.0%	0.6
41	131+059.239	131+072.751	546591.854	2871727.164	546604.5	2871722.408	350	Right	130+999.239	75	75	131+132.751	131065.995	546598.223	2871724.908	2	12	42.907	80	5.0%	-

Annexure-2.3A: Vertical Alignment Report (LME)

PVI	PVI			Grade		Diff. in Grade (%)	Chainage(m)		Level(m)		Type Of Curve	K Value
	Chainage (m)	Level (m)	Curve Length	IN (%)	OUT (%)		Start of Curve	End of Curve	Start of Curve	End of Curve		
Section-6												
1	113+933.269	145.367	180	3.295	-2.184	-5.479	113+843.269	114+023.269	142.401	143.402	Hog	32.852
2	114+111.032	141.485	110	-2.184	3.257	5.441	114+056.032	114+166.032	142.686	143.276	Sag	20.218
3	114+287.846	147.244	150	3.257	-3.246	-6.503	114+212.846	114+362.846	144.801	144.81	Hog	23.067
4	114+654.944	135.329	280	-3.246	3.265	6.511	114+514.944	114+794.944	139.873	139.9	Sag	43.005
5	115+007.356	146.836	70	3.265	4.458	1.193	114+972.356	115+042.356	145.693	148.396	Sag	58.665
6	115+266.924	158.409	125	4.458	0.738	-3.720	115+204.424	115+329.424	155.622	158.87	Hog	33.601
7	115+605.446	160.908	110	0.738	-4.991	-5.729	115+550.446	115+660.446	160.502	158.163	Hog	19.201
8	115+740.000	154.193	60	-4.991	-2.745	2.246	115+710.000	115+770.000	155.69	153.369	Sag	26.723
9	115+941.470	148.662	60	-2.745	-3.057	-0.312	115+911.470	115+971.470	149.486	147.745	Hog	192.508
10	116+117.427	143.283	60	-3.057	-2.248	0.809	116+087.427	116+147.427	144.2	142.609	Sag	74.158
11	116+393.238	137.083	170	-2.248	0.544	2.792	116+308.238	116+478.238	138.994	137.545	Sag	60.888
12	116+561.780	138	60	0.544	0	-0.544	116+531.780	116+591.780	137.837	138	Hog	110.278
13	116+809.322	138	60	0	0.351	0.351	116+779.322	116+839.322	138	138.105	Sag	170.885
14	116+941.140	138.463	75	0.351	2.403	2.052	116+903.640	116+978.640	138.331	139.364	Sag	36.551
15	117+088.336	142	100	2.403	0	-2.403	117+038.336	117+138.336	140.798	142	Hog	41.614
16	117+253.808	142	100	0	-1.039	-1.039	117+203.808	117+303.808	142	141.48	Hog	96.221
17	117+443.844	140.025	170	-1.039	2.517	3.556	117+358.844	117+528.844	140.908	142.165	Sag	47.8
18	117+890.691	151.273	200	2.517	-3.255	-5.772	117+790.691	117+990.691	148.756	148.018	Hog	34.65
19	118+210.133	140.876	180	-3.255	0.409	3.664	118+120.133	118+300.133	143.805	141.244	Sag	49.133
20	118+570.000	142.347	120	0.409	-1.306	-1.715	118+510.000	118+630.000	142.102	141.563	Hog	69.97
21	118+810.000	139.212	130	-1.306	0.563	1.869	118+745.000	118+875.000	140.061	139.578	Sag	69.545
22	119+127.559	141	150	0.563	0	-0.563	119+052.559	119+202.559	140.578	141	Hog	266.408
23	119+713.353	141	150	0	-1.516	-1.516	119+638.353	119+788.353	141	139.863	Hog	98.96
24	119+930.340	137.711	130	-1.516	0.498	2.014	119+865.340	119+995.340	138.696	138.034	Sag	64.57
25	120+089.114	138.501	60	0.498	-0.758	-1.256	120+059.114	120+119.114	138.352	138.273	Hog	47.773
26	120+246.403	137.308	60	-0.758	-0.57	0.188	120+216.403	120+276.403	137.536	137.137	Sag	318.416
27	120+315.911	136.912	60	-0.57	-0.536	0.034	120+285.911	120+345.911	137.083	136.751	Sag	1752.13
28	120+415.967	136.376	60	-0.536	0.363	0.899	120+385.967	120+445.967	136.537	136.485	Sag	66.76
29	120+629.438	137.151	60	0.363	1.145	0.782	120+599.438	120+659.438	137.042	137.494	Sag	76.761
30	120+721.952	138.21	115	1.145	-0.625	-1.770	120+664.452	120+779.452	137.552	137.851	Hog	64.997
31	121+075.765	136	60	-0.625	0	0.625	121+045.765	121+105.765	136.187	136	Sag	96.058
32	121+370.000	136	60	0	-0.33	-0.330	121+340.000	121+400.000	136	135.901	Hog	181.817
33	121+489.393	135.606	85	-0.33	1.1	1.430	121+446.893	121+531.893	135.746	136.074	Sag	59.426
34	121+744.495	138.413	60	1.1	-0.029	-1.129	121+714.495	121+774.495	138.083	138.404	Hog	53.126
35	122+130.102	138.301	60	-0.029	2.306	2.335	122+100.102	122+160.102	138.31	138.993	Sag	25.693
36	122+224.022	140.467	60	2.306	0.892	-1.414	122+194.022	122+254.022	139.775	140.735	Hog	42.429
37	122+367.170	141.744	60	0.892	3.082	2.190	122+337.170	122+397.170	141.476	142.669	Sag	27.397
38	122+519.144	146.428	210	3.082	-2.717	-5.799	122+414.144	122+624.144	143.192	143.575	Hog	36.211
39	122+737.015	140.508	190	-2.717	0.22	2.937	122+642.015	122+832.015	143.089	140.717	Sag	64.678
40	123+055.022	141.209	105	0.22	1.975	1.755	123+002.522	123+107.522	141.093	142.246	Sag	59.838
41	123+244.270	144.947	90	1.975	-0.303	-2.278	123+199.270	123+289.270	144.058	144.811	Hog	39.51
42	123+516.488	144.123	80	-0.303	1.844	2.147	123+476.488	123+556.488	144.244	144.861	Sag	37.267
43	123+696.210	147.437	140	1.844	-1.119	-2.963	123+626.210	123+766.210	146.146	146.654	Hog	47.255

44	123+975.251	144.315	90	-1.119	1.028	2.147	123+930.251	124+020.251	144.819	144.778	Sag	41.921
45	124+100.000	145.598	120	1.028	-1.476	-2.504	124+040.000	124+160.000	144.981	144.712	Hog	47.919
46	124+321.204	142.333	130	-1.476	0.918	2.394	124+256.204	124+386.204	143.292	142.929	Sag	54.311
47	124+949.895	148.102	150	0.918	-1.378	-2.296	124+874.895	125+024.895	147.414	147.068	Hog	65.337
48	125+250.000	143.966	80	-1.378	-0.523	0.855	125+210.000	125+290.000	144.517	143.757	Sag	93.6
49	125+414.284	143.106	160	-0.523	3.281	3.804	125+334.284	125+494.284	143.525	145.731	Sag	42.054
50	125+962.232	161.085	500	3.281	-3.26	-6.541	125+712.232	126+212.232	152.882	152.935	Hog	76.44
51	126+616.753	139.748	250	-3.26	-0.561	2.699	126+491.753	126+741.753	143.823	139.046	Sag	92.636
52	127+305.451	135.883	100	-0.561	0.314	0.875	127+255.451	127+355.451	136.164	136.04	Sag	114.29
53	127+754.517	137.292	150	0.314	-0.695	-1.009	127+679.517	127+829.517	137.057	136.77	Hog	148.634
54	127+981.859	135.711	200	-0.695	1.803	2.498	127+881.859	128+081.859	136.406	137.514	Sag	80.066
55	128+164.326	139	150	1.803	0	-1.803	128+089.326	128+239.326	137.648	139	Hog	83.217
56	128+508.270	139	80	0	-0.549	-0.549	128+468.270	128+548.270	139	138.78	Hog	145.683
57	128+646.487	138.241	120	-0.549	2.331	2.880	128+586.487	128+706.487	138.57	139.639	Sag	41.669
58	128+807.769	142	180	2.331	0	-2.331	128+717.769	128+897.769	139.902	142	Hog	77.23
59	129+133.370	142	100	0	1.465	1.465	129+083.370	129+183.370	142	142.733	Sag	68.24
60	129+284.658	144.217	160	1.465	-0.617	-2.082	129+204.658	129+364.658	143.045	143.723	Hog	76.824
61	129+515.513	142.792	160	-0.617	0.986	1.603	129+435.513	129+595.513	143.286	143.581	Sag	99.776
62	129+941.848	146.997	185	0.986	-0.35	-1.336	129+849.348	130+034.348	146.085	146.673	Hog	138.39
63	130+293.925	145.763	200	-0.35	1.551	1.901	130+193.925	130+393.925	146.113	147.314	Sag	105.191
64	130+618.979	150.804	250	1.551	-1.615	-3.166	130+493.979	130+743.979	148.865	148.786	Hog	78.974
65	130+817.519	147.598	85	-1.615	0.355	1.970	130+775.019	130+860.019	148.284	147.749	Sag	43.149
66	130+945.086	148.051	100	0.355	-1.704	-2.059	130+895.086	130+995.086	147.873	147.199	Hog	48.571
67	131+034.948	146.52	60	-1.704	-0.251	1.453	131+004.948	131+064.948	147.031	146.445	Sag	41.293
68	131+114.330	146.321	70	-0.251	0.621	0.872	131+079.330	131+149.330	146.409	146.539	Sag	80.259

Annexure-2.3B:-Vertical Alignment Report (RME)

PVI	PVI			Grade		Diff. in Grade (%)	Chainage(m)		Level(m)		Type Of Curve	K Value
	Chainage (m)	Level (m)	Curve Length	IN (%)	OUT (%)		Start of Curve	End of Curve	Start of Curve	End of Curve		
Section-6												
1	113+933.269	145.367	180	3.295	-2.818	-6.113	113+843.269	114+023.269	142.401	142.831	Hog	29.444
2	114+091.390	140.911	110	-2.818	3.224	6.042	114+036.390	114+146.390	142.461	142.684	Sag	18.207
3	114+287.846	147.244	150	3.224	-3.246	-6.470	114+212.846	114+362.846	144.826	144.81	Hog	23.187
4	114+654.944	135.329	280	-3.246	3.265	6.511	114+514.944	114+794.944	139.873	139.9	Sag	43.005
5	115+007.356	146.836	70	3.265	4.458	1.193	114+972.356	115+042.356	145.693	148.396	Sag	58.665
6	115+266.924	158.409	125	4.458	0.738	-3.720	115+204.424	115+329.424	155.622	158.87	Hog	33.601
7	115+605.446	160.908	110	0.738	-4.991	-5.729	115+550.446	115+660.446	160.502	158.163	Hog	19.201
8	115+740.000	154.193	60	-4.991	-2.745	2.246	115+710.000	115+770.000	155.69	153.369	Sag	26.723
9	115+941.470	148.662	60	-2.745	-3.057	-0.312	115+911.470	115+971.470	149.486	147.745	Hog	192.508
10	116+117.427	143.283	60	-3.057	-2.248	0.809	116+087.427	116+147.427	144.2	142.609	Sag	74.158
11	116+393.238	137.083	170	-2.248	0.544	2.792	116+308.238	116+478.238	138.994	137.545	Sag	60.888
12	116+561.780	138	60	0.544	0	-0.544	116+531.780	116+591.780	137.837	138	Hog	110.278
13	116+720.820	138	60	0	-0.844	-0.844	116+690.820	116+750.820	138	137.747	Hog	71.05
14	116+807.856	137.265	60	-0.844	0.399	1.243	116+777.856	116+837.856	137.518	137.385	Sag	48.241
15	116+937.326	137.782	75	0.399	2.793	2.394	116+899.826	116+974.826	137.632	138.829	Sag	31.329
16	117+088.336	142	100	2.793	0	-2.793	117+038.336	117+138.336	140.603	142	Hog	35.801
17	117+253.808	142	100	0	-1.039	-1.039	117+203.808	117+303.808	142	141.48	Hog	96.221
18	117+443.844	140.025	170	-1.039	2.517	3.556	117+358.844	117+528.844	140.908	142.165	Sag	47.8
19	117+890.691	151.273	200	2.517	-3.255	-5.772	117+790.691	117+990.691	148.756	148.018	Hog	34.65
20	118+210.133	140.876	180	-3.255	0.409	3.664	118+120.133	118+300.133	143.805	141.244	Sag	49.133
21	118+570.000	142.347	120	0.409	-1.306	-1.715	118+510.000	118+630.000	142.102	141.563	Hog	69.97
22	118+810.000	139.212	130	-1.306	0.563	1.869	118+745.000	118+875.000	140.061	139.578	Sag	69.545
23	119+127.559	141	150	0.563	0	-0.563	119+052.559	119+202.559	140.578	141	Hog	266.408
24	119+713.353	141	150	0	-1.516	-1.516	119+638.353	119+788.353	141	139.863	Hog	98.96
25	119+930.340	137.711	130	-1.516	0.498	2.014	119+865.340	119+995.340	138.696	138.034	Sag	64.57
26	120+089.114	138.501	60	0.498	-0.758	-1.256	120+059.114	120+119.114	138.352	138.273	Hog	47.773
27	120+246.403	137.308	60	-0.758	-0.57	0.188	120+216.403	120+276.403	137.536	137.137	Sag	318.416
28	120+315.911	136.912	60	-0.57	-0.536	0.034	120+285.911	120+345.911	137.083	136.751	Sag	1752.13
29	120+415.967	136.376	60	-0.536	0.363	0.899	120+385.967	120+445.967	136.537	136.485	Sag	66.76
30	120+629.438	137.151	60	0.363	1.145	0.782	120+599.438	120+659.438	137.042	137.494	Sag	76.761
31	120+721.952	138.21	115	1.145	-0.625	-1.770	120+664.452	120+779.452	137.552	137.851	Hog	64.997
32	121+075.765	136	60	-0.625	0	0.625	121+045.765	121+105.765	136.187	136	Sag	96.058
33	121+385.646	136	60	0	0.904	0.904	121+355.646	121+415.646	136	136.271	Sag	66.386
34	121+770.793	139.481	200	0.904	-0.328	-1.232	121+670.793	121+870.793	138.577	139.153	Hog	162.309
35	122+130.102	138.301	60	-0.328	2.306	2.634	122+100.102	122+160.102	138.4	138.993	Sag	22.774
36	122+224.022	140.467	60	2.306	0.892	-1.414	122+194.022	122+254.022	139.775	140.735	Hog	42.429
37	122+367.170	141.744	60	0.892	3.082	2.190	122+337.170	122+397.170	141.476	142.669	Sag	27.397
38	122+519.144	146.428	210	3.082	-2.717	-5.799	122+414.144	122+624.144	143.192	143.575	Hog	36.211
39	122+737.015	140.508	190	-2.717	0.22	2.937	122+642.015	122+832.015	143.089	140.717	Sag	64.678
40	123+055.022	141.209	105	0.22	1.975	1.755	123+002.522	123+107.522	141.093	142.246	Sag	59.838

PVI	PVI			Grade		Diff. in Grade (%)	Chainage(m)		Level(m)		Type Of Curve	K Value
	Chainage (m)	Level (m)	Curve Length	IN (%)	OUT (%)		Start of Curve	End of Curve	Start of Curve	End of Curve		
41	123+244.270	144.947	90	1.975	-0.303	-2.278	123+199.270	123+289.270	144.058	144.811	Hog	39.51
42	123+564.392	143.978	80	-0.303	2.624	2.927	123+524.392	123+604.392	144.099	145.028	Sag	27.334
43	123+696.210	147.437	140	2.624	-1.421	-4.045	123+626.210	123+766.210	145.6	146.442	Hog	34.608
44	123+965.562	143.609	90	-1.421	1.479	2.900	123+920.562	124+010.562	144.249	144.275	Sag	31.027
45	124+100.000	145.598	120	1.479	-1.171	-2.650	124+040.000	124+160.000	144.71	144.895	Hog	45.274
46	124+353.503	142.629	130	-1.171	0.918	2.089	124+288.503	124+418.503	143.391	143.226	Sag	62.241
47	124+949.895	148.102	150	0.918	-1.378	-2.296	124+874.895	125+024.895	147.414	147.068	Hog	65.333
48	125+250.000	143.966	14	-1.378	-0.523	0.855	125+243.160	125+256.840	144.06	143.93	Sag	15.999
49	125+414.284	143.106	160	-0.523	3.281	3.804	125+334.284	125+494.284	143.525	145.731	Sag	42.056
50	125+962.232	161.085	500	3.281	-3.26	-6.541	125+712.232	126+212.232	152.882	152.935	Hog	76.44
51	126+616.753	139.748	250	-3.26	-0.561	2.699	126+491.753	126+741.753	143.823	139.046	Sag	92.636
52	127+305.451	135.883	100	-0.561	0.314	0.875	127+255.451	127+355.451	136.164	136.04	Sag	114.29
53	127+754.517	137.292	150	0.314	-0.695	-1.009	127+679.517	127+829.517	137.057	136.77	Hog	148.634
54	127+981.859	135.711	200	-0.695	1.803	2.498	127+881.859	128+081.859	136.406	137.514	Sag	80.066
55	128+164.326	139	150	1.803	0	-1.803	128+089.326	128+239.326	137.648	139	Hog	83.217
56	128+508.270	139	80	0	-0.549	-0.549	128+468.270	128+548.270	139	138.78	Hog	145.683
57	128+646.487	138.241	120	-0.549	2.331	2.880	128+586.487	128+706.487	138.57	139.639	Sag	41.669
58	128+807.769	142	180	2.331	0	-2.331	128+717.769	128+897.769	139.902	142	Hog	77.23
59	129+133.370	142	100	0	1.465	1.465	129+083.370	129+183.370	142	142.733	Sag	68.24
60	129+284.658	144.217	160	1.465	-0.617	-2.082	129+204.658	129+364.658	143.045	143.723	Hog	76.824
61	129+515.513	142.792	160	-0.617	0.986	1.603	129+435.513	129+595.513	143.286	143.581	Sag	99.776
62	129+941.848	146.997	185	0.986	-0.35	-1.336	129+849.348	130+034.348	146.085	146.673	Hog	138.39
63	130+293.925	145.763	200	-0.35	1.551	1.901	130+193.925	130+393.925	146.113	147.314	Sag	105.191
64	130+618.979	150.804	250	1.551	-1.615	-3.166	130+493.979	130+743.979	148.865	148.786	Hog	78.974
65	130+817.519	147.598	85	-1.615	0.355	1.970	130+775.019	130+860.019	148.284	147.749	Sag	43.149
66	130+945.086	148.051	100	0.355	-1.704	-2.059	130+895.086	130+995.086	147.873	147.199	Hog	48.571
67	131+034.948	146.52	60	-1.704	-0.251	1.453	131+004.948	131+064.948	147.031	146.445	Sag	41.293
68	131+114.330	146.321	70	-0.251	0.621	0.872	131+079.330	131+149.330	146.409	146.539	Sag	80.258

Annexure 2.4C : Widening Scheme (Section 6)

Design Chainage		Length (m)	TCS Type	Description
From	To			
113830	114020	190	2	Eccentric Widening - Right side
114020	114470	450	2	Eccentric Widening - Left side
114470	114590	120	1A	New Construction - Realignment
114590	114640	50	2	Eccentric Widening - Right side
114640	114700	60	2	Eccentric Widening - Left side
114700	114890	190	1A	New Construction - Realignment
114890	114940	50	2	Eccentric Widening - Left side
114940	115170	230	1A	New Construction - Realignment
115170	115300	130	1	Concentric Widening
115300	115360	60	2	Eccentric Widening - Left side
115360	115520	160	2	Eccentric Widening - Right side
115520	115590	70	1	Concentric Widening
115590	115750	160	2	Eccentric Widening - Right side
115750	115850	100	2	Eccentric Widening - Left side
115850	115980	130	1A	New Construction - Realignment
115980	116110	130	2	Eccentric Widening - Right side
116110	116180	70	1	Concentric Widening
116180	116500	320	1A	New Construction - Realignment
116500	116634.7	134.7	2	Eccentric Widening - Left side
116634.7	116645.3	10.6	STR	MNB
116645.3	117050	404.7	2	Eccentric Widening - Left side
117050	117120	70	2	Eccentric Widening - Right side
117120	117162.5	42.5	1A	New Construction - Realignment
117162.5	117177.6	15.1	STR	MNB
117177.6	117260	82.4	1A	New Construction - Realignment
117260	117410	150	2	Eccentric Widening - Right side
117410	117470	60	2	Eccentric Widening - Left side
117470	117540	70	1	Concentric Widening
117540	117570	30	2	Eccentric Widening - Left side
117570	117710	140	1A	New Construction - Realignment
117710	117810	100	2	Eccentric Widening - Right side
117810	118150	340	1A	New Construction - Realignment
118150	118250	100	1	Concentric Widening
118250	118420	170	1A	New Construction - Realignment
118420	118460	40	2	Eccentric Widening - Left side
118460	118550	90	2	Eccentric Widening - Right side
118550	118610	60	1	Concentric Widening
118610	118710	100	2	Eccentric Widening - Right side
118710	118830	120	2	Eccentric Widening - Left side
118830	119490	660	1A	New Construction - Realignment
119490	119580	90	STR	MJB
119580	119840	260	1A	New Construction - Realignment
119840	120200	360	1	Concentric Widening
120200	120310	110	2	Eccentric Widening - Right side
120310	120420	110	1	Concentric Widening
120420	120590	170	2	Eccentric Widening - Left side
120590	120750	160	2	Eccentric Widening - Right side
120750	121100	350	1A	New Construction - Realignment
121100	121191.1	91.1	2	Eccentric Widening - Right side
121191.1	121208.9	17.8	STR	MNB
121208.9	121330	121.1	2	Eccentric Widening - Right side
121330	121470	140	1A	New Construction - Realignment
121470	121690	220	2	Eccentric Widening - Right side
121690	121730	40	1	Concentric Widening
121730	121830	100	2	Eccentric Widening - Left side
121830	122030	200	2	Eccentric Widening - Right side
122030	122210	180	2	Eccentric Widening - Left side
122210	122350	140	2	Eccentric Widening - Right side

122350	122390	40	1	Concentric Widening
122390	122490	100	2	Eccentric Widening - Left side
122490	122830	340	1	Concentric Widening
122830	122900	70	2	Eccentric Widening - Left side
122900	123150	250	1	Concentric Widening
123150	123650	500	1A	New Construction - Realignment
123650	124210	560	2	Eccentric Widening - Left side
124210	124480	270	2	Eccentric Widening - Right side
124480	124625	145	2	Eccentric Widening - Left side
124625	124780	155	2	Eccentric Widening - Right side
124780	124970	190	2	Eccentric Widening - Left side
124970	125340	370	2	Eccentric Widening - Right side
125340	126866.3	1526.3	1A	New Construction - Bypass
126866.3	126873.8	7.5	STR	MNB
126873.8	128292.5	1418.7	1A	New Construction - Bypass
128292.5	128307.5	15	STR	MNB
128307.5	128704	396.5	1A	New Construction - Bypass
128704	128716	12	STR	MNB
128716	129872	1156	1A	New Construction - Bypass
129872	129938	66	STR	MJB
129938	130630	692	1A	New Construction - Bypass
130630	130840	210	2	Eccentric Widening - Left side
130840	130990	150	1A	New Construction - Realignment
130990	131152	162	2	Eccentric Widening - Left side
Total =		**17322**		

PART-II

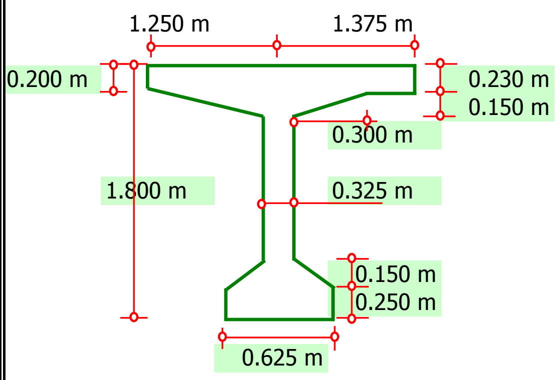
DESIGN OF STRUCTURES

DETAIL DESIGN CALCULATION OF RCC T GIRDER 22M .

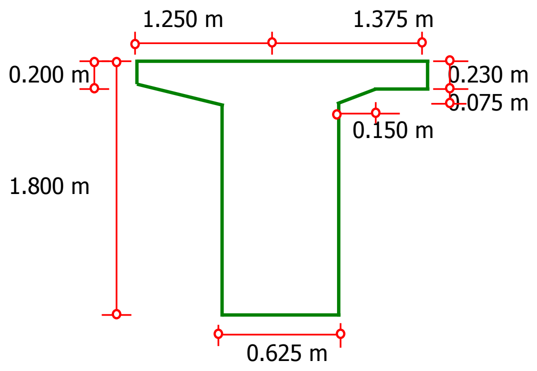
1.0 INPUT DATA

Span length c/c of Expansion Joint	=	22.000	m	
Span length c/c of Bearing	=	21.000	m	
Skew Angle	=	0.0	Degree	Clockwise (-)
Distance between C/L of Brg. and C/L of Exp. Joint	=	0.500	m	
No. of Longitudinal Girders	=	5	Nos	
No. of Cross Girders	=	3	Nos	
C/C spacing of Cross Girder	=	10.500	m	
C/C spacing of Longitudinal Girder	=	2.750	m	
Total Width of Superstructure	=	13.500	m	
Length of Girder Beyond Bearing	=	0.300	m	
Thickness of Deck slab	=	0.230	m	
Thickness of Deck slab at Exp. Joint	=	0.400	m	
Length of Solid Portion of Web	=	0.700	m	
Length of Tappering Portion of Web	=	0.900	m	
Depth of T Girder	=	1.500	m	
Depth of Cross Girder	=	1.250	m	
Cantilever length	=	1.250	m	
C/C spacing of Longitudinal Girder (Skew)	=	2.750	m	
Distance between C/L of Brg. and C/L of Exp. Joint (Normal)	=	0.500	m	
Bearing size in longitudinal direction (POT-PTFE/Elastomeric)	=	0.600	m	

1.1 Section Dimensions of Outer Girder

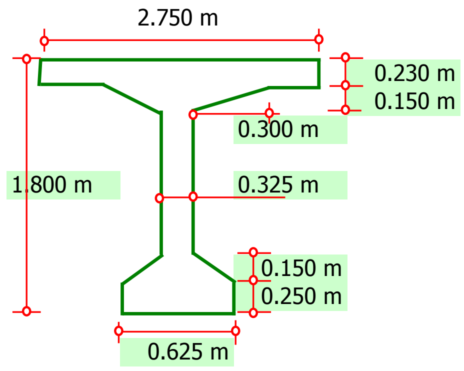


Mid Span Section

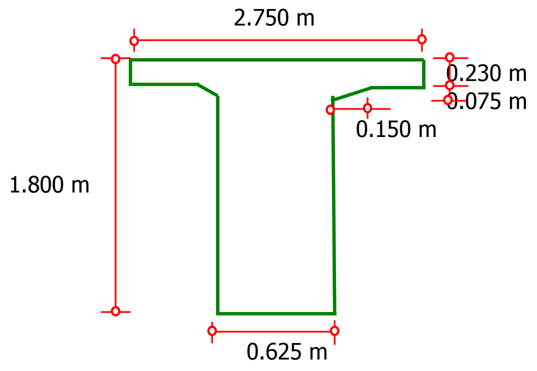


Support Section

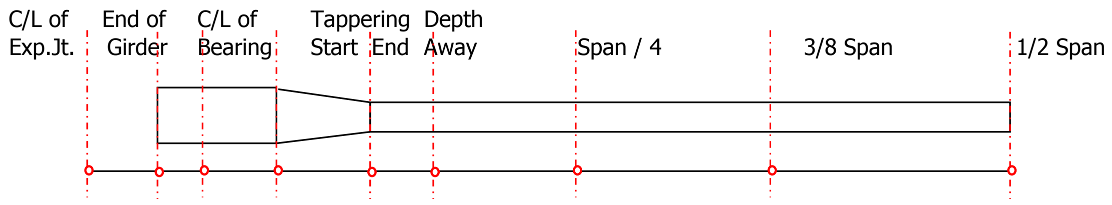
1.2 Section Dimensions of Inner Girder



Mid Span Section



Support Section

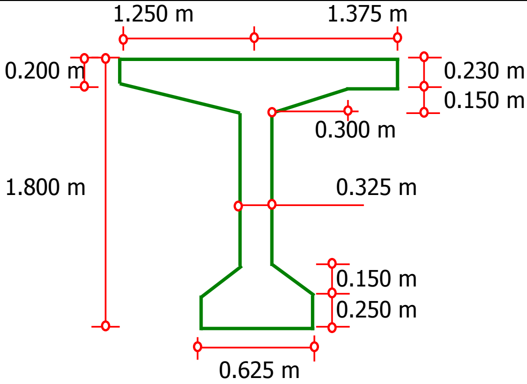


2.0 Section Properties of Longitudinal Members (Outer Girder)

C/C spacing of Bearing	=	21.000	m
C/C spacing of Longitudinal Girder	=	2.750	m
Cantilever length	=	1.250	m
Bearing size in longitudinal direction	=	0.600	m

2.1 Outer T - Girder at the Span

Member no: _GO-SPAN *



Effective Span,

l_0 = Min. { Distance from Brg. to Brg. ; Clear distance b/w supports + Eff. Depth* }

l_0 = Min. { 21.000 m + ; 20.400 m + 1.530 m }

* Effective depth assumed 0.85 times of Overall depth

l_0 = 21.000 m

b_1 = 1.088 m

b_2 = 1.213 m

$beff_1$ = Min. { $0.2\ b_1 + 0.1\ l_0$ = 2.318 m ; $0.2\ l_0$ = 4.200 m }

= 2.318 m

$beff_2$ = Min. { $0.2\ b_2 + 0.1\ l_0$ = 2.343 m ; $0.2\ l_0$ = 4.200 m }

= 2.343 m

beff = Min. { Σ beff,i + bw = 4.985 m ; b = 2.625 m }

beff = 2.625 m

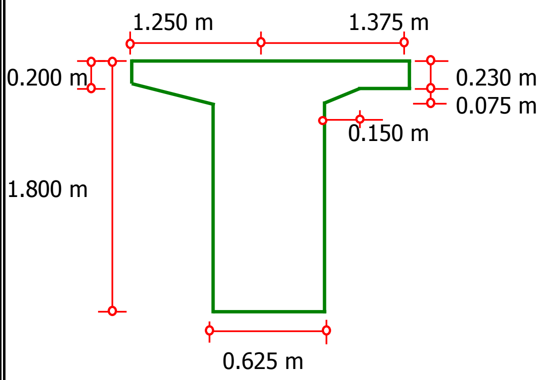
SL.No.	A	y	Ay	Ay^2	I_o	$I_o + Ay^2$	u
1. Rectangular top Flange Cantilever	0.25000	1.70000	0.42500	0.72250	0.000833	0.72333	1.45000
2. Rectangular top Flange Inner	0.31625	1.68500	0.53288	0.89790	0.001394	0.89930	2.51750
3. Triangular top Flange Cantilever	0.09788	1.54000	0.15073	0.23212	0.000203	0.23232	1.10230
4. Triangular top Flange Haunch	0.02250	1.52000	0.03420	0.05198	0.000028	0.05201	0.33541
5. Web	0.42900	0.91000	0.39039	0.35525	0.062291	0.41755	2.04000
5. Bottom Bulb, Triangle	0.02250	0.30000	0.00675	0.00203	0.000028	0.00205	0.42426
6. Bottom Bulb, Rectangle	0.15625	0.12500	0.01953	0.00244	0.000814	0.00326	1.12500
Total Section Σ	1.29438		1.55948	2.2642	0.066	2.32982	8.99447
	ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma (I_o + Ay^2)$	Σu
	0.7191	1.8000					

Section Properties of Total Section

Area	**ΣA**	=	1.2944	m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.2048	m
Moment of inertia of girder **(Iz) = $\Sigma(I_o + A.y^2) - \Sigma A.Y^2$**		=	0.4509	m^4
Perimeter in contact with atmoshphere **u**		=	8.9945	m

2.2 Outer T - Girder at the Support

Member no: _GO-SUP *



SL.No.	A	y	Ay	Ay^2	I_o	$I_o + Ay^2$		u
1. Rectangular top Flange Cantilever	0.25000	1.70000	0.42500	0.72250	0.000833	0.72333		1.45000
2. Rectangular top Flange Inner	0.31625	1.68500	0.53288	0.89790	0.001394	0.89930		2.51750
3. Triangular top Flange Cantilever	0.04922	1.56500	0.07703	0.12055	0.000040	0.12059		0.94336
4. Triangular top Flange Haunch	0.00563	1.54500	0.00869	0.01343	0.000002	0.01343		0.16771
5. Web	0.98125	0.78500	0.77028	0.60467	0.201557	0.80623		3.61500
Total Section Σ	1.60234		1.81388	2.3591	0.204	2.56288		8.69357
	ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma (I_o + Ay^2)$		Σu
	0.8902	1.8000						

Section Properties of Total Section

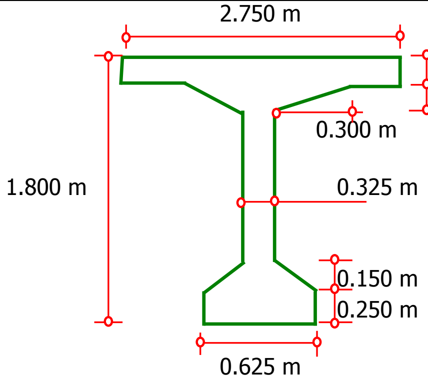
Area	**ΣA**	=	1.6023	m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.1320	m
Moment of inertia of Girder **(Iz) = $\Sigma(Io+A.y^2)-\Sigma A.Y^2$**		=	0.5095	m^4

3.0 Section Properties of Longitudinal Members (Inner Girder)

C/C spacing of Bearing	=	21.000	m
C/C spacing of Longitudinal Girder	=	2.750	m
Cantilever length	=	1.250	m
Bearing size in longitudinal direction	=	0.600	m

3.1 Inner T - Girder at the Span

Member no: _GI-SPAN *



0.0225

Effective Span,

l_0 = Min. { Distance from Brg. to Brg.
Clear distance b/w supports + Eff. Depth*

l_0 = Min. { 21.000 m +
20.400 m + 1.530 m

* Effective depth assumed 0.85 times of Overall depth

l_0 = 21.000 m

$b_{1,2}$ = 1.2125 m

$beff_{1,2}$ = Min. { $0.2\ b_{1,2} + 0.1\ l_0$ = 2.343 m
$0.2\ l_0$ = 4.200 m

= 2.343 m

beff = Min. { Σ beff,i + bw = 5.010 m
b = 2.750 m

beff = 2.750 m

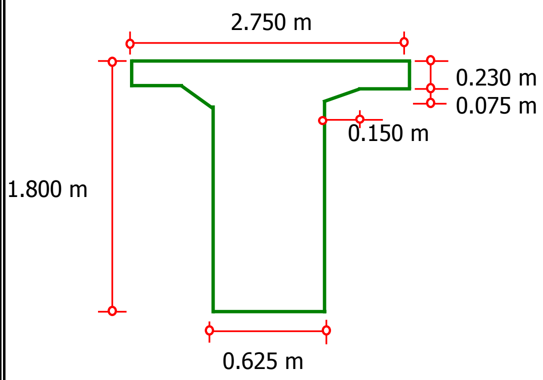
SL.No.			A	y	Ay	Ay^2	I_o	$I_o + Ay^2$		u	
1. Rectangular top Flange			0.63250	1.68500	1.06576	1.79581	0.002788	1.79860		5.03500	
2. Triangular top Flange			0.04500	1.52000	0.06840	0.10397	0.000056	0.10402		0.67082	
3. Web			0.42900	0.91000	0.39039	0.35525	0.062291	0.41755		2.04000	
4. Bottom Bulb, Triangle			0.02250	0.30000	0.00675	0.00203	0.000028	0.00205		0.42426	
5. Bottom Bulb, Rectangle			0.15625	0.12500	0.01953	0.00244	0.000814	0.00326		1.12500	
Total Section		Σ	**1.28525**		**1.55083**	**2.25950**	**0.06598**	**2.32548**		9.295084	0.13827
			ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma (I_o + Ay^2)$		Σu	
			0.7140	1.8000							

Section Properties of Total Section

Area	**ΣA**	=	1.2853	m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.2066	m
Moment of inertia of girder **(Iz) = $\Sigma(Io+A.y^2)-\Sigma A.Y^2$**		=	0.454179	m^4
Perimeter in contact with atmosphere **u**		=	9.2951	m

3.2 Inner T - Girder at the Support

Member no: _GI-SUP *



SL.No.	A	y	Ay	Ay^2	I_o	$I_o + Ay^2$		u	
1. Rectangular top Flange	0.63250	1.68500	1.06576	1.79581	0.002788	1.79860		5.03500	
2. Triangular top Flange	0.01125	1.54500	0.01738	0.02685	0.000004	0.02686		0.33541	
3. Web	0.98125	0.78500	0.77028	0.60467	0.201557	0.80623		3.61500	
Total Section Σ	1.62500		1.85343	2.4273	0.204	2.63168		8.985410	0.18085
	ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma (I_o + Ay^2)$		Σu	
	0.9028	1.8000							

Section Properties of Total Section

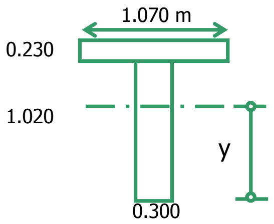
Area	**ΣA**	=	1.625	m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.141	m
Moment of inertia of Girder **(Iz) = $\Sigma(Io+A.y^2)-\Sigma A.Y^2$**		=	0.518	m^4

4.0 Section Properties of Transverse Members

Span length c/c of Bearing	=	21.000	m	
C/C spacing of Longitudinal Girder	=	2.750	m	
C/C spacing of Cross Girder	=	10.500	m	
Distance between C/L of Brg. and C/L of Exp. Joint	=	0.500	m	0.500 (Normal)
Cantilever length	=	1.250	m	
Depth of Cross-Girder	=	1.250	m	
Thickness of Deck slab	=	0.230	m	
Thickness of Deck slab at Exp. Joint	=	0.400	m	
Length of Solid Portion of Web	=	0.700	m	
Length of Tappering Portion of Web	=	0.900	m	
Web thickness of Inner Cross Girder	=	0.300	m	
Web thickness of External Cross Girder	=	0.400	m	
Depth of T Girder	=	1.500	m	
No. of Long Girders	=	5		
No. of Cross Girders	=	3		
Skew distance between c/c of bearing 0 Deg	=	2.750	m	

4.1 Inner Cross Girder

Member no: _INT-CG



Effective span l_o = 1.925 m

$b_{1,2}$ = 5.100 m

$beff_{1,2}$ = Min. $\begin{cases} 0.2\, b_{1,2} + 0.1\, l_0 = 1.213 \text{ m} \\ 0.2\, l_0 = 0.385 \text{ m} \end{cases}$

$beff_{1,2}$ = 0.385 m

beff = Min. $\begin{cases} \Sigma\, beff,i + bw = 1.070 \text{ m} \\ b = 10.500 \text{ m} \end{cases}$

beff = 1.070 m

Area = 0.55210 m^2 0.306 m^2

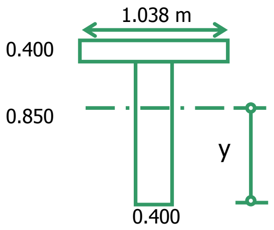
Distance of cg from bottom fibre (y) = 0.7886 m

Moment of inertia of end intermediate girder (Iz) = 0.08090 m^4

Moment of inertia girder (I_x) = 0.00947 m^4

4.2 End Cross Girder

Member no: _END-CG



Effective span l_o = 1.925 m

b_1 = 0.300 m

b_2 = 5.050 m

$beff_1$ = Min. $\begin{cases} 0.2\, b_1 + 0.1\, l_0 = 0.253 \text{ m} \\ 0.2\, l_0 = 0.385 \text{ m} \end{cases}$

$beff_1$ = 0.253 m

$beff_2$ = Min. $\begin{cases} 0.2\, b_2 + 0.1\, l_0 = 1.203 \text{ m} \\ 0.2\, l_0 = 0.385 \text{ m} \end{cases}$

$beff_2$ = 0.385 m

beff = Min. $\begin{cases} \Sigma\, beff,i + bw = 1.038 \text{ m} \\ b = 5.500 \text{ m} \end{cases}$

beff = 1.038 m

Area = 0.755 m^2 0.340 m^2

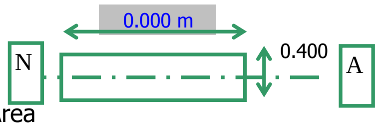
Distance of cg from bottom fibre (y) = 0.769 m

Moment of inertia of end intermediate girder (Iz) = 0.099 m^4

Moment of inertia girder (I_x) = 0.022 m^4

4.3 Edge Cantilever Slab beyound support

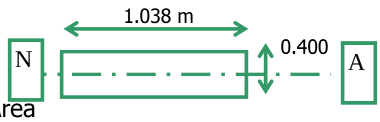
Member no: _DUMMY-T



Area	=	0.0000 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0000 m^4
Moment of inertia girder (I_x)	=	0.0000 m^4

4.4 Cantilever Slab (End of End Cross Girder)

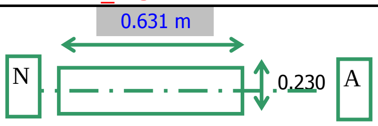
Member no: _T-SLAB1



Area	=	0.4150 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0055 m^4
Moment of inertia girder (I_x)	=	0.0087 m^4

4.5 Intermediate Slab Next to End Cross Girder

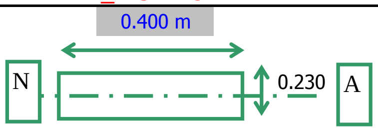
Member no: _T-SLAB2



Area	=	0.1452 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0006 m^4
Moment of inertia girder (I_x)	=	0.0010 m^4

4.6 Slab at Tapered portion

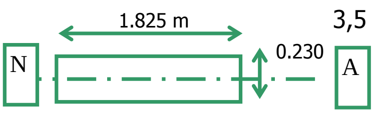
Member no: _T-SLAB3



Area	=	0.0920 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0004 m^4
Moment of inertia girder (I_x)	=	0.0005 m^4

4.7 Slab at Depth away from support

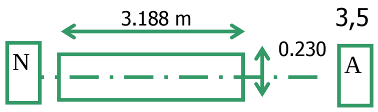
Member no: _T-SLAB4



Area	=	0.4198 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0019 m^4
Moment of inertia girder (I_x)	=	0.0033 m^4

4.8 Slab Adjacent to Intermediate Cross Girders

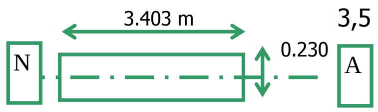
Member no: _T-SLAB5



Area	=	0.7331 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0032 m^4
Moment of inertia girder (I_x)	=	0.0058 m^4

4.9 Cantilever Slab Between Intermediate Cross Girders

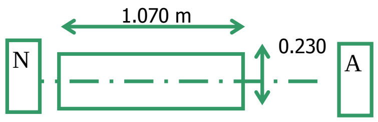
Member no: _T-SLAB6



Area	=	0.7826 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0034 m^4
Moment of inertia girder (I_x)	=	0.0062 m^4

4.10 Cantilever Slab (End of Central Intermediate Cross Girder

Member no: _T-SLAB7



Area	=	0.2461 m^2
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0011 m^4
Moment of inertia girder (I_x)	=	0.0019 m^4

4.11 ICG / Slab (Side Intermediate Cross Girder)

Member no: _T-SLAB8 { same as _T-SLAB5 (for 3 No's Cross Girder) & _INT-CG (for 5 No's Cross Girder)}

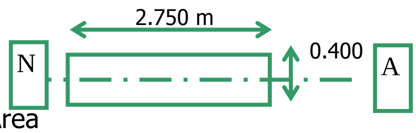
Area	=	0.733 m^2
Distance of cg from bottom fibre (y)	=	0.115 m
Moment of inertia (I_z)	=	0.003 m^4
Moment of inertia girder (I_x)	=	0.006 m^4

5.0 Section Properties of Longitudinal Members (Slab Portion)

C/C spacing of Longitudinal Girder	=	2.750 m
Cantilever length	=	1.250 m
Thickness of Deck slab at Exp. Joint	=	0.400 m

5.1 Slab at the end of Intermediate Girder

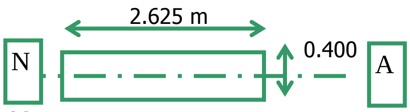
Member no: _L-SLAB1



Area	=	1.1000	m^2
Distance of cg from bottom fibre (y)	=	0.2000	m
Moment of inertia (I_z)	=	0.0147	m^4
Moment of inertia girder (I_x)	=	0.02585	m^4

5.2 Slab at the end of Outer Girder

Member no: _L-SLAB2



Area	=	1.0500	m^2
Distance of cg from bottom fibre (y)	=	0.2000	m
Moment of inertia (I_z)	=	0.0140	m^4
Moment of inertia girder (I_x)	=	0.02463	m^4

6.0 SIDL and Footpath Load Calculation

Span length c/c of Expansion Joint	=	22.000	m	
Span length c/c of Bearing	=	21.000	m	
Skew Angle	=	0.0	Degree	
Depth of Wearing Coat assumed for Design	=	0.100	m	
Density of Wearing Coat material	=	22.0	kN/m^3	
Density of footpath material	=	25.0	kN/m^3	
Weight of Crash Barrier (0.9 m height)per m run	=	8.5	kN/m	
Weight of Railing (1.1 m height) per m run	=	8.5	kN/m	
Width of Crash Barrier	=	0.50	m	
Width of Railing on both outer side of Bridge	=	0.50	m	
Width of Footpath on both side	=	1.50	m	
Thickness of Footpath	=	0.00	m	
Basic Footpath Live Load	=	0	kg/m^2	
Actual Footpath Live Load	=	0.00	kN/m^2	Cl. 206.3.(b) IRC 6-2014
Self weight of Footpath	=	0.00	kN/m^2	
Wearing Coat Load	=	2.20	kN/m^2	

6.1 Wind Load Calculation

Span length c/c of Expansion Joint		=	22.000	m
Span length c/c of Bearing		=	21.000	m
Overall width of superstructure		=	13.500	m
C/C spacing of Longitudinal Girder		=	2.750	m
Depth of superstructure		=	1.500	m
Depth of Wearing Coat		=	0.100	m
Solid area as seen in elevation	A_1	=	59.40	m^2
Plan area of the Superstructure	A_3	=	297.00	m^2
Type of Superstructure,			Two or more beam/ Box girder Bridge	
Width to depth ration i.e b / d		=	9.00	
Ration of clear distance bet.^{w} beams (for two or more beams/ box bridge) to depth		=	1.83	
Gust Factor	G	=	2.00	
Drag coefficient for the Superstructure	C_D	=	1.95	
Lift coefficient	C_L	=	0.75	

Wind load on loaded structure

As per clause 209.3.7 the bridges shall not be considered to be carrying any live load when the wind speed at deck level exceeds 36m/s.

Design Wind speed at any height V_b = 39.0 m/s

Bridge situated at, Plain Terrain

As per note no. 5 of cl no. 209 of IRC:6-2014, wind pressure shall be increased by 0 %

Wind Speed & design wind pressure for Basic wind speed of 33 m/s					Wind speed & design wind pressure for Basic wind speed of 39m/s				
H (m)	Plain Terrain		Terrain with Obstructic		H (m)	Plain Terrain		Terrain with Obstructior	
	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)		V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
10	27.8	463.7	17.8	190.5	< 10	32.9	647.6	21.0	266.1
15	29.2	512.5	19.6	230.5	15	34.5	715.8	23.2	321.9
20	30.3	550.6	21	265.3	20	35.8	769.0	24.8	370.5
30	31.4	590.2	22.8	312.2	30	37.1	824.3	26.9	436.0
50	33.1	659.2	24.9	370.4	50	39.1	920.7	29.4	517.3
60	33.6	676.3	25.6	392.9	60	39.7	944.6	30.3	548.8
70	34.0	693.6	26.2	412.8	70	40.2	968.7	31.0	576.6
80	34.4	711.2	26.9	433.3	80	40.7	993.3	31.8	605.2
90	34.9	729.0	27.5	454.2	90	41.2	1018.2	32.5	634.4
100	35.3	747.0	28.2	475.6	100	41.7	1043.3	33.3	664.3

Average Height of superstructure			=	10.0	m
Lower bound	Height = 10.0 m	P_z	=	647.6	N/m^2
Upper Bound	Height = 10.0 m	P_z	=	647.6	N/m^2
P_z = Design wind pressure at bridge for h =	10.0 m		=	647.6	N/m^2
Tranverse wind Force		F_T	=	P_Z x A_1 x G x C_D	
			=	150.0	KN	1.7
Longitudinal wind Force		F_L	=	25% of F_T	
			=	37.5	KN	0.1
Vertical wind Force		F_V	=	P_Z x A_3 x G x C_L	
				288.5	KN
UDL on girder			=	1.0	KN/m^2
	Outer girder		=	2.6	KN/m
	Inner girder		=	2.7	KN/m

Calculation of Effective Bridge Temperature loading:-

<u>Calculation of Uniform Temperature Rise & Fall effect:-</u>

Maximum Air Shade temperature		=	47.5	^{0}C
Minimum Air Shade temperature		=	2.5	^{0}C
Difference between Max. & Min. of Air Shade temperature		=	45.0	^{0}C
Maximum Effective Bridge temperature		=	32.50	^{0}C
Minimum Effective Bridge temperature		=	12.50	^{0}C
For Expansion, temperature Rise		=	35.00	^{0}C
For Contraction, temperature Fall		=	35.00	^{0}C
Coefficient of Thermal Expansion	α	=	0.000012	
For Expansion, Strain on Top Slab		=	0.000420	
For Contraction, Strain on Top Slab		=	0.000420	

Max. Longitudinal force:-

Class A	=	110.8	KN
70 R Wheel	=	200	KN
		227.7	KN
		57	KN

0.0 MATERIAL PROPERTY

Reinforcing Steel:-

Grade of Steel Reinforcement			Fe 500	MPa
Characteristic yield strength of Reinforcement	f_{yk}	=	500.0	MPa
Design yield of strength of Reinforcement	f_{yd}	=	434.78	MPa
Design yield of strength of shear Reinforcement	f_{ywd}	=	400.00	MPa
Modulus of Elasticity of Steel	E_s	=	200000	MPa

Cast in situ Deck Slab:-

Grade of Concrete			M 35	MPa
Characteristic compressive cube strength of Concrete at 28	f_{ck}	=	35	MPa
Mean value of Concrete cube compressive strength	f_{cm}	=	45	MPa
Modulus of Elasticity of Concrete	E_{cm}	=	32308	MPa
Design compressive strength of concrete	f_{cd}	=	15.63	MPa
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa
Mean value of tensile strength of Concrete at time of crack	$f_{ct.eff}$	=	2.90	MPa

Cast in situ RCC Girder:-

Grade of Concrete			M 35	MPa
Characteristic compressive cube strength of Concrete at 28	f_{ck}	=	35	MPa
Mean value of Concrete cube compressive strength	f_{cm}	=	45	MPa
Modulus of Elasticity of Concrete	E_{cm}	=	32308	MPa
Design compressive strength of concrete	f_{cd}	=	15.63	MPa
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa
Mean value of tensile strength of Concrete at time of crack	$f_{ct.eff}$	=	2.90	MPa

0.0 ANALYSIS ASSUMPTION

Environmental Parameters:-

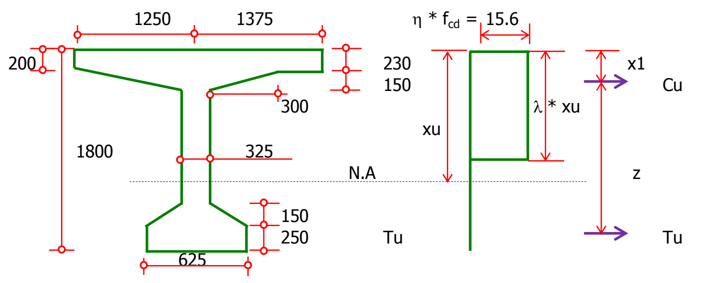
Relative Humidity	RH	=	80	%
Exposure condition considered		=	Moderate	
Coefficient of thermal expansion	α	=	1.2E-05	$/^oC$

9.0 ULS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT & SHEAR FORCE (*OUTER GIRDER*)

BY LIMIT STATE METHOD OF IRC : 112 - 2011

Design Parameters:-

Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa
Design yield of strength of Reinforcement	f_{yd}	=	434.8	MPa
Design yield of strength of shear Reinforcement	f_{ywd}	=	400.0	MPa
Characteristic compressive cube strength of Concrete at 28 day:	f_{ck}	=	35	MPa
Design compressive strength of concrete	f_{cd}	=	15.6	MPa
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa
Mean value of tensile strength of Concrete at time of cracks	$f_{ct.eff}$	=	2.90	MPa
Modulus of Elasticity of Concrete	E_{cm}	=	32308	MPa
Using Rectangular Stress block,				
Effective height factor	λ	=	0.8	
Compression zone factor	η	=	1	
	$\eta * f_{cd}$	=	15.63	MPa
Limiting value of Depth of N.A to effective depth, d	xu_{max} / d	=	0.464	
Moduler ration	E_{cmP} / E_{cmD}	=	1	(Precast Beam / Cast in situ Deck)



GIRDER DIMENSIONS RECTANGULAR STRESS BLOCK

Min. & Max. Longitudinal Reinforcement Percentage:-

Min. Reinforcement percentage for beam section As,min = max [$0.26*f_{ctm}/f_{yk}*b_t*d$, $0.0013*b_t*d$]

Max. Reinforcement percentage for beam section As,max = $0.025*A_c$ Clause 16.5.1.1

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
Bending Moment M_{ED}	*KN-m*	*480*	*1808*	*1808*	*2416*	*3100*	*3254*

*ULS Factored Moment

Check for Min. & Max. Longitudinal Reinforcement Percentage:-

No's of Reinforcement	No's	13	13	13	15	15	15
Diameter	mm	32	32	32	32	32	32
No's of Reinforcement	No's	0	0	0	0	0	0
Diameter	mm	25	25	25	25	25	25
Total As, provided	mm^2	10455	10455	10455	12064	12064	12064
C.G from Face	mm	122	122	122	132	132	132
d_{eff}	mm	1678	1678	1678	1668	1668	1668
b_t	mm	625	325	325	325	325	325
As,min	mm^2	1511	786	786	781	781	781
Check As,min < As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ac	mm^2	1602344	1294375	1294375	1294375	1294375	1294375
As,max	mm^2	40059	32359	32359	32359	32359	32359
Check As,max > As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
d_{oms}	mm	1728	1728	1728	1728	1728	1728

d_{oms} = deff upto c.g of Outer most Steel

Check for Ultimate Limit State Capacity:- Clause 8.2.1 & A2.9

λ * xu	mm	91	91	91	105	105	105
C_{Area}	mm^2	238525	238525	238525	275221	275221	275221
x1	mm	45	45	45	52	52	52
Cu	KN	3729	3729	3729	4303	4303	4303
Tu	KN	4546	4546	4546	5245	5245	5245
Check (Cu - Tu = 0)		-817	-817	-817	-942	-942	-942
xu	mm	114	114	114	131	131	131
xu/d_{eff}		0.068	0.068	0.068	0.079	0.079	0.079
Check xu/d_{eff} < xu_{max}/d_{eff}		**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**
z = d_{eff} - x1	mm	1633	1633	1633	1616	1616	1616
M_{RD} = Tu * z	KN-m	7421	7421	7421	8474	8474	8474
Check M_{ED} < M_{RD}		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
ΔF_d	KN	1003	786	786	642	531	498
$M_{ED}/z + \Delta F_d$	KN	1297	1893	1893	2137	2450	2512
M_{RD}/z	KN	4546	4546	4546	5245	5245	5245
Check $M_{RD}/z > M_{ED}/z + \Delta F_d$		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
As,cal	mm^2	676	2546	2546	3438	4411	4630
Check As,cal < As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
		28.52	41.65	41.65	40.75	46.71	47.89

Check for Shear Reinforcement Requirement:-

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*	
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*	
Shear Force V_{ED}	KN	802	656	656	536	425	398	
Reduction factor β		1.000	1.000	1.000	1.000	1.000	1.000	Clause10.3.3.3
Designed Torsion T_{Ed}	KN-m	0	0	0	0	0	0	
Cross sect. area, A	m^2	1.602	1.294	1.294	1.294	1.294	1.294	
Outer Perimeter, u	m	8.694	8.994	8.994	8.994	8.994	8.994	
Eff. Wall thk. $t_{ef,i}$=A/u	m	0.184	0.144	0.144	0.144	0.144	0.144	
Area enclosed by c/l of wall, A_k (m^2)		1.140	0.953	0.953	0.953	0.953	0.953	
Perimeter of the area Ak, u_k (m)		4.643	4.463	4.463	4.463	4.463	4.463	
Torsional Stress, $\tau_{t,i}$	KN/m^2	0	0	0	0	0	0	
z_i = h	m	1.800	1.800	1.800	1.800	1.800	1.800	
Torsional Shear, $V_{Ed,i}$	KN	0	0	0	0	0	0	
Shear Force V_{ED}'	KN	802	656	656	536	425	398	
Asl	mm^2	10455	10455	10455	12064	12064	12064	
d_{eff}	mm	1678	1678	1678	1668	1668	1668	
b_w	mm	625	325	325	325	325	325	
k		1.345	1.345	1.345	1.346	1.346	1.346	
$\rho 1$		0.010	0.019	0.019	0.020	0.020	0.020	
v_{min}		0.286	0.286	0.286	0.286	0.286	0.286	
σ_{cp}	N/mm^2	0.0	0.0	0.0	0.0	0.0	0.0	
$V_{Rd.c\ min}$	KN	300	156	156	155	155	155	
$V_{Rd.c}$	KN	508	328	328	331	331	331	
V_{Rdc}	KN	508	328	328	331	331	331	
Check for Shear Reinf. Requirement		**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	

Check if shear force V_{ED}' calculated with out reduction factor β Clause10.3.2 (5)

ν		0.532	0.532	0.532	0.532	0.532	0.532
0.5 * b_w * d * ν *f_{cd}	KN	4363	2269	2269	2255	2255	2255
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Check for Section Maximum Shear Capacity:- Clause10.3

Shear V_{ccd} Conc. Cord	KN	0	0	0	0	0	0	
Shear V_{td} Reinf. Cord	KN	0	0	0	0	0	0	
V_{NS} = V_{ED}' - V_{ccd} - V_{td}	KN	802	656	656	536	425	398	
d_{eff}	mm	1678	1678	1678	1668	1668	1668	
b_w	mm	625	325	325	325	325	325	
z = 0.9 * d_{eff}	mm	1510	1510	1510	1501	1501	1501	
α_{cw}		1	1	1	1	1	1	
$\nu 1$		0.6	0.6	0.6	0.6	0.6	0.6	
θ Calculated	deg.	16.1	22.7	22.7	22.7	18.7	18.7	
θ Adopted	deg.	21.8	22.7	22.7	22.7	21.8	21.8	
$V_{Rd.max}$ for θ Adopted	KN	3053	1637	1637	1627	1578	1578	
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
A_{sw} / s	mm^2/m	531	453	453	373	283	265	Clause10.3.3.2
ρ_{min}=(0.072*$f_{ck}^{0.5}$)/f_{yk}		0.00085	0.00085	0.00085	0.00085	0.00085	0.00085	Clause10.3.3.5
$A_{sw.min}$=ρ_{min}*s*b_w	mm^2/m	106	55	55	55	55	55	
Provide A_{sw} Legs	No's	2	2	2	2	2	2	
Dia.	mm	12	12	12	12	10	10	
Spacing	mm	200	200	200	200	200	200	
A_{sw} Provided	mm^2/m	1131	1131	1131	1131	785	785	
Check A_{sw} Provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
ΔF_d	KN	1003	786	786	642	531	498	Clause10.3.3.2
Torsional Long. Reinf. Asl (mm^2)		0	0	0	0	0	0	
Check for Asl provided for Torsion		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
Torsional Resistance, $T_{Rd.max}$ (KN-m)		1206	811	811	811	787	787	
$T_{Ed}/T_{Rd.max}$ + $V_{NS}/V_{Rd.max}$ <=1.0		0.26	0.40	0.40	0.33	0.27	0.25	
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
		3.81	2.49	2.49	3.04	3.71	3.96	
θ	deg.	16.1	22.7	22.7	22.7	18.7	18.7	
$V_{Rd.max}$	KN	2363	1637	1637	1627	1391	1391	
$V_{Rd.s}$	KN	2363	1637	1637	1627	1391	1391	
$V_{Rd.max}$ - $V_{Rd.s}$	KN	0	0	0	0	0	0	

Anchorage of Span Reinforcement at End:- Clause 16.5.1.4

Tensile force to be resisted	Fs	=	$V_{ED} * (a1/d_{eff}) + N_{ED}$
where,			
Axial force taken by reinforcement	N_{ED}	=	0.0 KN
Shear force at face of support	V_{ED}	=	802 KN
Effective depth at face of supprort	d_{eff}	=	1678 mm
	z	=	1510 mm
	a1	=	$z * (\cot \theta + \cot \alpha) / 2$
for θ = 21.8 deg. &	α =		90.0 deg.
	a1	=	1888 mm
Tensile force	Fs	=	902 KN
Area of tensile reinforcement at support section	As	=	10455 mm^2
Tensile capacity of achorage reinforcement	Ft	=	4546 KN **OK**

Support condition of girder, Direct support.

For Direct support, Anchorage to be provided upto 2/3 of $l_{b,net}$ distance from face of support. i.e = 541 mm

For Indirect support, Anchorage to be provided upto $l_{b,net}$ distance from face of support starting after leaving w/3,

Fo where w is the Bearing long. dimendion.

Check for Anchorage of Reinforcement rebars:- Clause 15.2.3 & 15.2.4

Characteristic yield strength of Reinforcement	f_{bd}	=	3.0 MPa
Basic anchorage length	l_b	=	$(\phi / 4) * (f_{yd} / f_{bd})$
Minimum anchorage length	$l_{b.min}$	=	Max. $(0.3 * l_b, 10\phi, 100mm)$
Design anchorage length	$l_{b.net}$	=	$\alpha_a * l_b * A_{s.req} / A_{s.pro}$
where,	α_a	=	0.7

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*

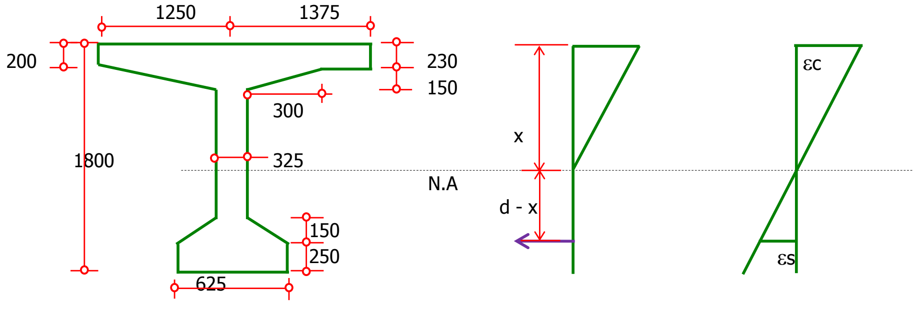
Dia. Of Rebar	mm	32	32	32	32	32	32
l_b	mm	1159	1159	1159	1159	1159	1159
$l_{b.min}$	mm	348	348	348	348	348	348
$A_{s.req} / A_{s.pro}$		1.0	1.0	1.0	1.0	1.0	1.0
$l_{b.net}$	mm	812	812	812	812	812	812

10.0 SLS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT (*OUTER GIRDER*)
BY LIMIT STATE METHOD OF IRC : 112 - 2011

Design Parameters:-

Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa	
Modulus of Elasticity of Steel	E_s	=	200000	MPa	
Characteristic compressive cube strength of Concrete at 28 days	f_{ck}	=	35	MPa	
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa	
Mean value of tensile strength of Concrete at time of cracks	$f_{ct.eff}$	=	2.90	MPa	
Creep factor	ϕ	=	1.76	Refer:- Creep coefficient calculation	
Modulus of Elasticity of Concrete,					
for Short term loading	E_{cm}	=	32308	MPa	
for long term loading	$E_{c.eff} = E_{cm} / (1 + \phi)$	=	11691	MPa	
Modular ratio,					
for Short term loading	$m = E_s / E_{cm}$	=	6.19		
for long term loading	$m' = E_s / E_{c.eff}$	=	17.11		14.0

		Rare Com.	**Q-P Com.**	
Max. Permissible Concrete Compressive Stress	=	16.8	12.6	MPa
Max. Permissible Tensile Stress in Steel	=	-400	-400	MPa



GIRDER DIMENSIONS STRESSES STRAINS

10.1 SLS STRESSES CHECK

Section	**Unit**	**c/L Brg.**	**Tappered**	**Deff**	**L/4**	**3L/8**	**L/2**
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*

Ast	mm^2	10455	10455	10455	12064	12064	12064	13672
d_{eff}	mm	1678	1678	1678	1668	1668	1668	1592
d_{oms}	mm	1728	1728	1728	1728	1728	1728	

Computation of Stresses in Girder section at SLS Rare Load Combination 3250

Bending Moment M_{ED}	KN-m	425	790	790	1850	2280	2309		
Depth of N.A, x	mm	415	415	415	440	440	440		378
I_{NA}	mm^4	3.07E+11	3.03E+11	3.03E+11	3.31E+11	3.31E+11	3.31E+11		
Stresses at									3.41E+11
Deck top D_{top}	N/mm^2	0.57	1.08	1.08	2.46	3.03	3.07	**OK**	0.34
Deck top D_{bottom}	N/mm^2	0.32	0.60	0.60	1.29	1.58	1.60		
Girder Top G_{top}	N/mm^3	0.32	0.60	0.60	1.29	1.58	1.60		
Girder Steel c.g G_{st_cg}	N/mm^2	-29.87	-56.27	-56.27	-117.49	-144.79	-146.64		
Girder Steel Outer G_{st_o}	N/mm^2	-31.05	-58.50	-58.50	-123.22	-151.87	-153.80	**OK**	

Computation of Stresses in Girder section at SLS Quassi-Permanent Load Combination

Bending Moment M_{ED}	KN-m	500	1242	1242	1968	2144	2203	
Depth of N.A, x	mm	415	415	415	440	440	440	
I_{NA}	mm^4	3.07E+11	3.03E+11	3.03E+11	3.31E+11	3.31E+11	3.31E+11	
Stresses at								
Deck top D_{top}	N/mm^2	0.67	1.70	1.70	2.61	2.85	2.93	**OK**
Deck top D_{bottom}	N/mm^2	0.37	0.94	0.94	1.37	1.49	1.53	
Girder Top G_{top}	N/mm^2	0.37	0.94	0.94	1.37	1.49	1.53	
Girder Steel c.g G_{st_cg}	N/mm^2	-35.14	-88.46	-88.46	-124.98	-136.16	-139.90	
Girder Steel Outer G_{st_o}	N/mm^2	-36.53	-91.97	-91.97	-131.08	-142.81	-146.74	**OK**

10.1 SLS CRACK WIDTH CHECK

Minimum area of reinforcement required to Crack control $A_{s,min}$ = $(kc * k * f_{ct,eff} * A_{ct}) / \sigma_c$, where

Coefficient kc = 0.40 for Bending member

Coefficient k = 0.65 for Web with h>800 mm

Mean value of tensile strength of Concrete $f_{ct,eff} = f_{ctm}$ = 2.77 MPa

Max. Stress permitted in reinforcement after crack $\sigma_c = f_{yk}$ = 500 MPa

Maximum Crack spacing $S_{r.max}$ = $3.4*c + (0.17*\phi / \rho_{p.eff})$

Clear cover of main reinforcement c = 40 mm

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

$A_{c.eff}$ $h_{c.eff} * b$

$h_{c.eff}$ = Min. $\left\{ \begin{array}{l} 2.5*(h - d) \\ (h - x/3) \\ h/2 \end{array} \right.$

$\varepsilon_{sm} - \varepsilon_{cm}$ = Max. $\left\{ \begin{array}{l} [\sigma_{sc} - k_t * f_{ct,eff} (1 + \alpha_e * \rho_{p.eff}) / \rho_{eff}] / \\ 0.6*\sigma_{sc}/E_s \end{array} \right.$

where, σ_{sc} is Stress in tension reinforcement assuming crack section

E_s = 200000 Mpa

E_{cm} = 32308 Mpa

α_e = E_s / E_{cm}

= 6.19

k_t = 0.5

Permissible Crack width (for Moderate Exposure) $w_{k.max}$ = 0.3 mm Table 12.1 of IRC 112:2011

Check for Min. Reinforcement for Crack control:-

Section At	*Unit*	*c/L Brg.*	*Tappered*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
As,pro	mm^2	10455	10455	10455	12064	12064	12064
h	mm	1800	1800	1800	1800	1800	1800
Depth of N.A, x	mm	264	264	264	281	281	281
bw	mm	625	325	325	325	325	325
Act = (h - x) * bw	mm^2	959967	499183	499183	493698	493698	493698
As,min	mm^2	1383	719	719	711	711	711
Check As,min < As,pro		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Calculation of Crack Width:-

d_{eff}	mm	1678	1678	1678	1668	1668	1668
$h_{c,eff}$	mm	305	305	305	330	330	330
$A_{c,eff}$	mm^2	190625	99125	99125	107250	107250	107250
$\rho_{p.eff}$		0.055	0.105	0.105	0.112	0.112	0.112
$S_{r.max}$	mm	235.2	187.6	187.6	184.4	184.4	184.4
$S_{pro.}$	mm	100.0	100.0	100.0	100.0	100.0	100.0
Check		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

1) Crack Width calculation at SLS Quassi-Pe 0

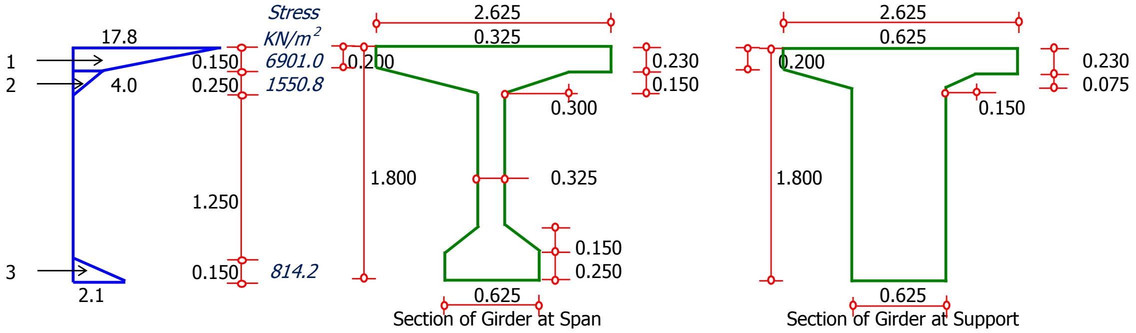
Section At	*Unit*	*c/L Brg.*	*Tappered*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
σ_{sc}	N/mm^2	-36.53	-91.97	-91.97	-131.08	-142.81	-146.74
$\varepsilon_{sm} - \varepsilon_{cm}$		0.00011	0.00035	0.00035	0.00055	0.00061	0.00063
w_k	mm	0.026	0.066	0.066	0.102	0.112	0.116
Check		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

10.1 CALCULATION OF LOAD DUE TO TEMPERATURE GRADIENT IN OUTER GIRDER

			At Span	At Support	
Total height of T-girder	h	=	1.800	1.800	m
Center of gravity (c.g) of T-girder from bottom	y	=	1.205	1.132	m
Moment of Inertia of girder section	I	=	0.451	0.510	m^4
Area of girder section	A	=	1.294	1.602	m^2
Modulus of Elasticity of Concrete	E_{cm}	=	3.23E+07		KN/m^2
Coefficient of thermal expansion of concrete	α	=	0.000012		^oC
Section Modulus at the top of Slab	Z_{ts}	=	0.758	0.763	m^3
Section Modulus at the top of Girder	Z_{tg}	=	1.235	1.163	m^3
Section Modulus at the bottom of Girder	Z_{bg}	=	0.374	0.450	m^3

T_1	=	17.8	^{o} C	h_1	=	0.150	m
T_2	=	4.0	^{o} C	h_2	=	0.250	m
T_3	=	2.1	^{o} C	h_3	=	0.150	m

1.0 Computation of Stresses due to Rise in Temperature



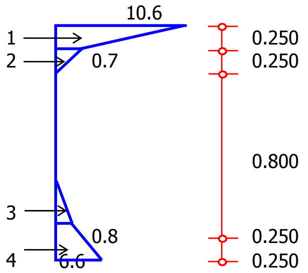
1.1 Stresses at Span section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KN-m	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	6901.0	2.625	1664.0	0.059	0.536	891.9	6901.0	-1548.9	-1278.1	4074.1
2	0.080	1550.8	2.625	273.6	0.187	0.408	111.5	1550.8	-1548.9	-784.2	-782.3
2'	0.170	1054.5	0.325	29.1	0.287	0.309	9.0	1054.5	-1548.9		
3	0.150	814.2	0.625	38.2	1.750	-1.155	-44.1	814.2	-1548.9	2587.2	1852.5
				2004.8			968.4				

1.1 Stresses at Support section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	6901.0	2.625	1664.0	0.059	0.609	1013.0	6901.0	-1268.0	-1474.3	4158.8
2	0.080	1550.8	2.625	273.6	0.187	0.481	131.5	1550.8	-1268.0	-966.6	-683.8
2'	0.170	1054.5	0.625	56.0	0.287	0.381	21.4	1054.5	-1268.0		
3	0.150	814.2	0.625	38.2	1.750	-1.082	-41.3	814.2	-1268.0	2498.4	2044.6
				2031.7			1124.6				

2.0 Computation of Stresses due to Fall in Temperature



Stress KN/m^2
-4109.6
-271.4
-310.2
-2558.8

h = 1.800 m

T_1	=	10.6	° C	h_1	=	0.250	m
T_2	=	0.7	° C	h_2	=	0.250	m
T_3	=	0.8	° C	h_3	=	0.250	m
T_4	=	6.6	° C	h_4	=	0.250	m

2.1 Stresses at Span section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.230	-4109.6	2.625	-1415.2	0.086	0.509	-720.4	-4109.6	1291.4	646.7	-2171.5
1'	0.020	-578.4	0.325	-2.8	0.239	0.356	-1.0	-578.4	1291.4	396.8	1109.7
2	0.130	-271.4	0.325	-8.5	0.307	0.288	-2.4	-271.4	1291.4		
2'	0.120	-130.3	0.325	-2.5	0.420	0.175	-0.4	-130.3	1291.4		
3	0.250	-310.2	0.475	-18.4	1.467	-0.871	16.0	-310.2	1291.4		
4	0.250	-2558.8	0.625	-224.1	1.569	-0.974	218.3	-2558.8	1291.4	-1309.0	-2576.5
				-1671.5			-489.9				

2.2 Stresses at Support section

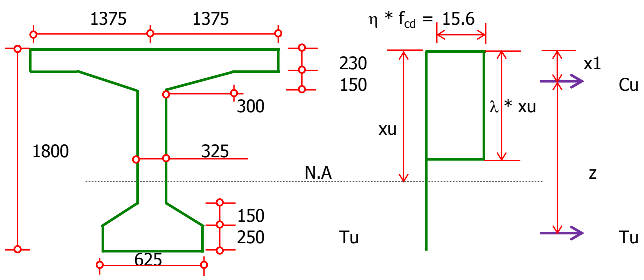
Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.230	-4109.6	2.625	-1415.2	0.086	0.582	-823.4	-4109.6	1054.8	801.6	-2253.2
1'	0.020	-578.4	0.625	-5.3	0.239	0.429	-2.3	-578.4	1054.8	525.6	1001.9
2	0.130	-271.4	0.625	-16.3	0.307	0.361	-5.9	-271.4	1054.8		
2'	0.120	-130.3	0.625	-4.9	0.420	0.248	-1.2	-130.3	1054.8		
3	0.250	-310.2	0.625	-24.2	1.467	-0.799	19.4	-310.2	1054.8		
4	0.250	-2558.8	0.625	-224.1	1.569	-0.901	202.0	-2558.8	1054.8	-1358.5	-2862.6
				-1690.1			-611.5				

11.0 ULS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT & SHEAR FORCE (*INNER GIRDER*)

BY LIMIT STATE METHOD OF IRC : 112 - 2011

Design Parameters:-

Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa
Design yield of strength of Reinforcement	f_{yd}	=	434.8	MPa
Design yield of strength of shear Reinforcement	f_{ywd}	=	400.0	MPa
Characteristic compressive cube strength of Concrete at 28 days	f_{ck}	=	35	MPa
Design compressive strength of concrete	f_{cd}	=	15.6	MPa
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa
Mean value of tensile strength of Concrete at time of cracks	$f_{ct.eff}$	=	2.90	MPa
Modulus of Elasticity of Concrete	E_{cm}	=	32308	MPa
Using Rectangular Stress block,				
Effective height factor	λ	=	0.8	
Compression zone factor	η	=	1	
	$\eta * f_{cd}$	=	15.63	MPa
Limiting value of Depth of N.A to effective depth, d	xu_{max} / d	=	0.464	
Moduler ration	E_{cmP} / E_{cmD}	=	1	(Precast Beam / Cast in situ Deck)



GIRDER DIMENSIONS RECTANGULAR STRESS BLOCK

Min. & Max. Longitudinal Reinforcement Percentage:-

Min. Reinforcement percentage for beam section As,min = max [$0.26*f_{ctm}/f_{yk}*b_t*d$, $0.0013*b_t*d$]

Max. Reinforcement percentage for beam section As,max = $0.025*A_c$ Clause 16.5.1.1

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
Bending Moment M_{ED}	*KN-m*	*480*	*1710*	*1710*	*2750*	*3010*	*3121*

*ULS Factored Moment

Check for Min. & Max. Longitudinal Reinforcement Percentage:-

No's of Reinforcement	No's	13	13	13	15	15	15
Diameter	mm	32	32	32	32	32	32
No's of Reinforcement	No's	0	0	0	0	0	0
Diameter	mm	25	25	25	25	25	25
Total As, provided	mm^2	10455	10455	10455	12064	12064	12064
C.G from Face	mm	122	122	122	132	132	132
d_{eff}	mm	1678	1678	1678	1668	1668	1668
b_t	mm	625	325	325	325	325	325
As,min	mm^2	1511	786	786	781	781	781
Check As,min < As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ac	mm^2	1625000	1285250	1285250	1285250	1285250	1285250
As,max	mm^2	40625	32131	32131	32131	32131	32131
Check As,max > As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
d_{oms}	mm	1728	1728	1728	1728	1728	1728

d_{oms} = deff upto c.g of Outer most Steel

Check for Ultimate Limit State Capacity:- Clause 8.2.1 & A2.9

$\lambda * xu$	mm	91	91	91	105	105	105
C_{Area}	mm^2	249883	249883	249883	288326	288326	288326
x1	mm	45	45	45	52	52	52
Cu	KN	3907	3907	3907	4508	4508	4508
Tu	KN	4546	4546	4546	5245	5245	5245
Check (Cu - Tu = 0)		-639	-639	-639	-738	-738	-738
xu	mm	114	114	114	131	131	131
xu/d_{eff}		0.068	0.068	0.068	0.079	0.079	0.079
Check $xu/d_{eff} < xu_{max}/d_{eff}$		**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**	**UR,OK**
$z = d_{eff} - x1$	mm	1633	1633	1633	1616	1616	1616
$M_{RD} = Tu * z$	KN-m	7421	7421	7421	8474	8474	8474
Check $M_{ED} < M_{RD}$		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
ΔF_d	KN	899	803	803	659	463	375
$M_{ED}/z + \Delta F_d$	KN	1193	1850	1850	2361	2326	2307
M_{RD}/z	KN	4546	4546	4546	5245	5245	5245
Check $M_{RD}/z > M_{ED}/z + \Delta F_d$		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
As,cal	mm^2	676	2408	2408	3913	4283	4441
Check As,cal < As, provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

		26.24	40.70	40.70	45.01	44.34	43.98

Check for Shear Reinforcement Requirement:-

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*	
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*	
Shear Force V_{ED}	KN	719	670	670	550	370	300	
Reduction factor β		1.000	1.000	1.000	1.000	1.000	1.000	Clause10.3.3.3
Designed Torsion T_{Ed}	KN-m	0	0	0	0	0	0	
Cross sect. area, A	m^2	1.625	1.285	1.285	1.285	1.285	1.285	
Outer Perimeter, u	m	8.985	9.295	9.295	9.295	9.295	9.295	
Eff. Wall thk. $t_{ef,i}$=A/u	m	0.181	0.138	0.138	0.138	0.138	0.138	
Area enclosed by c/l of wall, A_k (m^2)		1.169	0.957	0.957	0.957	0.957	0.957	
Perimeter of the area Ak, u_k (m)		4.682	4.475	4.475	4.475	4.475	4.475	
Torsional Stress, $\tau_{t,i}$	KN/m^2	0	0	0	0	0	0	
z_i = h	m	1.800	1.800	1.800	1.800	1.800	1.800	
Torsional Shear, $V_{Ed,i}$	KN	0	0	0	0	0	0	
Shear Force V_{ED}'	KN	719	670	670	550	370	300	
Asl	mm^2	10455	10455	10455	12064	12064	12064	
d_{eff}	mm	1678	1678	1678	1668	1668	1668	
b_w	mm	625	325	325	325	325	325	
k		1.345	1.345	1.345	1.346	1.346	1.346	
ρ1		0.010	0.019	0.019	0.020	0.020	0.020	
ν_{min}		0.286	0.286	0.286	0.286	0.286	0.286	
σ_{cp}	N/mm^2	0.0	0.0	0.0	0.0	0.0	0.0	
$V_{Rd.c\ min}$	KN	300	156	156	155	155	155	
$V_{Rd.c}$	KN	508	328	328	331	331	331	
V_{Rdc}	KN	508	328	328	331	331	331	
Check for Shear Reinf. Requirement		**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Reinf. Reqd**	**Pro. min. Reinf.**	

Check if shear force V_{ED}' calculated with out reduction factor β — Clause10.3.2 (5)

ν		0.532	0.532	0.532	0.532	0.532	0.532
0.5 * b_w * d * ν *f_{cd}	KN	4363	2269	2269	2255	2255	2255
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Check for Section Maximum Shear Capacity:- — Clause10.3

Shear V_{ccd} Conc. Cord	KN	0	0	0	0	0	0	
Shear V_{td} Reinf. Cord	KN	0	0	0	0	0	0	
V_{NS} = V_{ED}' - V_{ccd} - V_{td}	KN	719	670	670	550	370	300	
d_{eff}	mm	1678	1678	1678	1668	1668	1668	
b_w	mm	625	325	325	325	325	325	
z = 0.9 * d_{eff}	mm	1510	1510	1510	1501	1501	1501	
α_{cw}		1	1	1	1	1	1	
ν1		0.6	0.6	0.6	0.6	0.6	0.6	
θ Calculated	deg.	16.1	22.7	22.7	22.7	18.7	18.7	
θ Adopted	deg.	21.8	22.7	22.7	22.7	21.8	21.8	
$V_{Rd.max}$ for θ Adopted	KN	3053	1637	1637	1627	1578	1578	
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
A_{sw} / s	mm^2/m	476	463	463	382	246	200	Clause10.3.3.2
ρ_{min}=(0.072*$f_{ck}^{0.5}$)/f_{yk}		0.00085	0.00085	0.00085	0.00085	0.00085	0.00085	Clause10.3.3.5
$A_{sw.min}$=ρ_{min}*s*b_w	mm^2/m	106	55	55	55	55	55	
Provide A_{sw} Legs	No's	2	2	2	2	2	2	
Provide A_{sw} Dia.	mm	12	12	12	12	10	10	
Provide A_{sw} Spacing	mm	200	200	200	200	200	200	
A_{sw} Provided	mm^2/m	1131	1131	1131	1131	785	785	
Check A_{sw} Provided		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
ΔF_d	KN	899	803	803	659	463	375	Clause10.3.3.2
Torsional Long. Reinf. Asl (mm^2)		0	0	0	0	0	0	
Check for Asl provided for Torsion		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
Torsional Resistance, $T_{Rd.max}$ (KN-m)		1213	783	783	783	759	759	
$T_{Ed}/T_{Rd.max}$ + $V_{NS}/V_{Rd.max}$ <=1.0		0.24	0.41	0.41	0.34	0.23	0.19	
Check Section, **OK / REVISE**		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	
		4.25	2.44	2.44	2.96	4.26	5.26	
θ	deg.	16.1	22.7	22.7	22.7	18.7	18.7	
$V_{Rd.max}$	KN	2363	1637	1637	1627	1391	1391	
$V_{Rd.s}$	KN	2363	1637	1637	1627	1391	1391	
$V_{Rd.max}$ - $V_{Rd.s}$	KN	0	0	0	0	0	0	

Anchorage of Span Reinforcement at End:-

Clause 16.5.1.4

Tensile force to be resisted	Fs	=	V_{ED} * (a1/d_{eff}) + N_{ED}		
where,					
Axial force taken by reinforcement	N_{ED}	=	0.0	KN	
Shear force at face of support	V_{ED}	=	719	KN	
Effective depth at face of supprort	d_{eff}	=	1678	mm	
	z	=	1510.2	mm	
	a1	=	z * (cot θ + cot α) / 2		
for θ = 21.8 deg. &	α =		90.0 deg.		
	a1	=	1887.88468	mm	
Tensile force	Fs	=	809	KN	
Area of tensile reinforcement at support section	As	=	10455	mm^2	
Tensile capacity of achorage reinforcement	Ft	=	4546	KN	**OK**

Support condition of girder, Direct support.

For Direct support, Anchorage to be provided upto 2/3 of $l_{b,net}$ distance from face of support. i.e = 541 mm

For Indirect support, Anchorage to be provided upto $l_{b,net}$ distance from face of support starting after leaving w/3,

Fo where w is the Bearing long. dimendion.

Check for Anchorage of Reinforcement rebars:-

Clause 15.2.3 & 15.2.4

Characteristic yield strength of Reinforcement	f_{bd}	=	3.0	MPa
Basic anchorage length	l_b	=	(ϕ / 4) * (f_{yd} / f_{bd})	
Minimum anchorage length	$l_{b.min}$	=	Max. (0.3 * l_b , 10ϕ , 100mm)	
Design anchorage length	$l_{b.net}$	=	α_a * l_b *$A_{s.req}$ / $A_{s.pro}$	
where,	α_a	=	0.7	

Section At	*Unit*	*c/L Brg.*	*Tappered End*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*

Dia. Of Rebar	mm	32	32	32	32	32	32
l_b	mm	1159	1159	1159	1159	1159	1159
$l_{b.min}$	mm	348	348	348	348	348	348
$A_{s.req}$ / $A_{s.pro}$		1.0	1.0	1.0	1.0	1.0	1.0
$l_{b.net}$	mm	812	812	812	812	812	812

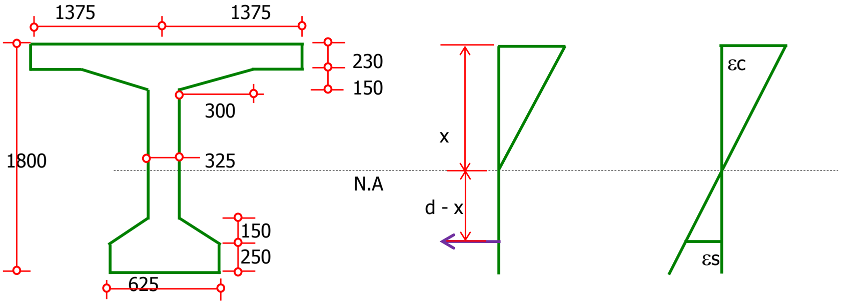
12.0 SLS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT (*INNER GIRDER*)

BY LIMIT STATE METHOD OF IRC : 112 - 2011

Design Parameters:-

Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa
Modulus of Elasticity of Steel	E_s	=	200000	MPa
Characteristic compressive cube strength of Concrete at 28 day:	f_{ck}	=	35	MPa
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.77	MPa
Mean value of tensile strength of Concrete at time of cracks	$f_{ct.eff}$	=	2.90	MPa
Creep factor	ϕ	=	1.76	Refer:- Creep coefficient calculation
Modulus of Elasticity of Concrete,				
for Short term loading	E_{cm}	=	32308	MPa
for long term loading	$E_{c.eff} = E_{cm} / (1 + \phi)$	=	11691	MPa
Modular ratio,				
for Short term loading	$m = E_s / E_{cm}$	=	6.19	
for long term loading	$m' = E_s / E_{c.eff}$	=	17.11	

		Rare Com.	**Q-P Com.**	
Max. Permissible Concrete Compressive Stress	=	16.8	12.6	MPa
Max. Permissible Tensile Stress in Steel	=	-400	-400	MPa



GIRDER DIMENSIONS STRESSES STRAINS

10.1 SLS STRESSES CHECK

Section At	**Unit**	**c/L Brg.**	**Tappered**	**Deff**	**L/4**	**3L/8**	**L/2**
Dist. from c/L brg.	m	0.000	1.600	1.500	5.250	7.875	10.500

Ast	mm^2	10455	10455	10455	12064	12064	12064
d_{eff}	mm	1678	1678	1678	1668	1668	1668
d_{oms}	mm	1728	1728	1728	1728	1728	1728

Computation of Stresses in Girder section at SLS Rare Load Combination

Bending Moment M_{ED}	KN-m	350	700	700	1250	2100	2252	
Depth of N.A, x	mm	407	407	407	431	431	431	
I_{NA}	mm^4	3.12E+11	3.07E+11	3.07E+11	3.35E+11	3.35E+11	3.35E+11	
Stresses at								
Deck top D_{top}	N/mm^2	0.46	0.93	0.93	1.61	2.70	2.90	**OK**
Deck top D_{bottom}	N/mm^2	0.26	0.52	0.52	0.86	1.44	1.55	
Girder Top G_{top}	N/mm^3	0.26	0.52	0.52	0.86	1.44	1.55	
Girder Steel c.g G_{st_cg}	N/mm^2	-24.42	-49.61	-49.61	-78.96	-132.66	-142.26	
Girder Steel Outer G_{st_o}	N/mm^2	-25.38	-51.56	-51.56	-82.79	-139.09	-149.16	**OK**

Computation of Stresses in Girder section at SLS Quassi-Permanent Load Combination

Bending Moment M_{ED}	KN-m	300	640	640	1550	1950	2180	
Depth of N.A, x	mm	407	407	407	431	431	431	
I_{NA}	mm^4	3.12E+11	3.07E+11	3.07E+11	3.35E+11	3.35E+11	3.35E+11	
Stresses at								
Deck top D_{top}	N/mm^2	0.39	0.85	0.85	1.99	2.51	2.80	**OK**
Deck top D_{bottom}	N/mm^2	0.22	0.48	0.48	1.06	1.34	1.50	
Girder Top G_{top}	N/mm^2	0.22	0.48	0.48	1.06	1.34	1.50	
Girder Steel c.g G_{st_cg}	N/mm^2	-20.93	-45.36	-45.36	-97.91	-123.18	-137.71	
Girder Steel Outer G_{st_o}	N/mm^2	-21.75	-47.14	-47.14	-102.66	-129.16	-144.39	**OK**

10.1 SLS CRACK WIDTH CHECK

Minimum area of reinforcement required to Crack control	$A_{s,min}$	=	$(kc * k * f_{ct,eff} * A_{ct}) / \sigma_c$	, where
Coefficient	kc	=	0.40	for Bending member
Coefficient	k	=	0.65	for Web with h>800 mm
Mean value of tensile strength of Concrete	$f_{ct,eff} = f_{ctm}$	=	2.77	MPa
Max. Stress permitted in reinforcement after crack	$\sigma_c = f_{yk}$	=	500	MPa
Maximum Crack spacing	$S_{r.max}$	=	$3.4*c + (0.17*\phi / \rho_{p.eff})$	
Clear cover of main reinforcement	c	=	40	mm
	$\rho_{p.eff}$	=	$A_s / A_{c,eff}$	
	$A_{c.eff}$		$h_{c.eff} * b$	
	$h_{c.eff}$	=	Min.	$2.5*(h - d)$ $(h - x/3)$ $h/2$
	$\varepsilon_{sm} - \varepsilon_{cm}$	=	Max.	$[\sigma_{sc} - k_t * f_{ct,eff} (1 + \alpha_e*\rho_{p.eff}) / \rho_{eff}] / E$ $0.6*\sigma_{sc}/E_s$

where, σ_{sc} is Stress in tension reinforcement assuming crack section

E_s	=	200000	Mpa
E_{cm}	=	32308	Mpa
α_e	=	E_s / E_{cm}	
	=	6.19	
k_t	=	0.5	

Permissible Crack width (for Moderate Exposure) $w_{k.max}$ = 0.3 mm — Table 12.1 of IRC 112:2011

Check for Min. Reinforcement for Crack control:-

Section At	*Unit*	*c/L Brg.*	*Tappered*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
As,pro	mm^2	10455	10455	10455	12064	12064	12064
h	mm	1800	1800	1800	1800	1800	1800
Depth of N.A, x	mm	258	258	258	275	275	275
bw	mm	625	325	325	325	325	325
Act = (h - x) * bw	mm^2	963444	500991	500991	495608	495608	495608
As,min	mm^2	1388	722	722	714	714	714
Check As,min < As,pro		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Calculation of Crack Width:-

d_{eff}	mm	1678	1678	1678	1668	1668	1668
$h_{c,eff}$	mm	305	305	305	330	330	330
$A_{c,eff}$	mm^2	190625	99125	99125	107250	107250	107250
$\rho_{p.eff}$		0.055	0.105	0.105	0.112	0.112	0.112
$S_{r.max}$	mm	235.2	187.6	187.6	184.4	184.4	184.4
$S_{pro.}$	mm	100.0	100.0	100.0	100.0	100.0	100.0
Status		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Crack Width calculation for Quasi-Permanent Load Combination:

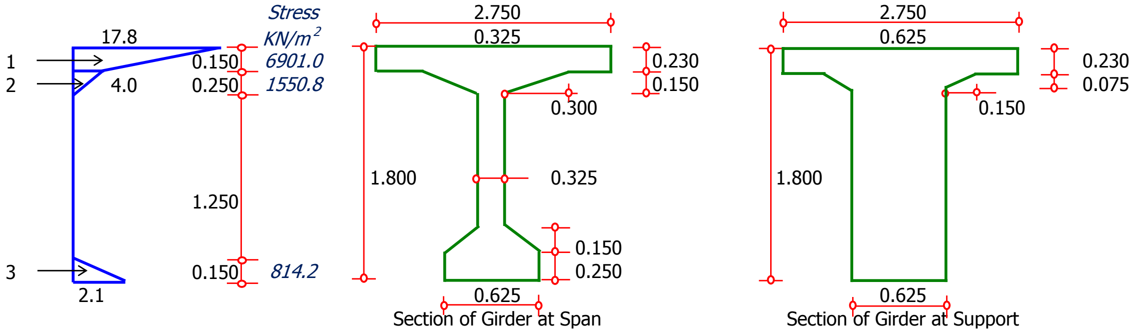
Section At	*Unit*	*c/L Brg.*	*Tappered*	*Deff*	*L/4*	*3L/8*	*L/2*
Dist. from c/L brg.	*m*	*0.000*	*1.600*	*1.500*	*5.250*	*7.875*	*10.500*
σ_{sc}	N/mm^2	-21.75	-47.14	-47.14	-102.66	-129.16	-144.39
$\varepsilon_{sm} - \varepsilon_{cm}$		0.00007	0.00014	0.00014	0.00041	0.00054	0.00062
w_k	mm	0.015	0.027	0.027	0.075	0.100	0.114
Status		**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

12.1 CALCULATION OF LOAD DUE TO TEMPERATURE GRADIENT IN INNER GIRDER

			At Span	*At Support*	
Total height of T-girder	h	=	1.800	1.800	m
Center of gravity (c.g) of T-girder from bottom	y	=	1.207	1.141	m
Moment of Inertia of girder section	I	=	0.454	0.518	m^4
Area of girder section	A	=	1.285	1.625	m^2
Modulus of Elasticity of Concrete	E_{cm}	=	3.23E+07		KN/m^2
Coefficient of thermal expansion of concrete	α	=	0.000012		^oC
Section Modulus at the top of Slab	Z_{ts}	=	0.765	0.785	m^3
Section Modulus at the top of Girder	Z_{tg}	=	1.250	1.206	m^3
Section Modulus at the bottom of Girder	Z_{bg}	=	0.376	0.454	m^3

T_1	=	17.8	$^o C$	h_1	=	0.150	m
T_2	=	4.0	$^o C$	h_2	=	0.250	m
T_3	=	2.1	$^o C$	h_3	=	0.150	m

1.0 Computation of Stresses due to Rise in Temperature



Section of Girder at Span   Section of Girder at Support

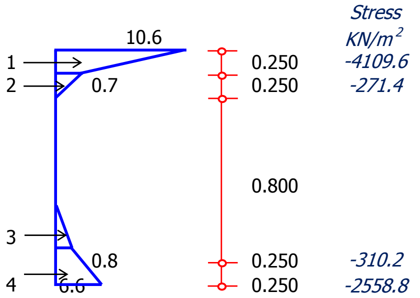
1.1 Stresses at Span section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KN-m	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	6901.0	2.750	1743.2	0.059	0.534	931.2	6901.0	-1631.6	-1322.5	3946.9
2	0.080	1550.8	2.750	286.6	0.187	0.406	116.3	1550.8	-1631.6	-809.9	-890.7
2'	0.170	1054.5	0.325	29.1	0.287	0.307	8.9	1054.5	-1631.6		
3	0.150	814.2	0.625	38.2	1.750	-1.157	-44.1	814.2	-1631.6	2689.4	1872.0
				2097.1			1012.3				

1.1 Stresses at Support section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	6901.0	2.750	1743.2	0.059	0.600	1046.4	6901.0	-1307.1	-1478.6	4115.4
2	0.080	1550.8	2.750	286.6	0.187	0.472	135.3	1550.8	-1307.1	-962.9	-719.2
2'	0.170	1054.5	0.625	56.0	0.287	0.373	20.9	1054.5	-1307.1		
3	0.150	814.2	0.625	38.2	1.750	-1.091	-41.6	814.2	-1307.1	2557.5	2064.6
				2124.0			1160.9				

2.0 Computation of Stresses due to Fall in Temperature



h = 1.800 m

T_1	=	10.6	° C	h_1	=	0.250	m
T_2	=	0.7	° C	h_2	=	0.250	m
T_3	=	0.8	° C	h_3	=	0.250	m
T_4	=	6.6	° C	h_4	=	0.250	m

2.1 Stresses at Span section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m²	m	KN	m	m	KNm	KN/m²	KN/m²	KN/m²	KN/m²
1	0.230	-4109.6	2.750	-1482.6	0.086	0.507	-752.0	-4109.6	1353.0	680.8	-2075.9
1'	0.020	-578.4	0.325	-2.8	0.239	0.355	-1.0	-578.4	1353.0	416.9	1191.4
2	0.130	-271.4	0.325	-8.5	0.307	0.286	-2.4	-271.4	1353.0		
2'	0.120	-130.3	0.325	-2.5	0.420	0.173	-0.4	-130.3	1353.0		
3	0.250	-310.2	0.475	-18.4	1.467	-0.873	16.1	-310.2	1353.0		
4	0.250	-2558.8	0.625	-224.1	1.569	-0.976	218.7	-2558.8	1353.0	-1384.4	-2590.2
				-1738.9			-521.1				

2.2 Stresses at Support section

Segment	Height	Stress	Width b	Force F_{te}	y from top	e	Moment $M_{te}=F_{te}*e$	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m²	m	KN	m	m	KNm	KN/m²	KN/m²	KN/m²	KN/m²
1	0.230	-4109.6	2.750	-1482.6	0.086	0.573	-850.0	-4109.6	1081.5	809.7	-2218.4
1'	0.020	-578.4	0.625	-5.3	0.239	0.421	-2.2	-578.4	1081.5	527.3	1030.3
2	0.130	-271.4	0.625	-16.3	0.307	0.352	-5.7	-271.4	1081.5		
2'	0.120	-130.3	0.625	-4.9	0.420	0.239	-1.2	-130.3	1081.5		
3	0.250	-310.2	0.625	-24.2	1.467	-0.807	19.6	-310.2	1081.5		
4	0.250	-2558.8	0.625	-224.1	1.569	-0.910	203.9	-2558.8	1081.5	-1400.4	-2877.7
				-1757.5			-635.7				

DETAIL DESIGN CALCULATION OF PIER WITH PILE FOUNDATION.

1.0 Preamble :

Design of Substructure & Foundation for Pier as Open foundation.

2.0 Design basis :

Codes of Indian Road Congress (IRCs)
IRC: 6 - 2017 :-Code of Practice for Road bridges (Load and Stresses)
IRC: 112 - 2011 :-Code of Practice for Concrete Road Bridge
IRC: 78 - 2014 :-Code of Practice for Road bridges (Foundation and Substructure)
IRC:SP: 114 - 2018 :-Guidelines for Seismic Design of Road Bridges

3.0 Design data :

Salient levels

Formation level / FRL	=	142.000	m	
Pier cap top level	=	139.348	m	
Highest flood level (HFL)	=	134.709	m	
Lowest bed (LBL) / ground level (GL)	=	132.670	m	
Pile cap top level	=	132.020	m	
Pile cap bottom level	=	130.220	m	
Scour level	=	124.220	m	
Founding level	=	100.220	m	

Material property

Density of PSC /RCC concrete		=	2.50	t/m^3
Dry density of soil/earth	γ_d	=	2.00	t/m^3
Submerged density of soil	γ_{sub}	=	1.00	t/m^3
Density of water	γ_w	=	1.00	t/m^3
Density of wearing coat/surfacing		=	2.20	t/m^3

Geotechnical data

Vertical pile capacity (Normal Case)	=	2950	kN	refer to Soil Investigation report
Horizontal pile capacity (Normal Case)	=	90	kN	
Uplift capacity of Pile		-848	kN	

Alignment detail

Radius of Curvature Structure	=	400	m
Design Speed	=	60	kmph
Effect of Centrifugal force	=	0	

Dimension Data

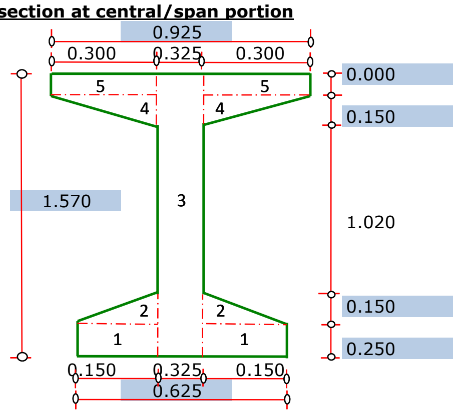
Diameter of cast-in-situ piles		=	1.2	m
Pile edge distance		=	0.15	m
Pile no's	long. direction (col.)	=	2	
	trans. direction (row)	=	4	
Pile spacing	long. direction (col.)	=	3.6	m
	trans. direction (row)	=	3.6	m
Pile cap depth		=	1.8	m
Pier dimension in long. direction		=	1.0	m
Pier dimension in trans. direction		=	6.0	m
Pier thickness for hollow type		=	0.0	m
Pier cap length (trans. direction)		=	12.5	m
Pier cap depth (rectangular portion)		=	0.50	m
Pier cap depth (inclined portion)		=	0.50	m
Pier cap projection beyond bearing		=	1.50	m

4.0 Dead load & SIDL calculation for Superstructure

Dimensions of Superstructure on LHS

Span c/L to c/L of Expansion joint	=	22.000	m	
Span c/L to c/L of Bearing	=	20.600	m	
Expansion joint	=	0.040	m	
Overall Span of girder	=	21.200	m	
Distance betw Exp. joint and c/L of Brg.	=	0.700	m	at deck slab level
Length of end thickning zone of girder	=	0.700	m	beyond c/Ll of bearing
Length of tapering zone of girder	=	0.900	m	
Spacing of girder (center to center)	=	2.750	m	
No's of Girder	=	5	Nos	
Web thickness of girder	=	0.325	m	
Total Carraigeway width of Deck Slab	=	13.500	m	
Overall depth of girder	=	1.800	m	
Deck slab Thickness	=	0.230	m	
Width of Crash Barrier	=	0.500	m	
Width of Foot Path	=	1.500	m	
Wearing Coat Thickness	=	0.100	m	

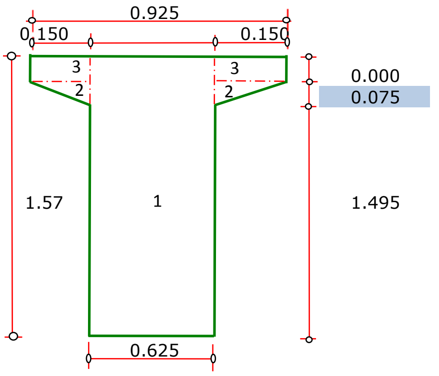
RCC T girder/PSC I girder section at central/span portion



Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

Girder E.No	Nos	B (m)	H (m)	Area(m^2)	No's of Girder	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
1	2	0.150	0.250	0.075	5	0.38			
2	2	0.150	0.150	0.023	5	0.11			
3	1	0.325	1.570	0.510	5	2.55			
4	2	0.300	0.150	0.045	5	0.23			
5	2	0.300	0.000	0.000	5	0.00			
Single Girder				0.653			17.40	2.5	28.4
Deck Slab	1	13.50	0.230	3.11	1	3.11	17.40	2.5	135.1
					Total	6.37	17.40	2.5	277.0

RCC T girder/PSC I girder section at support/end portion



Girder E.No	Nos	B (m)	H (m)	Area(m^2)	No's of Girder	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
1	1	0.625	1.570	0.981	5	4.91			
2	2	0.150	0.075	0.011	5	0.06			
3	2	0.150	0.000	0.000	5	0.00			
Single girder				0.993			2.00	2.5	4.96
Deck Slab	1	13.50	0.23	3.11	1	3.11	2.00	2.5	15.53
					Total	8.07	2.00	2.5	40.3

RCC T girder/PSC I girder section at tapering/transition zone

Avg. Area of midspan and end Section	Girder	0.82			1.80	2.5	3.70
	Deck	3.11			1.80	2.5	13.97
	Total			7.22	1.80	2.5	32.5

Dead Load of cantilever deck slab at ends

Description	Nos	Width(m)	Thick.(m)	Area(m^2)	Length(m)	Unit Wt (MT/m^3)	Total Wt (MT)
Cantilever Slab	2	13.50	0.400	10.80	0.8	2.5	21.6

Dead load of diaphram/cross girder

Description	Nos	Length(m)	Ht. in(m)	Area(m^2)	Thick.(m)	Unit Wt (t/m^3)	Total Wt (t)
1) At End	2	11.63	1.320	20.8	0.4	2.5	20.8
2) At Span	1	11.33	1.320	11.7	0.3	2.5	8.8
			Total	32.5			29.5
						Total Dead Load=	***401.0 t***

Surfacing calculation

a) Wearing Coat over Deck slab

Description	Nos	Width(m)	Depth(m)	Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
100 mm Thickness Wearing Coat	1	9.50	0.100	0.95	22.00	2.2	46.0
						Surfacing Load=	***46.0 t***

SIDL calculation

a) Crash Barrier and Kerb etc

Description	Nos	Area(m^2)/ Width(m)	Depth(m)	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
Crash Barrier	2	0.35		0.70	22.00	2.5	38.5
Raised Safety Kerb (250 mm thk.)	0	1.50	0.3	0.00	22.00	2.5	0.0

b) Hand Rail and Metal beam Crash Barrier

Description	Nos	Length(m)		Area(m^2)		Unit Wt (t/m^3)	Total Wt (t)
100 mm MS Dia. GI Pipe	0	22.00		0.001		7.85	0.0
W Beam crash barrier	0	22.00		0.020			0.0
						Total SIDL=	***38.5 t***

Footpath Dead Load calculation(FPDL)

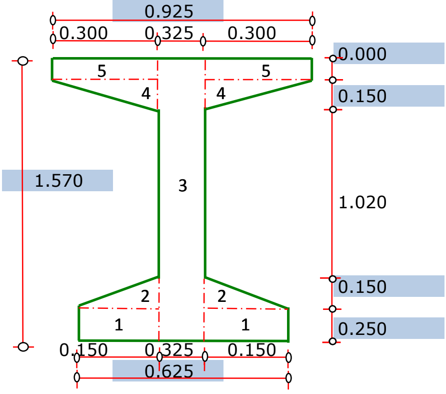
Description	Nos	Width(m)	Depth(m)	Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
300 mm thk. Foothpath	2	1.50	0.300	0.90	22.00	2.5	49.5
						Total FPDL =	***49.5 t***
						Total DL+Surfacing+ SIDL+FPDL=	***535.0 t***

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

Dimensions of Superstructure on RHS

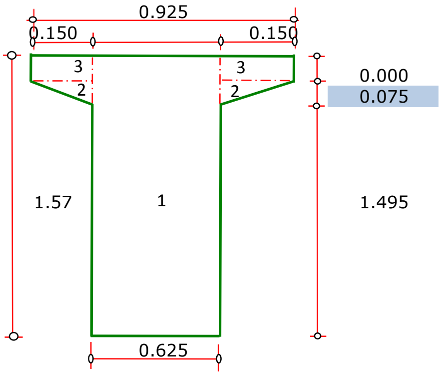
Span c/L to c/L of Expansion joint	=	22.000	m	
Span c/L to c/L of Bearing	=	20.600	m	
Expansion joint	=	0.040	m	
Overall Span of girder	=	21.200	m	
Dist bet. exp joint and c/l of bearing	=	0.700	m	at deck slab level
Length of end thickning zone of girder	=	0.700	m	beyond c/Ll of bearing
Length of tapering zone of girder	=	0.900	m	
Spacing of girder (center to center)	=	2.750	m	
No's of Girder	=	5	Nos	
Web thickness of girder	=	0.325	m	
Total Carraigeway width of Deck Slab	=	13.500	m	
Overall depth of girder	=	1.800	m	
Deck slab Thickness	=	0.230	m	
Width of Crash Barrier	=	0.500	m	
Width of Foot Path	=	1.500	m	
Wearing Coat Thickness	=	0.100	m	

RCC T girder/PSC I girder section at central/span portion



Girder E.No	Nos	B (m)	H (m)	Area(m^2)	No's of Girder	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
1	2	0.150	0.250	0.075	5	0.38			
2	2	0.150	0.150	0.023	5	0.11			
3	1	0.325	1.570	0.510	5	2.55			
4	2	0.300	0.150	0.045	5	0.23			
5	2	0.300	0.000	0.000	5	0.00			
Single Girder				0.653			17.40	2.5	28.4
Deck Slab	1	13.50	0.230	3.11	1	3.11	17.40	2.5	135.1
					Total	6.37	17.40	2.5	277.0

RCC T girder/PSC I girder section at support/end portion



Girder E.No	Nos	B (m)	H (m)	Area(m^2)	No's of Girder	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
1	1	0.625	1.570	0.981	5	4.91			
2	2	0.150	0.075	0.011	5	0.06			
3	2	0.150	0.000	0.000	5	0.00			
Single girder				0.993			2.00	2.5	4.96
Deck Slab	1	13.50	0.23	3.11	1	3.11	2.00	2.5	15.53
					Total	8.07	2.00	2.5	40.3

RCC T girder/PSC I girder section at tapering/transition zone

Avg. Area of Midspan and End Section	Girder	0.82			1.80	2.5	3.70
	Deck	3.11			1.80	2.5	13.97
	Total			7.22	1.80	2.5	32.5

Dead Load of cantilever deck slab at ends

Description	Nos	Width(m)	Thick.(m)	Area(m^2)	Length(m)	Unit Wt (MT/m^3)	Total Wt (MT)
Cantilever Slab	2	13.50	0.400	10.80	0.8	2.5	21.6

Dead load of diaphram/cross girder

Description	Nos	Length(m)	Ht. in(m)	Area(m^2)	Thick.(m)	Unit Wt (t/m^3)	Total Wt (t)
1) At End	2	11.63	1.320	20.8	0.4	2.5	20.8
2) At Span	1	11.33	1.320	11.7	0.3	2.5	8.8
			Total	32.5			29.5

Total Dead Load= 401.0 t

Surfacing calculation

a) Wearing Coat over Deck slab

Description	Nos	Width(m)	Depth(m)	Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
100 mm Thickness Wearing Coat	1	9.50	0.100	0.95	22.00	2.2	46.0

Surfacing Load= 46.0 t

SIDL calculation

a) Crash Barrier and Kerb etc

Description	Nos	Area(m^2)/ Width(m)	Depth(m)	Total Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
Crash Barrier	2	0.35		0.70	22.00	2.5	38.5
Raised Safety Kerb (250 mm thk.)	0	0.75	0.3	0.00	22.00	2.5	0.0

b) Hand Rail and Metal beam Crash Barrier

Description	Nos	Length(m)		Area(m^2)		Unit Wt (t/m^3)	Total Wt (t)
100 mm MS Dia. GI Pipe	0	22.00		0.001		7.85	0.0
W Beam crash barrier	0	22.00		0.020			0.0

Total SIDL= 38.5 t

Footpath Dead Load calculation(FPDL)

Description	Nos	Width(m)	Depth(m)	Area(m^2)	Length(m)	Unit Wt (t/m^3)	Total Wt (t)
300 mm thk. Foothpath	2	1.50	0.300	0.90	22.00	2.5	49.5

Total FPDL = 49.5 t

Total DL+Surfacing+ SIDL+FPDL= 535.0 t

4.1 Live Load Calculations

Footpath Live Load calculation

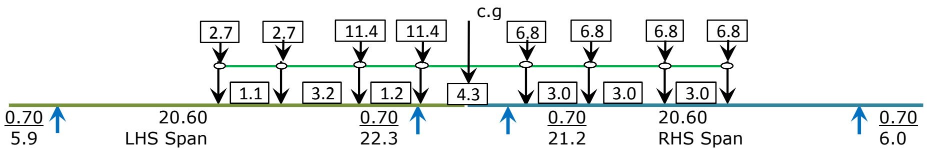
		LHS Span	RHS Span	Ref.
Basic Footpath live load	=	400 Kg/m^2	400 Kg/m^2	Cl.206.3 of
Actual Foothpath live load	=	342 Kg/m^2	342 Kg/m^2	IRC:6-2017
	=	0.34 t/m^2	0.34 t/m^2	
Footpath area	=	66.00 m^2	66.00 m^2	
Total Foothpath live load	=	22.6 t	22.6 t	

Case 1 - Max. Live Load Reaction & Max. Transverse Moment

1) Class A train of vehicle

Impact factor for Class A vehicle = $\frac{4.5}{26.6}$ = 0.169 (Cl.208.2 of IRC:6-2017)

For getting Maximum Reaction on Pier, c.g of Train load should coinside with c/L of Pier.



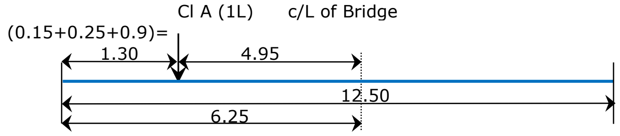
Reaction due to LL (Class A)

Single Lane Vehicle	=	22.3 t on LHS Span
	=	21.2 t on RHS Span
Two Lane Vehicle	=	44.6 t on LHS Span
	=	42.5 t on RHS Span
Three Lane Vehicle	=	60.2 t on LHS Span
	=	57.4 t on RHS Span
Four Lane Vehicle	=	71.3 t on LHS Span
	=	68.0 t on RHS Span

Reaction due to LL (Class A) with Impact factor

Single Lane Vehicle	=	50.9 t
Two Lane Vehicle	=	101.8 t
Three Lane Vehicle	=	137.4 t
Four Lane Vehicle	=	162.9 t

Eccentricity of Class A load (Single Lane Vehicle) in transverse direction placed in median side.



Eccentricity of LL = 4.95 m

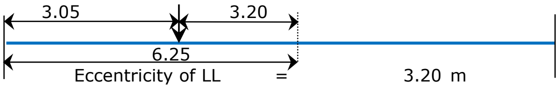
Transverse moment on Pier = 4.95 x 50.9

= 251.8 t-m

Eccentricity of Class A load (Two Lane Vehicle) in transverse direction placed in median side.

Cl A (2L) c/L of Bridge

(0.15+0.25+1.8+0.25+0.6)=



Eccentricity of LL = 3.20 m

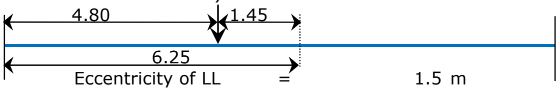
Transverse moment on Pier = 3.2 x 101.8

= 325.9 t-m

Eccentricity of Class A load (Three Lane Vehicle) in transverse direction placed in median side.

Cl A (3L) c/L of Bridge

(0.15+0.25+1.8+0.25+1.2+0.25+0.9)=



Eccentricity of LL = 1.5 m

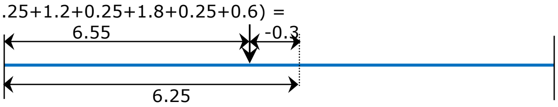
Transverse moment on Pier = 1.45 x 137.4

= 199.2 t-m

Eccentricity of Class A load (Four Lane Vehicle) in transverse direction placed in median side.

Cl A (3L) c/L of Bridge

).15+0.25+1.8+0.25+1.2+0.25+1.8+0.25+0.6) =



Eccentricity of LL = -0.30 m

Transverse moment on Pier = -0.3 x 162.9

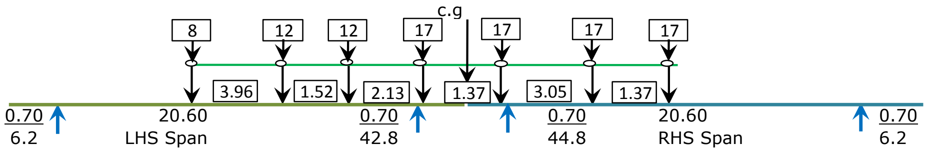
= -48.9 t-m

Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
Cl A(1lane)	26.0	24.8	50.9	4.950	0.700	0.700	251.9	0.8
Cl A(2lane)	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7
Cl A(3lane)	70.3	67.1	137.4	1.450	0.700	0.700	199.2	2.3
Cl A(4lane)	83.4	79.5	162.9	-0.300	0.700	0.700	-48.9	2.7

2) Class 70 R Wheeled vehicle

Impact factor for 70 R Wheel = 16.9% Cl.208.3 of IRC:6-2017

For getting Maximum Reaction on Pier, c.g of Train load should coinside with c/L of Pier.



Reaction due to LL 70 R Wheel = 42.8 t on LHS Span

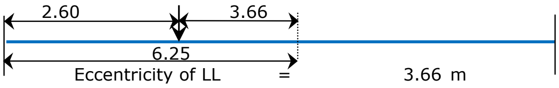
= 44.8 t on RHS Span

Reaction due to LL with Impact factor = 102.4 t

Eccentricity of 70 R Wheeled (First Lane Vehicle) load in transverse direction placed in median side.

70 R c/L of Bridge

(1.2+2.79/2)=



Eccentricity of LL = 3.66 m

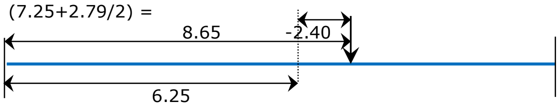
Transverse moment on Pier = 3.655 x 102.4

= 374.3 t-m

Eccentricity of 70 R Wheeled (Second Lane Vehicle) load in transverse direction placed in median side.

70 R c/L of Bridge

(7.25+2.79/2) =



Eccentricity of LL = -2.40 m

Transverse moment on Pier = -2.395 x 102.4

= -245.3 t-m

Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
70RW (1Lane	50.0	52.4	102.4	3.655	0.700	0.700	374.3	1.7
70RW (2Lane	80.0	83.9	163.8	-2.395	0.700	0.700	129.0	2.7

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

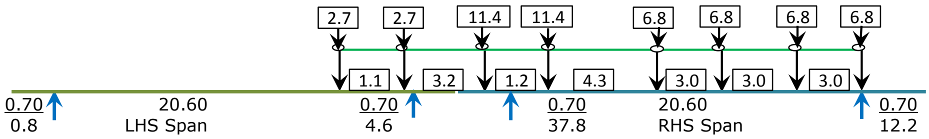
Eccentricity of combined Class A and 70 R Wheel load in transverse direction placed in median side (CL A +70 R)

Type	Load (t)			Eccentricity (m)			Moment (t-m)		
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.	
Cl A 1Lane	26.0	24.8	50.9	4.950	0.700	0.700	251.9	0.8	
70R Wheel	45.0	47.2	92.2	1.205	0.700	0.700	111.1	1.5	Multiplied by 0.9 for 3 Lane effect
Total	71.0	72.0	143.1	-	-	-	363.0	2.4	

Eccentricity of combined Class A 2 Lane and 70 R Wheel load in transverse direction placed in median side (CL A 2 Lane +70 R)

Type	Load (t)			Eccentricity (m)			Moment (t-m)		
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.	
Cl A 2Lane	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7	
70R Wheel	40.0	41.9	81.9	-2.295	0.700	0.700	-188.0	1.4	Multiplied by 0.8 for 4 Lane effect
Total	92.1	91.6	183.7	-	-	-	137.7	3.0	

Case 2 - Max. Longitudinal Moment Condition
1) Class A train of vehicle
For getting Maximum Longitudinal Moment on Pier, Train load should be placed on one side of span.



Reaction due to LL (Cl A)

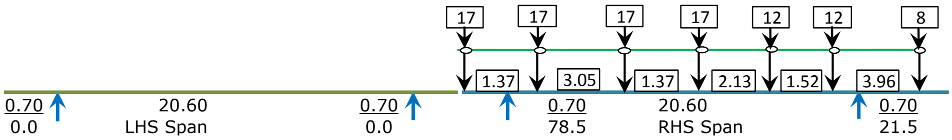
Single Lane Vehicle	=	4.6	t on LHS Span
	=	37.8	t on RHS Span
Two Lane Vehicle	=	9.2	t on LHS Span
	=	75.7	t on RHS Span
Three Lane Vehicle	=	12.4	t on LHS Span
	=	102.1	t on RHS Span
Four Lane Vehicle	=	14.7	t on LHS Span
	=	121.1	t on RHS Span

Reaction due to LL (Class A) with Impact factor

Single Lane Vehicle	=	49.6	t
Two Lane Vehicle	=	99.3	t
Three Lane Vehicle	=	134.0	t
Four Lane Vehicle		158.8	t

Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
Cl A(1lane)	5.4	44.2	49.6	4.950	0.700	0.700	245.6	27.2
Cl A(2lane)	10.8	88.5	99.2	3.200	0.700	0.700	317.5	54.4
Cl A(3lane)	14.5	119.4	133.9	1.450	0.700	0.700	194.2	73.4
Cl A(4lane)	17.2	141.5	158.8	-0.300	0.700	0.700	-47.6	87.0

2) Class 70 R Wheeled vehicle
For getting Maximum Longitudinal Moment on Pier, Train load should be placed on one side of span.



Reaction due to LL 70 R Wheel	=	0.0	t on LHS Span
	=	78.5	t on RHS Span
Reaction due to LL with Impact factor	=	91.8	t

Type	Load (t)			Eccentricity (m)			Moment (t-m)		
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.	
70RW (1Lane	0.0	91.8	91.8	3.655	0.700	0.700	335.6	64.3	
70RW (2Lane	0.0	146.9	146.9	-2.395	0.700	0.700	-16.3	102.8	Multiplied by 0.8 for 4 Lane effect

Ecc of combined Class A and 70R Wheel load in Transverse direction placed in median side(CL A +70R)

Type	Load (t)			Eccentricity (m)			Moment (t-m)		
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.	
Cl A 1Lane	5.4	44.2	49.6	4.950	0.700	0.700	245.6	27.2	
70R Wheel	0.0	82.6	82.6	1.205	0.700	0.700	99.6	57.8	Multiplied by 0.9 for 3 Lane effect
Total	5.4	126.9	132.2	-	-	-	310.6	76.5	

Ecc of combined Class A 2 Lane and 70R Wheel load in Transverse direction placed in median side(CL A 2 Lane +70R)

Type	Load (t)			Eccentricity (m)			Moment (t-m)		
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.	
Cl A 2Lane	10.8	88.5	99.2	3.2	0.700	0.700	317.5	54.4	
70R Wheel	0.0	73.4	73.4	-2.295	0.700	0.700	-168.6	51.4	Multiplied by 0.8 for 4 Lane effect
Total	10.8	161.9	172.7	-	-	-	134.0	95.2	

Case 3 - Max. Longitudinal Force Cl.211.2 of IRC:6-2017

Length of superstructure for breaking = 22.00 m

Vehicle Type	Load (t)	Length (m)	No's Train
Class A	55.4	38.8	1.0
70R	100	43.4	1.0

First Train

Load Case	% of Load to be taken			Load of Vehicle in				Breaking force (t)
	Lane 1	Lane 2	Lanes 3,4	Lane 1	Lane 2	Lane 3	Lane 4	
Cl A 1L	20	0	5	55.4	0	0	0	11.1
Cl A 2L	20	0	5	55.4	55.4	0	0	11.1
Cl A 3L	20	0	5	55.4	55.4	55.4	0	13.9
Cl A 4L	20	0	5	55.4	55.4	55.4	55.4	16.6
70R 1L	20	0	5	100	0	0	0	20.0
70R 2L	20	0	5	100	0	100	0	25.0
70R 1L+Cl A 1	20	0	5	100	0	55.4	0	22.8
70R 1L+Cl A 2	20	0	5	100	0	55.4	55.4	25.5

Other Trains

Load Case	% of Load to be taken			Load of Vehicle in				Breaking force (t)
	Lane 1	Lane 2	Lanes 3,4	Lane 1	Lane 2	Lane 3	Lane 4	
Cl A 1L	10	0	5	55.4	0	0	0	11.1
Cl A 2L	10	0	5	55.4	55.4	0	0	11.1
Cl A 3L	10	0	5	55.4	55.4	55.4	0	13.9
Cl A 4L	10	0	5	55.4	55.4	55.4	55.4	16.6
70R 1L	10	0	5	100	0	0	0	20.0
70R 2L	10	0	5	100	0	100	0	25.0
70R 1L+Cl A 1	10	0	5	100	0	55.4	0	22.8
70R 1L+Cl A 2	10	0	5	100	0	55.4	55.4	25.5

Summary of Live Load

Load case	Type	Load (t)			Moment (t-m)		Hor. Load (t)	
		LHS Span	RHS Span	Total	Long.	Trans.	Long.	Trans.
1	Cl A 1L	26.0	24.8	50.9	0.8	251.9	5.5	0.0
2	Cl A 2L	52.1	49.7	101.8	1.7	325.7	5.5	0.0
3	Cl A 3L	70.3	67.1	137.4	2.3	199.2	6.9	0.0
5	70RW 1L	50.0	52.4	102.4	1.7	374.3	10.0	0.0
7	Cl A 1L+70R	71.0	72.0	143.1	2.4	363.0	11.4	0.0
Max. LL_R	**LL1**	**71.0**	**72.0**	**143.1**	**2.4**	**363.0**	**11.4**	**0.0**
Max. M_T	**LL2**	**50.0**	**52.4**	**102.4**	**1.7**	**374.3**	**10.0**	**0.0**

Load case	Type	Load (t)			Moment (t-m)		Hor. Load (t)	
		LHS Span	RHS Span	Total	Long.	Trans.	Long.	Trans.
1	Cl A 1L	5.4	44.2	49.6	27.2	245.6	5.5	0.0
2	Cl A 2L	10.8	88.5	99.2	54.4	317.5	5.5	0.0
3	Cl A 3L	14.5	119.4	133.9	73.4	194.2	6.9	0.0
5	70RW 1L	0.0	91.8	91.8	64.3	335.6	10.0	0.0
7	Cl A 1L+70R	5.4	126.9	132.2	76.5	310.6	11.4	0.0
Max. M_L	**LL3**	**5.4**	**126.9**	**132.2**	**76.5**	**310.6**	**11.4**	**0.0**

5.0 Longitudnal Forces due to Bearing

			Sevice condition-		**One Span Dislodge condition-**	
Type of bearing used		=	**POT-PTFE**			
Elastomeric Bearing:-						
Movement of bearing due to Creep, Shrinkage & Temperature varriation		=	6.0	mm	6.0	mm
Size of Elastomeric Bearing						
Length (long.)	L	=	800	mm	800	mm
Width (trans.)	B	=	600	mm	600	mm
Height of Elastomer H	H	=	75	mm	75	mm
Shear Modulus of Elastomer G	G	=	1.0		1.0	
Shear Rating per bearing,	V = L*B*G/H	=	3.7	t	3.7	t
Nos of Bearing		=	10.0		5.0	
Bearing frictional force on Pier		=	37.1	t	18.5	t

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

			LL1	LL2	LL3	
Design horizontal frictional force			48.5 t	47.1 t	48.5 t	
POT-PTFE Bearing:			**Fixed**			**Fixed**
Coefficient of Friction	μ	=	0.05			0.05
			0.03			0.03
DL & SIDL reaction at free end, Rg		=	535.0 t			267.5 t
			LL1	LL2	LL3	
Live load reaction at at free end, Rq		=	143.1 t	102.4 t	132.2 t	0.0 t
Horizontal force due to Live load, F_h		=	22.8 t	20.0 t	22.8 t	0.0 t
a) Fixed Bearing						
Design horizontal force		=	45.3 t	41.9 t	44.7 t	13.4 t
b) Free Bearing						
Design horizontal force		=	33.9 t	31.9 t	33.4 t	13.4 t
Horizontal frictional force due to bearing		=	45.3 t	41.9 t	44.7 t	13.4 t
Design horizontal frictional force		=	45.3 t	41.9 t	44.7 t	13.4 t
Lever arm from base of foundation		=	9.279 m			
Long. Moment (M_L)about base of foundatio		=	420.2 t-m	388.5 t-m	415.2 t-m	124.1 t-m
Lever arm from shaft bottom		=	7.479 m			
Long. Moment (M_L)about shaft bottom		=	338.7 t-m	313.1 t-m	334.7 t-m	100.0 t-m

6.0 Seismic Coefficient calculation

Seismic zone of the area		=	IV	Fig:4.1 of IRC:SP:114 - 2018
Zone factor	Z	=	0.24	
Importance factor	I	=	1.2	

Seismic Coefficient in Longitudinal Direction

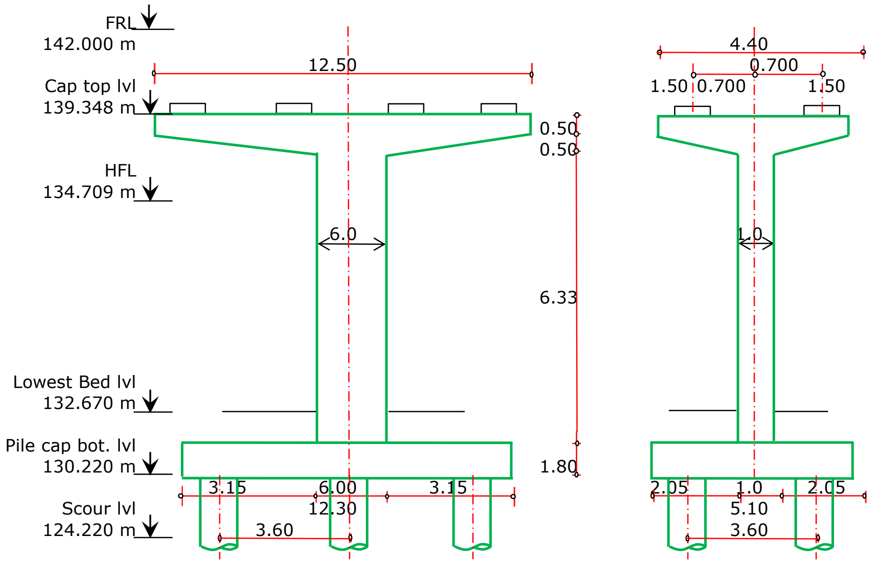
Horizontal seismic coefficient	A_{hL}	=	0.180
Vertical seismic coefficient	V_h	=	0.120

Seismic Coefficient in Transverse Direction

Horizontal seismic coefficient	A_{hT}	=	0.180
Vertical seismic coefficient	V_h	=	0.120

7.0 Load calculation of Substructure & Foundation

Dimensional details of Pier cap, Shaft & Footing

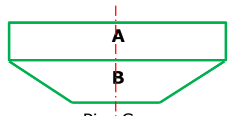


Section of Pier

Elevation of Pier

Height of pedestals,

PD1	=	0.200	m
PD2	=	0.269	m
PD3	=	0.338	m
PD4	=	0.406	m
PD5	=	0.475	m



Pier Cap

Area	Y from base	A x Y	
55.0	0.75	41.3	**A**
4.6	0.28	1.3	**B**
59.6	-	42.5	
	c.g from base =	0.713	m

Self weight of Pier Cap & Pedestal:-

Avg. size of each Pedestal on Pier	=	0.85 x 0.80	m^2
No's of Pedestal on Pier	=	10	
Avg. depth of Pedestal	=	0.303	m
Self weight of Pedestal on Pier	=	5.15	t
Self weight of Pier Cap	=	106.9	t
Self weight of Pier Cap with Pedestal	=	112.0	t

Self Weight of Pier Shaft:-

Radius of Cutwater Portion	=	0.5	m
Total height of Pier shaft	=	6.33	m
Area of Pier Shaft including Cutwater	=	6.57	m^2
Weight of Pier Shaft at Dry condition	=	103.9	t
Weight of Pier Shaft at HFL condition	=	86.3	t

Self Weight of Foundation:-

Total volume of foundation	=	112.9	m^3
Weight of foundation at Dry condition	=	282.3	t
Weight of foundation at HFL condition	=	169.4	t

Weight of Overburden Earth below LBL:-

Total volume of earth above foundation	=	36.5	m^3
Weight of earth at Dry condition	=	73.0	t
Weight of earth at HFL condition	=	36.5	t

8.0 Seismic Load Calculation

8.1 Summary of Dead load & SIDL of Superstructure:-

Description	Load (t)			Eccentricity e_L (m)		Moment	Eccentricity $e_{V\ Base}$ (m)		Eccentricity $e_{V\ Shaft}$ (m)	
	LHS Span	RHS Span	Total	Long. LHS	Long. RHS	M_L (t-m)	Trans.	Long.	Trans.	Long.
Superstructure DL & FPDL	225.2	225.2	450.5	0.700	0.700	0.0	11.187	9.431	9.387	7.631
Super Imposed Dead Load	19.3	19.3	38.5	0.700	0.700	0.0	12.230	9.431	10.430	7.631
Surfacinf / Wearing coat	23.0	23.0	46.0	0.700	0.700	0.0	11.730	9.431	9.930	7.631
Total DL, SIDL & Surfacing	***267.5***	***267.5***	***535.0***	***0.700***	***0.700***	***374.5***	***11.308***	***9.431***	***9.508***	***7.631***

			LHS Span		RHS Span	
Total load of DL, SIDL & Surfacing		=	267.5	t	267.5	t
Seismic Longitudinal force:-						
Longitudinal seismic coefficien	A_{hL}	=	0.180			
Seismic component of DL, SIDL & Surfacinc		=	48.1	t	48.1	t
Lever arm above base slab	$e_{V/Base}$	=	9.431	m	9.431	m
Longitudinal moment	M_L	=	454.1	t-m	454.1	t-m
Lever arm above Shaft	$e_{V/Shaft}$	=	7.631	m	7.631	m
Longitudinal moment	$M_{L/Shaft}$	=	367.4	t-m	367.4	t-m
Seismic Transverse force:-						
Transverse seismic coefficient	A_{hT}	=	0.180			
Seismic component of DL, SIDL & Surfacinc		=	48.1	t	48.1	t
Lever arm above base slab	$e_{V/Base}$	=	11.308	m	11.308	m
Transverse moment	M_T	=	544.5	t-m	544.5	t-m
Lever arm above Shaft	$e_{V/Shaft}$	=	9.508	m	9.508	m
Longitudinal moment	$M_{T/Shaft}$	=	457.8	t-m	457.8	t-m
Seismic Vertical force:-						
Vertical seismic coefficient	A_v	=	0.120			
Seismic component of DL, SIDL & Surfacinc		=	32.1	t	32.1	t
Lever arm about c/L of base sl	e_L	=	0.700	m	0.700	m
Longitudinal moment	M_L	=	22.5	t-m	22.5	t-m

8.1.1 Summary of Seismic component of Permanent load (DL, SIDL & Surfacing):-

At Fixed End	V	H_L	H_T	M_L	M_T	$M_{L/Shaft}$	$M_{T/Shaft}$
	t	t	t	t-m	t-m	t-m	t-m
Seismic Longitudinal force	-	***96.3***	-	***908.1***	-	***734.8***	-
Seismic Transverse force	-	-	***96.3***	-	***1088.9***	-	***915.6***
Seismic Vertical force	***64.2***	-	-	***0.0***	-	***0.0***	-

8.2 Summary of Live Load:-

Description	Load (t)			Moment (t-m)		Tran. force	Ecc. (m)	Eccentricity e_L (m)		Tran Mom.
	LHS Span	RHS Span	Total	Long. M_L	Trans. M_T	H_T (t)	$e_{V\ Base}$	Long. LHS	Long. RHS	M_T (t-m)
Max LL Reaction case LL1	71.0	72.0	143.1	2.4	363.0	0.0	12.980	0.700	0.700	0.0
Max Trans Moment case LL2	50.0	52.4	102.4	1.7	374.3	0.0	12.980	0.700	0.700	0.0
Max Long Moment case LL3	5.4	126.9	132.2	76.5	310.6	0.0	12.980	0.700	0.700	0.0

			LL1		LL2		LL3	
			LHS Span	RHS Span	LHS Span	RHS Span	LHS Span	RHS Span
Total Live load		=	71.0	72.0 t	50.0	52.4 t	5.4	126.9 t

Seismic Longitudinal force:- No Live load seismic component is considered in longitudinal direction

			LL1 LHS	LL1 RHS	LL2 LHS	LL2 RHS	LL3 LHS	LL3 RHS
Seismic Transverse force:-								
Transverse seismic coefficient	A_{hT}	=	0.180					
Seismic component of LL		=	12.8	13.0 t	9.0	9.4 t	1.0	22.8 t
Lever arm above base slab	$e_{V/Base}$	=	12.980	12.980 m	12.980	12.980 m	12.980	12.980 m
Transverse moment	M_T	=	166.0	168.2 t-m	116.8	122.4 t-m	12.6	296.4 t-m
Lever arm above Shaft	$e_{V/Shaft}$	=	11.180	11.180 m	11.180	11.180 m	11.180	11.180 m
Longitudinal moment	$M_{T/Shaft}$	=	143.0	144.9 t-m	100.6	105.5 t-m	10.8	255.3 t-m
Seismic Vertical force:-								
Vertical seismic coefficient	A_v	=	0.120					
Seismic component of LL		=	8.5	8.6 t	6.0	6.3 t	0.6	15.2 t
Lever arm about c/L of base sl	e_L	=	0.700	0.700 m	0.700	0.700 m	0.700	0.700 m
Longitudinal moment	M_L	=	6.0	6.0 t-m	4.2	4.4 t-m	0.5	10.7 t-m

8.2.1 Summary of Seismic component of Live Load:-

At Fixed End for LL1 Case	***V***	***H_L***	***H_T***	***M_L***	***M_T***	***$M_{L/Shaft}$***	***$M_{T/Shaft}$***
	t	***t***	***t***	***t-m***	***t-m***	***t-m***	***t-m***
Seismic Transverse force	***-***	***-***	***25.7***	***-***	***334.2***	***-***	***287.9***
Seismic Vertical force	***17.2***	***-***	***-***	***0.1***	***-***	***0.1***	***-***

At Fixed End for LL2 Case	***V***	***H_L***	***H_T***	***M_L***	***M_T***	***$M_{L/Shaft}$***	***$M_{T/Shaft}$***
	t	***t***	***t***	***t-m***	***t-m***	***t-m***	***t-m***
Seismic Transverse force	***-***	***-***	***18.4***	***-***	***239.2***	***-***	***206.1***
Seismic Vertical force	***12.3***	***-***	***-***	***0.2***	***-***	***0.2***	***-***

At Fixed End for LL3 Case	***V***	***H_L***	***H_T***	***M_L***	***M_T***	***$M_{L/Shaft}$***	***$M_{T/Shaft}$***
	t	***t***	***t***	***t-m***	***t-m***	***t-m***	***t-m***
Seismic Transverse force	***-***	***-***	***23.8***	***-***	***309.0***	***-***	***266.1***
Seismic Vertical force	***15.9***	***-***	***-***	***10.2***	***-***	***10.2***	***-***

8.3 Summary of Substructure & Foundation load:-

Description	LWL/Dry condition				HFL condition			
	V	$e_{V/Base}$	$e_{V/Shaft}$	e_L	V	$e_{V/Base}$	$e_{V/Shaft}$	e_L
	t	m	m	m	t	m	m	m
Pier cap with Pedestal	112.0	8.841	7.041	0.00	112.0	8.841	7.041	0.000
Pier shaft	103.9	4.964	3.164	0.00	86.3	4.964	3.164	0.000
Total Substructure	***216.0***	***6.975***	***5.175***	***0.000***	***198.3***	***7.154***	***5.354***	***0.000***
Foundation	282.3	0.900	-	0.00	169.4	0.900	-	0.000
Total Substructure & Foundation	***498.3***	***3.533***	-	***0.000***	***367.7***	***4.273***	-	***0.000***
Overburden earth	73.0	2.125	-	0.00	36.5	2.125	-	0.000

			LWL/Dry condition	HFL condition
Total load of Substructure & Foundation		=	498.3 t	367.7 t
Total load of Substructure		=	216.0 t	198.3 t
Seismic Longitudinal force:-				
Longitudinal seismic coefficien	A_{hL}	=	0.180	
Seismic component of Substructure & Foun		=	89.7 t	66.2 t
Lever arm above base slab	$e_{V/Base}$	=	3.533 m	4.273 m
Longitudinal moment	M_L	=	316.9 t-m	282.8 t-m
Seismic component of Substructure		=	38.9 t	35.7 t
Lever arm above Shaft	$e_{V/Shaft}$	=	5.175 m	5.354 m
Longitudinal moment	$M_{L/Shaft}$	=	464.1 t-m	354.3 t-m
Seismic Transverse force:-				
Transverse seismic coefficient	A_{hT}	=	0.180	
Seismic component of Substructure & Foun		=	89.7 t	66.2 t
Lever arm above base slab	$e_{V/Base}$	=	3.533 m	4.273 m
Transverse moment	M_T	=	316.9 t-m	282.8 t-m
Seismic component of Substructure		=	38.9 t	35.7 t
Lever arm above Shaft	$e_{V/Shaft}$	=	5.175 m	5.354 m
Longitudinal moment	$M_{L/Shaft}$	=	464.1 t-m	354.3 t-m
Seismic Vertical force:-				
Vertical seismic coefficient	A_v	=	0.120	
Seismic component of Substructure & Foun		=	59.8 t	44.1 t
Lever arm about c/L of base sl	e_L	=	0.000 m	0.000 m
Longitudinal moment	M_L	=	0.0 t-m	0.0 t-m
Seismic component of Substructure		=	25.9 t	23.8 t
Lever arm about c/L of base sl	e_L	=	0.000 m	0.000 m
Longitudinal moment	M_L	=	0.0 t-m	0.0 t-m

8.3.1 Summary of Seismic component of Substructure & Foundation:- (At Founding Level)

At Fixed End for LWL Condition	***V***	***H_L***	***H_T***	***M_L***	***M_T***
	t	***t***	***t***	***t-m***	***t-m***
Seismic Longitudinal force	***-***	***89.7***	***-***	***316.9***	***-***
Seismic Transverse force	***-***	***-***	***89.7***	***-***	***316.9***
Seismic Vertical force	***59.8***	***-***	***-***	***0.0***	***-***

(At Shaft Bottom Level)

V	***H_L***	***H_T***	***$M_{L/Shaft}$***	***$M_{T/Shaft}$***
t	***t***	***t***	***t-m***	***t-m***
-	***38.9***	***-***	***464.1***	***-***
-	***-***	***38.9***	***-***	***464.1***
25.9	***-***	***-***	***0.0***	***-***

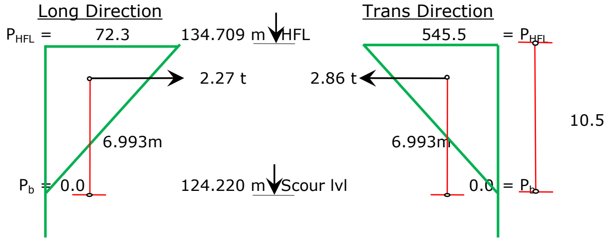
At Fixed End for HFL Condition	***V***	***H_L***	***H_T***	***M_L***	***M_T***
	t	***t***	***t***	***t-m***	***t-m***
Seismic Longitudinal force	***-***	***66.2***	***-***	***282.8***	***-***
Seismic Transverse force	***-***	***-***	***66.2***	***-***	***282.8***
Seismic Vertical force	***44.1***	***-***	***-***	***0.0***	***-***

V	***H_L***	***H_T***	***$M_{L/Shaft}$***	***$M_{T/Shaft}$***
t	***t***	***t***	***t-m***	***t-m***
-	***35.7***	***-***	***354.3***	***-***
-	***-***	***35.7***	***-***	***354.3***
23.8	***-***	***-***	***0.0***	***-***

9.0 Water Current force calculation at HFL condition

Mean velocity at HFL,	V	=	3.0	m/sec
Type of cut water		=	Semi circular	
Possible variation in water current force		=	20	^{0}
K value for Pier	Long direct	=	0.66	
	Trans direct		0.66	
	V_{max}	=	$\sqrt{2}$ x	V
			4.24	m/sec
Max. current effect in long dire	V_L	=	4.24 x	Sin (20)
		=	1.45	m/sec
	V_L^2	=	2.11	m/sec
Max. current effect in trans dir	V_T	=	4.24 x	Cos(20)
		=	3.99	m/sec
	V_T^2	=	15.89	m/sec

Pressure due to water current, P = $52 \times K \times V^2_{max}$ Kg/m^2



Velocity Profile

9.1 Summary of Water Current force calculation at HFL condition:-

Water Current force	H_L	H_T	$e_{V/Base}$ (m)	M_L	M_T
At Pile cap bottom level	2.27	2.86	0.993	2.3	2.8
At Shaft bottom level	2.27	2.86	-0.807	-1.8	-2.3

10.0 Wind Load calculation

Wind Load on Superstructure:-

Basic Wind Speed = 50 m/sec
Depth of Superstructure = 1.80 m
Height of Crash Barrier = 0.90 m
Height of Pier (GL to Cap top) = 6.68 m
height of Vehicle = 3.0 m

Hourly mean speed, V_z = 42.1 m/sec
Wind pressure, P_z = 1064.5 N/m^2

Vertical Wind Load:-
Area in plan, A_3 = 297.0 m^2
Lift Coefficient, C_L = 0.75
Gust factor, G = 2.00
Vertical wind load, Fv = 47.4 t

Transverse Wind Load:-
Solid Area, A_1 = 59.4 m^2
Gust factor, G = 2.00
Drag Coefficient, C_D = 1.30 b / t = 7.50 where, Width=b & Depth=t
Transverse wind load, F_T = 16.4 t

Longitudinal Wind Load
Longitudinal wind load, F_L = 25% of F_T
= 4.1 t
Eccentricity of load from Founding level = 10.78 m

Wind Load on Live Load:-

Transverse Wind Load
Solid Area, A_1 = 46.2 m^2
Gust factor, G = 2.00
Drag Coefficient, C_D = 1.20
Transverse wind load, F_T = 11.8 t

Longitudinal Wind Load
Longitudinal wind load, F_L = 25% of F_T
= 3.0 t
Eccentricity of load from Founding level = 13.28 m

Wind Load on Substructure

Solid Area, A_{1T} = 6.7 m^2
Solid Area, A_{1L} = 40.1 m^2
Gust factor, G = 2.00
Drag Coefficient, C_D = 1.30 h / b = 1.11 , t / b = 0.17 where Height=h

Transverse wind load, F_T = 1.8 t
Longitudinal wind load, F_L = 11.1 t

Eccentricity of load from Founding level = 5.789 m

Wind Load	V (t)	H_L (t)	H_T(t)	$e_{V/Base}$ (m)	M_L (t-m)	M_T (t-m)
on Superstructure	47.4	4.1	16.4	10.781	44.3	177.2
on Substructure	-	11.1	1.8	5.789	64.2	10.7
on Live load	-	3.0	11.8	13.280	39.2	156.7

$e_{V/Shaft}$ (m)	M_L (t-m)	M_T (t-m)
8.981	36.9	147.6
3.989	44.2	7.4
11.480	33.9	135.5

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

11.0 Summary of Forces at Founding Level

Summary of Forces Founding level (LWL)	Forces about c/L of foundation					Span Dislodge Condition Forces about c/L of foundation				
	V	H_L	H_T	M_L	M_T	V	H_L	H_T	M_L	M_T
Description	t	t	t	t-m	t-m	t	t	t	t-m	t-m
1) DL of Superstructure	450.5			0.0	0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier	38.5			0.0	0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat	46.0			0.0	0.0	23.0			16.1	0.0
4) Carriageway Live load, LL1	143.1		0.0	2.4	363.0					
LL2	102.4		0.0	1.7	374.3					
LL3	132.2		0.0	76.5	310.6					
5) Bearing frictional force, LL1		45.3		420.2			13.4		124.1	
LL2		41.9		388.5						
LL3		44.7		415.2						
6) Substructure & Foundation	498.3					498.3				
7) Overburden earth	73.0					73.0				
8) Wind load	47.4	15.2	18.3	108.5	187.9					
Seismic Longitudinal										
10) DL, SIDL component of Superstructure		96.3		908.1			48.1		454.1	
11) Substructure & Foundation component		89.7		316.9			89.7		316.9	
Seismic Transverse										
12) DL, SIDL component of Superstructure			96.3		1088.9			48.1		544.5
13) Cariageway Live load, LL1			25.7		334.2					
LL2			18.4		239.2					
LL3			23.8		309.0					
14) Substructure & Foundation component			89.7		316.9			89.7		316.9
Seismic Vertical										
15) DL, SIDL component of Superstructure	64.2			0.0		32.1			22.5	
16) Cariageway Live load, LL1	17.2			0.1						
LL2	12.3			0.2						
LL3	15.9			10.2						
17) Substructure & Foundation component	59.8			0.0		59.8			0.0	

Summary of Forces Founding level (HFL)	Forces about c/L of foundation					Span Dislodge Condition Forces about c/L of foundation				
	V	H_L	H_T	M_L	M_T	V	H_L	H_T	M_L	M_T
Description	t	t	t	t-m	t-m	t	t	t	t-m	t-m
1) DL of Superstructure	450.5			0.0	0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier	38.5			0.0	0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat	46.0			0.0	0.0	23.0			16.1	0.0
4) Carriageway Live load, LL1	143.1		0.0	2.4	363.0					
LL2	102.4		0.0	1.7	374.3					
LL3	132.2		0.0	76.5	310.6					
5) Bearing frictional force, LL1		45.3		420.2			13.4		124.1	
LL2		41.9		388.5						
LL3		44.7		415.2						
6) Substructure & Foundation	367.7					367.7				
7) Overburden earth	36.5					36.5				
8) Wind load	47.4	15.2	18.3	108.5	187.9					
9) Water current force		2.27	2.86	2.3	2.8		2.3	2.9	2.3	2.8
Seismic Longitudinal										
10) DL, SIDL component of Superstructure		96.3		908.1			48.1		454.1	
11) Substructure & Foundation component		66.2		282.8			66.2		282.8	
Seismic Transverse										
12) DL, SIDL component of Superstructure			96.3		1088.9			48.1		544.5
13) Cariageway Live load, LL1			25.7		334.2					
LL2			18.4		239.2					
LL3			23.8		309.0					
14) Substructure & Foundation component			66.2		282.8			66.2		282.8
Seismic Vertical										
15) DL, SIDL component of Superstructure	64.2			0.0		32.1			22.5	
16) Cariageway Live load, LL1	17.2			0.1						
LL2	12.3			0.2						
LL3	15.9			10.2						
17) Substructure & Foundation component	44.1			0.0		44.1			0.0	

11.1 Load Combination for Pile Capacity Check (as per IRC:78-2014)

Load Case Description	Forces about c/L of pile cap					
	V	H_L	H_T	M_L	M_T	
	t	t	t	t-m	t-m	
LC-01: NC,LWL	1106	0	0	0	0	LC-01
LC-02: NC,LWL,LL1	1249	45	0	423	363	LC-02
LC-03: NC,LWL,LL2	1209	42	0	390	374	LC-03
LC-04: NC,LWL,LL3	1238	45	0	492	311	LC-04
LC-05: SS,LWL,LL1,SL=1,ST=0.3,SV=0.3	1173	231	57	1646	514	LC-05
LC-06: SS,LWL,LL2,SL=1,ST=0.3,SV=0.3	1165	228	57	1614	511	LC-06
LC-07: SS,LWL,LL3,SL=1,ST=0.3,SV=0.3	1171	231	57	1656	502	LC-07
LC-08: SS,LWL,LL1,SL=0.3,ST=1,SV=0.3	1173	101	191	788	1545	LC-08
LC-09: SS,LWL,LL2,SL=0.3,ST=1,SV=0.3	1165	98	190	756	1529	LC-09
LC-10: SS,LWL,LL3,SL=0.3,ST=1,SV=0.3	1171	101	191	799	1530	LC-10
LC-11: SS,LWL,LL1,SL=0.3,ST=0.3,SV=1	1262	101	57	788	514	LC-11
LC-12: SS,LWL,LL2,SL=0.3,ST=0.3,SV=1	1253	98	57	756	511	LC-12
LC-13: SS,LWL,LL3,SL=0.3,ST=0.3,SV=1	1260	101	57	800	502	LC-13
LC-14: NC,HFL	939	2	3	2	3	LC-14
LC-15: NC,HFL,LL1	1082	48	3	425	366	LC-15
LC-16: NC,HFL,LL2	1042	44	3	392	377	LC-16
LC-17: NC,HFL,LL3	1071	47	3	494	313	LC-17
LC-18: SS,HFL,LL1,SL=1,ST=0.3,SV=0.3	934	210	53	1614	507	LC-18
LC-19: SS,HFL,LL2,SL=1,ST=0.3,SV=0.3	926	207	53	1582	504	LC-19

LC-20: SS,HFL,LL3,SL=1,ST=0.3,SV=0.3	932	209	53	1623	495	LC-20
LC-21: SS,HFL,LL1,SL=0.3,ST=1,SV=0.3	934	96	170	780	1514	LC-21
LC-22: SS,HFL,LL2,SL=0.3,ST=1,SV=0.3	926	93	169	748	1497	LC-22
LC-23: SS,HFL,LL3,SL=0.3,ST=1,SV=0.3	932	96	170	789	1498	LC-23
LC-24: SS,HFL,LL1,SL=0.3,ST=0.3,SV=1	856	96	53	780	507	LC-24
LC-25: SS,HFL,LL2,SL=0.3,ST=0.3,SV=1	849	93	53	748	504	LC-25
LC-26: SS,HFL,LL3,SL=0.3,ST=0.3,SV=1	854	96	53	788	495	LC-26
LC-27: NC,LWL,SD	839	13	0	311	0	LC-27
LC-28: SS,LWL,SD,SL=1,ST=0.3,SV=0.3	859	117	31	895	194	LC-28
LC-29: SS,LWL,SD,SL=0.3,ST=1,SV=0.3	859	44	103	490	646	LC-29
LC-30: SS,LWL,SD,SL=0.3,ST=0.3,SV=1	908	44	31	502	194	LC-30
LC-31: NC,HFL,SD	672	16	3	314	3	LC-31
LC-32: SS,HFL,SD,SL=1,ST=0.3,SV=0.3	655	101	29	861	189	LC-32
LC-33: SS,HFL,SD,SL=0.3,ST=1,SV=0.3	655	41	89	474	623	LC-33
LC-34: SS,HFL,SD,SL=0.3,ST=0.3,SV=1	614	41	29	463	189	LC-34
	1	2	3	4	5	6

Critical Load Combination		V	H_L	H_T	M_L	M_T	LC No.
		t	t	t	t-m	t-m	
NC: Max. Axial load,	V_{max}	1249	45	0	423	363	LC-02
NC: Min. Axial load,	V_{min}	672	16	3	314	3	LC-31
NC: Max. Long. Moment,	M_{L-max}	1071	47	3	494	313	LC-17
NC: Max. Trans. Moment,	M_{T-max}	1042	44	3	392	377	LC-16
NC: Max. Long. Force,	$H_{L,Max}$	1082	48	3	425	366	LC-15
NC: Max. Trans. Force,	$H_{T,Max}$	939	2	3	2	3	LC-14
SS: Max. Axial load,	V_{max}	1262	101	57	788	514	LC-11
SS: Min. Axial load,	V_{min}	614	41	29	463	189	LC-34
SS: Max. Long. Moment,	M_{L-max}	1171	231	57	1656	502	LC-07
SS: Max. Trans. Moment,	M_{T-max}	1173	101	191	788	1545	LC-08
SS: Max. Long. Force,	$H_{L,Max}$	1173	231	57	1646	514	LC-05
SS: Max. Trans. Force,	$H_{T,Max}$	1173	101	191	788	1545	LC-08

11.2 Load Combination for Design of Piles & Pile Cap at ULS (Table B.2 & B.4 of IRC:6 -2017)

Load Case Description	Forces about c/L of pile cap					
	V	H_L	H_T	M_L	M_T	
	t	t	t	t-m	t-m	
LC-01: NC,LWL	1554	14	16	98	169	LC-01
LC-02: NC,LWL,LL1*	1769	54	16	479	714	LC-02
LC-03: NC,LWL,LL2*	1708	51	16	450	731	LC-03
LC-04: NC,LWL,LL3*	1753	54	16	586	635	LC-04
LC-05: NC,LWL,LL1	1719	82	16	731	587	LC-05
LC-06: NC,LWL,LL2	1672	76	16	682	600	LC-06
LC-07: NC,LWL,LL3	1707	81	16	808	526	LC-07
LC-08: SS,LWL,LL1,SL=1,ST=0.3,SV=0.3	1598	302	86	2048	735	LC-08
LC-09: SS,LWL,LL2,SL=1,ST=0.3,SV=0.3	1589	300	85	2032	729	LC-09
LC-10: SS,LWL,LL3,SL=1,ST=0.3,SV=0.3	1595	301	86	2061	723	LC-10
LC-11: SS,LWL,LL1,SL=0.3,ST=1,SV=0.3	1598	106	287	762	2282	LC-11
LC-12: SS,LWL,LL2,SL=0.3,ST=1,SV=0.3	1589	105	285	746	2255	LC-12
LC-13: SS,LWL,LL3,SL=0.3,ST=1,SV=0.3	1595	106	286	775	2264	LC-13
LC-14: SS,LWL,LL1,SL=0.3,ST=0.3,SV=1	1732	106	86	762	735	LC-14
LC-15: SS,LWL,LL2,SL=0.3,ST=0.3,SV=1	1722	105	85	746	729	LC-15
LC-16: SS,LWL,LL3,SL=0.3,ST=0.3,SV=1	1729	106	86	777	723	LC-16
LC-17: NC,HFL	977	14	17	89	153	LC-17
LC-18: NC,HFL,LL1*	1163	51	17	428	625	LC-18
LC-19: NC,HFL,LL2*	1110	48	17	402	640	LC-19
LC-20: NC,HFL,LL3*	1149	50	17	521	557	LC-20
LC-21: NC,HFL,LL1	1120	51	17	428	516	LC-21
LC-22: NC,HFL,LL2	1079	48	17	402	527	LC-22
LC-23: NC,HFL,LL3	1109	50	17	498	464	LC-23
LC-24: SS,HFL,LL1,SL=1,ST=0.3,SV=0.3	1265	269	78	1999	723	LC-24
LC-25: SS,HFL,LL2,SL=1,ST=0.3,SV=0.3	1257	267	78	1983	717	LC-25
LC-26: SS,HFL,LL3,SL=1,ST=0.3,SV=0.3	1263	268	78	2011	710	LC-26
LC-27: SS,HFL,LL1,SL=0.3,ST=1,SV=0.3	1265	98	254	749	2233	LC-27
LC-28: SS,HFL,LL2,SL=0.3,ST=1,SV=0.3	1257	96	252	733	2207	LC-28
LC-29: SS,HFL,LL3,SL=0.3,ST=1,SV=0.3	1263	98	254	760	2215	LC-29
LC-30: SS,HFL,LL1,SL=0.3,ST=0.3,SV=1	1147	98	78	749	723	LC-30
LC-31: SS,HFL,LL2,SL=0.3,ST=0.3,SV=1	1141	96	78	733	717	LC-31
LC-32: SS,HFL,LL3,SL=0.3,ST=0.3,SV=1	1145	98	78	758	710	LC-32
LC-33: NC,LWL,SD	1142	20	0	445	0	LC-33
LC-34: SS,LWL,SD,SL=1,ST=0.3,SV=0.3	1162	110	31	905	194	LC-34
LC-35: SS,LWL,SD,SL=0.3,ST=1,SV=0.3	1162	38	103	500	646	LC-35
LC-36: SS,LWL,SD,SL=0.3,ST=0.3,SV=1	1210	38	62	512	388	LC-36
LC-37: NC,HFL,SD	916	22	3	448	3	LC-37
LC-38: SS,HFL,SD,SL=1,ST=0.3,SV=0.3	899	95	29	871	189	LC-38
LC-39: SS,HFL,SD,SL=0.3,ST=1,SV=0.3	899	35	89	484	623	LC-39
LC-40: SS,HFL,SD,SL=0.3,ST=0.3,SV=1	859	35	54	472	375	LC-40
	1	2	3	4	5	6

Critical Load Combination		V	H_L	H_T	M_L	M_T	LC No.
		t	t	t	t-m	t-m	
Max. Axial load,	V_{max}	1769	54	16	479	714	LC-02
Min. Axial load,	V_{min}	859	35	54	472	375	LC-40
Max. Long. Moment,	M_{L-max}	1595	301	86	2061	723	LC-10
Max. Trans. Moment,	M_{T-max}	1598	106	287	762	2282	LC-11
Max. Long. Force,	$H_{L,Max}$	1598	302	86	2048	735	LC-08
Max. Trans. Force,	$H_{T,Max}$	1598	106	287	762	2282	LC-11

11.3 Load Combination for Design of Piles & Pile Cap at SLS (Table B.3 of IRC:6 -2017))

Load Case Description	Forces about c/L of foundation					
	V	H_L	H_T	M_L	M_T	
	t	t	t	t-m	t-m	
SR-01: Rare,LWL,WL*	1163	15	18	108	188	SR-01
SR-02: Rare,LWL,LL1*	1287	36	11	320	476	SR-02
SR-03: Rare,LWL,LL2*	1246	34	11	300	487	SR-03
SR-04: Rare,LWL,LL3*	1276	36	11	391	423	SR-04
SR-05: Rare,LWL,LL1	1251	54	11	487	385	SR-05
SR-06: Rare,LWL,LL2	1221	51	11	455	393	SR-06
SR-07: Rare,LWL,LL3	1243	54	11	538	346	SR-07
SR-08: Rare,HFL,WL*	996	17	21	111	191	SR-08
SR-09: Rare,HFL,LL1*	1120	39	14	322	479	SR-09
SR-10: Rare,HFL,LL2*	1079	37	14	302	490	SR-10
SR-11: Rare,HFL,LL3*	1109	38	14	393	426	SR-11
SR-12: Rare,HFL,LL1	1084	57	14	489	388	SR-12
SR-13: Rare,HFL,LL2	1054	53	14	457	396	SR-13
SR-14: Rare,HFL,LL3	1076	56	14	540	349	SR-14
SR-15: Rare,LWL,SD	843	13	0	315	0	SR-15
SR-16: Rare,HFL,SD	676	16	3	317	3	SR-16
SQ-P-1: Q-P,LWL	1115	23	0	210	0	SQ-P-1
SQ-P-2: Q-P,LWL	1115	21	0	194	0	SQ-P-2
SQ-P-3: Q-P,LWL	1115	22	0	208	0	SQ-P-3
SQ-P-4: Q-P,HFL	948	23	0	210	0	SQ-P-4
SQ-P-5: Q-P,HFL	948	21	0	194	0	SQ-P-5
SQ-P-6: Q-P,HFL	948	22	0	208	0	SQ-P-6
	1	2	3	4	5	6

Critical Load Combination		V	H_L	H_T	M_L	M_T	LC No.
		t	t	t	t-m	t-m	
SR: Max. Axial load,	V_{max}	1287	36	11	320	476	SR-02
SR:Min. Axial load,	V_{min}	676	16	3	317	3	SR-16
SR:Max. Long. Moment,	M_{L-max}	1076	56	14	540	349	SR-14
SR:Max. Trans. Moment,	M_{T-max}	1079	37	14	302	490	SR-10
SQ-P: Max. Axial load,	V_{max}	1115	23	0	210	0	SQ-P-1
SQ-P:Min. Axial load,	V_{min}	948	23	0	210	0	SQ-P-4
SQ-P:Max. Long. Moment,	M_{L-max}	1115	23	0	210	0	SQ-P-1
SQ-P:Max. Trans. Moment,	M_{T-max}	843	13	0	315	0	SR-15

12.0 Summary of Forces at Pier Shaft Bottom Level

Summary of Forces Pier Shaft Bottom level (LWL)	Forces about c/L of Shaft					Span Dislodge Condition Forces about c/L of Shaft				
	V	H_L	H_T	M_L	M_T	V	H_L	H_T	M_L	M_T
Description	t	t	t	t-m	t-m	t	t	t	t-m	t-m
1) DL of Superstructure	450.5			0.0	0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier	38.5			0.0	0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat	46.0			0.0	0.0	23.0			16.1	0.0
4) Carriageway Live load, LL1	143.1		0.0	2.4	363.0					
LL2	102.4		0.0	1.7	374.3					
LL3	132.2		0.0	76.5	310.6					
5) Bearing frictional force, LL1		45.3		338.7			13.4		100.0	
LL2		41.9		313.1						
LL3		44.7		334.7						
6) Substructure	216.0					216.0				
7) Wind load	47.4	15.2	18.3	81.1	155.0					
Seismic Longitudinal										
10) DL, SIDL component of Superstructure		96.3		734.8			48.1		367.4	
11) Substructure component		38.9		464.1			38.9		464.1	
Seismic Transverse										
12) DL, SIDL component of Superstructure			96.3		915.6			48.1		457.8
13) Cariageway Live load, LL1			25.7		287.9					
LL2			18.4		206.1					
LL3			23.8		266.1					
14) Substructure component			38.9		464.1			38.9		464.1
Seismic Vertical										
15) DL, SIDL component of Superstructure	64.2			0.0		32.1			22.5	
16) Cariageway Live load, LL1	17.2			0.1						
LL2	12.3			0.2						
LL3	15.9			10.2						
17) Substructure component	25.9			0.0		25.9			0.0	

12.1 Load Combination for Design of Shaft at ULS (Table B.2 of IRC:6 -2017)

Load Case Description	Forces about c/L of Shaft					
	V	H_L	H_T	M_L	M_T	
	t	t	t	t-m	t-m	
LC-01: NC,LWL	1075	14	16	73	140	LC-01
LC-02: NC,LWL,LL1*	1289	54	16	381	684	LC-02
LC-03: NC,LWL,LL2*	1228	51	16	357	701	LC-03
LC-04: NC,LWL,LL3*	1273	54	16	489	605	LC-04
LC-05: NC,LWL,LL1	1239	82	16	584	557	LC-05
LC-06: NC,LWL,LL2	1193	76	16	545	570	LC-06
LC-07: NC,LWL,LL3	1227	81	16	663	497	LC-07
LC-08: SS,LWL,LL1,SL=1,ST=0.3,SV=0.3	1103	225	63	1968	719	LC-08
LC-09: SS,LWL,LL2,SL=1,ST=0.3,SV=0.3	1094	224	62	1955	714	LC-09
LC-10: SS,LWL,LL3,SL=1,ST=0.3,SV=0.3	1101	225	63	1982	707	LC-10
LC-11: SS,LWL,LL1,SL=0.3,ST=1,SV=0.3	1103	83	210	709	2229	LC-11
LC-12: SS,LWL,LL2,SL=0.3,ST=1,SV=0.3	1094	82	208	696	2206	LC-12
LC-13: SS,LWL,LL3,SL=0.3,ST=1,SV=0.3	1101	83	210	723	2212	LC-13
LC-14: SS,LWL,LL1,SL=0.3,ST=0.3,SV=1	1201	83	63	709	719	LC-14
LC-15: SS,LWL,LL2,SL=0.3,ST=0.3,SV=1	1191	82	62	696	714	LC-15
LC-16: SS,LWL,LL3,SL=0.3,ST=0.3,SV=1	1199	83	63	725	707	LC-16
LC-17: NC,LWL,SD	662	20	0	409	0	LC-17
LC-18: SS,LWL,SD,SL=1,ST=0.3,SV=0.3	675	72	20	938	207	LC-18
LC-19: SS,LWL,SD,SL=0.3,ST=1,SV=0.3	675	26	65	501	691	LC-19
LC-20: SS,LWL,SD,SL=0.3,ST=0.3,SV=1	705	26	39	513	415	LC-20

	1	2	3	4	5	6	
Critical Load Combination	V	H_L	H_T	M_L	M_T	LC No.	
	t	t	t	t-m	t-m		
NS: Max. Axial load, V_{max}	1289	54	16	381	684	LC-02	NS: Vmax
NS: Min. Axial load, V_{min}	662	20	0	409	0	LC-17	NS: Vmin
NS: Max. Long. Moment, M_{L-max}	1227	81	16	663	497	LC-07	NS: ML-max
NS: Max. Trans. Moment, M_{T-max}	1228	51	16	357	701	LC-03	NS: MT-max
SS: Max. Axial load, V_{max}	1201	83	63	709	719	LC-14	SS: Vmax
SS: Min. Axial load, V_{min}	675	72	20	938	207	LC-18	SS: Vmin
SS: Max. Long. Moment, M_{L-max}	1101	225	63	1982	707	LC-10	SS: ML-max
SS: Max. Trans. Moment, M_{T-max}	1103	83	210	709	2229	LC-11	SS: MT-max

12.2 Load Combination for Design of Shaft at SLS (Table B.3 of IRC:6 -2017)

Load Case Description	Forces about c/L of Shaft					
	V	H_L	H_T	M_L	M_T	
	t	t	t	t-m	t-m	
SR-1: Rare,LWL,WL*	808	15	18	81	155	SR-1
SR-2: Rare,LWL,LL1*	932	36	11	254	456	SR-2
SR-3: Rare,LWL,LL2*	891	34	11	238	467	SR-3
SR-4: Rare,LWL,LL3*	921	36	11	326	404	SR-4
SR-5: Rare,LWL,LL1	896	54	11	389	365	SR-5
SR-6: Rare,LWL,LL2	865	51	11	363	374	SR-6
SR-7: Rare,LWL,LL3	888	54	11	441	326	SR-7
SR-8: Rare,LWL,SD	488	13	0	290	0	SR-8
SQ-P-1: Q-P,LWL	760	23	0	169	0	SQ-P-1
SQ-P-2: Q-P,LWL	760	21	0	157	0	SQ-P-2
SQ-P-3: Q-P,LWL	760	22	0	167	0	SQ-P-3

	1	2	3	4	5	6	
Critical Load Case	V	H_L	H_T	M_L	M_T	LC No.	
	t	t	t	t-m	t-m		
SR: Max. Axial load, V_{max}	932	36	11	254	456	SR-2	SR: Vmax
SR: Min. Axial load, V_{min}	488	13	0	290	0	SR-8	SR: Vmin
SR: Max. Long. Moment, M_{L-max}	888	54	11	441	326	SR-7	SR: ML-max
SR: Max. Trans. Moment, M_{T-max}	891	34	11	238	467	SR-3	SR: MT-max
SQ-P: Max. Long. Moment, M_{L-max}	760	23	0	169	0	SQ-P-1	SQ-P: ML-max

13.0 Calculation of Loads and Moments for Pier Cap Design

For LHS Span :-

Eff. deck width of outer girder	=	2.63	m
Eff. deck width of inner girder	=	2.75	m

For RHS Span :-

Eff. deck width of outer girder	=	2.63	m
Eff. deck width of inner girder	=	2.75	m

For LHS Span :-

DL reaction (Inner girder)	=	40.4	t
DL reaction (Outer girder)	=	39.6	t
SIDL reaction (Inner girder)	=	6.7	t
SIDL reaction (Outer side Outer girder)	=	23.5	t
SIDL reaction (Median side Outer girder)	=	23.5	t
DL+SIDL from left (Inner girder)	=	**47.1**	t
DL+SIDL from left (Outer side Outer girder	=	**63.2**	t
DL+SIDL from left (Median side Outer girde	=	**63.2**	t

For RHS Span :-

DL reaction (Inner girder)	=	40.4	t
DL reaction (Outer girder)	=	39.6	t
SIDL reaction (Inner girder)	=	6.7	t
SIDL reaction (Outer side Outer girder)	=	23.5	t
SIDL reaction (Median side Outer girder)	=	23.5	t
DL+SIDL from left (Inner girder)	=	**47.1**	t
DL+SIDL from left (Outer side Outer girder	=	**63.2**	t
DL+SIDL from left (Median side Outer girde	=	**63.2**	t

Live Load :-

Live Load	**Vertical Load (MT)**			**M-T**
	LHS Span	**RHS Span**	**Total**	**MT-m**
Load Case 1	71.0	72.0	143.1	363.0
Load Case 2	50.0	52.4	102.4	374.3
Load Case 3	5.4	126.9	132.2	310.6

Seismic coefficient Calculation (ESAM) :-

As per IRC:SP:114-2018, Seismic effects need not to be checked for structures up to 10.0 m span in all Seismic zones and for structures in Zone II & III, satisfying both limits stated below.
i) Span is less than 15.0 m.
ii) Total length of the Bridge/ Structure is less than 60.0 m

All other bridges shall be designed for seismic effects.

Seismic zone = V Fig:4.1 of IRC:SP:114 - 2018
Span length of Bridge = 30.0 m > 15.0 m
Total length of Bridge = 70.0 m > 60.0 m

Hence Bridge/ Structure is to be designed for Seismic effects.

Since Bridge/ Structure are in Zone- V, Vertical component of Seismic force is to be considered to act simultaneously with Horizontal component in design. (Refer Cl. 4.2.1 of IRC:SP:114-2018)

A_V = 0.667 x A_h
Horizontal seismic force, F_{eq} = A_h x (DL + b x LL) (Refer Cl. 5.2.1 of IRC:SP:114-2018)
where, F_{eq} = Seismic Force to be resisted
DL = Dead Load from superstructure & substructure
LL = Appropriate Live Load
b = Multiplying factor for Live Load as per Cl. 4.6 of IRC:SP:114-2018
= 0.2 (when seismic force is acting perpendicular to traffic)
= 0.0 (when seismic force is acting parallel to traffic)

Tables as per IRC:SP:114-2018, for Zone Factor, Importance Factor and Foundation Strata:

As per Table 4.2

Zone Number	Zone Factor (Z)
II	0.10
III	0.16
IV	0.24
V	0.36

As per Table 4.3

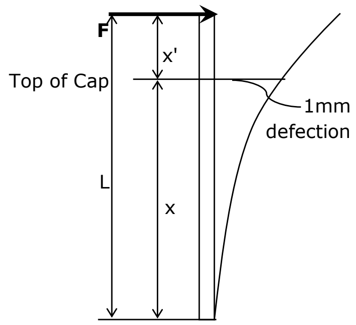
Seismic Class	Importance Factor (I)
Normal	1.00
Important	1.20
Critical	1.50

As per Table 5.1

Foundation Strara Soil Type	Notation
Rock/Hard Soil (I)	HS
Medium/ Stiff Soil (II)	MS
Soft Soil (III)	SS

Horizontal force, F = 6 EI / {x² (3L - x)}
where,
F = Force applied at cg of Superstructure or top of Bearing to produce 1mm deflection at Pier/Autment top for Seismic effect in Transverse or Longitudinal direction respectively
x = Distance of Pier top from support where 1mm deflection is required

x'_L = Distance between top of Bearing to Pier top
x'_T = Distance between cg of Superstructure to Pier top



Horizontal Seismic coefficient, A_h = (Z/2) x (Sa/g) / (R/I)
where,
Zone Factor, Z = 0.36
Importance Factor, I = 1.2
Resonce Reduction Factor, R
Longitudinal direction, R_L = 3.0
Transverse direction, R_T = 3.0
Elastic Seismic Acceleration Coefficient (ESAM), S_a/g
Foundation soil strata = SS
Elastic Seismic Acceleration Coefficient (ESAM) for 5% damping,

$$S_a/g = \begin{cases} 2.5 & \rightarrow 0 <= T < 0.67 \text{ s} \\ 1.67/T & \rightarrow 0.67 \text{ s} < T <= 4.0 \text{ s} \\ 0.42 & \rightarrow T > 4.0 \text{ s} \end{cases}$$

Type of Pier/ Abutment = RC
Grade of Concrete = M 35
Youngs Modulus, E = 3.2E+07 KN/m^2

x = 7.328 m
x'_L = 0.303 m
x'_T = 1.207 m

Minimum DL & LL has been considered for calculating fundamental time period on conservative side as the same coefficient is used for all max load, min load and max longitudinal moment cases.

Seismic Coefficient in Longitudinal Direction in Service Condition:

Size of Rectangular Pier/ Abutment = 1.00 x 6.00 m
Moment of Inertia I = 6 x 1^3 /12
= 0.5 m^4
Dia of Solid Pier D = 1.00 m
Moment of Inertia I = pi() x 1^4 /64
= 0.0 m^4
Outer Dia of Hollow Pier OD = 1.00 m
Inner Dia of Hollow Pier ID = 0.00 m
Moment of Inertia I = pi() x (1^4 - 0^4) /64
= 0.0 m^4
Cracked Moment of Inertia of Pier (75% of gross uncracked section) as per Cl. 219.5.1 of IRC:6-2017
I = 0.4 m^4
F = 87 KN/mm

Fundamental time period, T = 2 x (D / (1000 x F)) ^0.5
where D = $D_1 + D_2$
D_1 = DL + SIDL from the superstructure
= 5350 KN
D_2 = Live Load
= 0 KN
T = 0.50 sec
Elastic Seismic Acceleration Coefficient for 5% damping,
S_a/g = 2.50

HS	2.02
MS	2.50
SS	2.50

Horizontal Seismic Coefficient, A_{hL} = (0.36/ 2) x 2.5 / (3 / 1.2)
= 0.180
Vertical Seismic Coefficient, A_v = $A_h*2/3$ or 0
= 0.120

Seismic Coefficient in Transverse Direction in Service Condition:

Size of Rectangular Pier/ Abutment = 1.00 x 6.00 m
Moment of Inertia I = 1 x 6 ^ 3 /12
= 18.0 m^4
Dia of Solid Pier D = 1.00 m
Moment of Inertia I = pi() x 1 ^ 4 /64
= 0.0 m^4
Outer Dia of Hollow Pier OD = 1.00 m
Inner Dia of Hollow Pier ID = 0.00 m
Moment of Inertia I = pi() x (1 ^ 4 - 0 ^4) /64
= 0.0 m^4
Cracked Moment of Inertia of Pier (75% of gross uncracked section) as per Cl. 219.5.1 of IRC:6-2017
I = 13.5 m^4
F = 2667 KN/mm

Fundamental time period T = 2 x (D / (1000 x F)) ^0.5
where D = $D_1 + D_2$
D_1 = DL + SIDL from the superstructure
= 5350 KN
D_2 = Live Load
= 205 KN
T = 0.09 sec
Elastic Seismic Acceleration Coefficient for 5% damping,
S_a/g = 2.50

HS	2.50
MS	2.50
SS	2.50

Horizontal Seismic Coefficient, A_{hT} = (0.36/ 2) x 2.5 / (3 / 1.2)
= 0.18
Vertical Seismic Coefficient, A_v = $A_h*2/3$ or 0
= 0.12

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

14.0 Pile Geotechnical Capacity Checking :-

Grade of concrete of cast-insitu Pile, = M 35

Salient dimensions: (Pile & Pile cap)

Diameter of pile		=	1.20	m
Thickness of pile cap		=	1.80	m
Pile no's	long. direction (col.)	=	2	
	trans. direction (row)	=	4	
Pile spacing	long. direction (col.)	=	3.60	m
	trans. direction (row)	=	3.60	m
No. of pile		=	8	
Pile edge distance		=	0.15	m
Length of pile cap along long. direction		=	5.10	m
Length of pile cap along trans. direction		=	12.30	m

Salient dimensions: (Pier / Abutment)

Eff. length of pier along long. direction	=	1.00	m
Eff. length of pier along trans. direction	=	6.00	m
Ecc. of c/l of pier & pile cap (long. direct)	=	0.00	m
Ecc. of c/l of pier & pile cap (trans. direct)	=	0.00	m

Other Salient Design Data

Total length of pile (below pile cap)	=	30.000	m
Length of pile (upto depth of fixity)	=	14.054	m
Self weight of pile	=	848	KN
Self weight of pile (upto depth of fixity)	=	397	KN

Lateral load capacity of Piles as per IS : 2911 (I) / Sec 2 2010

Area of pile, $A = \pi * d^2/4$ = 1.131 m^2

Moment of inertia pile, $I = \pi * d^4/64$ = 0.102 m^4

Radius of gyration, $r = \sqrt{(I/A)}$ = 0.300

Concrete design strength of pile, f_{ck} = 35 MPa

Modulus of elasticity, $E_c = 22*((f_{ck}+10)/12.5)^{0.3}$ = 3.23E+07 KN/m^2 3.29E+06

The depth of fixity (in m) is determined from the codal provisions and graphs,

$$T = \sqrt[5]{\frac{EI}{\eta_b}}$$

,where η_b = Modulus of Subgrade Reaction for Granular Soil, in KN/m^3 (see Table 3) η_b

$$R = \sqrt[4]{\frac{EI}{KB}}$$

,where B = width of Pile Shaft (Dia. for Circular Piles), in m & K = 0.2 * k1 /B

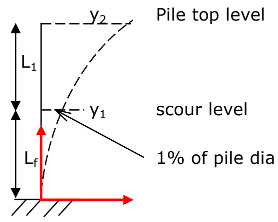
$$\text{Thus,} \quad R = \sqrt[4]{\frac{EI}{0.2 * k_1}}$$

,k_1 = Modulus of Subgrade Reaction for Cohesive Soil, in KN/m^3 (see Table 4) k_1

Type of top layer soil providing lateral resistance = Granular soil Silty sand

From Table we get,	η_b	=	5000	KN/m^3
	T	=	3.661	m
Free Length of pile,	L_1 / e	=	6.000	m
	L_1 / T	=	1.639	
From graph corresponding value of	L_f / T	=	2.20	
	L_f / z_f	=	8.054	m
	e + z_f	=	14.054	m
Allowable deflection at scour level,	y_1	=	0.012	m
Considering equation of parabola,	y	=	αL^2	
	α	=	0.000185	
	y_2	=	0.037	m

i.e Pile cap bottom Level - Scour Level



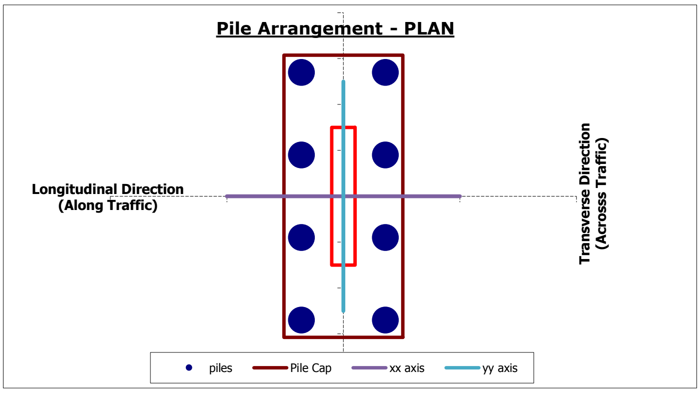
Cl. 79.3.5.1 of IRC:78

Design lateral load for fixed head pile, H $= \dfrac{12\,E\,I\,y}{(e + z_f)^3}$ = 519 kN

Design moment for fixed head pile, M_F = $m * H * (e + z_f)/2$

Moment reduction factor, m = 0.82

Pile No.	Long. xi (m)	Trans. yi (m)	n	xi^2	yi^2
P1	-1.80	-5.40	1	3.24	29.16
P2	1.80	-5.40	1	3.24	29.16
P3	-1.80	-1.80	1	3.24	3.24
P4	1.80	-1.80	1	3.24	3.24
P5	-1.80	1.80	1	3.24	3.24
P6	1.80	1.80	1	3.24	3.24
P7	-1.80	5.40	1	3.24	29.16
P8	1.80	5.40	1	3.24	29.16
			8	25.9	129.6



Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

Critical Load Combination		V	M_L	M_T	H_L	H_T	$H_{R/Pile}$	LC No.
		kN	kN-m	kN-m	kN	kN	kN	
NC: Max. Axial load,	Vmax	12493	4226	3630	453	0	57	LC-02
NC: Min. Axial load,	Vmin	6717	3136	28	156	29	20	LC-31
NC: Max. Long. Moment,	ML-max	10714	4940	3135	470	29	59	LC-17
NC: Max. Trans. Moment,	MT-max	10415	3924	3771	441	29	55	LC-16
NC: Max. Long. Force,	HL,Max	10822	4248	3658	476	29	60	LC-15
NC: Max. Trans. Force,	HT,Max	9391	23	28	23	29	5	LC-14
SS: Max. Axial load,	Vmax	12623	7882	5144	1011	573	145	LC-11
SS: Min. Axial load,	Vmin	6145	4625	1890	414	286	63	LC-34
SS: Max. Long. Moment,	ML-max	11708	16561	5024	2307	572	297	LC-07
SS: Max. Trans. Moment,	MT-max	11731	7882	15453	1011	1911	270	LC-08
SS: Max. Long. Force,	HL,Max	11731	16457	5144	2313	573	298	LC-05
SS: Max. Trans. Force,	HT,Max	11731	7882	15453	1011	1911	270	LC-08

Comb. Type	NC	NC	NC	NC	NC	NC	SS	SS	SS	SS	SS	SS
Pile No.	Vmax(kN)	Vmin(kN)	ML-max(kN)	MT-max(kN)	HL,Max(kN)	HT,Max(kN)	Vmax(kN)	Vmin(kN)	ML-max(kN)	MT-max(kN)	HL,Max(kN)	HT,Max(kN)
P1	1117	621	866	872	905	1171	816	368	104	275	109	275
P2	1704	1056	1552	1417	1495	1174	1911	1011	2404	1370	2395	1370
P3	1218	621	953	977	1007	1172	959	421	244	704	252	704
P4	1805	1057	1639	1522	1597	1175	2054	1063	2544	1799	2538	1799
P5	1319	622	1040	1082	1109	1173	1102	473	383	1134	395	1134
P6	1905	1058	1726	1627	1699	1176	2197	1116	2683	2228	2681	2228
P7	1419	623	1127	1187	1210	1174	1245	526	523	1563	538	1563
P8	2006	1059	1813	1732	1800	1177	2340	1168	2823	2658	2823	2658

Max. values	2006	1059	1813	1732	1800	1177	2340	1168	2823	2658	2823	2658
Min. values	1117	621	866	872	905	1171	816	368	104	275	109	275

Check for pile capacity

Max. values	2855	1907	2661	2580	2648	2025	3188	2016	3671	3506	3672	3506
Min. values	1965	1469	1714	1720	1754	2019	1664	1216	952	1123	957	1123
Vertical Capacity	2950	2950	2950	2950	2950	2950	3688	3688	3688	3688	3688	3688
Status	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

14.1 Pile ULS Capacity Check :-

Critical Load Combination		V	M_L	M_T	H_L	H_T	$H_{R/Pile}$	LC No.
		kN	kN-m	kN-m	kN	kN	kN	
Max. Axial load,	Vmax	17691	4794	7136	544	165	71	LC-02
Min. Axial load,	Vmin	8588	4725	3751	347	543	81	LC-40
Max. Long. Moment,	ML-max	15955	20613	7225	3013	858	392	LC-10
Max. Trans. Moment,	MT-max	15978	7618	22816	1063	2867	382	LC-11
Max. Long. Force,	HL,Max	15978	20481	7353	3016	860	392	LC-08
Max. Trans. Force,	HT,Max	15978	7618	22816	1063	2867	382	LC-11

Pile No.	Vmax(kN)	Vmin(kN)	ML-max(kN)	MT-max(kN)	HL,Max(kN)	HT,Max(kN)
P1	1581	589	262	517	269	517
P2	2247	1245	3125	1576	3113	1576
P3	1779	693	463	1151	473	1151
P4	2445	1349	3325	2209	3317	2209
P5	1978	797	663	1785	677	1785
P6	2643	1454	3526	2843	3522	2843
P7	2176	902	864	2419	881	2419
P8	2842	1558	3727	3477	3726	3477

Max. Vert. load (kN)	2842	1558	3727	3477	3726	3477
Min. Vert. load (kN)	1581	589	262	517	269	517
Hor. Load (kN)	71	81	392	382	392	382
Moment (kN-m)	235	266	1293	1262	1295	1262

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

Material data:

Charecteristics concrete comp. streng	f_{ck}	=	35	MPa
Modulus of Elasticity of Concrete,	E_{cm}	=	32308	MPa
Strength reduction factor,	α	=	0.67	
Concrete material factor,	γ_m	=	1.50	
Design concrete comp. strength,	$f_{cd} = \alpha f_{ck}/ \gamma_m$	=	15.63	MPa
Peak concrete strain at failure,	ε_{cu2}	=	0.0035	
Concrete strain at peak stress,	ε_{c2}		0.002	
Grade of Reinforcing steel,		=	Fe 500	
Charecteristic strength of r/f,	f_{yk}	=	500	MPa
Partial factor of safety for r/f,	γ_s	=	1.15	
Design tensile strength,	$f_{yd} = f_{yk}/ \gamma_s$	=	435	MPa
Design yield of strength of shear r/f,	f_{ywd}	=	400	MPa
Modulus of Elasticity of Steel,	E_s	=	200000	MPa
Yield strain of steel,	$\varepsilon_s = f_{yd} /E_s$	=	0.00217	
Max. strain of steel,	$\varepsilon_{s.max} = 0.002 + f_{yd} /E_s$	=	0.00417	
Creep Coefficient,	ϕ	=	1.00	

Geometric data:

Diameter of section,	D	=	1200	mm
Actual column length,	l_0	=	14.054	m
Effective length factor,		=	1.00	
Effective length of column	l_e	=	14.054	m
Radius of Gyration,	i	=	0.300	m
Slenderness ratio,	$\lambda = l_e/ i$	=	46.85	

R/f detail at column at potential plastic hinge location:

Diameter of long bars (L1)	1st layer	=	20	mm	
	2nd layer	=	20	mm	
Number of long bars	1st layer	=	16		
	2nd layer	=	16		
Area of steel provided,	A_{sc}	=	10053	mm^2	
Percentage of steel provided		=	0.89	%	**OK**
Diameter of spiral/ hoop,		=	16	mm	
Spacing spiral/ hoop,	S_L	=	95	mm	
Cover to spiral/ hoop,	c	=	75	mm	
Effective cover to long r/f,		=	101	mm	
Effective diameter of r/f,		=	998	mm	
Effective depth of the section,		=	1099	mm	

R/f detail at column at outside of potential plastic hinge location:

Diameter of long bars (L1)	1st layer	=	20	mm	
Number of long bars	1st layer	=	16		
Area of steel provided,			5027	mm^2	
Percentage of steel provided		=	0.44	%	**OK**

Calculation of demand points incorporating second order effects:

Parameters for limiting slenderness ratio:

Limiting Slenderness ration,	λ_{lim}	=	$20 * A * B * C / \sqrt{n}$		Eq.11.1
where,	A	=	$1 / (1+0.2 * \phi_{ef})$		of IRC:112
Creep Coefficient,	$\phi(\infty,0)$	=	1.00		
First order BM Quasi-Permanent load	M_{oEqp}	=	93 kN-m		
First order BM in design load in ULS	M_{oEd}	=	235 kN-m	2.51	
Effective creep ratio,	$\phi_{ef} = \phi(\infty,0) * M_{oEqp}/M_{oEd}$	=	0.40		
	A	=	0.93		
where,	B	=	$\sqrt{(1 + 2\omega)}$		
Mechanical r/f ratio,	$\omega = A_s * f_{yd} / A_c * f_{cd}$	=	0.25		
	B	=	1.22		
where,	C	=	$1.7 - r_m$		
Moment Ratio,	$r_m = M_{01} / M_{02}$	=	1		
	C	=	0.7		
Relative normal force,	n	=	$N_{Ed} / A_c * f_{cd}$		

Eccentricity due to slenderness:

Mominal second order moment,	M_2	=	$N_{Ed} * e_2$
Deflection,	e_2	=	$(1/ r) * l_e^2 / c$
Factor for constant cross section,	c	=	10
Curvature,	1/r	=	$K_r * K_\varphi * 1/r_0$
Factor depending on axial load,	K_r	=	$(n_u - n)/(n_u - n_{bal})$ <=1
	$n_u = 1 + \omega$	=	1.25
	n_{bal}	=	0.4
Factor for taking account of creep,	K_φ	=	$1+ \beta * \varphi_{ef}$
	$\beta = 0.35 + (f_{ck}/200) -(\lambda/150)$	=	0.21
	K_φ	=	1.08
	$1/ r_0 = f_{yd} / (E_s * 0.45d)$	=	0.0053 1/m

Pile Moment Capacity Check at ULS :-

* check for α value

Load case	N_{Ed} kN	M_{0Ed} kN-m	$n = N_{Ed}/ (A_c * f_{cd})$	λ_{lim}	K_r	e_2 (m)	M_{Ed} kN-m	$P_U / f_{ck} * D^2$	$M_U / f_{ck} * D^3$	M_U kN-m	M_{Ed} / M_u	
Vmax(kN)	2842	235	0.16	39.5	1.0	0.1	555	0.056	0.045	2704	0.21	**OK**
Vmin(kN)	1558	266	0.09	53.4	1.0	0.0	266	0.031	0.040	2404	0.11	**OK**
ML-max(kN)	3727	1293	0.21	34.5	1.0	0.1	1714	0.074	0.047	2868	0.60	**OK**
MT-max(kN)	3477	1262	0.20	35.7	1.0	0.1	1654	0.069	0.047	2826	0.59	**OK**
HL,Max(kN)	3726	1295	0.21	34.5	1.0	0.1	1715	0.074	0.047	2868	0.60	**OK**
HT,Max(kN)	3477	1262	0.20	35.7	1.0	0.1	1654	0.069	0.047	2826	0.59	**OK**
Vmax(kN)	71	235	0.00	250.0	1.0	0.0	235	0.001	0.032	1958	0.12	**OK**
Vmin(kN)	81	266	0.00	234.9	1.0	0.0	266	0.002	0.032	1962	0.14	**OK**
ML-max(kN)	392	1293	0.02	106.5	1.0	0.0	1293	0.008	0.034	2063	0.63	**OK**
MT-max(kN)	382	1262	0.02	107.8	1.0	0.0	1262	0.008	0.034	2060	0.61	**OK**
HL,Max(kN)	392	1295	0.02	106.5	1.0	0.0	1295	0.008	0.034	2063	0.63	**OK**
HT,Max(kN)	382	1262	0.02	107.8	1.0	0.0	1262	0.008	0.034	2060	0.61	**OK**

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
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		Design by	D.S.R	
		Check by	S.G	

Check for Ultimate Shear strength of Pile at ULS :-

Design Shear Force of the concrete section, $V_{NS} = V_{Ed}$ = 392 kN

Design of section without shear reinforcement:-

Factor for concrete cracked in shear, $\nu=0.6(1-f_{ck}/310)$ = 0.53
$K = Min.[1 + (200/ d)^{0.5} , 2]$ = 1.43
$\rho_1 = Min.[A_{sl} / b_w * d , 0.02]$ = 0.008
$\nu_{min} = 0.031 * K^{3/2} * f_{ck}^{1/2}$ = 0.312 MPa
Mean Comp. stress in concrete, $\sigma_{cp}=Min[N_{Ed}/A_c, 0.2f_{cd}]$ Mpa 0 MPa
Min. Design Shear resistance, $V_{Rd.c1}=(\nu_{min} + 0.15*\sigma_{cp})b_w*d$ = 412 kN
$V_{Rd.c2}=[0.12*K*(80*\rho1*f_{ck})^{0.33}+ 0.15*\sigma_{cp}]b_w*d$ = 620 kN
Design Shear resistance, $V_{Rdc} = Max.[V_{Rd.c1} \& V_{Rd.c2}]$ = 620 kN
Check for Shear R/f (SR) requirement, **SR NRQ**

*RQ - Required
*NRQ - Not Required

Design of section with shear reinforcement:-

Dia. of Shear r/f, stirrups/ links ϕ = 16 mm
Number of stirrups/ links, n = 1
Spacing of stirrups/ links s = 95 mm
Area of Shear r/f provided, A_{sw} = 201 mm^2
A_{sw} / s = 2.1 mm^2/mm
Minimum area of Shear r/f required, Min. A_{sw} / s = 1.0 mm^2/mm
Status, Area of Shear r/f, **OK**
Lever arm factor, = 0.9
Coeff. depends on stress in compression cord, α_{cw} = 1.0
Factor for concrete cracked in shear, $\nu_1 = \nu$ = 0.53
Shear capacity of concrete strut corresponding, θ = 45 degree
$V_{Rd.max}=\alpha_{cw}*b_w*z*\nu1*f_{cd}/(\cot\theta+\tan\theta)$ = 4938 kN
Status, Check for Redesign of section, **OK**
Angle of concrete strut due shear, θ = 2.3 degree
Shear capacity of strut, $V_{Rd}=\alpha_{cw}*b_w*z*\nu1*f_{cd}/(\cot\theta+\tan\theta)$ 392 kN
0

Angle of concrete strut considered, θ = 21.8 degree
Shear resistance provided by r/f, $V_{Rd.s} = A_{sw}/s*z*f_{ywd}*\cot\theta$ = 2094 kN
Status, Check for shear capacity of section, **OK**
5.3

14.3 Pile Stress & Crack Width Check at SLS :-

Critical Load Combination		V	M_L	M_T	H_L	H_T	$H_{R/Pile}$	LC No.
		kN	kN-m	kN-m	kN	kN	kN	
SR: Max. Axial load,	Vmax	12869	3196	4757	363	110	47	SR-02
SR:Min. Axial load,	Vmin	6763	3168	28	156	29	20	SR-16
SR:Max. Long. Moment,	ML-max	10760	5400	3486	561	138	72	SR-14
SR:Max. Trans. Moment,	MT-max	10792	3021	4899	365	138	49	SR-10
SQ-P: Max. Axial load,	Vmax	11154	2101	0	226	0	28	SQ-P-1
SQ-P:Min. Axial load,	Vmin	9483	2101	0	226	0	28	SQ-P-4
SQ-P:Max. Long. Moment,	ML-max	11154	2101	0	226	0	28	SQ-P-1
SQ-P:Max. Trans. Moment,	MT-max	8433	3146	0	134	0	17	SR-15

Comb. Type	SR	SR	SR	SR	SQ-P	SQ-P	SQ-P	SQ-P
Pile No.	Vmax	Vmin	ML-max	MT-max	Vmax	Vmin	ML-max	MT-max
P1	1189	624	825	935	1248	1040	1248	836
P2	1632	1064	1575	1355	1540	1331	1540	1273
P3	1321	625	922	1071	1248	1040	1248	836
P4	1765	1065	1672	1491	1540	1331	1540	1273
P5	1453	626	1018	1207	1248	1040	1248	836
P6	1897	1066	1768	1627	1540	1331	1540	1273
P7	1585	626	1115	1343	1248	1040	1248	836
P8	2029	1067	1865	1763	1540	1331	1540	1273

Max. Vert. load (kN)	2029	1067	1865	1763	1540	1331	1540	1273
Min. Vert. load (kN)	1189	624	825	935	1248	1040	1248	836
Hor. Load (kN)	47	20	72	49	28	28	28	17
Moment (kN-m)	157	66	239	161	93	93	93	55

Actual column Length, L_0 = 8.054 m
Effective length factor, = 1.00
Effective length of column L_e = 8.054 m
Radius of Gyration, r = 0.300 m
L_e / r = 26.8
Short column

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		Design by	D.S.R	
		Check by	S.G	

Confinment Reinforcement check:

Ductile detailing shall be carried out for bridges located in zones III, IV and V of seismic zone map of IRC:6-2017.

For Circular Column

As per Clause No. 7.4 of IS:13920-1993

Area of trans r/f,	A_{sh} = $0.09SD_k f_{ck}/f_y[A_q/A_k-1.0)$	=	201	mm^2
Dia. of column ,	D	=	1200	mm
Dia. of core from outside of spiral/ ho	D_k	=	1050	mm
Gross area of column cross section,	A_q	=	1130973	mm^2
Area of the concrete core,	A_k	=	865901	mm^2
Spacing of spiral/ hoop,	S	=	99	mm

As per Clause No. 17.2.1 of IRC:112-2011

Confining r/f ratio,	ω_{wd}	=	$\rho_w f_{yd} / f_{cd}$		
Where, Volumetric ratio,	ρ_w	=	$4 A_{sp}/(D_{sp} * S_L)$		
Area of spiral/ hoop bar,	A_{sp}	=	201 mm^2		
Dia. of spiral or hoop bar,	D_{sp}	=	1050 mm		
For circular section,	$\omega_{wd.c}$	≥	$\max(1.4\ \omega_{.\omega req}\ ;\ 0.18)$		Eq. 17.7
Normalised axial force,	$n_k = N_{Ed}/(A_c * f_{ck}) > 0.08$	=	0.08	0.08	of IRC:112
R/f ratio of longitudinal r/f,	$p_L = A_s / A_c$	=	0.009		
	$\omega_{w.req} = 0.37 * A_c/A_{cc} * n_k + 0.13 * f_{yd}/f_{cd} * (p_L - 0.01)$	=	0.035		
	$\omega_{wd.c}$	=	0.18		
Confining r/f ratio,	$4 A_{sp}/(D_{sp} * S_L) * f_{yd}/f_{cd}$	=	0.18		
Spacing of spiral/ hoop,	S_L	=	118 mm		

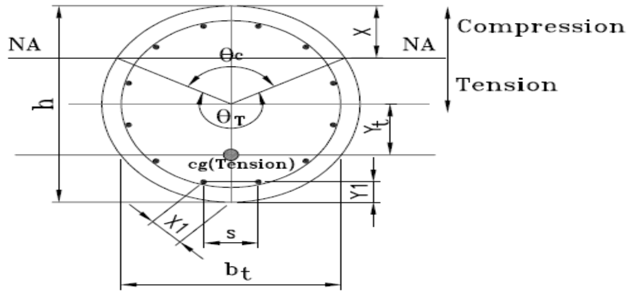
Provide, 16 ϕ @ 95 mm upto 1200 mm from pile cap bottom

Flexure design of Pile at SLS :-

Material Data

Concrete strength of cast-insitu Pile,	f_{ck}	=	35	MPa
Modulus of Elasticity of Concrete,	E_{cm}	=	32308	MPa
Charecteristic strength of r/f,	f_{yk}	=	500	MPa
Modulus of Elasticity of Steel,	E_s		200000	Mpa
Mean tensile strength of concrete,	f_{ctm}	=	2.77	Mpa
Per. concrete compressive stress,	$0.48*f_{ck}$	=	16.80	MPa
Per. tensile stress in steel,	$0.8*f_{yk}$	=	400	MPa
Creep Coefficient,	ϕ	=	1.00	
Eq. modulus of elasticity,	$E_{c,eff} = E_{cm}/(1+\phi)$	=	16154	MPa
Modular Ratio,	m		12.4	

at SLS Rare load combination



Geometric data

Diameter of column,	d or h	=	1200	mm
Cover to spiral/ hoop,	d''	=	75	mm
Effective cover,	d'	=	101	mm
Diameter (f) of long bars,	1st layer	=	20	mm
	2nd layer	=	20	mm
Number of long bars	1st layer	=	16	Nos.
	2nd layer	=	16	Nos.
Area of steel provided,	A_{st}	=	10053	mm^2
Percentage of steel provided		=	0.9	%
Cross-sectional area of uncracked section, A		=	1245385	mm^2
Moment of inertia of uncracked section, I		=	1.303E+11	mm^4

Stress check at SLS :-

* check formula

Load	V	M	Compressive stress	Bending stress	Check for section	N.A depth (n)	Angle at centre of circle	Dist. bet. centre of cracked section to	Cross-sectional area of cracked	MoI of cracked section, I_{NA}	Effective area of cracked section
case	kN	kN-m	Mpa	Mpa		mm	radian	mm	mm^2	mm^4	mm^2
SR:Vmax	2029	157	2.3	0.9	Uncracked	1824	6.3	0.0	1130973	1.018E+11	1245385
SR:Vmin	1067	66	1.2	0.6	Uncracked	2133	6.3	0.0	1130973	1.018E+11	1245385
SR:ML-max	1865	239	2.6	0.4	Uncracked	1338	6.3	0.0	1130973	1.018E+11	1245385
SR:MT-max	1763	161	2.2	0.7	Uncracked	1632	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:Vmax	1540	93	1.7	0.8	Uncracked	2155	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:Vmin	1331	93	1.5	0.6	Uncracked	1945	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:ML-ma	1540	93	1.7	0.8	Uncracked	2155	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-ma	1273	55	1.3	0.8	Uncracked	2776	6.3	0.0	1130973	1.018E+11	1245385
SR:Vmax	1189	157	1.7	0.2	Uncracked	1317	6.3	0.0	1130973	1.018E+11	1245385
SR:Vmin	624	66	0.8	0.2	Uncracked	1497	6.3	0.0	1130973	1.018E+11	1245385
SR:ML-max	825	239	1.8	-0.4	Uncracked	839	4.0	131.8	844096	4.141E+10	958507
SR:MT-max	935	161	1.5	0.0	Uncracked	1146	5.4	9.2	1113015	9.59E+10	1227427
SQ-P:Vmax	1248	93	1.4	0.6	Uncracked	1861	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:Vmin	1040	93	1.3	0.4	Uncracked	1650	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:ML-ma	1248	93	1.4	0.6	Uncracked	1861	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-ma	836	55	0.9	0.4	Uncracked	2029	6.3	0.0	1130973	1.018E+11	1245385

* check formula

* check formula

Load	Ecentricity	Net eccentricity	Eff. MoI of cracked section	Calculated N.A	y_{assume}	$y_{cal} - y_{assum}$	Stress at N.A	Compressive stress in con.	Tensile stress in steel
case	mm	mm	mm^4	mm	mm	mm	MPa	σ_c (MPa)	σ_s (MPa)
SR:Vmax	0.0	77.1	1.176E+11	1224	1224	0	0.0	2.4	11.9
SR:Vmin	0.0	61.6	1.176E+11	1533	1533	0	0.0	1.2	7.2
SR:ML-max	0.0	128.0	1.176E+11	738	738	0	0.0	2.7	6.0
SR:MT-max	0.0	91.4	1.176E+11	1032	1032	0	0.0	2.2	9.1
SQ-P:Vmax	0.0	60.7	1.176E+11	1555	1555	0	0.0	1.7	10.4
SQ-P:Vmin	0.0	70.2	1.176E+11	1345	1345	0	0.0	1.5	8.3
SQ-P:ML-ma	0.0	60.7	1.176E+11	1555	1555	0	0.0	1.7	10.4
SQ-P:MT-ma	0.0	43.4	1.176E+11	2176	2176	0	0.0	1.3	9.8
SR:Vmax	0.0	131.7	1.176E+11	717	717	0	0.0	1.8	3.6
SR:Vmin	0.0	105.2	1.176E+11	897	897	0	0.0	0.8	2.8
SR:ML-max	116.0	173.3	5.892E+10	355	355	0	0.0	2.0	-7.8
SR:MT-max	8.3	164.1	1.117E+11	555	555	0	0.0	1.6	0.8

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		Check by	S.G	

SQ-P:Vmax	0.0	74.9	1.176E+11	1261	1261	0	0.0	1.5	7.5
SQ-P:Vmin	0.0	89.9	1.176E+11	1050	1050	0	0.0	1.3	5.4
SQ-P:ML-ma	0.0	74.9	1.176E+11	1261	1261	0	0.0	1.5	7.5
SQ-P:MT-ma	0.0	66.1	1.176E+11	1429	1429	0	0.0	1.0	5.4

SLS Rare load combination

Max. Concrete Compressive Stress = 2.7 MPa **OK**

Max. Tensile Stress in Steel = -7.8 MPa **OK**

SLS Q-P load combination

Max. Concrete Compressive Stress = 1.7 MPa **OK**

Max. Tensile Stress in Steel = 0 MPa **OK**

Crack width Check at SLS :- Cl 12.3 of IRC:112

Calculation of Crack Width

Permissible Crack width, = 0.30 mm for Moderate Exposure

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

where, $\varepsilon_{sm} - \varepsilon_{cm}$ = $Max\{[\sigma_{sc} - k_t * f_{ct,eff}(1 + \alpha_e * \rho_{peff})/\rho_{eff}]/ E_s \ \& \ (0.6*\sigma_{sc}/ E_s)\}$

$\alpha_e = E_s / E_{cm}'$ = 6.19

Mean tensile strength of concrete, f_{ctm} = 2.77 MPa

Bar diameter, ϕ_{eq} = 20 mm

Calculated spacing b/w bars, $5 *(c + \phi_{eq}/2)$ = 425 mm **OK**

Effective depth in tension, $h_{c,eff}$ = Min.[2.5*(h-d), (h-x)/3, h/2]

k_1 = 0.80

k_2 = 0.50

c = 75 mm

Max. Crack spacing, $S_{r.max}$ = $3.4c + 0.425 * k_1 * k_2 * \phi/\rho_{p,eff}$

Coefficient depends on duration of loa k_t = 0.50

Description/ Load case >		SQ-P:Vmax	SQ-P:Vmin	SQ-P:ML-max	SQ-P:MT-max	SQ-P:Vmax	SQ-P:Vmin	SQ-P:ML-max	SQ-P:MT-max	
Diameter of section, d	=	1200	1200	1200	1200	1200	1200	1200	1200	mm
Depth of Neutral axis, x	=	2155	1945	2155	2776	1861	1650	1861	2029	mm
Tensile stress in R/f, σ_{sc}	=	0	0	0	0	0	0	0	0	MPa
Area of steel, A_s	=	10053	10053	10053	10053	10053	10053	10053	10053	mm^2
$h_{c,eff}$	=	253	248	253	253	220	150	220	253	mm
$A_{c,eff}$	=	173141	168920	173141	173141	142363	81546	142363	173141	mm^2
$\rho_{peff} = A_s / A_{c,eff}$	=	0.06	0.06	0.06	0.06	0.07	0.12	0.07	0.06	
$\varepsilon_{sm} - \varepsilon_{cm}$	=	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
$S_{r.max}$	=	314	312	314	314	303	283	303	314	
W_k	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	mm
Status	=	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	

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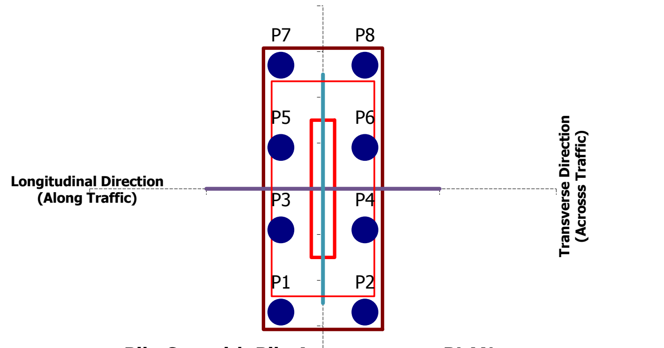
		Check by	S.G	

15.0 Design of Pile Cap by Bending theory :-

Grade of concrete of Pile cap,		=	M 35	
Salient dimensions: (Pile cap)				
Diameter of pile		=	1.20	m
Thickness of pile cap		=	1.80	m
Pile no's long. direction (col.)		=	2	
trans. direction (row)		=	4	
Pile spacing long. direction (col.)		=	3.60	m
trans. direction (row)		=	3.60	m
No. of pile		=	8	
Pile edge distance		=	0.15	m
Length of pile cap along long. direction		=	5.10	m
Length of pile cap along trans. direction		=	12.30	m
Salient dimensions: (Pier / Abutment)				
Eff. length of pier along long. direction		=	1.00	m
Eff. length of pier along trans. direction		=	6.00	m
Ecc. of c/l of pier & pile cap (long. direct)		=	0.00	m
Ecc. of c/l of pier & pile cap (trans. direct)		=	0.00	m
Material property				
Density of PSC /RCC concrete		=	25	KN/m^3
Dry density of soil/ earth	γ_d	=	20	KN/m^3
Density of water	γ_w	=	10	KN/m^3
Thickness of earth over Pile cap		=	0.65	m
Reinforcement details				
Clear cover to reinforcement		=	75	mm
Dia. of longitudinal r/f at bottom		=	25	mm
Dia. of tranverse r/f at bottom		=	25	mm
Dia. of shear r/f		=	12	mm
Eff. depth at longitudinal direction		=	1.701	m
Eff. depth at transverse direction		=	1.676	m



Pile Cap with Pile Arrangement - PLAN

Bending moment (BM) & shear force (SF) due to s/w of Pile cap & Earth fill :

Longitudinal direction:-

Length from face of shaft for Bending moment	=	2.050	m
Length from $d_{eff.}$ from shaft for Shear force	=	0.350	m
Downward Bending moment (incl. 15% submerger	=	1404	kN-m
Downward Shear force (incl. 15% submergence)	=	234	kN

Transverse direction:-

Length from face of shaft for Bending moment	=	3.150	m
Length from $d_{eff.}$ from shaft for Shear force	=	1.475	m
Downward Bending moment (incl. 15% submerger	=	1375	kN-m
Downward Shear force (incl. 15% submergence)	=	409	kN

Load Case	Factor	Long. direction		Trans. direction	
		BM_L (kN-m)	SF_L (kN)	BM_T (kN-m)	SF_T (kN)
ULS case	1.35	1895	315	1856	552
SLS case	1.0	1404	234	1375	409

Lever arm of piles from face,

along Longitudinal direction	=	1.300	m, Pile no.	P1	P3	P5	P7		
		0.000	m, Pile no.						
along Transverse direction	=	2.400	m, Pile no.	P1	P2				
	=	-1.200	m, Pile no.	P3	P4				

For opposite face check:-

Lever arm of piles from face,

along Longitudinal direction	=	1.300	m, Pile no.	P2	P4	P6	P8		
		0.000	m, Pile no.						
along Transverse direction	=	2.400	m, Pile no.	P7	P8				
	=	-1.200	m, Pile no.	P5	P6				

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		Check by	S.G	

15.1 Moment & shear force on Pile cap at ULS :-

Pile No.	Vmax(kN)	Vmin(kN)	ML-max(kN)	MT-max(kN)	HL,Max(kN)	HT,Max(kN)
P1	1581	589	262	517	269	517
P2	2247	1245	3125	1576	3113	1576
P3	1779	693	463	1151	473	1151
P4	2445	1349	3325	2209	3317	2209
P5	1978	797	663	1785	677	1785
P6	2643	1454	3526	2843	3522	2843
P7	2176	902	864	2419	881	2419
P8	2842	1558	3727	3477	3726	3477

BM_L (kN-m)	7872	1980	1031	5739	1094	5739
SF_L (kN)	2557	824	545	1929	564	1929
	7198					
BM^*_L (kN-m)	11335					
SF^*_L (kN)	3575					

BM_T (kN-m)	2262	95	1727	-865	1712	-865
SF_T (kN)	2190	762	1874	948	1870	948

BM^*_T (kN-m	4641					
SF^*_T (kN)	3042					

Check for Ultimate Strength Capacity of Pile cap in Flexure at ULS :-

Material data:

Charecteristics concrete comp. streng	f_{ck}	=	35	MPa
Modulus of Elasticity of Concrete,	E_{cm}	=	32308	MPa
Strength reduction factor,	α	=	0.67	
Concrete material factor,	γ_m	=	1.50	
Design concrete comp. strength,	$f_{cd} = \alpha f_{ck}/\gamma_m$	=	15.63	MPa
Mean tensile strength of concrete,	f_{ctm}	=	2.77	MPa
Peak concrete strain at failure,	ε_{cu2}	=	0.0035	
Concrete strain at peak stress,	ε_{c2}		0.002	
Grade of Reinforcing steel,		=	Fe 500	
Charecteristic strength of r/f,	f_{yk}	=	500	MPa
Partial factor of safety for r/f,	γ_s	=	1.15	
Design tensile strength,	$f_{yd} = f_{yk}/\gamma_s$	=	435	MPa
Design yield of strength of shear r/f,	f_{ywd}	=	400	MPa
Modulus of Elasticity of Steel,	E_s	=	200000	
Yield strain of steel,	$\varepsilon_s = f_{yd}/E_s$	=	0.00217	
Max. strain of steel,	$\varepsilon_{s.max} = 0.002 + f_{yd}/E_s$	=	0.00417	
Balanced/ limiting depth of N.A,	$x_{u.max}/d$		0.456	
Creep Coefficient,	ϕ	=	1.00	

			Long. Direction		**Trans. Direction**	
Design bending moment of Pile cap,	M_{Ed}		11335	kN-m	4641	kN-m
Width of Pile cap,	b	=	5100	mm	12300	mm
Effective depth of Pile cap,	d_{eff}	=	1701	mm	1676	mm
Ultimate design constant,	$R = M_{Ed}/bd^2$	=	0.77	N/mm^2	0.13	N/mm^2
% of steel required,	$p_{t.req} = f_{ck}/(2f_y)*(1-(1-(4.598*R/f_{ck})^2)$	=	0.0018		0.0003	
Min. % of steel required,	p_t = max.[$0.26*f_{ctm}/f_{yk}$, 0.0013]	=	0.0014		0.0014	
Area of steel required for bending,	$A_{st.req}$	=	15732	mm^2	29698	mm^2
R/f provided, Dia.	1st layer	=	25	mm	25	mm
Spacing		=	125	mm c/c	125	mm c/c
Dia.	2nd layer	=	0	mm	0	mm
Spacing		=	0	mm c/c	0	mm c/c
Area of steel provided for bending,	$A_{st.pro}$	=	19246	mm^2	47521	mm^2
Percentage of steel provided,	$p_{t.pro}$	=	0.0022		0.0023	
Status of reinforcement steel,			**OK**		**OK**	
Balanced/ limiting depth of N.A,	$x_{u.max} = 0.456*d$	=	776	mm	764	mm
Actual depth of N.A,	$x=(0.87*f_{yk}*A_{st})/(0.362*f_{ck}*b)$	=	130	mm	133	mm
Actual depth of N.A,	x	=	130	mm	133	mm
Status of section i.e Under R/f (UR) or Over R/f (OR),			**UR,OK**		**UR,OK**	
Strain in r/f steel in tension,	ε_s		0.0425		0.0407	
Stress in r/f steel in tension,	f_s	=	434.8	MPa	434.8	
Total tensile force in steel,	$T=A_{st}*f_s$	=	8368	kN	20661	kN
Total compressive force in concrete,	$C= 0.362f_{ck}*x*b$	=	8368	kN	20661	kN
Difference of force,	C - T	=	0		0	
Lever arm of compressive force in concrete,	z= (d-0.416*x)	=	1647	mm	1620	mm
Moment of resistence,	$M_{Rd}=C*z$	=	13779	kN-m	33478	kN-m
Factor of Safety,	M_{Rd}/M_{Ed}	=	1.2		7.2	
Status Ultimate strength of Pile cap in Flexure,			**OK**		**OK**	

Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

Check for Ultimate Shear strength of Pile cap at ULS :-

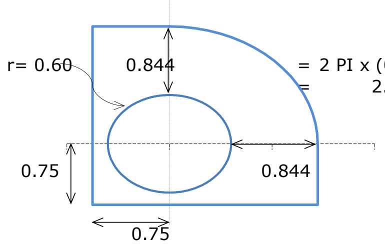
			Long. Direction	**Trans. Direction**
Design Shear Force of the concrete section,	$V_{NS} = V_{Ed}$	=	3575 kN	3042 kN
Design of section without shear reinforcement:-				
Factor for concrete cracked in shear,	ν=0.6(1-f_{ck}/310)	=	0.53	0.53
	Max. V_{Ed} = 0.5 b_w*d*ν*f_{cd}	=	36082 kN	85742 kN
	Status,		**OK**	**OK**
	K = Min.[1 + $(200/d)^{0.5}$, 2]	=	1.34	1.35
	ρ_1 = Min.[A_{sl} / b_w*d , 0.02]	=	0.002	0.002
	ν_{min} = 0.031 * $K^{3/2}$ * $f_{ck}^{1/2}$	=	0.285 MPa	0.286 MPa
Mean Comp. stress in concrete,	σ_{cp}=Min[N_{Ed}/A_c, 0.2f_{cd}] Mpa		0 MPa	0 MPa
Min. Design Shear resistance,	$V_{Rd.c1}$=(ν_{min} + 0.15*σ_{cp})b_w*d	=	2475 kN	5899 kN
	$V_{Rd.c2}$=[0.12*K*(80*ρ1*f_{ck} $)^{0.33}$+ 0.15*σ_{cp}]b_w*d	=	2554 kN	6158 kN
Design Shear resistance,	V_{Rdc} =Max.[$V_{Rd.c1}$ & $V_{Rd.c2}$]	=	2554 kN	6158 kN
Check for Shear R/f (SR) requirement,			**SR RQ**	**SR NRQ**
Design of section with shear reinforcement:-				
Dia. of Shear r/f, stirrups/ links	ϕ	=	12 mm	12 mm
Number of stirrups/ links,	n	=	12	16
Spacing of stirrups/ links	s	=	150 mm	125 mm
Area of Shear r/f provided,	A_{sw}	=	1357 mm^2	1810 mm^2
	A_{sw} / s	=	9.0 mm^2/mm	14.5 mm^2/mm
Minimum area of Shear r/f required,	Min. A_{sw} / s	=	5.4 mm^2/mm	13.1 mm^2/mm
Status, Area of Shear r/f,			**OK**	**OK**
Lever arm factor,		=	0.9	0.9
Coeff. depends on stress in compression cord,	α_{cw}	=	1.0	1.0
Factor for concrete cracked in shear,	ν_1 = ν	=	0.53	0.53
Shear capacity of concrete strut corresponding,	θ	=	45 degree	45 degree
	$V_{Rd.max}$=α_{cw}*b_w*z*ν1*f_{cd}/(cotθ+tanθ)	=	32474 kN	77168 kN
Status, Check for Redesign of section,			**OK**	**OK**
Angle of concrete strut due shear,	θ	=	3.2 degree	1.1 degree
Shear capacity of strut,	V_{Rd}=α_{cw}*b_w*z*ν1*f_{cd}/(cotθ+tanθ)		3575 kN	3042 kN
			0	0
Angle of concrete strut considered,	θ	=	21.8 degree	21.8 degree
Shear resistance provided by r/f,	$V_{Rd.s}$ = A_{sw}/s*z*f_{ywd}*cotθ	=	13848 kN	21831 kN
Status, Check for shear capacity of section,			**OK**	**OK**
			3.9	7.2

*RQ - Required
*NRQ - Not Required

Check for Punching Shear stress of Pile cap for Edge pile at ULS:-

Max. Pile Reaction at ULS,	V_{Ed}	=	3727 kN
Max. moment on Pile at ULS,	M_{Ed}	=	1295 kN-m
Average effective depth of pila cap,	d	=	1688 mm
Check at Pile perimeter,			
Factored Shear due to s/w of Cap (incl. 15% subm	ΔV_{Ed}	=	89 kN
Net applied Shear force,	$V_{Ed.red}$ = V_{Ed} - Δ_{VEd}	=	3637 kN
Eccentricity,	e = $M_{Ed.red}$ / $V_{Ed.red}$	=	356 mm
Perimeter at face of Pile,	u_o	=	2142 mm
Factor depends on moment & axial load,	β	=	1.56
Punching Shear stress,	v_{Ed} = β $V_{Ed.red}$ / (u_o*d)	=	1.57 MPa
Max. Punching shear resistance,	$v_{Rd.max}$=1/2*ν*f_{cd}	=	4.16 MPa
			OK
Check at a distance 'd/2' from Pile perimeter,	d/2	=	844 mm
Factored Shear due to s/w of Cap (incl. 15% subm	ΔV_{Ed}	=	249 kN
Net applied Shear force,	$V_{Ed.red}$ = V_{Ed} - Δ_{VEd}	=	3478 kN
Control perimeter,	u_i	=	2268 mm
Punching Shear stress,	v_{Ed} = $V_{Ed.red}$ / (u_i*d)	=	0.91 MPa
	K = Min.[1 + $(200/d)^{0.5}$, 2]	=	1.34
	ν_{min} = 0.031 * $K^{3/2}$ * $f_{ck}^{1/2}$	=	0.286 MPa
Mean value of percentage of steel provided,	p_l	=	0.003
Punching shear resistance at control perimeter,			
	v_{Rd} = Min.[0.12K(80p1$f_{ck}$$)^{0.33}$*2d/a, v_{min}*2d/a]	=	1.14 MPa
			OK

where, ν=0.6(1-f_{ck}/310)



= 2 PI x (0.6+0.844)/4
= 2.268 m

15.2 Pile cap SLS Stress & Crack Width Check :-

Comb. Type	SR	SR	SR	SR	SQ-P	SQ-P	SQ-P	SQ-P
Pile No.	Vmax	Vmin	ML-max	MT-max	Vmax	Vmin	ML-max	MT-max
P1	1189	624	825	935	1248	1040	1248	836
P2	1632	1064	1575	1355	1540	1331	1540	1273
P3	1321	625	922	1071	1248	1040	1248	836
P4	1765	1065	1672	1491	1540	1331	1540	1273
P5	1453	626	1018	1207	1248	1040	1248	836
P6	1897	1066	1768	1627	1540	1331	1540	1273
P7	1585	626	1115	1343	1248	1040	1248	836
P8	2029	1067	1865	1763	1540	1331	1540	1273

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		Design by	D.S.R	
		Check by	S.G	

BM_L (kN-m)	5807	1848	3640	4520	5088	4001	5088	2942
BM^*_L (kN-m)								
BM_T (kN-m)	1693	649	1272	1047	1972	1470	1972	1155
BM^*_T (kN-m)								

Flexure design of Pile Cap at SLS:-

Material Data

Concrete strength of cast-insitu Pile,	f_{ck}	=	35 MPa	
Modulus of Elasticity of Concrete,	E_{cm}	=	32308 MPa	
Charecteristic strength of r/f,	f_{yk}	=	500 MPa	
Modulus of Elasticity of Steel,	E_s		200000 Mpa	
Mean tensile strength of concrete,	f_{ctm}	=	2.77 Mpa	
Per. concrete compressive stress,	$0.48*f_{ck}$	=	16.80 MPa	at SLS Rare load combination
Per. tensile stress in steel,	$0.8*f_{yk}$	=	400 MPa	
Creep Coefficient,	ϕ	=	1.00	
Eq. modulus of elasticity,	$E_{c,eff} = E_{cm}/(1+\phi)$	=	16154 MPa	
Modular Ratio,	m		12.4	

Stress check at SLS:-

			Long. Direction			Trans. Direction			
			SLS Rare	SLS Q-P		SLS Rare	SLS Q-P		
Design bending moment of Pile cap,	M		5807	5088	kN-m	1693	1972	kN-m	
Width of Pile cap,	b	=	5100	5100	mm	12300	12300	mm	
Overall depth of Pile cap,	D	=	1800	1800	mm	1800	1800	mm	
Effective depth of Pile cap,	d_{eff}	=	1701	1701	mm	1676	1676	mm	
Area of steel provided,	$A_{st, pro}$	=	19246	19246	mm^2	47521	47521	mm^2	
Actual depth of N.A,	x	=	355	355	mm	355	355	mm	
Comp. stress of concrete,	σ_c	=	4.1	3.6	MPa	0.5	0.6	MPa	
Tensile stress in steel,	σ_s	=	190.7	167.1	MPa	22.9	26.6	MPa	
Total tensile force in steel,	$T=A_{st}*\sigma_s$	=	3670	3215	kN	1088	1266	kN	from Stress diagram
Total comp. force in concrete,	$C= 0.5*\sigma_c*x*b$	=	3670	3215	kN	1088	1266	kN	from Stress diagram
Difference of force,	C - T	=	0	0		0	0		
c.g of comp. force from extreme face,	x_1	=	118	118		118	118		from Stress diagram
Moment due to comp. force,	$M_s = T*(d - x_1)$	=	5807	5088	kN-m	1693	1972	kN-m	from Stress diagram
Difference of moment,	$M - M_s$	=	0	0		0	0		
Status			**OK**	**OK**		**OK**	**OK**		

Moment of Inertia,	I	=	2.09E+12	2.09E+12	mm^4	4.821E+12	4.821E+12	mm^4
	Y/2	=	900	900		900	900	mm
Stress in Concrete,	σ_c	=	2.50	2.19		0.32	0.37	MPa
Section Cracked or Uncracked			Uncracked	Uncracked		Uncracked	Uncracked	
Depth of N.A,	x	=	355	355		355	355	mm
Transformed Moment of Inertia,	I_{NA}	=	5.074E+11	5.074E+11		1.209E+12	1.209E+12	mm^4
Compressive stress in concrete,	σ_c	=	4.1	3.6		0.5	0.6	MPa
Tensile stress in Steel	σ_s	=	190.7	167.1		22.9	26.6	MPa
Status Check,			**OK**	**OK**		**OK**	**OK**	

Crack width Check at SLS :-

Calculation of Crack Width

Permissible Crack width,		=	0.30 mm	for	Moderate	Exposure
	$\alpha_e = E_s / E_{cm}'$	=	6.19			
Mean tensile strength of concrete,	f_{ctm}	=	2.77 MPa			
	k_1	=	0.80			
	k_2	=	0.50			
	c	=	75 mm			
Coefficient depends on duration of loa	k_t	=	0.50			

			Long. Direction		Trans. Direction	
			SLS Q-P		SLS Q-P	
Equivalent bar diameter,	ϕ_{eq}	=	25	mm	25	mm
Calculated spacing b/w bars,	$5*(c + \phi_{eq}/2)$	=	437.5	mm	438	mm
Status			**OK**		**OK**	
Overall depth of Pile cap,	D	=	1800	mm	1800	mm
Depth of Neutral axis,	x	=	355	mm	355	mm
Tensile stress in R/f,	σ_{sc}	=	167.1	MPa	26.6	MPa
Area of steel,	A_s	=	19246	mm^2	47521	mm^2
Eff. depth in tension,	$h_{c.eff} = Min.[2.5*(h-d), (h-x)/3, h/2]$	=	249	mm	311	mm
Eff. area of concrete within tensile zone,	$A_{c,eff} = h_{c,eff} * b$	=	1268625	mm^2	3828375	mm^2
	$\rho_{peff} = A_s / A_{c,eff}$	=	0.015		0.012	
	$\varepsilon_{sm} - \varepsilon_{cm} = Max\{[\sigma_{sc}-k_t*f_{ct,eff}(1+\alpha_e*\rho_{peff})/\rho_{eff}]/E_s \&(0.6*\sigma_{sc}/E_s)\}$	=	0.0005		0.0001	
	$S_{r.max} = 3.4c + 0.425*k_1*k_2*\phi/\rho_{p,eff}$	=	535		597	
	$W_k = S_{r.max}*(\varepsilon_{sm} - \varepsilon_{cm})$	=	0.268	mm	0.048	mm
Status			**OK**		**OK**	

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		Design by	D.S.R	
		Check by	S.G	

16.0 Design of Pier Shaft:-

Grade of concrete of cast-insitu Pile,		=	M 35				
Material data:							
Charecteristics concrete comp. streng	f_{ck}	=	35	MPa			
Modulus of Elasticity of Concrete,	E_{cm}	=	32308	MPa			
Strength reduction factor,	α	=	0.67				
Concrete material factor,	γ_m	=	1.50				
Design concrete comp. strength,	$f_{cd} = \alpha f_{ck} / \gamma_m$	=	15.63	MPa			
Peak concrete strain at failure,	ε_{cu2}	=	0.0035				
Concrete strain at peak stress,	ε_{c2}		0.002				
Grade of Reinforcing steel,		=	Fe 500				
Charecteristic strength of r/f,	f_{yk}	=	500	MPa			
Partial factor of safety for r/f,	γ_s	=	1.15				
Design tensile strength,	$f_{yd} = f_{yk} / \gamma_s$	=	435	MPa			
Design yield of strength of shear r/f,	f_{ywd}	=	400	MPa			
Modulus of Elasticity of Steel,	E_s	=	200000	MPa			
Yield strain of steel,	$\varepsilon_s = f_{yd} / E_s$	=	0.00217				
Max. strain of steel,	$\varepsilon_{s.max} = 0.002 + f_{yd} / E_s$	=	0.00417				
Creep Coefficient,	ϕ	=	1.50				

Geometric data: (Pier / Abutment)			**Long. Direction**		**Trans. Direction**			
Dimension of Pier/ Abutment,		=	1000	mm	6000	mm	**Wall type Pier**	6000000
Actual column length,	l_0	=	7.328	m	7.328	m		
Effective length factor,		=	2.30		2.30			
Effective length of column	l_e	=	16.853	m	16.853	m		
Radius of Gyration,	i	=	289	mm	1732	mm		
Slenderness ratio,	$\lambda = l_e / i$	=	58.38		9.73			
Vertical longitudinal r/f detail of Column on each face:								
Diameter of long. Bars	1^{st} layer	=	25	mm	25	mm		
	2^{nd} layer	=	25	mm	25	mm		
Number of long. bars	1^{st} layer	=	16		16			
	2^{nd} layer	=	16		16			
Area of r/f provided on each face,	$A_{sc\|Face}$	=	15708	mm^2	15708	mm^2		
Percentage of r/f provided at each face,			0.26	%	0.26	%	**OK**	
Total area of long. r/f provided,	A_{sc}	=	62832	mm^2				
Total percentage of long. r/f provided,	p	=	1.05	%			**OK**	
Horizontal r/f detail of Column:								
Dia. of Shear r/f, stirrups/ links	ϕ	=	16	mm	20	mm		
Number of stirrups/ links,	n	=	8		30			
Spacing of stirrups/ links	s	=	-30	mm	-30	mm		
Area of Shear r/f provided,	A_{sw}	=	1608	mm^2	9425	mm^2		
Clear cover to r/f,	c	=	50	mm	50	mm		
Effective cover to long r/f,		=	79	mm	83	mm		
Effective depth of the section,		=	922	mm	5918	mm		

Calculation of demand points incorporating second order effects:

Parameters for limiting slenderness ratio:						
Limiting Slenderness ration,	λ_{lim}	=	$20 * A * B * C / \sqrt{n}$			Eq.11.1
where,	A	=	$1 / (1+0.2 * \phi_{ef})$			of IRC:112
Creep Coefficient,	$\phi(\infty,0)$	=	1.50			
First order BM Quasi-Permanent load	M_{oEqp}	=	1693	kN-m		
First order BM in design load in ULS	M_{oEd}	=	7831	kN-m		
Effective creep ratio,	$\phi_{ef} = \phi(\infty,0) * M_{oEqp}/M_{oEd}$	=	0.32			
	A	=	0.94			
where,	B	=	$\sqrt{(1 + 2\omega)}$			
Mechanical r/f ratio,	$\omega = A_s * f_{yd} / A_c * f_{cd}$	=	0.29			
	B	=	1.26			
where,	C	=	$1.7 - r_m$			
Moment Ratio,	$r_m = M_{01} / M_{02}$	=	1			
	C	=	0.7			
Relative normal force,	n	=	$N_{Ed} / A_c * f_{cd}$			
Eccentricity due to slenderness:						
Mominal second order moment,	M_2	=	$N_{Ed} * e_2$			
Deflection,	e_2	=	$(1/r) * l_e^2 / c$			
Factor for constant cross section,	c	=	10			
Curvature,	1/r	=	$K_r * K_\phi * 1/r_0$			
Factor depending on axial load,	K_r	=	$(n_u - n)/(n_u - n_{bal}) <=1$			
	$n_u = 1 + \omega$	=	1.29			
	n_{bal}	=	0.4			
Factor for taking account of creep,	K_ϕ	=	$1 + \beta * \varphi_{ef}$			
	$\beta = 0.35 + (f_{ck}/200) - (\lambda/150)$	=	0.14		0.46	
	K_ϕ	=	1.04		1.15	
	$1/r_0 = f_{yd} / (E_s * 0.45d)$	=	0.0030	1/m		

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		Design by	D.S.R	
		Check by	S.G	

Pier Moment Capacity Check at ULS :-

Load case	1 N_{Ed} kN	2 M_{0EdlL} kN-m	3 M_{0EdlT} kN-m	4 H_L kN	5 H_T kN	$n = N_{Ed}/(A_c * f_{cd})$	λ_{lim}	K_r	β_l	β_T	$K_{\phi L}$	$K_{\phi T}$
NS: Vmax	12894	3814	6840	544	165	0.14	44.6	1.0	0.14	0.46	1.04	1.15
NS: Vmin	6619	4092	0	201	0	0.07	62.3	1.0	0.14	0.46	1.04	1.15
NS: ML-max	12269	6630	4967	808	165	0.13	45.7	1.0	0.14	0.46	1.04	1.15
NS: MT-max	12284	3574	7009	514	165	0.13	45.7	1.0	0.14	0.46	1.04	1.15
SS: Vmax	12011	7094	7194	835	631	0.13	46.2	1.0	0.14	0.46	1.04	1.15
SS: Vmin	6749	9379	2074	720	196	0.07	61.7	1.0	0.14	0.46	1.04	1.15
SS: ML-max	11006	19819	7070	2251	630	0.12	48.3	1.0	0.14	0.46	1.04	1.15
SS: MT-max	11029	7093	22285	835	2105	0.12	48.2	1.0	0.14	0.46	1.04	1.15

Load case	e_{2L} (m)	e_{2T} (m)	M_{EdlL} kN-m	M_{EdlT} kN-m	M_{UL} kN-m	M_{UT} kN-m	$\frac{N_{Ed}}{N_{Rd}}$	α	$(M_{Ed} / M_u)^{\alpha}$	
NS: Vmax	0.09	0.00	4977	6840	15575	98899	0.11	1.01	0.39	**OK**
NS: Vmin	0.00	0.00	4092	0	13789	88259	0.06	1.00	0.30	**OK**
NS: ML-max	0.09	0.00	7737	4967	15412	97934	0.10	1.00	0.55	**OK**
NS: MT-max	0.09	0.00	4682	7009	15417	97959	0.10	1.00	0.37	**OK**
SS: Vmax	0.09	0.00	8177	7194	15342	97513	0.10	1.00	0.61	**OK**
SS: Vmin	0.00	0.00	9379	2074	13830	88500	0.06	1.00	0.70	**OK**
SS: MT-max	0.09	0.00	8088	22285	15071	95902	0.09	1.00	0.77	**OK**

Check for Ultimate Shear strength of Pier at ULS :-

			Long. Direction	Trans. Direction	
Design Shear Force of the concrete section,	$V_{NS} = V_{Ed}$	=	2251 kN	2105 kN	
Applied compressive Axial force,	N_{Ed}	=	11006 kN	11029 kN	
Design of section without shear reinforcement:-					
Factor for concrete cracked in shear,	$\nu=0.6(1-f_{ck}/310)$	=	0.53	0.53	
	$K = \text{Min.}[1 + (200/d)^{0.5}, 2]$	=	1.47	1.18	
	$\rho_1 = \text{Min.}[A_{sl} / b_w * d , 0.02]$	=	0.003	0.003	
	$\nu_{min} = 0.031 * K^{3/2} * f_{ck}^{1/2}$	=	0.325 MPa	0.236 MPa	
Mean Comp. stress in concrete,	$\sigma_{cp}=\text{Min}[N_{Ed}/A_c, 0.2f_{cd}]$ Mpa		1.8 MPa	1.8 MPa	
Min. Design Shear resistance,	$V_{Rd.c1}=(\nu_{min} + 0.15*\sigma_{cp})b_w*d$	=	3321 kN	3029 kN	
	$V_{Rd.c2}=[0.12*K*(80*\rho1*f_{ck})^{0.33}+ 0.15*\sigma_{cp}]b_w*d$	=	3449 kN	3261 kN	
Design Shear resistance,	$V_{Rdc} = \text{Max.}[V_{Rd.c1} \& V_{Rd.c2}]$	=	3449 kN	3261 kN	
Check for Shear R/f (SR) requirement,			**SR NRQ**	**SR NRQ**	*RQ - Required
Design of section with shear reinforcement:-					*NRQ - Not Required
Dia. of Shear r/f, stirrups/ links	ϕ	=	16 mm	20 mm	
Number of stirrups/ links,	n	=	8	30	
Spacing of stirrups/ links	s	=	-30 mm	-30 mm	
Area of Shear r/f provided,	A_{sw}	=	1608 mm^2	9425 mm^2	
	A_{sw} / s	=	-53.6 mm^2/mm	-314.2 mm^2/mm	
Minimum area of Shear r/f required,	Min. A_{sw} / s	=	0.9 mm^2/mm	5.1 mm^2/mm	
Lever arm factor,		=	0.9	0.9	
Coeff. depends on stress in compression cord,	α_{cw}	=	1.0	1.0	
Factor for concrete cracked in shear,	$\nu_1 = \nu$	=	0.53	0.53	
Shear capacity of concrete strut corresponding,	θ	=	45 degree	45 degree	
	$V_{Rd.max}=\alpha_{cw}*b_w*z*\nu1*f_{cd}/(\cot\theta+\tan\theta)$	=	20703 kN	22158 kN	
Status, Check for Redesign of section,			**OK**	**OK**	
Angle of concrete strut due shear,	θ	=	20.4 degree	2.7 degree	
Shear capacity of strut,	$V_{Rd}=\alpha_{cw}*b_w*z*\nu1*f_{cd}/(\cot\theta+\tan\theta)$		2251 kN	2105 kN	
			0	0	
Angle of concrete strut considered,	θ	=	21.8 degree	21.8 degree	
Shear resistance provided by r/f,	$V_{Rd.s} = A_{sw}/s*z*f_{vwd}*\cot\theta$	=	-44470 kN	-1673253 kN	
			-19.8	-795.0	

Confinment Reinforcement check:

Ductile detailing shall be carried out for bridges located in zones III, IV and V of seismic zone map of IRC:6-2017.

For Rectangular Column

As per Clause No. 7.4 of IS:13920-1993

Area of trans r/f, $A_{sh} = 0.18S\ D_k\ f_{ck}/f_y[A_g/A_k-1.0)$		=	1608 mm^2
Larger dimension of column ,	D	=	6000 mm
Larger dimension of r/f core,	D_k	=	5900 mm
Smaller dimension of r/f core,	B_k	=	5900 mm
Gross area of column cross section,	A_g	=	6000000 mm^2
Area of the concrete core,	A_k	=	34810000 mm^2
Spacing of spiral/ hoop,	S	=	-26 mm

As per Clause No. 17.2.1 of IRC:112-2011

Confining r/f ratio,	ω_{wd}	=	$\rho_w\ f_{yd} / f_{cd}$		
Where,	ρ_w	=	$A_{sw} /(b * S_L)$		
Area of spiral/ hoop,	A_{sw}	=	1608 mm^2		
Dia. of spiral or hoop	b	=	5900 mm		
For rectangular section,	$\omega_{wd.c}$	≥	max($\omega_{.\omega req}$; 0.12)		
Confining r/f ratio,	$\omega_{.\omega req}$	=	$0.37*A_c/A_{cc}*n_k+0.13*f_{yd}/f_{cd}*(p_L-0.01)$		Eq. 17.5
Where, $n_k =$	$N_{Ed}/(A_c*f_{ck}) > 0.08$	=	0.08	0.08 `	of IRC:112
$p_L =$	$A_s/(b * D)$	=	0.01		
	$\omega_{wd.c}$		0.12		
Confining r/f ratio,	$A_{sw} /(b*w_{wd.c})*f_{yd} /f_{cd}$	=	63 mm		

Provide, 16 ϕ @ -30 mm upto 6000 mm from pile cap top

DESIGN OF 35.3M PSC I-GIRDER

Introduction

The following design note pertains to the details design of longitudinal PSC T girders type superstructure. The analysis is done with Grillage analogy using STAAD pro. software. Design parameters used in the Grillage model & details design of longitudinal PSC T girder has been presented in the following pages.

1.0 Design Parameters

1)	Str. No./ Design Chainage of Structure	=	Minor Bridge		
2)	Crossing/ Skew angle	=	0.0	Deg.	
3)	Span Arrangement	=	1 x 35.0	m	
4)	c/c of Expansion joints	=	35.000	m	
5)	c/c of Bearing - Exp. joint on Abutment side	=	0.750	m	
6)	c/c of Bearing - Exp. joint on Pier side	=	0.750	m	
7)	c/c of Bearings (Brg.)	=	33.500	m	

1.1 Geometry:

1)	Carriageway Width	=	12.500	m	
2)	Overall Width	=	13.500	m	
3)	Width of Crash Barrier, on Median side	=	0.500	m	
	on Outer side	=	0.500	m	
	on Inner side of Foothpath/ Kerb	=	0.000	m	
4)	Width of Foothpath/ Footway Kerb	=	0.000	m	(On one side)
5)	Cross Slope	=	2.5 %		
6)	Thickness of Wearing Coat	=	65	mm	
	For design purpose considering future overlay	=	100	mm	
7)	c/c Spacing of Longitudinal girders	=	2.750	m	
8)	No's of Longitudinal girder	=	5		
9)	Cantilever overhang, on Median side	=	1.250	m	on G-I side
	on Outer side		1.250	m	on G-V side
10)	No's of Intermediate Cross girder	=	1		
11)	Total Depth of Superstructure	=	2.020	m	
12)	Depth of Deck Slab	=	0.220	m	
13)	Depth of Longitudinal girder	=	1.800	m	
14)	Width of Intermediate Cross girder/ Diaphram	=	0.300	m	
15)	Width of End Cross girder/ Diaphram	=	0.400	m	
16)	Girder length beyond c/l of Brg. on each side	=	0.450	m	

1.2 Material Properties:

1)	Concrete Grade for Deck slab	=	M 45	
2)	Concrete Grade for Cross girder	=	M 45	
3)	Concrete Grade of PSC Longitudinal girder	=	M 45	
4)	Reinforcement	=	Fe 500	
5)	Clear Cover	=	40	mm
6)	Condition of Exposure	=	Moderate	
7)	Cable/ Strands	=	19 T 13 conforming to IS 14268 (Stress-relieved multiply strands of low relaxat with Corrugated HDPE sheathing for prestress	

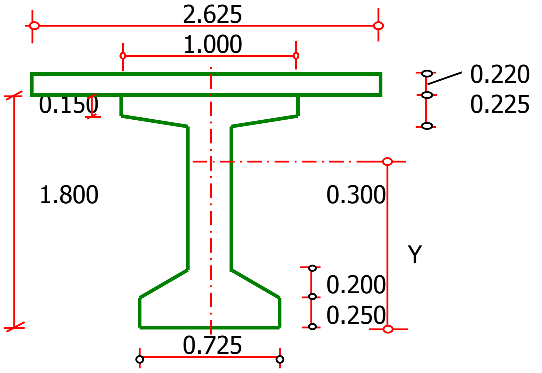
1.3 Proposed construction sequence

1)	Casting of PSC Longitudinal girder	=	0	Days
2)	1st stage prestressing	=	14	Days
3)	2nd stage prestressing	=	28	Days
4)	Casting of deck slab & Cross girders	=	35	Days
5)	Removal of shuttering	=	49	Days
6)	Casting of Crash barrier, Wearing coat etc	=	56	Days
7)	Live load	=	63	Days

1.0 Section Properties of Outer Longitudinal Girder OG (G-I to G-V)

c/c spacing of Bearing	=	33.500 m
c/c spacing of Longitudinal Girder	=	2.750 m
c/c spacing of Cross Girder	=	16.750 m
Distance between c/l of Brg. and c/l of Exp. joint	=	0.750 m
Cantilever overhang length	=	1.250 m
Depth of Longitudinal girder	=	1.800 m
Thickness of Deck slab	=	0.220 m
Thickness of Deck slab at Expansion joint	=	0.400 m
Grade of Concrete of Deck Slab	=	45 Mpa
Grade of Concrete of Longitudinal girder	=	45 Mpa
Length of Solid Portion	=	1.500 m
Length of Splayed Portion	=	1.500 m

Girder Section at the Span



Calculation of Torsional Constant				
Component	Width	Depth	Area	Iz
Top Slab	2.625	0.220	0.578	0.0047
Top Flange	0.883	0.225	0.199	0.0028
Web	1.125	0.300	0.338	0.0085
Bottom flang	0.631	0.450	0.284	0.0114
			Iz =	0.0274

Modulus of Elasticity of Deck Slab	=	34313 MPa
Modulus of Elasticity of Girder	=	34313 MPa
beff = Σ beff,i + bw	=	7.465 m
c/c Spacing of Longitudinal girder, b	=	2.625 m

SL.No.		A	y	Ay	Ay^2	I_o	$I_o + Ay^2$	u
1. Deck Slab		0.57750	1.91000	1.10303	2.10678	0.002329	2.10911	4.690
2. Rctangular top Flange		0.15000	1.72500	0.25875	0.44634	0.000281	0.44663	0.300
3. Triangular top Flange		0.02625	1.62500	0.04266	0.06932	0.000008	0.06932	0.026
4. Web + Top & Bottom Rect. Haunch		0.42000	0.95000	0.39900	0.37905	0.068600	0.44765	2.250
5. Bottom Bulb, Triangle		0.04250	0.31667	0.01346	0.00426	0.000094	0.00436	0.043
6. Bottom Bulb, Rectangle		0.18125	0.12500	0.02266	0.00283	0.000944	0.00378	1.225
Composite Section	Σ	1.39750		1.83955	3.0086	0.072	3.08084	8.534
Girder Only	Σ	0.82000		0.73652	0.9018	0.070	0.97173	3.844
		ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma(I_o + Ay^2)$	Σu

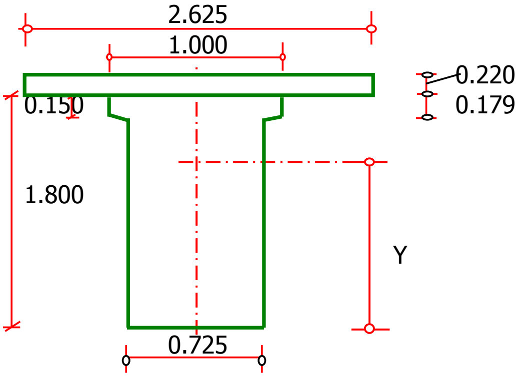
Section Properties of Composite Section

Area	**ΣA**	=	1.3975 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.3163 m
Moment of inertia of composite girder	**$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	0.6594 m^4

Section Properties of Girder

Area	**ΣA**	=	0.8200 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	0.8982 m
Moment of inertia of girder	**$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	0.3102 m^4

Girder Section at the Support



Calculation of Torsional Constant				
Component	Width	Depth	Area	Iz
Top Slab	2.625	0.220	0.578	0.0047
Top Flange	0.977	0.179	0.175	0.0009
Web	1.171	0.725	0.849	0.0967
Bottom flang	0.725	0.450	0.326	0.0143
			Iz =	0.1166

Modulus of Elasticity of Deck Slab = 34313 MPa
Modulus of Elasticity of Girder = 34313 MPa

beff = Σ beff,i + bw = 7.805 m
c/c Spacing of Longitudinal girder, b = 2.625 m

SL.No.		A	y	Ay	Ay^2	I_o	$I_o + Ay^2$	u
1. Deck Slab		0.57750	1.91000	1.10303	2.10678	0.002329	2.10911	4.690
2. Rctangular top Flange		0.15000	1.72500	0.25875	0.44634	0.000281	0.44663	0.300
3. Triangular top Flange		0.00405	1.64018	0.00664	0.01090	0.000000	0.01090	0.004
4. Web + Top Rect. Haunch		1.19625	0.82500	0.98691	0.81420	0.271399	1.08560	3.241
Composite Section	Σ	1.92780		2.35533	3.37822	0.27401	3.65223	8.235
Girder Only	Σ	1.35030		1.25230	1.27144	0.27168	1.54312	3.545
		ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma(I_o + Ay^2)$	Σu

Section Properties of Composite Section

Area — **ΣA** = 1.9278 m^2

Distance of cg from bottom fibre (Y) — **Y =Σ(A.y) / ΣA** = 1.2218 m

Moment of inertia of end intermediate girder — **$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$** = 0.7746 m^4

Section Properties of Girder

Area — **ΣA** = 1.3503 m^2

Distance of cg from bottom fibre (Y) — **Y =Σ(A.y) / ΣA** = 0.9274 m

Moment of inertia of end intermediate girder — **$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$** = 0.3817 m^4

1.1 Summary of Section Properties of Longitudinal Girder (Outer)

A. Section Properties of Composite Section at the Span

Area **1.3975** m^2 — Y_b = **1.3163** m — Y_{ts} = **0.7037** m
I_z = **0.6594** m^4 — Z_b = **0.5010** m^3 — Z_{ts} = **0.9371** m^3
Y_{tG} = **0.4837** m
Z_{tG} = **1.3633** m^3

B. Section Properties of Girder at the Span

Area **0.8200** m^2 — Y_b = **0.8982** m — Y_t = **0.9018** m
I_z = **0.3102** m^4 — Z_b = **0.3453** m^3 — Z_t = **0.3440** m^3

C. Section Properties of Composite Section at the Support

Area **1.9278** m^2 — Y_b = **1.2218** m — Y_{ts} = **0.7982** m
I_z = **0.7746** m^4 — Z_b = **0.6340** m^3 — Z_{ts} = **0.9704** m^3
Y_{tG} = **0.5782** m
Z_{tG} = **1.3395** m^3

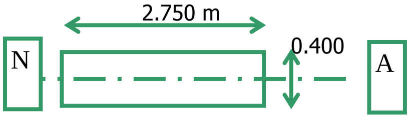
D. Section Properties of Girder at the Support

Area **1.3503** m^2 — Y_b = **0.9274** m — Y_t = **0.8726** m
I_z = **0.3817** m^4 — Z_b = **0.4116** m^3 — Z_t = **0.4374** m^3

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphram Members

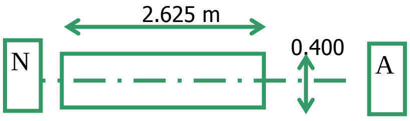
c/c spacing of Bearing		=	33.500 m	
c/c spacing of Longitudinal girder		=	2.750 m	
c/c spacing of Cross girder		=	16.750 m	
Distance between c/l of Brg. and c/l of Exp. joint		=	0.750 m	0.750
Cantilever length on Median side (G-I side)		=	1.250 m	
Cantilever length on Outer side (G-IV side)		=	1.250 m	
Depth of Long girder		=	1.800 m	
Thickness of Deck slab		=	0.220 m	
Depth of Cross girder		=	1.550 m	
No's of Longitudinal girders		=	5	
Thickness of Deck slab at Exp. joint		=	0.400 m	
Web thickness of Inner Cross girder		=	0.300 m	
Web thickness of External Cross girder		=	0.400 m	
Skew distance between c/c of bearing	0.0	=	2.750 m	
Length of Solid Portion		=	1.500 m	
Length of Splayed Portion		=	1.500 m	
Grade of Concrete of Deck Slab		=	45 Mpa	
Grade of Concrete of Girder		=	45 Mpa	
Modular Ratio		=	1.000	
Effective width of Slab for Inner Long. girder		=	2.750 m	
Effective width of Slab for Outer Long. girder		=	2.625 m	

Cantilever Slab Inner (At Expansion End) **_LONGCANTIN**
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.1000 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0147 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

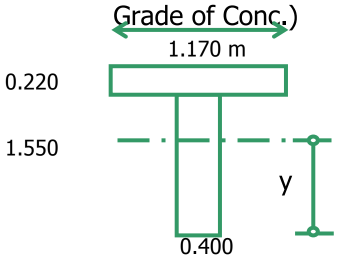
Cantilever Slab Outer (At Expansion End) **_LONGCANTOUT**
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.0500 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0140 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

End Cross Girder **_ENDCG**
(with reduced width for diff. in Grade of Conc.)

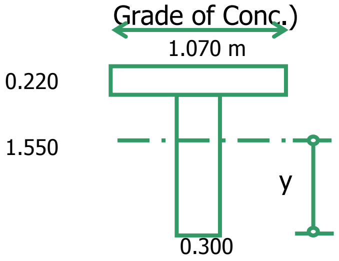
beff = Min{ Σ beff,i + bw, b}	=	1.170 m
Modulus of Elasticity ratio	=	1.000



Area	=	0.8774 m^2
Distance of cg from bottom fibre (y)	=	1.0346 m
Moment of inertia of end x-girder (Iz)	=	0.2676 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate/ Inner Cross Girder **_INTCG**

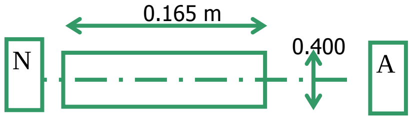
(with reduced width for diff. in Grade of Conc.)



beff = Min{ Σ beff,i + bw, b}	=	1.070 m
Modulus of Elasticity ratio	=	1.000
Area	=	0.7004 m^2
Distance of cg from bottom fibre (y)	=	1.0724 m
Moment of inertia of end x-girder (Iz)	=	0.2165 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At Expansion End) **_EXPTRANSLAB**

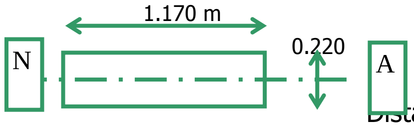
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.0660 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0009 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At End Cross Girder) **_TRANSCANT1**

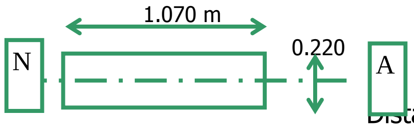
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.2574 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0010 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At Inner Cross Girder) **_TRANSCANT2**

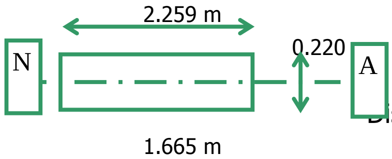
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.2354 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0009 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Next to End Cross Girder) **_TRANSLAB1**

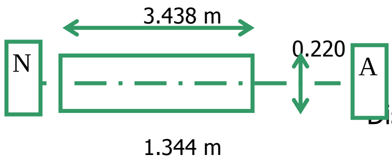
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.4969 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0020 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (At Splayed/ Tapered Portion) **_TRANSLAB2**

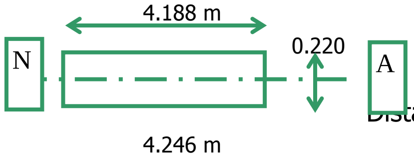
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.7563 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0031 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Adjacent to First Inner Cross Girder) **_TRANSLAB3**

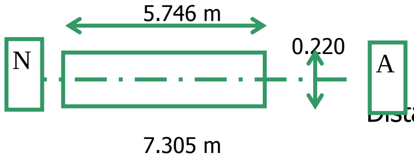
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.9213 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0037 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Between Inner Cross Girders) **_TRANSLAB4**

(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.2642 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0051 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

1.2 SIDL and Footpath Load Calculation

Span length c/c of Expansion Joint	=	35.000	m	
Span length c/c of Bearing	=	33.500	m	
Skew Angle	=	0.0	Deg.	
Depth of Wearing Coat assumed for Design	=	0.100	m	
Density of Wearing Coat material	=	22.0	kn/m^3	
Density of footpath material	=	25.0	kn/m^3	
Weight of Crash Barrier (0.9 m height)per m run	=	8.5	kn/m	
Weight of Railing (1.1 m height) per m run	=	8.5	kn/m	
Width of Crash Barrier	=	0.50	m	
Width of Railing on both outer side of Bridge	=	0.50	m	
Width of Footpath on both side	=	0.00	m	
Thickness of Footpath	=	0.00	m	
Basic Footpath Live Load	=	400	kg/m^2	
Actual Footpath Live Load	=	2.84	kn/m^2	Cl. 206.3.(b) IRC 6-201
Self weight of Footpath	=	0.00	kn/m^2	
Wearing Coat Load	=	2.20	kn/m^2	

3.1 Wind Load Calculation

Span length c/c of Expansion Joint		=	35.000	m
Span length c/c of Bearing		=	33.500	m
Overall width of superstructure		=	13.500	m
C/C spacing of Longitudinal Girder		=	2.750	m
Depth of superstructure		=	2.020	m
Depth of Wearing Coat		=	0.100	m
Solid area as seen in elevation	A_1	=	112.70	m^2
Plan area of the Superstructure	A_3	=	472.50	m^2
Type of Superstructure,			Two or more beam/ Box girder Bridge	
Width to depth ration i.e b / d		=	6.68	
Ration of clear distance bet.^{w} beams (for two or more beams/ box bridge) to depth		=	1.36	
Gust Factor	G	=	2.00	
Drag coefficient for the Superstructure	C_D	=	1.95	
Lift coefficient	C_L	=	0.75	

Wind load on loaded structure

As per clause 209.3.7 the bridges shall not be considered to be carrying any live load when the wind speed at deck level exceeds 36m/s.

Design Wind speed at any height	V_b	=	39.0	m/s
Bridge situated at,			Plain Terrain	

As per note no. 5 of cl no. 209 of IRC:6-2014, wind pressure shall be increased by 0 %

Wind Speed & design wind pressure for Basic wind speed of 33 m/s | **Wind speed & design wind pressure for Basic wind speed of 39m/s**

H (m)	Plain Terrain		Terrain with Obstructio		H (m)	Plain Terrain		Terrain with Obstruction	
	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)		V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
10	27.8	463.7	17.8	190.5	< 10	38.8	647.6	24.9	266.1
15	29.2	512.5	19.6	230.5	15	40.8	715.8	27.4	321.9
20	30.3	550.6	21	265.3	20	42.3	769.0	29.3	370.5
30	31.4	590.2	22.8	312.2	30	43.9	824.3	31.8	436.0
50	33.1	659.2	24.9	370.4	50	46.2	920.7	34.8	517.3
60	33.6	676.3	25.6	392.9	60	46.9	944.6	35.8	548.8
70	34.0	693.6	26.2	412.8	70	47.5	968.7	36.6	576.6
80	34.4	711.2	26.9	433.3	80	48.0	993.3	37.6	605.2
90	34.9	729.0	27.5	454.2	90	48.7	1018.2	38.4	634.4
100	35.3	747.0	28.2	475.6	100	49.3	1043.3	39.4	664.3

Average Height of superstructure				=	15.0	m		
Lower bound	Height =	15.0 m	P_z	=	715.8	N/m^2		
Upper Bound	Height =	15.0 m	P_z	=	715.8	N/m^2		
P_z = Design wind pressure at bridge for h =		15.0 m		=	715.8	N/m^2		
Tranverse wind Force			F_T	=	P_Z x A_1 x G x C_D			
				=	314.6	KN	1.80	KN/m
Longitudinal wind Force			F_L	=	25% of F_T			
				=	78.7	KN	0.17	KN/m
Vertical wind Force			F_V	=	P_Z x A_3 x G x C_L			
					507.3	KN		
UDL on girder				=	1.1	KN/m^2		
Outer girder				=	2.8	KN/m	2.82	KN/m
Inner girder				=	3.0	KN/m	2.95	KN/m

1.3 Live Load Idealization

	Span length c/c of Expansion Joint	=	35.000	m	
	Span length c/c of Bearing	=	33.500	m	
	Skew Angle	=	0.0	Degree	
	Total Width of Superstructure	=	13.500	m	
	Width of Railing / Crash barrier on right side	=	0.500	m	
	Width of Railing / Crash barrier on left side	=	0.500	m	
	Width of Footpath on both side (considered for design o	=	0.000	m	
	Cantilever length	=	1.250	m	
	C/C spacing of Longitudinal Girder	=	2.750	m	
	Maximum Length of Live Load	=	18.8	m	13.4, 18.8
	Effect of Skew	=	0.0		
	Travelling Length of Live Load	=	54	m	
	Increment of Live Load	=	1	m	
	Number of Load Case for each Live Load	=	55		
	Carriageway width for LL (considered for design only)	=	12.5	m	
1 *Load Case 1*	70R W 1 Lane Placed most Eccentric (Left side)				
	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130	m	
	Z2 = 2.13 + 1.93	=	4.060	m	
2 *Load Case 2*	70R W 1 Lane Placed at Center of Superstructure				
	Z1 = 13.5/2 -1.93/2	=	5.785	m	
	Z2 = 5.785 + 1.93	=	7.715	m	
3 *Load Case 3*	70R W 1 Lane Placed most Eccentric (Right side)				
	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.44	m	
	Z2 = 9.44 + 1.93	=	11.37	m	
4 *Load Case 4*	70R W 1 Lane Placed at Center of Inner girder				
	Z1 = 1.25 + 2.75 - 1.93/2	=	3.035	m	
	Z2 = 3.035 + 1.93	=	4.965	m	
5 *Load Case 5*	70R W 2 Lane Placed most Eccentric (Left side)				From CB
1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130	m	
70R	Z2 = 2.13 + 1.93	=	4.060	m	3.095
2nd Lane	Z3 = 0.5 + 0 + 7.25 + 0.86/2	=	8.180	m	
70R	Z4 = 8.18 + 1.93	=	10.110	m	9.145
6 *Load Case 6*	70R W 2 Lane Placed at near Center of Superstructure				
1st Lane	Z1 = 12.5/2 - 1.2/2 - 0.86/2 - 1.93		3.29	m	From CL
70R	Z2 = 3.29 + 1.93	=	5.22	m	1.995
2nd Lane	Z3 = 5.22 + 0.86/2 + 1.2 + 0.86/2	=	7.28	m	
70R	Z4 = 7.28 + 1.93	=	9.21	m	-1.995
7 *Load Case 7*	Class A 1 Lane Placed most Eccentric (Left side)				
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
	Z2 = 0.9 + 1.8	=	2.70	m	
8 *Load Case 8*	Class A 1 Lane Placed most Eccentric (Right side)				
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.80	m	
	Z2 = 10.8 + 1.8	=	12.60	m	
9 *Load Case 9*	Class A 2 Lane Placed most Eccentric (Left side)				
1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
Class A	Z2 = 0.9 + 1.8	=	2.70	m	
2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m	
Class A	Z4 = 4.4 + 1.8	=	6.20	m	

10 *Load Case 10*	Class A 2 Lane Placed most Eccentric (Right side)				
1st Lane	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8 -0.25 - 1.2 - 0.25 - 1.8	=	7.30	m	
Class A	Z2 = 7.3 + 1.8	=	9.10	m	
2nd Lane	Z3 = 9.1 + 0.5 + 1.2	=	10.80	m	
Class A	Z4 = 10.8 + 1.8	=	12.60	m	
11 *Load Case 11*	Class A 3 Lane Placed most Eccentric (Left side)				
1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
Class A	Z2 = 0.9 + 1.8	=	2.70	m	
2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m	
Class A	Z4 = 4.4 + 1.8	=	6.20	m	
3rd Lane	Z5 = 6.2 + 0.5 + 1.2	=	7.90	m	
Class A	Z6 = 7.9 + 1.8	=	9.70	m	
12 *Load Case 12*	Class A 3 Lane Placed at Center of Superstructure				
1st Lane	Z1 = 13.5/2 - 1.8/2 - 0.5 - 1.2 - 1.8	=	2.35	m	
Class A	Z2 = 2.35 + 1.8	=	4.15	m	
2nd Lane	Z3 = 4.15 + 0.5 + 1.2	=	5.85	m	
Class A	Z4 = 5.85 + 1.8	=	7.65	m	
3rd Lane	Z5 = 7.65 + 0.5 + 1.2	=	9.35	m	
Class A	Z6 = 9.35 + 1.8	=	11.15	m	
13 *Load Case 13*	Class A 3 Lane Placed most Eccentric (Right side)				
1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8	=	3.80	m	
Class A	Z2 = 3.8 + 1.8	=	5.60	m	
2nd Lane	Z3 = 5.6 + 0.5 + 1.2	=	7.30	m	
Class A	Z4 = 7.3 + 1.8	=	9.10	m	
3rd Lane	Z5 = 9.1 + 0.5 + 1.2	=	10.80	m	
Class A	Z6 = 10.8 + 1.8	=	12.60	m	
14 *Load Case 14*	Class A 4 Lane Placed most Eccentric (Left side)				From CB
1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
Class A	Z2 = 0.9 + 1.8	=	2.70	m	1.800
2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m	
Class A	Z4 = 4.4 + 1.8	=	6.20	m	5.300
3rd Lane	Z5 = 6.2 + 0.5 + 1.2	=	7.90	m	
Class A	Z6 = 7.9 + 1.8	=	9.70	m	8.800
4th Lane	Z7 = 9.7 + 0.5 + 1.2	=	11.40	m	
Class A	Z8 = 11.4 + 1.8	=	13.20	m	12.300
15 *Load Case 15*	Class A 4 Lane Placed at Center of Superstructure				
1st Lane	Z1 = 13.5/2 - (0.25+1.2+0.25)/2 - 1.8 - 0.5 - 1.2 - 1.8	=	0.60	m	
Class A	Z2 = 0.6 + 1.8	=	2.40	m	
2nd Lane	Z3 = 2.4 + 0.5 + 1.2	=	4.10	m	
Class A	Z4 = 4.1 + 1.8	=	5.90	m	
3rd Lane	Z5 = 5.9 + 0.5 + 1.2	=	7.60	m	
Class A	Z6 = 7.6 + 1.8	=	9.40	m	
4th Lane	Z7 = 9.4 + 0.5 + 1.2	=	11.10	m	
Class A	Z8 = 11.1 + 1.8	=	12.90	m	
16 *Load Case 16*	Class A 4 Lane Placed most Eccentric (Right side)				
1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8		0.30	m	
Class A	Z2 = 0.3 + 1.8	=	2.10	m	
2nd Lane	Z3 = 2.1 + 0.5 + 1.2	=	3.80	m	
Class A	Z4 = 3.8 + 1.8	=	5.60	m	
3rd Lane	Z5 = 5.6 + 0.5 + 1.2	=	7.30	m	
Class A	Z6 = 7.3 + 1.8	=	9.10	m	
4th Lane	Z7 = 9.1 + 0.5 + 1.2	=	10.80	m	
Class A	Z8 = 10.8 + 1.8	=	12.60	m	

17	*Load Case 17*	Class A for combination of 70R Eccentric on Left	70R + Class A 1 Lane				
	1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.13	m		
	70R	Z2 = 2.13 + 1.93	=	4.06	m		
	1st Lane	Z1 = 0.5 + 0 + 7.25 + 0.5/2	=	8.00	m		
	Class A	Z2 = 8 + 1.8	=	9.80	m		
18	*Load Case 18*	Class A for combination of 70R Eccentric on Right	70R + Class A 1 Lane				
	1st Lane	Z1 = 13.5 - 0.5 - 7.25 - 0.5/2 - 1.8	=	3.70	m		
	Class A	Z2 = 3.7 + 1.8	=	5.50	m		
	1st Lane	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.44	m		
	70R	Z2 = 9.44 + 1.93	=	11.37	m		
19	*Load Case 19*	Class A 2 Lane for combination of 70R Eccentric on Left	70R + Class A 2 Lane				
	1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.13	m		
	70R	Z2 = 2.13 + 1.93	=	4.06	m		
	1st Lane	Z1 = 0.5 + 0 + 7.25 + 0.5/2	=	8.00	m		
	Class A	Z2 = 8 + 1.8	=	9.80	m		
	2nd Lane	Z3 = 9.8 + 0.5 + 1.2	=	11.50	m		
	Class A	Z4 = 11.5 + 1.8	=	13.30	m		
20	*Load Case 20*	Class A 2 Lane for combination of 70R placed near Cent	70R + Class A 2 Lane				
	1st Lane	Z1 = 7.25 - 0.86/2 - 1.93	=	4.89	m		
	70R	Z2 = 4.89 + 1.93	=	6.82	m		
	1st Lane	Z1 = 6.82 + 0.86/2 + 1.2 + 0.5/2	=	8.70	m		
	Class A	Z2 = 8.7 + 1.8	=	10.50	m		
	2nd Lane	Z3 = 10.5 + 0.5 + 1.2	=	12.20	m		
	Class A	Z4 = 12.2 + 1.8	=	14.00	m		
21	*Load Case 21*	70R W for combination of Class A 2 Lane Eccentric on Le	Class A 2 Lane + 70R			From CB	
	1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m		
	Class A	Z2 = 0.9 + 1.8	=	2.70	m	1.800	
	2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m		
	Class A	Z4 = 4.4 + 1.8	=	6.20	m	5.300	
	1st Lane	Z5 = 6.2 + 0.5/2 + 1.2 + 0.86/2	=	8.080	m		
	70R	Z6 = 8.08 + 1.93	=	10.010	m	9.045	
22	*Load Case 22*	70R W for combination of Class A Eccentric on Right	Class A 1 Lane + 70R				
	1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 2.3 - 1.2 - 2.79 + 0.86/2	=	6.99	m		
	70R	Z2 = 6.99 + 1.93	=	8.92	m		
	1st Lane	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.80	m		
	Class A	Z2 = 10.8 + 1.8	=	12.60	m		

Note : Z1 to Z6 indicates Distance of 1st to 6th Axle from Left

1.3 Live Load Idealization

Span length c/c of Expansion Joint	=	35.000	m
Span length c/c of Bearing	=	33.500	m
Skew Angle	=	0.0	Degree
Total Width of Superstructure	=	13.500	m
Width of Crash Barrier on right side	=	0.500	m
Width of Railing / Crash barrier on left side	=	0.500	m
Width of Footpath on left side (considered for design onl	=	0.000	m
Cantilever length	=	1.250	m
C/C spacing of Longitudinal Girder	=	2.750	m
Maximum Length of Live Load	=	18.8	m
Effect of Skew	=	0.0	
Travelling Length of Live Load	=	54	m
Increment of Live Load	=	1	m
Number of Load Case for each Live Load	=	55	
Carriageway width for LL (considered for design only)	=	12.5	m

Load Case 1	70R W Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130	m
	Z2 = 2.13 + 1.93	=	4.060	m
Load Case 2	70R W Placed at center			
	Z1 = 13.5 /2 -1.93/2	=	5.785	m
	Z2 = 5.785 + 1.93	=	7.715	m
Load Case 3	70R W Placed most eccentric (Right side)			
	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.440	m
	Z2 = 9.44 + 1.93	=	11.370	m
Load Case 4	70R W 1 Lane Placed at Center of Inner girder			
	Z1 = 1.25 + 2.75 - 1.93/2	=	3.035	m
	Z2 = 3.035 + 1.93	=	4.965	m
Load Case 5	Class A 1 Lane Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.900	m
	Z2 = 0.9 + 1.8	=	2.700	m
Load Case 6	Class A 1 Lane Placed most eccentric (Right side)			
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.800	m
	Z2 = 10.8 + 1.8	=	12.600	m
Load Case 7	Class A 3 Lane Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.900	m
	Z2 = 0.9 + 1.8	=	2.700	m
	Z3 = 2.7 + 0.5 + 1.2	=	4.400	m
	Z4 = 4.4 + 1.8	=	6.200	m
	Z5 = 6.2 + 0.5 + 1.2	=	7.900	m
	Z6 = 7.9 + 1.8	=	9.700	m
Load Case 8	Class A 3 Lane Placed at center			
	Z1 = 13.5 /2 - 1.8/2 - 0.5 - 1.8	=	2.350	m
	Z2 = 2.35 + 1.8	=	4.150	m
	Z3 = 4.15 + 0.5 + 1.2	=	5.850	m
	Z4 = 5.85 + 1.8	=	7.650	m
	Z5 = 7.65 + 0.5 + 1.2	=	9.350	m
	Z6 = 9.35 + 1.8	=	11.150	m

Load Case 9	Class A 3 Lane Placed most eccentric (Right side)		
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8-1.7-1.8-1.7-1.8	=	3.800 m
	Z2 = 3.8 + 1.8	=	5.600 m
	Z3 = 5.6 + 0.5 + 1.2	=	7.300 m
	Z4 = 7.3 + 1.8	=	9.100 m
	Z5 = 9.1 + 0.5 + 1.2	=	10.800 m
	Z6 = 10.8 + 1.8	=	12.600 m
Load Case 10	Class A for combination of 70R Eccentric on left (Case 1)		
	Z1 = 0.5 + 0 + 7.25 + 0.15 + 0.5/2	=	8.150 m
	Z2 = 8.15 + 1.8	=	9.950 m
Load Case 11	Class A for combination of 70R Eccentric on Right (Case 2)		
	Z1 = 13.5 - 0.5 - 9.85 + 0.15 + 0.5/2	=	3.550 m
	Z2 = 3.55 + 1.8	=	5.350 m
Load Case 12	70R W for combination of Class A Eccentric on left (Case 3)		
	Z1 = 0.5 + 0 + 2.45 + 1.2 + 0.86/2	=	4.580 m
	Z2 = 4.58 + 1.93	=	6.510 m
Load Case 13	70R W for combination of Class A Eccentric on right (Case 4)		
	Z1 = 13.5 - 0.5 - 2.45 - 1.2 -2.79 + 0.86/2	=	6.990 m
	Z2 = 6.99 + 1.93	=	8.920 m

Note : Z1 to Z6 indicates Distance of 1st to 6th Axle from Left

1.4 Bending Moment & Shear Force of Longitudinal Members

Group-2 Girder(G-I & G-IV) : Outer Girder

Total length of the Precast Girder unit		=	34.400 m
Position of Bearing from Girder End		=	0.450 m
c/c spacing of Bearing		=	33.500 m
c/c spacing of Longitudinal Girder		=	2.750 m
Length of cantilever portion		=	1.250 m
Area of Girder at the Span		=	0.820 m^2
Area of Girder at the Support		=	1.350 m^2
Length of the normal section		=	28.400 m
Length of the tapered section		=	1.500 m
Length of the thickened section		=	1.500 m
Thickness of Intermediate X-Girder		=	0.300 m
Thickness of End X-Girder		=	0.400 m
Thickness of Deck slab		=	0.220 m
Thickness of Precast Deck slab placed on Girder top		=	0.000 m
Width of the Precast Deck slab placed on Girder top		=	0.000 m
Desity of Concrete Girder		=	25.0 KN/m^3
Desity of Concrete Top Slab		=	26.0 KN/m^3
Longitudinal Cantilever Length		=	0.750 m

Self Weight of the Girder during Stage-1 Loading

Weight of Girder at Center section	= 0.82 x 25	=	20.5 KN/m
Weight of Girder at End section	= 1.35 x 25	=	33.8 KN/m
Weight of Girder at Variable section	=(20.5 + 33.8) / 2	=	27.1 KN/m
Weight of End X-girder		=	2.0 KN
Weigt of Intermediate X-girder		=	3.6 KN

Weight of the Shuttering Slab during Stage-2 Loading

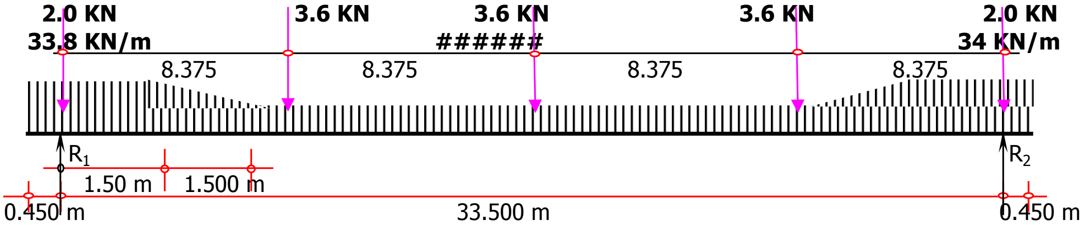
Weight of Shuttering		=	5.0 KN/m

Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	=2.8 x 0.22 x 26	=	15.7 KN/m
Wt. of end x-girder		=	6.8 KN
Wt. of intermediate x-girder		=	5.1 KN

Calculation of Moment & Shear Force at different section

Stage-1 loading **Self weight of girder**

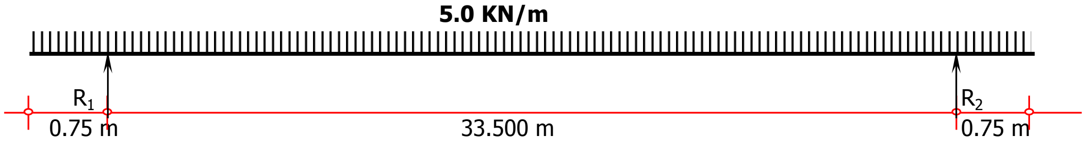


Support Reaction **R_1 = 395.7 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.450	4.638	8.825	13.013	17.200
Support load	395.71	395.71	395.71	395.71	395.71
End Load	15.19	65.83	65.83	65.83	65.83
Varing Load	0.00	40.69	40.69	40.69	40.69
Center Load	0.00	24.34	110.19	196.03	281.88
Point Load	1.98	1.98	5.53	5.53	9.09
Support C.G	0.00	4.19	8.38	12.56	16.75
End C.G	0.23	3.66	7.85	12.04	16.23
Varing C.G	0.00	2.00	6.19	10.37	14.56
Center C.G	0.00	0.59	2.69	4.78	6.88
Point C.g	0.00	4.19	3.00	7.18	6.93
Support Mom.	0.00	1657.02	3314.05	4971.07	6628.09
End Moment	3.42	241.09	516.74	792.39	1068.05
Varing Moment	0.00	81.33	251.73	422.13	592.54
Center Moment	0.00	14.45	296.13	937.27	1937.89
Point Moment	0.00	8.30	16.59	39.77	62.94
BM (KNm)	-3.4	1311.9	2232.9	2779.5	2966.7
SF (KN)	378.5	262.9	173.5	87.6	-1.8

Stage-2 loading **Shuttering Load for Deck Slab**

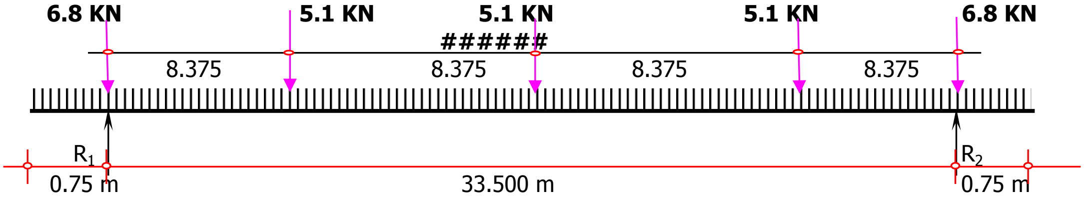
5.0 KN/m



Support Reaction **R_1 = 87.5 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.750	4.938	9.125	13.313	17.500
Support load	87.50	87.50	87.50	87.50	87.50
Center Load	3.75	24.69	45.63	66.56	87.50
Support C.G	0.00	4.19	8.38	12.56	16.75
Center C.G	0.38	2.47	4.56	6.66	8.75
Support Momer	0.00	366.41	732.81	1099.22	1465.63
Center Moment	1.41	60.95	208.16	443.06	765.63
BM (KNm)	-1.4	305.5	524.6	656.2	700.0
SF (KN)	83.8	62.8	41.9	20.9	0.0

Stage-3 loading **Deck Slab Load**



Support Reaction **R_1 = 289.7 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.750	4.938	9.125	13.313	17.500
Support load	289.69	289.69	289.69	289.69	289.69
Center Load	11.80	77.67	143.54	209.41	275.28
Point Load	6.78	6.78	11.87	11.87	16.95
Support C.G	0.00	4.19	8.38	12.56	16.75
Center C.G	0.38	2.47	4.56	6.66	8.75
Point C.G	0.00	4.19	4.79	8.97	9.21
Support Momer	0.00	1213.06	2426.11	3639.17	4852.23
Center Moment	4.42	191.74	654.88	1393.86	2408.66
Point Moment	0.00	28.40	56.79	106.49	156.18
BM (KNm)	-4.4	992.9	1714.4	2138.8	2287.4
SF (KN)	271.1	205.2	134.3	68.4	-2.5

1.5 Bending Moments & Shear Forces at Various Sections

A. MOMENT AT DIFFERENT SECTION (KNm) for OUTER GIRDER

Span Length (c/c of Brg.)	=	33.50	m
I.F for 70 R wheel	=	1.130	
I.F for Class A [1+4.5/(6+E_{ff} Span)]	=	1.114	
Warping Effect	=	1.000	

Note:- Value of Bending Moments are taken upto support section where as shear force is considered @ a distance 'd' effective from support

LOADINGS	c/L Brg.	L/8	L/4	3L/8	L/2	Factor SLS Rare
	0m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	0.0	1311.9	2232.9	2779.5	2966.7	1.0
2nd Stage DL	0.0	992.9	1714.4	2138.8	2287.4	1.0
Shuttering Load	0.0	305.5	524.6	656.2	700.0	1.0
SIDL (Crash Barrier)	0.0	297.0	452.0	479.0	383.7	1.0
SIDL (Surfcaing/ Wearing coat	0.0	274.3	478.2	608.7	664.8	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	0.0	1231.2	2105.4	2653.2	2908.2	1.0
	0	4107	6983	8659	9211	

B. SHEAR AT DIFFERENT SECTION (KN) FOR OUTER GIRDER

LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	378.5	262.9	173.5	87.6	0.0	1.0
2nd Stage DL	271.1	205.2	134.3	68.4	0.0	1.0
SIDL (Crash Barrier)	94.0	76.3	43.8	7.3	26.7	1.0
SIDL (Surfcaing/ Wearing coat	76.3	67.8	47.2	26.3	5.1	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	339.9	336.6	289.9	223.7	128.3	1.0

1.6 Bending Moments & Shear Forces at Various Sections

A. MOMENT AT DIFFERENT SECTION (KNm) for INNER GIRDER

Span Length (c/c of Brg.) = 33.500 m

Note:- Value of Bending Moments are taken upto support section where as shear force is considered @ a distance 'd' effective from support

LOADINGS	c/L Brg.	L/8	L/4	3L/8	L/2	Factor SLS Rare
	0m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	0.0	1311.9	2232.9	2779.5	2966.7	1.0
2nd Stage DL	0.0	992.9	1714.4	2138.8	2287.4	1.0
Shuttering Load	0.0	305.5	524.6	656.2	700.0	1.0
SIDL (Crash Barrier)	0.0	60.0	141.3	250.5	377.8	1.0
SIDL (Surfcaing/ Wearing coat	0.0	302.6	517.8	644.4	682.9	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	0.0	1397.6	2091.9	2373.9	2581.8	1.0
	0	4065	6698	8187	8897	

B. SHEAR AT DIFFERENT SECTION (KN) FOR INNER GIRDER

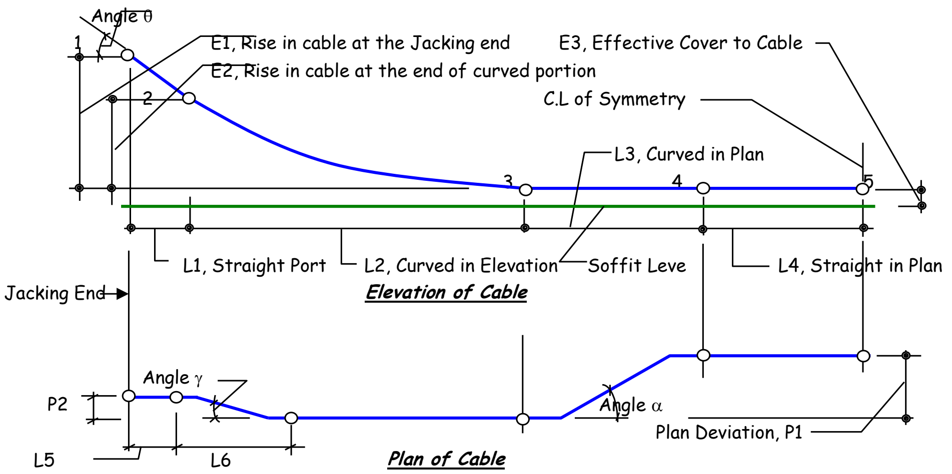
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	378.5	262.9	173.5	87.6	0.0	1.0
2nd Stage DL	271.1	205.2	134.3	68.4	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	383.1	366.3	290.2	197.7	275.1	1.0

3.0 Effect of Prestressing (Outer Girder):-

A. Basic Prestressing Data

1) Nominal Diameter	Ds	12.7	mm	19	
2) Nominal Area	A	98.7	sq.mm		
3) Nominal Mass	Pu	0.775	Kg/m		
4) Yield Strength	Fy	1670	MPa		
5) Tensile Strength	Fu	1860	MPa		1395
6) Minimum Breaking Load	Pn	183.7	KN		138
7) Characteristic co-efficient value for Prestressing Force	γ_{sup}	1.0			
7a) Characteristic co-efficient value for Prestressing Force	γ_{inf}	1.0			
8) Multiplied by the co-efficient		183.7	KN		
9) Young's Modulus of Elasticity	Eps	195	Gpa		
10) Jacking Force at Transfer (% of Breaking Load)	Pj	75	%		
11) Slip at Jacking end	s	6	mm		
12) Coefficient of Friction	μ	0.17	per radian	Refer Table 7.1 of IRC:112	
13) Wobble Friction Coefficient	k	0.002	per metre	Corrugated HDPE sheathing	
14) Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		(Refer Table 6.2 , IRC:112)	
15) Relaxation loss at 1000 hrs at 50% UTS (as % of initial stress)	Re2	0		(Refer Table 6.2 , IRC:112)	
16) Age of concrete for 1st Stage prestressing	t_{d1}	14	days		
17) Dia of Prestressing Duct	q_d	90	mm		
18) Concrete Grade	Fcu_1	45	MPa		
19) Modulus of Elasticity of Concrete for Girder (28 days)	Ec_1	34313	Mpa	(Refer Table 6.2 and Eq 6.2,	
20) Concrete Grade at the time of 1st Stege stressing in 14 days	Fcj	40.6	MPa		
21) Modulus of Elasticity of Concrete at 1st Stage stressing	Ecj	33463	Mpa		
22) Modulus of Elasticity of Concrete for Girder (28 days)	Ec_2	34313	MPa		
23) Concrete Grade for Deck Slab	Fcu_2	45	MPa		
24) Overall depth of PSC composite girder	D	2.020	m		

B. Details of Prestressing Cables



Elevation of Cable

Plan of Cable

Cable No.	Strands per cable	Stage of Prestressing	L1 (L5+L6<=L1)	L2	L3	L4	P1	P2	L5	
1	18	1	2.200	3.000	0.000	12.000	0.000	0.000	0.000	2.200
2	18	1	2.200	3.000	0.000	12.000	0.000	0.000	0.000	2.200
3	17	1	4.700	3.000	0.000	9.500	0.000	0.000	0.000	4.700
4	18	1	5.200	4.000	0.000	8.000	0.000	0.000	0.000	5.200
5	18	1	5.700	5.000	0.000	6.500	0.000	0.000	0.000	5.700
6	0.0001	0	5.700	5.000	0.000	6.500	0.000	0.000	0.000	5.700
Cable No.	Strands per cable	Stage of Prestressing	E1	E2	Angle θ (degrees)	Angle α (degrees)	E3	Angle γ (degrees)	L6	E2+E3
1	18	1	0.265	0.107	4.099	0.000	0.135	0.000	0.000	0.242
2	18	1	0.265	0.107	4.099	0.000	0.135	0.000	0.000	0.242
3	17	1	0.485	0.117	4.475	0.000	0.315	0.000	0.000	0.432
4	18	1	0.705	0.195	5.597	0.000	0.495	0.000	0.000	0.690
5	18	1	0.925	0.281	6.444	0.000	0.675	0.000	0.000	0.956
6	0.0001	0	0.745	0.227	5.196	0.000	0.855	0.000	0.000	1.082

C. Force in Cables at nodal points after Friction & Slip Losses

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	1	0.000	2.206	5.208	5.208	17.208
$\Sigma\theta$ (rad)		0.00000	0.00000	0.07154	0.07154	0.07154
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.99560	0.97768	0.97768	0.95449
$P_x = P_o * Z$ (KN)		2479.95	2469.03	2424.59	2424.59	2367.09
P_x^1		2230.29	2241.20	2285.65	2285.65	2343.15

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	2	0.000	2.206	5.208	5.208	17.208
$\Sigma\theta$ (rad)		0.00000	0.00000	0.07154	0.07154	0.07154
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.99560	0.97768	0.97768	0.95449
$P_x = P_o * Z$ (KN)		2479.95	2469.03	2424.59	2424.59	2367.09
P_x^1		2230.29	2241.20	2285.65	2285.65	2343.15

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	3	0.000	4.714	7.717	7.717	17.217
$\Sigma\theta$ (rad)		0.00000	0.00000	0.07811	0.07811	0.07811
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.99062	0.97170	0.97170	0.95341
$P_x = P_o * Z$ (KN)		2342.18	2320.20	2275.88	2275.88	2233.05
P_x^1		2109.95	2131.93	2176.24	2176.24	2219.07

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	4	0.000	5.225	9.230	9.230	17.230
$\Sigma\theta$ (rad)		0.00000	0.00000	0.09769	0.09769	0.09769
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.98960	0.96554	0.96554	0.95022
$P_x = P_o * Z$ (KN)		2479.95	2454.17	2394.49	2394.49	2356.49
P_x^1		2228.87	2254.65	2314.33	2314.33	2352.34

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	5	0.000	5.736	10.744	10.744	17.244
$\Sigma\theta$ (rad)		0.00000	0.00000	0.11247	0.11247	0.11247
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.98859	0.96021	0.96021	0.94780
$P_x = P_o * Z$ (KN)		2479.95	2451.66	2381.26	2381.26	2350.50
P_x^1		2227.25	2255.54	2325.94	2325.94	2350.50

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	6	0.000	5.724	10.729	10.729	17.229
$\Sigma\theta$ (rad)		0.00000	0.00000	0.09068	0.09068	0.09068
Z= (exp)$^{-(\mu\Sigma\theta+k\Sigma x)}$		1.00000	0.98862	0.96380	0.96380	0.95135
$P_x = P_o * Z$ (KN)		0.01	0.01	0.01	0.01	0.01
P_x^1		-11.11	-11.11	-11.11	-22.23	-11.11

34.200

D. Force in Cable at chosen sections after Friction & Slip Losses

Cable No.	Notation	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
1	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	2.206	5.208	5.208	5.208
	x2	2.206	5.208	17.208	17.208	17.208
	y1	0.400	0.242	0.135	0.135	0.135
	y2	0.242	0.135	0.135	0.135	0.135
	y_{ord}	0.375	0.168	0.135	0.135	0.135
	θ	4.099	1.275	0.000	0.000	0.000
	p_{f1}	2479.950	2469.034	2424.586	2424.586	2424.586
	p_{f2}	2469.034	2424.587	2367.089	2367.089	2367.089
	p_{s1}	2230.285	2241.201	2285.649	2285.649	2285.649
	p_{s2}	2241.201	2285.649	2343.146	2343.146	2343.146
	$\mathbf{P_f}$	**2478.218**	**2438.394**	**2408.571**	**2388.088**	**2367.605**
	$\mathbf{P_s}$	**2232.018**	**2271.841**	**2301.664**	**2322.147**	**2342.631**
2	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	2.206	5.208	5.208	5.208
	x2	2.206	5.208	17.208	17.208	17.208
	y1	0.400	0.242	0.135	0.135	0.135
	y2	0.242	0.135	0.135	0.135	0.135
	y_{ord}	0.375	0.168	0.135	0.135	0.135
	θ	4.099	1.275	0.000	0.000	0.000
	p_{f1}	2479.950	2469.034	2424.586	2424.586	2424.586
	p_{f2}	2469.034	2424.587	2367.089	2367.089	2367.089
	p_{s1}	2230.285	2241.201	2285.649	2285.649	2285.649
	p_{s2}	2241.201	2285.649	2343.146	2343.146	2343.146
	$\mathbf{P_f}$	**2478.218**	**2438.394**	**2408.571**	**2388.088**	**2367.605**
	$\mathbf{P_s}$	**2232.018**	**2271.841**	**2301.664**	**2322.147**	**2342.631**
3	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	0.000	7.717	7.717	7.717
	x2	4.714	4.714	17.217	17.217	17.217
	y1	0.800	0.800	0.315	0.315	0.315
	y2	0.432	0.432	0.315	0.315	0.315
	y_{ord}	0.773	0.466	0.315	0.315	0.315
	θ	4.475	4.475	0.000	0.000	0.000
	p_{f1}	2342.175	2342.175	2275.885	2275.885	2275.885
	p_{f2}	2320.195	2320.195	2233.051	2233.051	2233.051
	p_{s1}	2109.946	2109.946	2176.236	2176.236	2176.236
	p_{s2}	2131.926	2131.926	2219.070	2219.070	2219.070
	$\mathbf{P_f}$	**2340.543**	**2322.244**	**2272.127**	**2252.852**	**2233.577**
	$\mathbf{P_s}$	**2111.577**	**2129.877**	**2179.993**	**2199.268**	**2218.544**

4	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	0.000	5.225	9.230	9.230
	x2	5.225	5.225	9.230	17.230	17.230
	y1	1.200	1.200	0.690	0.495	0.495
	y2	0.690	0.690	0.495	0.495	0.495
	y_{ord}	1.166	0.783	0.528	0.495	0.495
	θ	5.597	5.597	0.952	0.000	0.000
	p_{f1}	2479.950	2479.950	2454.170	2394.493	2394.493
	p_{f2}	2454.170	2454.170	2394.493	2356.486	2356.486
	p_{s1}	2228.874	2228.874	2254.654	2314.331	2314.331
	p_{s2}	2254.654	2254.654	2314.331	2352.338	2352.338
	P_f	**2478.223**	**2458.857**	**2404.621**	**2377.412**	**2357.102**
	P_s	**2230.601**	**2249.967**	**2304.203**	**2331.412**	**2351.722**

5	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	0.000	5.736	10.744	10.744
	x2	5.736	5.736	10.744	17.244	17.244
	y1	1.600	1.600	0.956	0.675	0.675
	y2	0.956	0.956	0.675	0.675	0.675
	y_{ord}	1.561	1.120	0.798	0.675	0.675
	θ	6.444	6.444	2.832	0.000	0.000
	p_{f1}	2479.950	2479.950	2451.661	2381.261	2381.261
	p_{f2}	2451.661	2451.661	2381.261	2350.505	2350.505
	p_{s1}	2227.247	2227.247	2255.536	2325.937	2325.937
	p_{s2}	2255.536	2255.536	2325.937	2350.505	2350.505
	P_f	**2478.224**	**2458.868**	**2412.106**	**2371.415**	**2351.187**
	P_s	**2228.974**	**2248.330**	**2295.092**	**2333.802**	**2349.960**

6	L_x^1	0.350	4.275	8.550	12.825	17.100
	x	**0.35**	**4.275**	**8.55**	**12.825**	**17.1**
	x1	0.000	0.000	5.724	10.729	10.729
	x2	5.724	5.724	10.729	17.229	17.229
	y1	1.600	1.600	1.082	0.855	0.855
	y2	1.082	1.082	0.855	0.855	0.855
	y_{ord}	1.568	1.213	0.954	0.855	0.855
	θ	5.196	5.196	2.812	0.000	0.000
	p_{f1}	0.014	0.014	0.014	0.013	0.013
	p_{f2}	0.014	0.014	0.013	0.013	0.013
	p_{s1}	-11.110	-11.110	-11.110	-22.231	-22.231
	p_{s2}	-11.110	-11.110	-11.109	-11.109	-11.109
	P_f	**0.014**	**0.014**	**0.013**	**0.013**	**0.013**
	P_s	**-11.110**	**-11.110**	**-11.109**	**-18.644**	**-11.329**

(*) Notations Used :

Σx (m) = Cumulative Length of Cable from jacking end in metres
$\Sigma\theta$ (rad) = Cumulative angle of deviation in radian from jacking end
P_o = Force at Jacking end before transfer (KN)
P_x = Force at the nodal points before transfer (KN)
P_x^1 = Force at the nodal points after transfer (KN)
L_x^1 = Cumulative Length of Cable from jacking end (in metres) at various sections (m)
P_f = Force at the critical sections before transfer (i,e before slip at anchorage) (KN)
P_s = Force at the critical sections after transfer (i,e after slip at anchorage)(KN)

E. Elongation Calculation Grip Length = **600** mm

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
P_x	1	2479.95	2469.03	2424.59	2424.59	2367.09
Σx (m)		0.000	2.206	5.208	5.208	17.208
Elongation (mm)		**122.1**				
P_x	2	2479.95	2469.03	2424.59	2424.59	2367.09
Σx (m)		0.000	2.206	5.208	5.208	17.208
Elongation (mm)		**122.1**				
P_x	3	2342.18	2320.20	2275.88	2275.88	2233.05
Σx (m)		0.000	4.714	7.717	7.717	17.217
Elongation (mm)		**122.3**				
P_x	4	2479.95	2454.17	2394.49	2394.49	2356.49
Σx (m)		0.000	5.225	9.230	9.230	17.230
Elongation (mm)		**122.2**				
P_x	5	2479.95	2451.66	2381.26	2381.26	2350.50
Σx (m)		0.000	5.736	10.744	10.744	17.244
Elongation (mm)		**122.3**				
P_x	6	0.01	0.01	0.01	0.01	0.01
Σx (m)		0.000	5.724	10.729	10.729	17.229
Elongation (mm)		**0.0**				

F. Horizontal & Vertical Component of Prestress Force

Section	Stage of Prestressing	Cable No.	No. of Cables	Y_{ord} (m)	θ (deg)	$P_s.\cos\theta$ (KN)	$P_s.\cos\theta.\ Y_{ord}$ (KN.m)	$P_s.\sin\theta$ (KN)
SUPPORT SECTION	1	1	0.95	0.375	4.099	2109.1	790.9	151.2
	1	2	0.95	0.375	4.099	2109.1	790.9	151.2
	1	3	0.89	0.773	4.475	1883.5	1455.4	147.4
	1	4	0.95	1.166	5.597	2103.1	2452.0	206.1
	1	5	0.95	1.561	6.444	2098.3	3274.9	237.0
	0	6	0.00	1.568	5.196	0.0	0.0	0.0
	Total Stage 1					**10303.3**	**8764.0**	**892.8**
	Eff. Eccentricity of Cable			**0.0768**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
1/8th SPAN SECTION	1	1	0.95	0.168	1.275	2151.7	362.2	47.88555853
	1	2	0.95	0.168	1.275	2151.7	362.2	47.88555853
	1	3	0.89	0.466	4.475	1899.9	886.2	148.691821
	1	4	0.95	0.783	5.597	2121.4	1661.1	207.9023948
	1	5	0.95	1.120	6.444	2116.5	2370.9	239.0583733
	0	6	0.00	1.120	6.444	0.0	0.0	-6.56253E-06
	Total Stage 1					**10441.3**	**5642.7**	**691.4**
	Eff. Eccentricity of Cable			**0.3578**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
1/4TH SPAN SECTION	1	1	0.95	0.135	0.000	2180.5	294.4	0.0
	1	2	0.95	0.135	0.000	2180.5	294.4	0.0
	1	3	0.89	0.315	0.000	1950.5	614.4	0.0
	1	4	0.95	0.52816	0.952	2182.6	1152.8	36.3
	1	5	0.95	0.798	2.832	2171.6	1733.4	107.4
	0	6	0.00	0.954	2.812	0.0	0.0	0.0
	Total Stage 1					**10665.8**	**4089.3**	**143.7**
	Eff. Eccentricity of Cable			**0.5148**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
3/8TH SPAN SECTION	1	1	0.95	0.135	0.000	2199.9	297.0	0.0
	1	2	0.95	0.135	0.000	2199.9	297.0	0.0
	1	3	0.89	0.315	0.000	1967.8	619.8	0.0
	1	4	0.95	0.495	0.952	2208.4	1093.2	36.7
	1	5	0.95	0.675	0.000	2211.0	1492.4	0.0
	0	6	0.00	0.855	0.000	0.0	0.0	0.0
	Total Stage 1					**10787.0**	**3799.4**	**36.7**
	Eff. Eccentricity of Cable			**0.5460**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
MID SPAN SECTION	1	1	0.95	0.135	0.000	2219.3	299.6	0
	1	2	0.95	0.135	0.000	2219.3	299.6	0
	1	3	0.89	0.315	0.000	1985.0	625.3	0
	1	4	0.95	0.495	0.000	2227.9	1102.8	0
	1	5	0.95	0.675	0.000	2226.3	1502.7	0
	0	6	0.00	0.855	0.000	0.0	0.0	0
	Total Stage 1					**10877.9**	**3830.1**	**0.0**
	Eff. Eccentricity of Cable			**0.5461**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				

(*) Notations Used :

Y_{ord} (m) = Vertical Ordinate of Cable from soffit of deck

θ (deg) = Cumulative angle of deviation in radian from jacking end

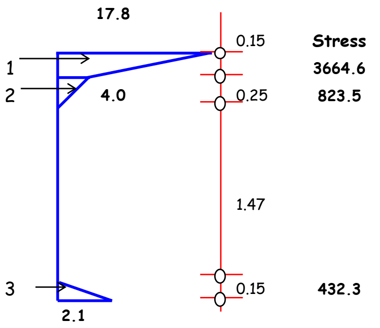
P_s = Force at the critical sections after transfer (i,e after slip at anchorage) (KN)

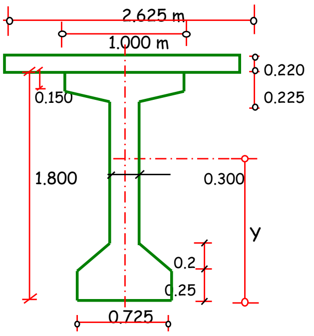
4.0 Effect of Temperature Gradient (As per Clause 218.3 of IRC:6 - 2010) - Outer Girder

			At Span	At Support	
Total Height of the girder	h	=	2.020	2.020	m
C.G. of Girder from bottom	Y_b	=	1.3163	1.2218	m
M.O.I. of the Section	I	=	0.6594	0.7746	m^4
Area of the Section	A	=	1.3975	1.9278	m^2
Modulus of Elasticity of Concrete	Ec	=	1.72E+07		KN/m^2
Coefficient of thermal expansion of concrete	α	=	1.20E-05		°C
Section Modulus at the top of Slab	Z_{TS}	=	0.9371	0.9704	m^3
Section Modulus at the top of Girder	Z_{TG}	=	1.3633	1.3395	m^3
Section Modulus at the bottom of Girder	Z_{BG}	=	0.5010	0.6340	m^3

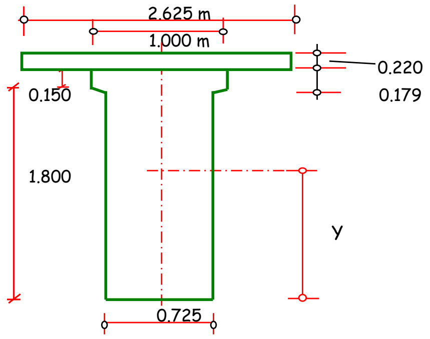
T_1	=	17.8	° C	h_1	=	0.15	m
T_2	=	4.0	° C	h_2	=	0.25	m
T_3	=	2.1	° C	h_3	=	0.15	m

4.1 COMPUTATION OF STRESSES DUE TO RISE IN TEMPERATURE





Section of Girder at Span



Section of Girder at Support

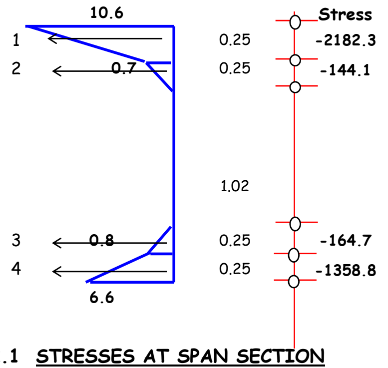
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	3664.6	2.625	883.60	0.059	0.645	569.49	3664.63	-780.40	-672.38	**2211.8**
2	0.070	823.5	2.625	130.14	0.183	0.521	67.75	823.51	-780.40	-462.17	**-419.1**
3	0.180	592.9	1.000	53.36	0.280	0.424	22.61	592.93	-780.40		
4	0.150	432.3	0.725	23.51	1.970	-1.266	-29.77	432.34	-780.40	1257.74	**909.7**
				1090.61			**630.08**				

4.1.2 STRESSES AT SUPPORT SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	3664.6	2.625	883.60	0.059	0.739	653.03	3664.63	-565.73	-771.50	**2327.4**
2	0.070	823.5	2.625	130.14	0.183	0.615	80.05	823.51	-565.73	-558.87	**-301.1**
3	0.180	592.9	1.000	53.36	0.280	0.518	27.65	592.93	-565.73		
4	0.150	432.3	0.725	23.51	1.313	-0.515	-12.11	432.34	-565.73	1180.86	**1047.5**
				1090.61			**748.63**				

4.2 COMPUTATION OF STRESSES DUE TO FALL IN TEMPERATURE



h	=	2.020	m				
h_1	=	0.250	m	T_1	=	10.6	°C
h_2	=	0.250	m	T_2	=	0.7	°C
h_3	=	0.250	m	T_3	=	0.8	°C
h_4	=	0.250	m	T_4	=	6.6	°C

4.2.1 STRESSES AT SPAN SECTION

								Stresses			
Segment	Height	Stress	b	Force	y from top	e	Moment	Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.220	-2182.3	2.625	-742.38	0.084	0.619	-459.73	-2182.31	655.74	336.02	**-1190.6**
2	0.030	-388.7	1.000	-7.99	0.233	0.471	-3.76	-388.70	655.74	230.97	**498.0**
3	0.195	-144.1	1.000	-17.14	0.327	0.377	-6.46	-144.11	655.74		
4	0.055	-31.7	0.300	-0.26	0.463	0.240	-0.06	-31.71	655.74		
5	0.250	-164.7	0.513	-10.55	1.687	-0.983	10.37	-164.70	655.74		
4	0.250	-1358.8	0.725	-138.07	1.752	-1.049	144.77	-1358.79	655.74	-628.55	**-1331.6**
				-916.39			**-314.88**				

2.2 STRESSES AT SUPPORT SECTION

								Stresses			
Segment	Height	Stress	b	Force	y from top	e	Moment	Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.220	-2182.3	2.625	-742.38	0.084	0.714	-529.92	-2182.31	477.82	409.91	**-1294.6**
2	0.030	-388.7	1.000	-7.99	0.233	0.566	-4.52	-388.70	477.82	296.94	**386.1**
3	0.195	-144.1	1.000	-17.14	0.327	0.472	-8.08	-144.11	477.82		
4	0.055	-31.7	0.725	-0.63	0.463	0.335	-0.21	-31.71	477.82		
5	0.250	-164.7	0.725	-14.93	1.687	-0.888	13.26	-164.70	477.82		
4	0.250	-1358.8	0.725	-138.07	1.752	-0.954	131.71	-1358.79	477.82	-627.41	**-1508.4**
				-921.14			**-397.76**				

5.0 Check for Flexural Stresses at Various Sections (Outer Girder)

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.275**	**8.550**	**12.825**	**17.100**
A. Section property of Girder only						
Area of the section, A	m^2	1.3503	0.8200	0.8200	0.8200	0.8200
Depth of the section, d	m	1.800	1.800	1.800	1.800	1.800
CG of section from bottom, Y_b	m	0.9274	0.8982	0.8982	0.8982	0.8982
Inertia of section, I_{x-x}	m^4	0.3817	0.3102	0.3102	0.3102	0.3102
Top Section Modulus, Z_t	m^3	0.4374	0.3440	0.3440	0.3440	0.3440
Bottom Section Modulus, Z_b	m^3	0.4116	0.3453	0.3453	0.3453	0.3453
B. Dead Load Moments	KNm	0.0	1311.9	2232.9	2779.5	2966.7
Stress at Top Fibre, σ_t	KN/m^2	0.0	3813.9	6491.5	8080.7	8624.9
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-3798.6	-6465.5	-8048.4	-8590.4
C. Prestress after 7 days	KN	10303.3	10441.3	10665.8	10787.0	10877.9
Eff. Eccentricity of Cable, **e**	m	0.077	0.358	0.515	0.546	0.546
CG of Tendons from Bottom Y_{ord}	m	0.851	0.540	0.383	0.352	0.352
Prestressing Factor (Top) $(1/A-e/Z_t)$	m^{-2}	0.56497	0.17938	-0.27711	-0.36778	-0.36813
Prestressing Factor (Bottom) $(1/A+e/Z_b)$	m^{-2}	0.92722	2.25548	2.71015	2.80045	2.80081
Stress at Top Fibre due to prestress	KN/m^2	5821.0	1872.9	-2955.6	-3967.2	-4004.5
Stress at Bottom Fibre due to prestress	KN/m^2	9553.4	23550.1	28906.0	30208.5	30466.9
Cumulative Stress at Top Fibre, σ_t	KN/m^2	**5821.0**	**5686.8**	**3535.8**	**4113.5**	**4620.3**
Cumulative Stress at Bottom Fibre, σ_b	KN/m^2	**9553.4**	**19751.5**	**22440.5**	**22160.1**	**21876.6**
D. Elastic Shortening Loss	KN	377.5	377.5	377.5	377.5	377.5
Stress after P-E loss						
Top	KN/m^2	5607.8	1805.2	-2851.0	-3828.4	-3865.6
Bottom	KN/m^2	9203.4	22698.7	27883.0	29151.3	29409.7
Cumulative Stress after E. S. loss						
Top Fibre, σ_t	KN/m^2	**5607.8**	**5619.1**	**3640.4**	**4252.3**	**4759.3**
Bottom Fibre, σ_b	KN/m^2	**9203.4**	**18900.1**	**21417.5**	**21103.0**	**20819.3**
Stress at CG of Tendon	KN/m^2	7504.2	14912.7	17630.9	17805.7	17677.8
Avg stress at CG of Tendon	KN/m^2	15735.1				
Check for loss due to elastic shortening	=	1/2 * 15735.1 * (195000/33463) * 98.7*83/1000000 KN				
	=	377.5				
E. Losses in prestress, 7-28days						
1) Relaxation loss (14-28 days)						
Prestressing force after E Loss	KN	9925.8	10063.8	10288.4	10409.5	10500.4
Initial Stress as % of UTS (f_p)	f_p	0.648	0.657	0.671	0.679	0.685
Relaxation loss for low relaxation steel at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.846	1.959	2.142	2.241	2.315
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	83.99				
Factor For time dependent loss		**1.0**				
Relaxation loss at mid-span for 7 **to 28**	KN	= 83.99/100 * 2.315/100 * (0.75*183.7*83) * 1				
days	KN	**178.2**	**189.1**	**206.8**	**216.3**	**223.5**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.275**	**8.550**	**12.825**	**17.100**
2) Shrinkage Loss (14-28 days)						
Residual Shrinkage Strain at	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Factor For time dependent loss		**1.0**				
Relaxation loss for 21 to 28 days	KN	=-(0.000034 - 0.000058) * 98.7 * 83 * 195000 * 1/1000				
	KN	**38.1**	**46.1**	**46.1**	**46.1**	**46.1**
3) Creep Loss (7-28 days)						
Creep Co-efficient at	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at	28 days	2.127	2.194	2.194	2.194	2.194
Factor For time dependent loss		**1.0**				
Creep Loss	KN	**170.5**	**175.8**	**175.8**	**175.8**	**175.8**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**386.8**	**411.0**	**428.6**	**438.2**	**445.3**
Stress after (R + S + C) loss						
Top	KN/m^2	-218.5	-73.7	118.8	161.2	163.9
Bottom	KN/m^2	-358.6	-926.9	-1161.7	-1227.1	-1247.3
Cumulative Stress after R + S + C loss						
Top Fibre, σ_t	KN/m^2	**5389.3**	**5545.4**	**3759.2**	**4413.5**	**4923.2**
Bottom Fibre, σ_b	KN/m^2	**8844.7**	**17973.2**	**20255.8**	**19875.9**	**19572.0**
Stress at CG of Tendon	KN/m^2	7211.8	14241.9	16742.0	16850.2	16706.6
Avg stress at CG of Tendon	KN/m^2	14948.3				
Check for loss due to creep of concrete	=	(2.426245 - 2.1275) * 14948.3/1000 * 98.7 * 83 * 195000 * 1 /1000 KN				
	=	170.5	175.8	175.8	175.8	175.8
F. Shuttering Load Moments	KNm	0.0	305.5	524.6	656.2	700.0
Stress at Top Fibre, σ_t	KN/m^2	0.0	888.0	1525.3	1907.6	2035.1
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-884.5	-1519.2	-1900.0	-2026.9
G. 2nd Stage DL Moments						
1) Sterss due to 2nd Stege DL	KNm	0.0	992.9	1714.4	2138.8	2287.4
Stress at Top Fibre, σ_t	KN/m^2	0.0	2886.7	4984.3	6218.1	6650.0
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-2875.1	-4964.3	-6193.2	-6623.4
2) Effect in prestress due to 2nd Stage DL						
Stersses at CG of Tendon	KN/m^2	0.0	-1145.2	-2845.3	-3764.6	-4027.0
Gain due to dead load of Deck Slab	KN	0.0	-54.9	-136.5	-180.6	-193.2
Stress at Top Fibre, σ_t	KN/m^2	0.0	9.9	-37.8	-66.4	-71.1
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	123.9	370.0	505.8	541.2
3) Stersses after 2nd Stage DL						
Stress at Top Fibre, σ_t	KN/m^2	**5389.3**	**9330.0**	**10231.0**	**12472.8**	**13537.2**
Stress at Bottom Fibre, σ_b	KN/m^2	**8844.7**	**14337.5**	**14142.2**	**12288.5**	**11462.8**
I. Section property of Composite Section						
Area of the section, A	m^2	1.9278	1.3975	1.3975	1.3975	1.3975
Depth of the section, d	m	2.020	2.020	2.020	2.020	2.020
CG of section from bottom, Y_b	m	1.2218	1.3163	1.3163	1.3163	1.3163
Inertia of section, I_{x-x}	m^4	0.7746	0.6594	0.6594	0.6594	0.6594
Top Section Modulus, Z_{tS}	m^3	0.9704	0.9371	0.9371	0.9371	0.9371
Top Section Modulus, Z_{tG}	m^3	1.3395	1.3633	1.3633	1.3633	1.3633
Bottom Section Modulus, Z_b	m^3	0.6340	0.5010	0.5010	0.5010	0.5010

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.275**	**8.550**	**12.825**	**17.100**
CG of Tendons from Bottom Y_{ord}	m	0.851	0.540	0.383	0.352	0.352
Revised Eff. Eccentricity of Cable, **e**	m	0.371	0.776	0.933	0.964	0.964
Prestressing Factor (Top) $(1/A-e/Z_{tS})$	m^{-2}	0.13622	-0.11241	-0.27997	-0.31325	-0.31338
Prestressing Factor (Top) $(1/A-e/Z_{tG})$	m^{-2}	0.24164	0.14645	0.03127	0.00840	0.00831
Prestressing Factor (Bottom) $(1/A+e/Z_b)$	m^{-2}	1.10418	2.26436	2.57779	2.64004	2.64029
J. Stress due to release of shuttering load						
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	-326.0	-559.9	-700.2	-747.0
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	-224.1	-384.8	-481.3	-513.5
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	609.7	1047.3	1309.8	1397.3
K. Stress after release of shuttering load						
Stress at Top of Deck, σ_{tS}	KN/m^2	**0.0**	**-326.0**	**-559.9**	**-700.2**	**-747.0**
Stress at Top of Girder, σ_{tG}	KN/m^2	**5389.3**	**9105.9**	**9846.1**	**11991.5**	**13023.7**
Stress at Bottom of Girder, σ_b	KN/m^2	**8844.7**	**14947.3**	**15189.5**	**13598.3**	**12860.1**
L. Losses in prestress, 28-49days						
1) Relaxation loss (28-49 days)						
Prestressing force after E Loss	KN	9925.8	10063.8	10288.4	10409.5	10500.4
Initial Stress as % of UTS (f_p)	f_p	0.648	0.657	0.671	0.679	0.685
Relaxation loss for low relaxation steel at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.846	1.959	2.142	2.241	2.315
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.3, IRC:112 - 2011)	%	12.81				
Factor For time dependent loss		**1.0**				
Relaxation loss at mid-span for **28 to 49**	KN	= 12.81/100 * 2.315/100 * (0.75*183.7*83) * 1				
days	KN	**27.2**	**28.8**	**31.5**	**33.0**	**34.1**
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at	49 days	0.00009	0.00010	0.00010	0.00010	0.00010
Factor For time dependent loss		**1.0**				
Relaxation loss for **28 to 49 days**	KN	=- (0.000058 - 0.000093) * 98.7 * 83 * 195000 * 1/1000				
	KN	**56.3**	**52.9**	**52.9**	**52.9**	**52.9**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.275**	**8.550**	**12.825**	**17.100**
3) Creep Loss (28-49 days)						
Creep Co-efficient at	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at	49 days	1.912	1.972	1.972	1.972	1.972
Factor For time dependent loss		**1.0**				
Creep Loss	KN	**101.2**	**104.4**	**104.4**	**104.4**	**104.4**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**184.7**	**186.1**	**188.8**	**190.2**	**191.3**
Stress after (R + S + C) loss						
Top of Deck	KN/m^2	-25.2	20.9	52.8	59.6	60.0
Top of Girder	KN/m^2	-44.6	-27.2	-5.9	-1.6	-1.6
Bottom of Girder	KN/m^2	-203.9	-421.3	-486.6	-502.2	-505.1
Cumulative Stress after R + S + C loss						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-25.2**	**-305.0**	**-507.0**	**-640.6**	**-687.0**
Stress at Top of Girder, σ_{tG}	KN/m^2	**5344.6**	**9078.7**	**9840.2**	**11989.9**	**13022.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**8640.8**	**14525.9**	**14702.9**	**13096.1**	**12355.0**
Stress at CG of Tendon	KN/m^2	7083.2	12890.5	13667.2	12879.6	12485.5
Avg stress at CG of Tendon	KN/m^2	12305.4				
Check for loss due to creep of concrete	=	(2.1275 - 1.912101) * 12305.4/1000 * 98.7 * 83 * 195000 * 1 /1000 KN				
	=	101.2	104.4	104.4	104.4	104.4
M. SIDL+Crash Barrier DL Moments	KNm	0.0	626.2	1025.8	1209.4	1181.5
1) Sterss due to SIDL						
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	668.2	1094.7	1290.6	1260.8
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	459.3	752.5	887.1	866.6
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	-1249.9	-2047.7	-2414.2	-2358.4
2) Effect in prestress due to SIDL						
Stersses at CG of Tendon	KN/m^2	0.0	-736.8	-1451.3	-1768.2	-1727.5
Gain due to SIDL	KN	0.0	-35.3	-69.6	-84.8	-82.9
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	-4.0	-19.5	-26.6	-26.0
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	5.2	2.2	0.7	0.7
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	80.0	179.5	224.0	218.8
3) Stersses after SIDL						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-25.2**	**359.2**	**568.2**	**623.4**	**547.8**
Stress at Top of Girder, σ_{tG}	KN/m^2	**5344.6**	**9543.1**	**10594.9**	**12877.7**	**13889.4**
Stress at Bottom of Girder, σ_b	KN/m^2	**8640.8**	**13356.1**	**12834.7**	**10905.9**	**10215.5**
O. Losses in prestress, 49days onwards						
1) Relaxation loss (49 days onwards)						
Prestressing force after E Loss	KN	9925.8	10063.8	10288.4	10409.5	10500.4
Initial Stress as % of UTS (f_p)	f_p	0.648	0.657	0.671	0.679	0.685
Relaxation loss at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.846	1.959	2.142	2.241	2.315
Time after prestressing	hr.	Infinity				
Residual Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.3, IRC:112 - 2011)	%	203.20				
Factor For time dependent loss		1				
Relaxation loss at mid-span for **49 days**	KN	= 203.2/100 * 2.315/100 * (0.75*183.7*83) * 1				
to Infinity	KN	**431.2**	**457.5**	**500.3**	**523.4**	**540.7**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.275**	**8.550**	**12.825**	**17.100**
2) Shrinkage Loss (49 days onwards)						
Residual Shrinkage Strain at	49 days	0.00009	0.00010	0.00010	0.00010	0.00010
Residual Shrinkage Strain at	infinity	0.00025	0.00027	0.00027	0.00027	0.00027
Factor For time dependent loss		1.0				
Relaxation loss for **49 days to Infinity**	KN	=- (0.000093 - 0.000247) * 98.7 * 83 * 195000 * 1/1000				
	KN	**246.8**	**280.1**	**280.1**	**280.1**	**280.1**
3) Creep Loss (49 days onwards)						
Creep Co-efficient at	49 days	1.912	1.972	1.972	1.972	1.972
Creep Co-efficient at	infinity	1.298	1.339	1.339	1.339	1.339
Factor For time dependent loss		1				
Creep Loss	KN	**220.4**	**227.3**	**227.3**	**227.3**	**227.3**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**898.4**	**964.9**	**1007.7**	**1030.8**	**1048.1**
Stress after (R + S + C) loss						
Top of Deck	KN/m^2	-122.4	108.5	282.1	322.9	328.5
Top of Girder	KN/m^2	-217.1	-141.3	-31.5	-8.7	-8.7
Bottom of Girder	KN/m^2	-992.0	-2184.9	-2597.7	-2721.3	-2767.3
Cumulative Stress after R + S + C loss i.e. at service without LL						
Top of Deck, σ_{tS}	KN/m^2	**-147.5**	**467.6**	**850.3**	**946.3**	**876.2**
Top of Girder, σ_{tG}	KN/m^2	**5127.5**	**9401.8**	**10563.3**	**12869.1**	**13880.7**
Bottom of Girder, σ_b	KN/m^2	**7648.8**	**11171.1**	**10237.0**	**8184.5**	**7448.2**
Stress at CG of Tendon	KN/m^2	6457.4	10639.9	10306.5	9101.2	8706.4
Avg stress at CG of Tendon	KN/m^2	9407.4				
Check for loss due to creep of concrete	=	(1.912101 - 1.298361)/10 * 9407.4/1000 * 98.7 * 83 * 195000 * 1 /1000 KN				
	=	220.4	227.3	227.3	227.3	227.3
P. Live Load Moments	KNm	0.0	1231.2	2105.4	2653.2	2908.2
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	1313.9	2246.7	2831.3	3103.4
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	903.1	1544.3	1946.2	2133.1
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	-2457.8	-4202.7	-5296.3	-5805.2
Cumulative Stress at Service with LL						
Top of Deck, σ_{tS}	KN/m^2	**-147.5**	**1781.5**	**3097.1**	**3777.7**	**3979.6**
Top of Girder, σ_{tG}	KN/m^2	**5127.5**	**10304.9**	**12107.7**	**14815.2**	**16013.9**
Bottom of Girder, σ_b	KN/m^2	**7648.8**	**8713.4**	**6034.3**	**2888.2**	**1643.0**
Q. Total time dependent Losses + Elastic Shortening Loss in cables						
	KN	**1847.3**	**1849.1**	**1796.4**	**1771.2**	**1786.1**
% Loss	%	**17.9**	**17.7**	**16.8**	**16.4**	**16.4**
R. Stress due to Differential Shrinkage & Creep						
Strain due to differential shrinkage and creep		**2.00E-04**				
Reduction factor due to creep		**0.43**				
Axial force **P**	KN	1704.2	1704.2	1704.2	1704.2	1704.2
Eccentricity **e**	m	0.688	0.594	0.594	0.594	0.594
Stress due to Shrinkage & Creep	KN/m^2	-858.2	-651.8	-651.8	-651.8	-651.8
	KN/m^2	1759.6	1961.5	1961.5	1961.5	1961.5
	KN/m^2	-966.0	-800.2	-800.2	-800.2	-800.2

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	0.350	4.275	8.550	12.825	17.100
1) Stress at service with Differential Shrinkage and Differential Creep effect (without Live Load)						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-1005.8**	**-184.2**	**198.5**	**294.5**	**224.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**6887.1**	**11363.3**	**12524.9**	**14830.6**	**15842.3**
Stress at Bottom of Girder, σ_b	KN/m^2	**6682.8**	**10371.0**	**9436.9**	**7384.3**	**6648.0**
2) Stress at service with Differential Shrinkage and Differential Creep effect (with Live Load)						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-1005.8**	**1129.7**	**2445.2**	**3125.8**	**3327.8**
Stress at Top of Girder, σ_{tG}	KN/m^2	**6887.1**	**12266.5**	**14069.2**	**16776.7**	**17975.4**
Stress at Bottom of Girder, σ_b	KN/m^2	**6682.8**	**7913.2**	**5234.1**	**2088.1**	**842.9**
S. Sterss due to Temperature Rise and Fall						
Stress due to Temperature Rise Case	KN/m^2	2327.4	2211.8	2211.8	2211.8	2211.8
	KN/m^2	-301.1	-419.1	-419.1	-419.1	-419.1
	KN/m^2	1047.5	909.7	909.7	909.7	909.7
Stress due to Temperature Fall Case	KN/m^2	-1294.6	-1190.6	-1190.6	-1190.6	-1190.6
	KN/m^2	386.1	498.0	498.0	498.0	498.0
	KN/m^2	-1508.4	-1331.6	-1331.6	-1331.6	-1331.6
1) Stress at Service without Live Load + Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**1321.6**	**2027.7**	**2410.3**	**2506.3**	**2436.2**
Stress at Top of Girder, σ_{tG}	KN/m^2	**6586.0**	**10944.3**	**12105.8**	**14411.5**	**15423.2**
Stress at Bottom of Girder, σ_b	KN/m^2	**7730.3**	**11280.7**	**10346.6**	**8294.0**	**7557.7**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-2300.4**	**-1374.7**	**-992.1**	**-896.1**	**-966.2**
Stress at Top of Girder, σ_{tG}	KN/m^2	**7273.1**	**11861.4**	**13022.9**	**15328.6**	**16340.3**
Stress at Bottom of Girder, σ_b	KN/m^2	**5174.4**	**9039.4**	**8105.3**	**6052.7**	**5316.4**
2) Stress at Service with 75% Live Load + Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**1321.6**	**3013.1**	**4095.4**	**4629.8**	**4763.8**
Stress at Top of Girder, σ_{tG}	KN/m^2	**6586.0**	**11621.6**	**13264.1**	**15871.2**	**17023.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**7730.3**	**9437.3**	**7194.5**	**4321.8**	**3203.8**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-2300.4**	**-389.3**	**693.0**	**1227.4**	**1361.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**7273.1**	**12538.7**	**14181.1**	**16788.2**	**17940.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**5174.4**	**7196.0**	**4953.2**	**2080.5**	**962.5**
2) Stress at Service with 100% Live Load + 60%Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**390.7**	**2456.8**	**3772.3**	**4452.9**	**4654.9**
Stress at Top of Girder, σ_{tG}	KN/m^2	**6706.4**	**12015.0**	**13817.8**	**16525.3**	**17724.0**
Stress at Bottom of Girder, σ_b	KN/m^2	**7311.3**	**8459.0**	**5779.9**	**2633.9**	**1388.7**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-1782.5**	**415.4**	**1730.9**	**2411.5**	**2613.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**7118.7**	**12565.3**	**14368.0**	**17075.6**	**18274.2**
Stress at Bottom of Girder, σ_b	KN/m^2	**5777.7**	**7114.3**	**4435.2**	**1289.1**	**43.9**

6.0 Summary Of Extreme Fiber Stress (Outer Girder)

STAGES	Stress at top of Deck		Stress at top of Girder		Stress at bottom of Girder		Allowable Stress in Slab		Allowable Stress in Girder		Status	
	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve		
Stress after 49 days before (SIDL i.e Crash Barrier & Surfacing)	-0.03	-0.69	13.02	5.34	14.70	8.64	21.60	-2.70	21.60	-2.98	OK	OK
Stress at service without LL	0.95	-0.15	13.88	5.13	11.17	7.45	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with LL	3.98	-0.15	16.01	5.13	8.71	1.64	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with Diff. Shr.& Creep (without LL)	0.29	-1.01	15.84	6.89	10.37	6.65	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with Diff. Shr.& Creep (with LL)	3.33	-1.01	17.98	6.89	7.91	0.84	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service without LL + Diff. Shr.& Creep + Temp rise	2.51	1.32	15.42	6.59	11.28	7.56	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service without LL + Diff. Shr.& Creep + Temp fall	-0.90	-2.30	16.34	7.27	9.04	5.17	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with75% LL + Diff. Shr.& Creep + Temp rise	4.76	1.32	17.02	6.59	9.44	3.20	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 75% LL + Diff. Shr.& Creep + Temp fall	1.36	-2.30	17.94	7.27	7.20	0.96	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 100% LL + Diff. Shr.& Creep + Temp rise with 60% temp. gradient	4.65	0.39	17.72	6.71	8.46	1.39	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 100% LL + Diff. Shr.& Creep + Temp fall with 60% temp. gradient	2.61	-1.78	18.27	7.12	7.11	0.04	21.60	-2.70	21.60	0.00	OK	OK

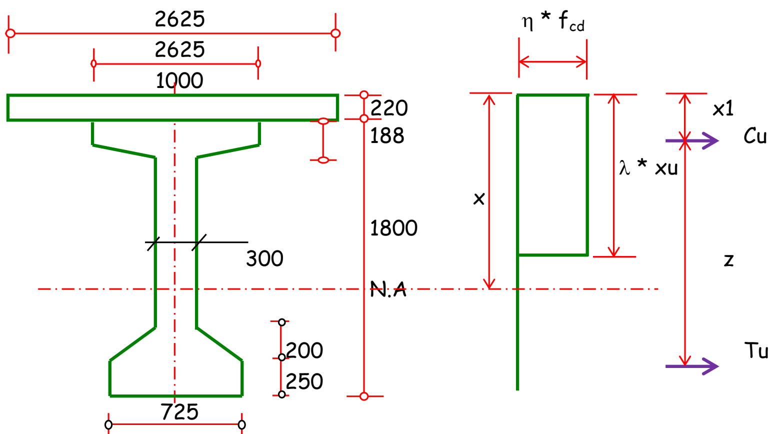
8.0 ULTIMATE MOMENT CHECK (ULS) FOR PSC T-GIRDER (OUTER) AS PER IRC : 112 - 2011 :-

Span Length (c/c of Brg.)		=	33.500	m
Design Parameters				
Grade of of Concrete of Longitudinal PSC Girder		=	M 45	MPa
Modulus of Elasticity of Concrete of PSC Girder	$E_{cm/g}$	=	34	GPa
Grade of of Concrete of RCC Deck Slab		=	M 45	Mpa
Modulus of Elasticity of Concrete of RCC Deck Slab	$E_{cm/d}$	=	34	GPa
Partial factor for Concrete	γ_m	=	1.5	
Longterm Strength factor for Concrete	α	=	0.67	
Characteristic yield strength of Reinforcement		=	Fe 500	Mpa
Modulus of Elasticity of Reinforcement Steel	E_s	=	200	GPa
Partial factor for Reinforcement Steel	γ_s	=	1.15	
Nomonal Diameter of Prestressing Strands		=	12.7	mm
Nominal Area of each Strands		=	98.7	mm^2
Nos of Strands		=	89	
Modulus of Elasticity of Prestressing Steel	E_p	=	195	GPa
Partial factor for Prestressing Steel	γ_p	=	1.15	
Charecteristic tensile strength of Prestressing steel	f_{pk}	=	1860	Mpa
Dia. of Untensioned Reinforcement	ϕ	=	12	mm
c.g of UntensionedReinforcement from soffit girder	c	=	58	mm

Using Rectangular Stress block,

Effective height factor	λ	=	0.8
Compression zone factor	η	=	1.0

Dimension of Inner Girder



GIRDER DIMENSIONS RECTANGULAR STRESS BLOCK

Bending Moment at Different Section (KN-m) for Inner Girder

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor
	0m	4.1875m	8.375m	12.5625m	16.75m	ULS
Moment due to 1st Stage Dead Load (DL)	0.0	1311.9	2232.9	2779.5	2966.7	1.35
Moment due to 2nd Stage Dead Load (DL)	0.0	992.9	1714.4	2138.8	2287.4	1.35
Moment due to SIDL (Crash Barrier)	0.0	297.0	452.0	479.0	383.7	1.35
Moment due to SIDL (Surfcaing/ Wearing coat)	0.0	274.3	478.2	608.7	664.8	1.75
Moment due to Pedestrian Live Load (LL)	0.0	0.0	0.0	0.0	0.0	1.5
Moment due to Live Load (LL)	0.0	1231.2	2105.4	2653.2	2908.2	1.5
Total Moment at ULS	**0.0**	**5839.3**	**9934.0**	**12331.5**	**13136.6**	

Prestressing Forces & Losess

Initial Prestress applied (KN)	10303	10441	10666	10787	10878
Total / Final Loses (KN)	1847	1849	1796	1771	1786
Final Prestress after all Loses (KN)	8456	8592	8869	9016	9092
c.g of Strands from Girder soffit (mm)	851	540	383	352	352

Material Data

Grade of Reinforcement Steel	Fe 500	Fe 500	Fe 500	Fe 500	Fe 500	
Charecteristic strength of Reinforcement, f_{yk} (Mpa)	500	500	500	500	500	Table 18.1, IRC 112
Partial factor for Reinforcement Steel, γ_s	1.15	1.15	1.15	1.15	1.15	Cl 6.2.2, IRC 112
Design value for Tensile Strength, $f_{yd} = f_{yk}/\gamma_s$ (Mpa)	434.8	434.8	434.8	434.8	434.78	Cl 6.2.2, IRC 112
Modulus of Elasticity of Reinforcement, Es (Gpa)	200	200	200	200	200	Cl 6.2.2, IRC 112
Grade of Concrete	M 45	M 45	M 45	M 45	M 45	
Charecteristics compressive strength, fck (Mpa)	45	45	45	45	45	Table 6.5, IRC 112
α	0.67	0.67	0.67	0.67	0.67	Cl 6.4.2.8, IRC 112
Concrete material factor, γ_m	1.5	1.5	1.5	1.5	1.5	Cl 6.4.2.8, IRC 112
Design value for concrete compressive strength, $f_{cd} = \alpha f_{ck}/\gamma_m$ (MPa)	20.1	20.1	20.1	20.1	20.1	Cl 6.4.2.8, IRC 112
Modulus of Elasticity of Concrete, E_{cm} (GPa)	34	34	34	34	34	Table 6.5, IRC 112
Modulus of Elasticity of Prestressing Steel, Ep (GPa)	195	195	195	195	195	
Charecteristic tensile strength of Prestressing steel, f_{pk} (Mpa)	1860	1860	1860	1860	1860	Cl 6.3.5, IRC 112
Partial factor for Prestressing Steel, γ_s	1.15	1.15	1.15	1.15	1.15	Cl 6.3.5, IRC 112
Design tensile strength of Prestressing steel, f_{pd} (MPa)	1407.1	1407.1	1407.1	1407.1	1407.1	Cl 6.3.5, IRC 112
Initial prestrain of prestressing steel	0.00494	0.00502	0.00518	0.00526	0.00531	

Geometric Data

Width of Bottom Flange of Girder, b (mm)	725	725	725	725	725
Total Depth of Girder, D (mm)	2020	2020	2020	2020	2020
c.g of Reinforcement from soffit girder, c (mm)	58	58	58	58	58
Dia. of Untensioned Reinforcement, ϕ (mm)	12	12	12	12	12
Effective depth of Girder, d (mm)	1962	1962	1962	1962	1962
Spacing of Reinforcement, s (mm) or	0	0	0	0	0
Number of Reinforcement Bar, n	6	6	6	6	6
Area of Untensioned Reinforcement, A_t (mm^2)	679	679	679	679	679
Number of Strands	89	89	89	89	89
Area of Single Strand (mm^2)	98.7	98.70	98.70	98.70	98.70
Total area of Strand (mm^2)	8784	8784	8784	8784	8784
c.g of Strands from soffit of Girder (mm)	851	540	383	352	352
Reinforcement (%)	0.0463	0.0463	0.0463	0.0463	0.0463

Ultimate Moment check

Position of N.A from Compression face, x (mm)	340	340	340	340	340
Total Compressive Force, C (N)	12655706	12655706	12655706	12655706	12655706
Total Tensile Force, T (N)	12655706	12655706	12655706	12655706	12655706
Difference, C-T	0	0	0	0	0
Strain in Concrete	0.0035	0.0035	0.0035	0.0035	0.0035
Strain in Reinforcement	0.01669	0.01669	0.01669	0.01669	0.01669
Strain in Prestressing Steel	0.01347	0.01674	0.01852	0.01892	0.01897
So, Stress in Reinforcement (MPa)	434.8	434.8	434.8	434.8	434.8
So, Stress in Prestressing Steel (MPa)	1407.1	1407.1	1407.1	1407.1	1407.1
Lever Arm from c/g of Reinforcement (mm)	1825.9	1825.9	1825.9	1825.9	1825.9
Lever Arm from c/g of Prestressing Steel (mm)	1033.3	1343.5	1500.5	1531.7	1531.8
Tensile Force in Reinforcement (KN)	295.0	295.0	295.0	295.0	295.0
Tensile Force in Prestressing Steel (KN)	12360.7	12360.7	12360.7	12360.7	12360.7
Moment of Resistance of the Section (KN-m)	13311	17145	19086	19472	19473
Status	**OK**	**OK**	**OK**	**OK**	**OK**

9.0 CHECK FOR SHEAR (ULS) FOR PSC T-GIRDER (OUTER) AS PER IRC : 112 - 2011 :-

Span Length (c/c of Bearing)		=	33.500	m		
Grade of Concrete of Girder	f_{ck}	=	45	Mpa		
Axial Tensile Stress of Concrete	$f_{ctk,0.05}$	=	2.5	Mpa		(Table 6.5 of IRC: 112 - 2011)
Longterm Strength factor for Concrete	α	=	0.67			
Partial factor for Concrete	γ_m	=	1.5			
Characteristic strength of Steel	f_{yk}	=	500	Mpa		
Partial factor for Reinforcement Steel	γ_s		1.15			
Nomonal Diameter of Prestressing Strands		=	12.7	mm		
Nominal Area of each Strands		=	98.7	mm^2		
Nos of Strands		=	89		8784.31	

Shear at Different Section (KN) for Inner Girder

Location	d' away from Support	L/8	L/4	3L/8	L/2	Factor ULS	As per table 3.2 of IRC 6-2014
Distance from Support in (m)	2.00	4.188	8.375	12.563	16.750		
Loadings							
1st Stage DL (KN)	378.5	262.9	173.5	87.6	0.0	1.35	
2nd Stage DL (KN)	271.1	205.2	134.3	68.4	0.0	1.35	
SIDL (Crash Barrier) (KN)	94.0	76.3	43.8	7.3	26.7	1.35	
SIDL (Surfcaing/ Wearing coat) (KN)	76.3	67.8	47.2	26.3	5.1	1.75	
Carriageway LL (KN)	0.0	0.0	0.0	0.0	0.0	1.5	
Carriageway LL (KN)	339.9	336.6	289.9	223.7	128.3	1.5	

Total loss in Prestress	1847.3	1849.1	1796.4	1771.2	1786.1

Shear Resistance (refer Cl. 10.3.2 of IRC:112-2011)

Location	d' away from Support	L/8	L/4	3L/8	L/2	Remarks
Ultimate Shear, V_u (KN)	1647.3	1358.6	992.1	602.1	237.4	
1. Elements not requiring Design Shear Reinforcement						
a) Effective Width, b_{wc} (m)	0.68	0.255	0.255	0.255	0.255	= web thk. - 0.5 x dia of duct.
b) Overall Depth, D (m)	2.020	2.020	2.020	2.020	2.020	= Depth of Girder + Depth of Deck Slab
c) CG of Tendons from Bottom Y_{ord} (m)	0.851	0.540	0.383	0.352	0.352	
d) Depth , d_b (m)	1.169	1.480	1.637	1.668	1.668	d_b = D - Y_{ord} of cable from soffit of girder
e) Axial Tensile Strength of Concrete, $f_{ctk,005}$ (Mpa)	2.50	2.50	2.50	2.50	2.50	Ref. Cl 10.3.2, IRC:112
f) Partial Safety factor fo Concrete, γ_m	1.50	1.50	1.50	1.50	1.50	
g) Partial Safety factor fo Steel, γ_m	1.15	1.15	1.15	1.15	1.15	
h) Design Tensile Strength of Concrete, f_{ctd} (Mpa)	1.67	1.67	1.67	1.67	1.67	= f_{ctd}/γ_m, where γ_m = 1.5
i) Applied Longitudinal Force,N_{ED} (KN)	8455.9	8592.1	8869.4	9015.8	9091.8	= Axial Prestressing Force - Total loss due to prestress
j) Compressive Stress due to prestress, σ_{cp} (Mpa)	4.02	4.02	4.02	4.02	4.02	Min. value of N_{ED}/Ac or 0.2 x fcd
k) Second Moment of Area, I (m^4)	0.7746	0.6594	0.6594	0.6594	0.6594	Calculated from section property
l) First Moment of Area, S (m^3)	0.5411	0.4290	0.4290	0.4290	0.4290	Considered from section property
m) Value of k_1	1.0	1.0	1.0	1.0	1.0	Ref. Cl 10.3.1, IRC:112
n) Effect of Vertical Prestress, V_{pd} (KN)	697.8	542.62	114.59	29.48	0.00	= Vertical Prestressing Force x (1 - Ratio of loss due to prestress with Applied lobgitudinal Force)
o) Area of Prestressing Steel, A_s(mm^2)	8784	8784	8784	8784	8784	
p) Value of K	1.41	1.37	1.35	1.35	1.35	Ref. Cl 10.3.1, IRC:112
q) $v_{min} = 0.031 K^{3/2} fck^{1/2}$	0.349	0.333	0.326	0.325	0.325	Eq. 10.3
r) % of Prestressing Steel, ρ_1	0.011	0.023	0.021	0.021	0.021	= Area of Prestressing Steel/(b_{wc} x d)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk,0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk,0.5}/\gamma_m$	-1.67	-1.67	-1.67	-1.67	-1.67	- denotes for "tensile stress"
u) Flexural tensile strength (Mpa)	5.778	7.114	4.435	1.289	0.044	Flexural Tensile strength in service condition
v) Shear Zone	B	B	C	C	D	Applicable for Shear Zone as per fig. 10.1(a) of IRC:111
w) Is flexural tensile strength under max. BM is smaller than $f_{ctk0.5}/\gamma_m$	No	No	No	No	No	
x) Shear Resistance, V_{Rdc} = $(I*b_{wc}/s)*((f_{ctd})^2+k1*\sigma_{cp}*f_{ctd})^{0.5}$	2996.6	1206.8	1206.8	1206.8	1206.8	Ref. Eq. 10.4 of, IRC:113
y) Shear Resistance, $V_{Rdc} = [0.12*K*(80*\rho_1*f_{ck})^{0.33} + 0.15*\sigma_{cp}]\ b_w * d$	934.3	494.5	533.6	541.2	541.3	Ref. Cl 10.3.2, IRC:112
z) Design Shear resistance	934.3	494.5	533.6	541.2	541.3	
aa) Min. Conc. Shear capacity $v_{Rd.c}$ min = $(v_{min} + 0.15 * \sigma_{cp})\ b_w * d$	757.4	353.0	387.7	394.6	394.6	Ref. Eq. 10.1 of IRC:112
Governing Values of, V_{Rdc}	**934.3**	**494.5**	**533.6**	**541.2**	**541.3**	

2. Elements requiring Design Shear Reinforcement

a) **Net Design Shear Force, V_{NS}(KN)**	**949.5**	**815.9**	**877.5**	**572.7**	**237.4**	**V_{NS} = Vu - Vpd**
b) Design yeild strength of Shear Reinforcement	347.8	347.8	347.8	347.8	347.8	
c) cotθ	2.50	2.50	2.50	2.50	2.50	Assumed value
d) Lever arm, z	1.17	1.48	1.64	1.67	1.67	= Difference between cg of tension force and compression force
e) Steel Shear capacity VRd.s = (Asw / s) z fywd cot θ, KN	2726.0	3449.1	3815.2	2186.9	2187.1	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity VRd.max = αcw*b*z*v1*[fcd/(cot θ + Tan θ)], KN	3968.3	1882.8	2082.6	2122.3	2122.5	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of VRd.s & VRd.max	2726.0	1882.8	2082.6	2122.3	2122.5	
h) Design value for concrete compressive strength (fcd = αfck/γm), Mpa	20.1	20.1	20.1	20.1	20.1	
i) v1 =	0.60	0.60	0.60	0.60	0.60	as fck is less than 80 Mpa
j) αcw =	1.20	1.20	1.20	1.20	1.20	
m) Effective Width, b_{wc} (m)	0.68	0.255	0.255	0.255	0.255	
n) Depth , d (m)	1.169	1.480	1.637	1.668	1.668	
o) Dia. of Shear reinforcement (mm)	16	16	16	12	12	
p) No. of Links	2	2	2	2	2	
t) Spacing of Reinforcement	150	150	150	150	150	
r) Minimum shear reinf. ρ_{min} (mm^2/m)	768	364	403	411	411	Ref. Cl 10.3.3.5, IRC:112
t) **Shear Reinforcement, Asw**=	**402**	**402**	**402**	**226**	**226**	in mm2
u) V_{RDC} + V_{RDS}	**3660**	**2377**	**2616**	**2664**	**2664**	
v) Is (V_{RDC} + V_{RDS}) > V_{NS}	***OK***	***OK***	***OK***	***OK***	***OK***	
w) Actual shear reinf. provided(mm^2/m)	2681	2681	2681	1508	1508	
x) Is Shear reinf. Pro. > Min. shear reinf. ρ_{min}	***OK***	***OK***	***OK***	***OK***	***OK***	
	3.9	2.9	3.0	4.7	11.2	
	2.2	1.7	2.6	4.4	11.2	

10.0 DESIGN OF SHEAR CONNECTOR FOR OUTER GIRDER

(Refer Clause 10.3.4 of IRC : 112 - 2011)

Span Length (c/c of Bearing)		=	33.500	m
Grade of Concrete	f_{ck}	=	45	Mpa
Characteristic Strength of Steel	f_{yk}	=	500	Mpa

Shear at Different Section (KN) for Inner Girder

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Factor ULS
Distance from Support in (m)	2.000	4.188	8.375	12.563	16.750	
Loadings						
1st Stage DL (KN)	378.5	262.9	173.5	87.6	0.0	1.35
2nd Stage DL (KN)	271.1	205.2	134.3	68.4	0.0	1.35
SIDL (Crash Barrier) (KN)	94.0	76.3	43.8	7.3	26.7	1.35
SIDL (Surfcaing/ Wearing coat) (KN)	76.3	67.8	47.2	26.3	5.1	1.75
Carriageway LL (KN)	0.0	0.0	0.0	0.0	0.0	1.5
Carriageway LL (KN)	339.9	336.6	289.9	223.7	128.3	1.5

A. Check for Limiting Shear

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Remarks
1. Ultimate Shear, V_{ED} (KN)	1647.3	1358.6	992.1	602.1	237.4	
2. Thk. Of Deck Slab (mm)	220	220	220	220	220	
3. Depth, d_b (m)	1.169	1.480	1.637	1.668	1.668	

B. Check for Interface Shear Reinforcement

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Remarks
1. Ultimate Shear Capacity of Section uncracked in Bending (refer Cl. 10.3.2 of IRC:112-2011):-						
a) Width of Interface, b_i (mm)	1000	1000	1000	1000	1000	
b) Lever Arm, z (mm)	1059	1370	1527	1558	1558	
c) β	1.0	1.0	1.0	1.0	1.0	
d) Interface Shear Stress, V_{EDi} (Mpa)	1.555	0.992	0.650	0.387	0.152	
e) Surface Factor, μ	0.6	0.6	0.6	0.6	0.6	
f) Angle of Reinf. To the Interface, α	90	90	90	90	90	
g) $f_{yd,}$ (MPa)	400	400	400	400	400	
h) Max Co-existing Normal Stress, σ_n (Mpa)	0.000	0.000	0.000	0.000	0.000	
i) Required Reinforcement, ρ (%)	0.00648	0.00413	0.00271	0.00161	0.00063	
j) Minimum Reinforcement, ρ (%)	0.0015	0.0015	0.0015	0.0015	0.0015	
k) Required Area of Interface Reinforcement (mm^2/m)	6479	4133	2708	1611	1500	
l) Dia. of Shear reinforcement (mm)	16	16	16	12	12	Provide at web of Girder
m) No. of Legs	2	2	2	2	2	
n) Addl.Shear reinforcement dia (mm)	16	16	12	12	12	Provide at top of Girder
o) No. of Legs	4	2	2	2	2	
p) Spacing required (mm)	186	195	232	281	302	
q) Spacing provided (mm)	150	150	150	150	150	
r) Check, Spacing pro. > Spacing req.	**OK**	**OK**	**OK**	**OK**	**OK**	

DESIGN OF 37.0M PSC I-GIRDER

Introduction

The following design note pertains to the details design of longitudinal PSC T girders type superstructure. The analysis is done with Grillage analogy using STAAD pro. software. Design parameters used in the Grillage model & details design of longitudinal PSC T girder has been presented in the following pages.

1.0 Design Parameters

1)	Str. No./ Design Chainage of Structure	=	Minor Bridge		
2)	Crossing/ Skew angle	=	0.0	Deg.	
3)	Span Arrangement	=	1 x 37.0	m	
4)	c/c of Expansion joints	=	37.000	m	
5)	c/c of Bearing - Exp. joint on Abutment side	=	0.750	m	
6)	c/c of Bearing - Exp. joint on Pier side	=	0.750	m	
7)	c/c of Bearings (Brg.)	=	35.500	m	

1.1 Geometry:

1)	Carriageway Width	=	12.500	m	
2)	Overall Width	=	13.500	m	
3)	Width of Crash Barrier, on Median side	=	0.500	m	
	on Outer side	=	0.500	m	
	on Inner side of Foothpath/ Kerb	=	0.000	m	
4)	Width of Foothpath/ Footway Kerb	=	0.000	m	(On one side)
5)	Cross Slope	=	2.5 %		
6)	Thickness of Wearing Coat	=	65	mm	
	For design purpose considering future overlay	=	100	mm	
7)	c/c Spacing of Longitudinal girders	=	2.750	m	
8)	No's of Longitudinal girder	=	5		
9)	Cantilever overhang, on Median side	=	1.250	m	on G-I side
	on Outer side		1.250	m	on G-V side
10)	No's of Intermediate Cross girder	=	1		
11)	Total Depth of Superstructure	=	2.220	m	
12)	Depth of Deck Slab	=	0.220	m	
13)	Depth of Longitudinal girder	=	2.000	m	
14)	Width of Intermediate Cross girder/ Diaphram	=	0.300	m	
15)	Width of End Cross girder/ Diaphram	=	0.400	m	
16)	Girder length beyond c/l of Brg. on each side	=	0.450	m	

1.2 Material Properties:

1)	Concrete Grade for Deck slab	=	M 45	
2)	Concrete Grade for Cross girder	=	M 45	
3)	Concrete Grade of PSC Longitudinal girder	=	M 45	
4)	Reinforcement	=	Fe 500	
5)	Clear Cover	=	40	mm
6)	Condition of Exposure	=	Moderate	
7)	Cable/ Strands	=	19 T 13 conforming to IS 14268 (Stress-relieved multiply strands of low relaxat with Corrugated HDPE sheathing for prestress	

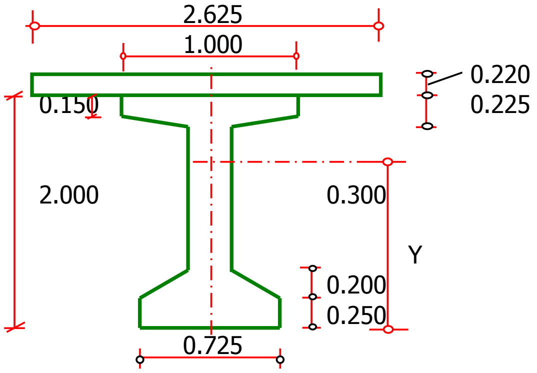
1.3 Proposed construction sequence

1)	Casting of PSC Longitudinal girder	=	0	Days
2)	1st stage prestressing	=	14	Days
3)	2nd stage prestressing	=	28	Days
4)	Casting of deck slab & Cross girders	=	35	Days
5)	Removal of shuttering	=	49	Days
6)	Casting of Crash barrier, Wearing coat etc	=	56	Days
7)	Live load	=	63	Days

1.0 Section Properties of Outer Longitudinal Girder OG (G-I to G-V)

c/c spacing of Bearing	=	35.500 m
c/c spacing of Longitudinal Girder	=	2.750 m
c/c spacing of Cross Girder	=	17.750 m
Distance between c/l of Brg. and c/l of Exp. joint	=	0.750 m
Cantilever overhang length	=	1.250 m
Depth of Longitudinal girder	=	2.000 m
Thickness of Deck slab	=	0.220 m
Thickness of Deck slab at Expansion joint	=	0.400 m
Grade of Concrete of Deck Slab	=	45 Mpa
Grade of Concrete of Longitudinal girder	=	45 Mpa
Length of Solid Portion	=	1.500 m
Length of Splayed Portion	=	1.500 m

Girder Section at the Span



Calculation of Torsional Constant				
Component	Width	Depth	Area	Iz
Top Slab	2.625	0.220	0.578	0.0047
Top Flange	0.883	0.225	0.199	0.0028
Web	1.325	0.300	0.398	0.0102
Bottom flang	0.631	0.450	0.284	0.0114
			Iz =	0.0291

Modulus of Elasticity of Deck Slab	=	34313 MPa
Modulus of Elasticity of Girder	=	34313 MPa
beff = Σ beff,i + bw	=	7.865 m
c/c Spacing of Longitudinal girder, b	=	2.625 m

SL.No.		A	y	Ay	Ay^2	I_o	$I_o + Ay^2$	u
1. Deck Slab		0.57750	2.11000	1.21853	2.57109	0.002329	2.57342	4.690
2. Rctangular top Flange		0.15000	1.92500	0.28875	0.55584	0.000281	0.55613	0.300
3. Triangular top Flange		0.02625	1.82500	0.04791	0.08743	0.000008	0.08744	0.026
4. Web + Top & Bottom Rect. Haunch		0.48000	1.05000	0.50400	0.52920	0.102400	0.63160	2.650
5. Bottom Bulb, Triangle		0.04250	0.31667	0.01346	0.00426	0.000094	0.00436	0.043
6. Bottom Bulb, Rectangle		0.18125	0.12500	0.02266	0.00283	0.000944	0.00378	1.225
Composite Section	Σ	1.45750		2.09530	3.7507	0.106	3.85671	8.934
Girder Only	Σ	0.88000		0.87677	1.1796	0.104	1.28329	4.244
		ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma(I_o + Ay^2)$	Σu

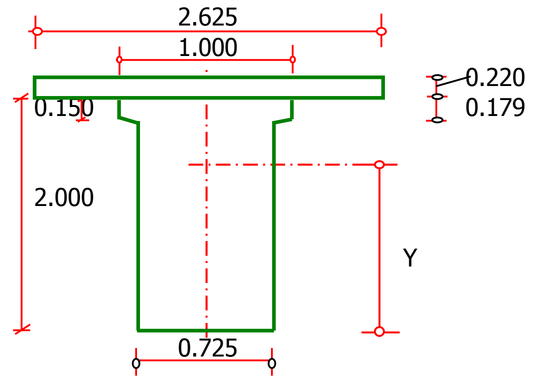
Section Properties of Composite Section

Area	ΣA	=	1.4575 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.4376 m
Moment of inertia of composite girder	**Iz = $\Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	0.8445 m^4

Section Properties of Girder

Area	ΣA	=	0.8800 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	0.9963 m
Moment of inertia of girder	**Iz = $\Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	0.4097 m^4

Girder Section at the Support



Calculation of Torsional Constant				
Component	Width	Depth	Area	Iz
Top Slab	2.625	0.220	0.578	0.0047
Top Flange	0.977	0.179	0.175	0.0009
Web	1.371	0.725	0.994	0.1224
Bottom flang	0.725	0.450	0.326	0.0143
			Iz =	0.1423

Modulus of Elasticity of Deck Slab		=	34313 MPa
Modulus of Elasticity of Girder		=	34313 MPa
beff =	Σ beff,i + bw	=	8.205 m
c/c Spacing of Longitudinal girder, b		=	2.625 m

SL.No.		**A**	**y**	**Ay**	**Ay^2**	**I_o**	**$I_o + Ay^2$**	**u**
1. Deck Slab		0.57750	2.11000	1.21853	2.57109	0.002329	2.57342	4.690
2. Rctangular top Flange		0.15000	1.92500	0.28875	0.55584	0.000281	0.55613	0.300
3. Triangular top Flange		0.00405	1.84018	0.00746	0.01372	0.000000	0.01372	0.004
4. Web + Top Rect. Haunch		1.34125	0.92500	1.24066	1.14761	0.382536	1.53014	3.641
Composite Section	Σ	2.07280		2.75539	4.28826	0.38515	4.67340	8.635
Girder Only	Σ	1.49530		1.53686	1.71717	0.38282	2.09999	3.945
		ΣA		ΣAy	ΣAy^2	ΣI_o	$\Sigma(I_o + Ay^2)$	Σu

Section Properties of Composite Section

Area	**ΣA**	=	2.0728 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.3293 m
Moment of inertia of end intermediate girder	**$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	1.0107 m^4

Section Properties of Girder

Area	**ΣA**	=	1.4953 m^2
Distance of cg from bottom fibre (Y)	**Y =Σ(A.y) / ΣA**	=	1.0278 m
Moment of inertia of end intermediate girder	**$Iz = \Sigma(Io+A.y^2)-\Sigma A.Y^2$**	=	0.5204 m^4

1.1 Summary of Section Properties of Longitudinal Girder (Outer)

A. Section Properties of Composite Section at the Span

Area	**1.4575** m^2	**Y_b =**	**1.4376** m	**Y_{ts} =**	**0.7824** m
I_z =	**0.8445** m^4	**Z_b =**	**0.5875** m^3	**Z_{ts} =**	**1.0794** m^3
				Y_{tG} =	**0.5624** m
				Z_{tG} =	**1.5016** m^3

B. Section Properties of Girder at the Span

Area	**0.8800** m^2	**Y_b =**	**0.9963** m	**Y_t =**	**1.0037** m
I_z =	**0.4097** m^4	**Z_b =**	**0.4112** m^3	**Z_t =**	**0.4082** m^3

C. Section Properties of Composite Section at the Support

Area	**2.0728** m^2	**Y_b =**	**1.3293** m	**Y_{ts} =**	**0.8907** m
I_z =	**1.0107** m^4	**Z_b =**	**0.7603** m^3	**Z_{ts} =**	**1.1347** m^3
				Y_{tG} =	**0.6707** m
				Z_{tG} =	**1.5069** m^3

D. Section Properties of Girder at the Support

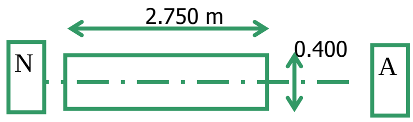
Area	**1.4953** m^2	**Y_b =**	**1.0278** m	**Y_t =**	**0.9722** m
I_z =	**0.5204** m^4	**Z_b =**	**0.5063** m^3	**Z_t =**	**0.5353** m^3

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphram Members

c/c spacing of Bearing		=	35.500 m	
c/c spacing of Longitudinal girder		=	2.750 m	
c/c spacing of Cross girder		=	17.750 m	
Distance between c/l of Brg. and c/l of Exp. joint		=	0.750 m	0.750
Cantilever length on Median side (G-I side)		=	1.250 m	
Cantilever length on Outer side (G-IV side)		=	1.250 m	
Depth of Long girder		=	2.000 m	
Thickness of Deck slab		=	0.220 m	
Depth of Cross girder		=	1.750 m	
No's of Longitudinal girders		=	5	
Thickness of Deck slab at Exp. joint		=	0.400 m	
Web thickness of Inner Cross girder		=	0.300 m	
Web thickness of External Cross girder		=	0.400 m	
Skew distance between c/c of bearing	0.0	=	2.750 m	
Length of Solid Portion		=	1.500 m	
Length of Splayed Portion		=	1.500 m	
Grade of Concrete of Deck Slab		=	45 Mpa	
Grade of Concrete of Girder		=	45 Mpa	
Modular Ratio		=	1.000	
Effective width of Slab for Inner Long. girder		=	2.750 m	
Effective width of Slab for Outer Long. girder		=	2.625 m	

Cantilever Slab Inner (At Expansion End) **_LONGCANTIN**

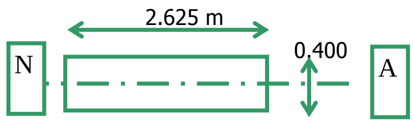
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.1000 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0147 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab Outer (At Expansion End) **_LONGCANTOUT**

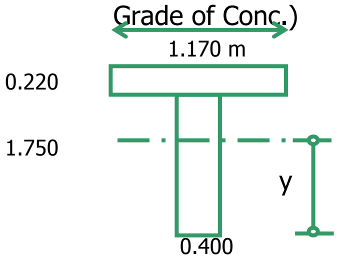
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.0500 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0140 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

End Cross Girder **_ENDCG**

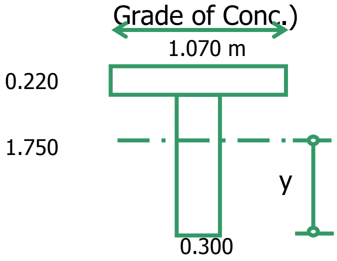
(with reduced width for diff. in Grade of Conc.)



beff = Min{ Σ beff,i + bw, b}	=	1.170 m
Modulus of Elasticity ratio	=	1.000
Area	=	0.9574 m^2
Distance of cg from bottom fibre (y)	=	1.1398 m
Moment of inertia of end x-girder (Iz)	=	0.3623 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate/ Inner Cross Girder **_INTCG**

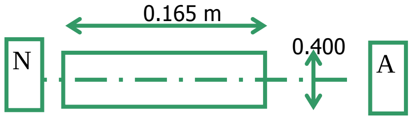
(with reduced width for diff. in Grade of Conc.)



beff = Min{ Σ beff,i + bw, b}	=	1.070 m
Modulus of Elasticity ratio	=	1.000
Area	=	0.7604 m^2
Distance of cg from bottom fibre (y)	=	1.1799 m
Moment of inertia of end x-girder (Iz)	=	0.2926 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At Expansion End) **_EXPTRANSLAB**

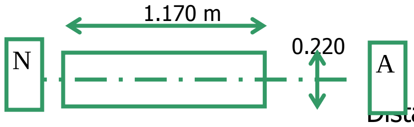
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.0660 m^2
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0009 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At End Cross Girder) **_TRANSCANT1**

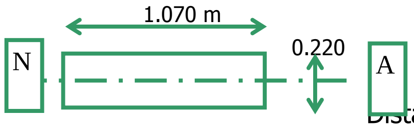
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.2574 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0010 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Cantilever Slab (At Inner Cross Girder) **_TRANSCANT2**

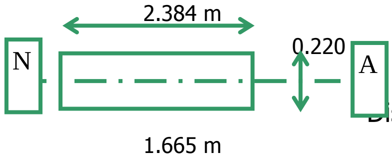
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.2354 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0009 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Next to End Cross Girder) **_TRANSLAB1**

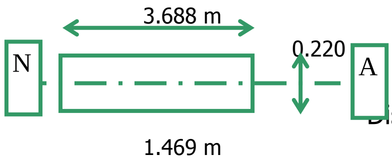
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.5244 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0021 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (At Splayed/ Tapered Portion) **_TRANSLAB2**

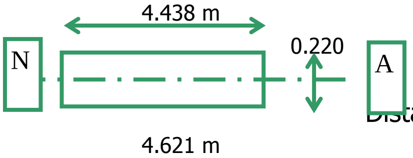
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.8113 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0033 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Adjacent to First Inner Cross Girder) **_TRANSLAB3**

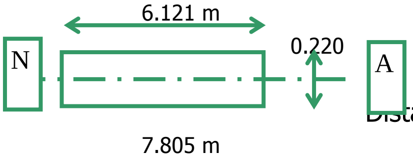
(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	0.9763 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0039 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

Intermediate Slab (Between Inner Cross Girders) **_TRANSLAB4**

(with reduced width for diff. in Grade of Conc.)



Modulus of Elasticity ratio	=	1.000
Area	=	1.3467 m^2
Distance of cg from bottom fibre (y)	=	0.1100 m
Moment of inertia (I_z)	=	0.0054 m^4
Moment of inertia girder (Ix)	=	0.0001 m^4

1.2 SIDL and Footpath Load Calculation

Span length c/c of Expansion Joint	=	37.000	m	
Span length c/c of Bearing	=	35.500	m	
Skew Angle	=	0.0	Deg.	
Depth of Wearing Coat assumed for Design	=	0.100	m	
Density of Wearing Coat material	=	22.0	kn/m^3	
Density of footpath material	=	25.0	kn/m^3	
Weight of Crash Barrier (0.9 m height)per m run	=	8.5	kn/m	
Weight of Railing (1.1 m height) per m run	=	8.5	kn/m	
Width of Crash Barrier	=	0.50	m	
Width of Railing on both outer side of Bridge	=	0.50	m	
Width of Footpath on both side	=	0.00	m	
Thickness of Footpath	=	0.00	m	
Basic Footpath Live Load	=	400	kg/m^2	
Actual Footpath Live Load	=	2.76	kn/m^2	Cl. 206.3.(b) IRC 6-201
Self weight of Footpath	=	0.00	kn/m^2	
Wearing Coat Load	=	2.20	kn/m^2	

3.1 Wind Load Calculation

Span length c/c of Expansion Joint		=	37.000	m
Span length c/c of Bearing		=	35.500	m
Overall width of superstructure		=	13.500	m
C/C spacing of Longitudinal Girder		=	2.750	m
Depth of superstructure		=	2.220	m
Depth of Wearing Coat		=	0.100	m
Solid area as seen in elevation	A_1	=	126.54	m^2
Plan area of the Superstructure	A_3	=	499.50	m^2
Type of Superstructure,			Two or more beam/ Box girder Bridge	
Width to depth ration i.e b / d		=	6.08	
Ration of clear distance bet.^{w} beams (for two or more beams/ box bridge) to depth		=	1.24	
Gust Factor	G	=	2.00	
Drag coefficient for the Superstructure	C_D	=	1.95	
Lift coefficient	C_L	=	0.75	

Wind load on loaded structure

As per clause 209.3.7 the bridges shall not be considered to be carrying any live load when the wind speed at deck level exceeds 36m/s.

Design Wind speed at any height V_b = 39.0 m/s

Bridge situated at, Plain Terrain

As per note no. 5 of cl no. 209 of IRC:6-2014, wind pressure shall be increased by 0 %

Wind Speed & design wind pressure for Basic wind speed of 33 m/s

H (m)	Plain Terrain		Terrain with Obstructio	
	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
10	27.8	463.7	17.8	190.5
15	29.2	512.5	19.6	230.5
20	30.3	550.6	21	265.3
30	31.4	590.2	22.8	312.2
50	33.1	659.2	24.9	370.4
60	33.6	676.3	25.6	392.9
70	34.0	693.6	26.2	412.8
80	34.4	711.2	26.9	433.3
90	34.9	729.0	27.5	454.2
100	35.3	747.0	28.2	475.6

Wind speed & design wind pressure for Basic wind speed of 39m/s

H (m)	Plain Terrain		Terrain with Obstruction	
	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
< 10	38.8	647.6	24.9	266.1
15	40.8	715.8	27.4	321.9
20	42.3	769.0	29.3	370.5
30	43.9	824.3	31.8	436.0
50	46.2	920.7	34.8	517.3
60	46.9	944.6	35.8	548.8
70	47.5	968.7	36.6	576.6
80	48.0	993.3	37.6	605.2
90	48.7	1018.2	38.4	634.4
100	49.3	1043.3	39.4	664.3

Average Height of superstructure				=	15.0	m	
Lower bound	Height =	15.0 m	P_z	=	715.8	N/m^2	
Upper Bound	Height =	15.0 m	P_z	=	715.8	N/m^2	
P_z = Design wind pressure at bridge for h =		15.0 m		=	715.8	N/m^2	
Tranverse wind Force			F_T	=	P_Z x A_1 x G x C_D		
				=	353.3	KN	1.91 KN/m
Longitudinal wind Force			F_L	=	25% of F_T		
				=	88.3	KN	0.18 KN/m
Vertical wind Force			F_V	=	P_Z x A_3 x G x C_L		
					536.3	KN	
UDL on girder				=	1.1	KN/m^2	
Outer girder				=	2.8	KN/m	2.82 KN/m
Inner girder				=	3.0	KN/m	2.95 KN/m

1.3 Live Load Idealization

	Span length c/c of Expansion Joint	=	37.000 m	
	Span length c/c of Bearing	=	35.500 m	
	Skew Angle	=	0.0 Degree	
	Total Width of Superstructure	=	13.500 m	
	Width of Railing / Crash barrier on right side	=	0.500 m	
	Width of Railing / Crash barrier on left side	=	0.500 m	
	Width of Footpath on both side (considered for design o	=	0.000 m	
	Cantilever length	=	1.250 m	
	C/C spacing of Longitudinal Girder	=	2.750 m	
	Maximum Length of Live Load	=	18.8 m	13.4, 18.8
	Effect of Skew	=	0.0	
	Travelling Length of Live Load	=	56 m	
	Increment of Live Load	=	1 m	
	Number of Load Case for each Live Load	=	57	
	Carriageway width for LL (considered for design only)	=	12.5 m	
1 *Load Case 1*	70R W 1 Lane Placed most Eccentric (Left side)			
	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130 m	
	Z2 = 2.13 + 1.93	=	4.060 m	
2 *Load Case 2*	70R W 1 Lane Placed at Center of Superstructure			
	Z1 = 13.5/2 -1.93/2	=	5.785 m	
	Z2 = 5.785 + 1.93	=	7.715 m	
3 *Load Case 3*	70R W 1 Lane Placed most Eccentric (Right side)			
	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.44 m	
	Z2 = 9.44 + 1.93	=	11.37 m	
4 *Load Case 4*	70R W 1 Lane Placed at Center of Inner girder			
	Z1 = 1.25 + 2.75 - 1.93/2	=	3.035 m	
	Z2 = 3.035 + 1.93	=	4.965 m	
5 *Load Case 5*	70R W 2 Lane Placed most Eccentric (Left side)			From CB
1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130 m	
70R	Z2 = 2.13 + 1.93	=	4.060 m	3.095
2nd Lane	Z3 = 0.5 + 0 + 7.25 + 0.86/2	=	8.180 m	
70R	Z4 = 8.18 + 1.93	=	10.110 m	9.145
6 *Load Case 6*	70R W 2 Lane Placed at near Center of Superstructure			
1st Lane	Z1 = 12.5/2 - 1.2/2 - 0.86/2 - 1.93		3.29 m	From CL
70R	Z2 = 3.29 + 1.93	=	5.22 m	1.995
2nd Lane	Z3 = 5.22 + 0.86/2 + 1.2 + 0.86/2	=	7.28 m	
70R	Z4 = 7.28 + 1.93	=	9.21 m	-1.995
7 *Load Case 7*	Class A 1 Lane Placed most Eccentric (Left side)			
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90 m	
	Z2 = 0.9 + 1.8	=	2.70 m	
8 *Load Case 8*	Class A 1 Lane Placed most Eccentric (Right side)			
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.80 m	
	Z2 = 10.8 + 1.8	=	12.60 m	
9 *Load Case 9*	Class A 2 Lane Placed most Eccentric (Left side)			
1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90 m	
Class A	Z2 = 0.9 + 1.8	=	2.70 m	
2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40 m	
Class A	Z4 = 4.4 + 1.8	=	6.20 m	

10	*Load Case 10*	Class A 2 Lane Placed most Eccentric (Right side)				
	1st Lane	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8 -0.25 - 1.2 - 0.25 - 1.8	=	7.30	m	
	Class A	Z2 = 7.3 + 1.8	=	9.10	m	
	2nd Lane	Z3 = 9.1 + 0.5 + 1.2	=	10.80	m	
	Class A	Z4 = 10.8 + 1.8	=	12.60	m	
11	*Load Case 11*	Class A 3 Lane Placed most Eccentric (Left side)				
	1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
	Class A	Z2 = 0.9 + 1.8	=	2.70	m	
	2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m	
	Class A	Z4 = 4.4 + 1.8	=	6.20	m	
	3rd Lane	Z5 = 6.2 + 0.5 + 1.2	=	7.90	m	
	Class A	Z6 = 7.9 + 1.8	=	9.70	m	
12	*Load Case 12*	Class A 3 Lane Placed at Center of Superstructure				
	1st Lane	Z1 = 13.5/2 - 1.8/2 - 0.5 - 1.2 - 1.8	=	2.35	m	
	Class A	Z2 = 2.35 + 1.8	=	4.15	m	
	2nd Lane	Z3 = 4.15 + 0.5 + 1.2	=	5.85	m	
	Class A	Z4 = 5.85 + 1.8	=	7.65	m	
	3rd Lane	Z5 = 7.65 + 0.5 + 1.2	=	9.35	m	
	Class A	Z6 = 9.35 + 1.8	=	11.15	m	
13	*Load Case 13*	Class A 3 Lane Placed most Eccentric (Right side)				
	1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8	=	3.80	m	
	Class A	Z2 = 3.8 + 1.8	=	5.60	m	
	2nd Lane	Z3 = 5.6 + 0.5 + 1.2	=	7.30	m	
	Class A	Z4 = 7.3 + 1.8	=	9.10	m	
	3rd Lane	Z5 = 9.1 + 0.5 + 1.2	=	10.80	m	
	Class A	Z6 = 10.8 + 1.8	=	12.60	m	
14	*Load Case 14*	Class A 4 Lane Placed most Eccentric (Left side)				From CB
	1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m	
	Class A	Z2 = 0.9 + 1.8	=	2.70	m	1.800
	2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m	
	Class A	Z4 = 4.4 + 1.8	=	6.20	m	5.300
	3rd Lane	Z5 = 6.2 + 0.5 + 1.2	=	7.90	m	
	Class A	Z6 = 7.9 + 1.8	=	9.70	m	8.800
	4th Lane	Z7 = 9.7 + 0.5 + 1.2	=	11.40	m	
	Class A	Z8 = 11.4 + 1.8	=	13.20	m	12.300
15	*Load Case 15*	Class A 4 Lane Placed at Center of Superstructure				
	1st Lane	Z1 = 13.5/2 - (0.25+1.2+0.25)/2 - 1.8 - 0.5 - 1.2 - 1.8	=	0.60	m	
	Class A	Z2 = 0.6 + 1.8	=	2.40	m	
	2nd Lane	Z3 = 2.4 + 0.5 + 1.2	=	4.10	m	
	Class A	Z4 = 4.1 + 1.8	=	5.90	m	
	3rd Lane	Z5 = 5.9 + 0.5 + 1.2	=	7.60	m	
	Class A	Z6 = 7.6 + 1.8	=	9.40	m	
	4th Lane	Z7 = 9.4 + 0.5 + 1.2	=	11.10	m	
	Class A	Z8 = 11.1 + 1.8	=	12.90	m	
16	*Load Case 16*	Class A 4 Lane Placed most Eccentric (Right side)				
	1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8		0.30	m	
	Class A	Z2 = 0.3 + 1.8	=	2.10	m	
	2nd Lane	Z3 = 2.1 + 0.5 + 1.2	=	3.80	m	
	Class A	Z4 = 3.8 + 1.8	=	5.60	m	
	3rd Lane	Z5 = 5.6 + 0.5 + 1.2	=	7.30	m	
	Class A	Z6 = 7.3 + 1.8	=	9.10	m	
	4th Lane	Z7 = 9.1 + 0.5 + 1.2	=	10.80	m	
	Class A	Z8 = 10.8 + 1.8	=	12.60	m	

17	*Load Case 17*	Class A for combination of 70R Eccentric on Left	70R + Class A 1 Lane				
	1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.13	m		
	70R	Z2 = 2.13 + 1.93	=	4.06	m		
	1st Lane	Z1 = 0.5 + 0 + 7.25 + 0.5/2	=	8.00	m		
	Class A	Z2 = 8 + 1.8	=	9.80	m		
18	*Load Case 18*	Class A for combination of 70R Eccentric on Right	70R + Class A 1 Lane				
	1st Lane	Z1 = 13.5 - 0.5 - 7.25 - 0.5/2 - 1.8	=	3.70	m		
	Class A	Z2 = 3.7 + 1.8	=	5.50	m		
	1st Lane	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.44	m		
	70R	Z2 = 9.44 + 1.93	=	11.37	m		
19	*Load Case 19*	Class A 2 Lane for combination of 70R Eccentric on Left	70R + Class A 2 Lane				
	1st Lane	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.13	m		
	70R	Z2 = 2.13 + 1.93	=	4.06	m		
	1st Lane	Z1 = 0.5 + 0 + 7.25 + 0.5/2	=	8.00	m		
	Class A	Z2 = 8 + 1.8	=	9.80	m		
	2nd Lane	Z3 = 9.8 + 0.5 + 1.2	=	11.50	m		
	Class A	Z4 = 11.5 + 1.8	=	13.30	m		
20	*Load Case 20*	Class A 2 Lane for combination of 70R placed near Cent	70R + Class A 2 Lane				
	1st Lane	Z1 = 7.25 - 0.86/2 - 1.93	=	4.89	m		
	70R	Z2 = 4.89 + 1.93	=	6.82	m		
	1st Lane	Z1 = 6.82 + 0.86/2 + 1.2 + 0.5/2	=	8.70	m		
	Class A	Z2 = 8.7 + 1.8	=	10.50	m		
	2nd Lane	Z3 = 10.5 + 0.5 + 1.2	=	12.20	m		
	Class A	Z4 = 12.2 + 1.8	=	14.00	m		
21	*Load Case 21*	70R W for combination of Class A 2 Lane Eccentric on Le	Class A 2 Lane + 70R				From CB
	1st Lane	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.90	m		
	Class A	Z2 = 0.9 + 1.8	=	2.70	m		1.800
	2nd Lane	Z3 = 2.7 + 0.5 + 1.2	=	4.40	m		
	Class A	Z4 = 4.4 + 1.8	=	6.20	m		5.300
	1st Lane	Z5 = 6.2 + 0.5/2 + 1.2 + 0.86/2	=	8.080	m		
	70R	Z6 = 8.08 + 1.93	=	10.010	m		9.045
22	*Load Case 22*	70R W for combination of Class A Eccentric on Right	Class A 1 Lane + 70R				
	1st Lane	Z1 = 13.5 - 0.5 - 0.15 - 2.3 - 1.2 - 2.79 + 0.86/2	=	6.99	m		
	70R	Z2 = 6.99 + 1.93	=	8.92	m		
	1st Lane	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.80	m		
	Class A	Z2 = 10.8 + 1.8	=	12.60	m		

Note : Z1 to Z6 indicates Distance of 1st to 6th Axle from Left

1.3 Live Load Idealization

Span length c/c of Expansion Joint	=	37.000	m
Span length c/c of Bearing	=	35.500	m
Skew Angle	=	0.0	Degree
Total Width of Superstructure	=	13.500	m
Width of Crash Barrier on right side	=	0.500	m
Width of Railing / Crash barrier on left side	=	0.500	m
Width of Footpath on left side (considered for design onl	=	0.000	m
Cantilever length	=	1.250	m
C/C spacing of Longitudinal Girder	=	2.750	m
Maximum Length of Live Load	=	18.8	m
Effect of Skew	=	0.0	
Travelling Length of Live Load	=	56	m
Increment of Live Load	=	1	m
Number of Load Case for each Live Load	=	57	
Carriageway width for LL (considered for design only)	=	12.5	m

Load Case 1	70R W Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 1.2 + 0.86/2	=	2.130	m
	Z2 = 2.13 + 1.93	=	4.060	m
Load Case 2	70R W Placed at center			
	Z1 = 13.5 /2 -1.93/2	=	5.785	m
	Z2 = 5.785 + 1.93	=	7.715	m
Load Case 3	70R W Placed most eccentric (Right side)			
	Z1 = 13.5 - 0.5 - 1.2 - 2.79 + 0.86/2	=	9.440	m
	Z2 = 9.44 + 1.93	=	11.370	m
Load Case 4	70R W 1 Lane Placed at Center of Inner girder			
	Z1 = 1.25 + 2.75 - 1.93/2	=	3.035	m
	Z2 = 3.035 + 1.93	=	4.965	m
Load Case 5	Class A 1 Lane Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.900	m
	Z2 = 0.9 + 1.8	=	2.700	m
Load Case 6	Class A 1 Lane Placed most eccentric (Right side)			
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8	=	10.800	m
	Z2 = 10.8 + 1.8	=	12.600	m
Load Case 7	Class A 3 Lane Placed most eccentric (Left side)			
	Z1 = 0.5 + 0 + 0.15 + 0.50/2	=	0.900	m
	Z2 = 0.9 + 1.8	=	2.700	m
	Z3 = 2.7 + 0.5 + 1.2	=	4.400	m
	Z4 = 4.4 + 1.8	=	6.200	m
	Z5 = 6.2 + 0.5 + 1.2	=	7.900	m
	Z6 = 7.9 + 1.8	=	9.700	m
Load Case 8	Class A 3 Lane Placed at center			
	Z1 = 13.5 /2 - 1.8/2 - 0.5 - 1.8	=	2.350	m
	Z2 = 2.35 + 1.8	=	4.150	m
	Z3 = 4.15 + 0.5 + 1.2	=	5.850	m
	Z4 = 5.85 + 1.8	=	7.650	m
	Z5 = 7.65 + 0.5 + 1.2	=	9.350	m
	Z6 = 9.35 + 1.8	=	11.150	m

Load Case 9	Class A 3 Lane Placed most eccentric (Right side)		
	Z1 = 13.5 -0.5 - 0.15 - 0.25 - 1.8-1.7-1.8-1.7-1.8	=	3.800 m
	Z2 = 3.8 + 1.8	=	5.600 m
	Z3 = 5.6 + 0.5 + 1.2	=	7.300 m
	Z4 = 7.3 + 1.8	=	9.100 m
	Z5 = 9.1 + 0.5 + 1.2	=	10.800 m
	Z6 = 10.8 + 1.8	=	12.600 m
Load Case 10	Class A for combination of 70R Eccentric on left (Case 1)		
	Z1 = 0.5 + 0 + 7.25 + 0.15 + 0.5/2	=	8.150 m
	Z2 = 8.15 + 1.8	=	9.950 m
Load Case 11	Class A for combination of 70R Eccentric on Right (Case 2)		
	Z1 = 13.5 - 0.5 - 9.85 + 0.15 + 0.5/2	=	3.550 m
	Z2 = 3.55 + 1.8	=	5.350 m
Load Case 12	70R W for combination of Class A Eccentric on left (Case 3)		
	Z1 = 0.5 + 0 + 2.45 + 1.2 + 0.86/2	=	4.580 m
	Z2 = 4.58 + 1.93	=	6.510 m
Load Case 13	70R W for combination of Class A Eccentric on right (Case 4)		
	Z1 = 13.5 - 0.5 - 2.45 - 1.2 -2.79 + 0.86/2	=	6.990 m
	Z2 = 6.99 + 1.93	=	8.920 m

Note : Z1 to Z6 indicates Distance of 1st to 6th Axle from Left

1.4 Bending Moment & Shear Force of Longitudinal Members

Group-2 Girder(G-I & G-IV) : Outer Girder

Total length of the Precast Girder unit		=	36.400 m
Position of Bearing from Girder End		=	0.450 m
c/c spacing of Bearing		=	35.500 m
c/c spacing of Longitudinal Girder		=	2.750 m
Length of cantilever portion		=	1.250 m
Area of Girder at the Span		=	0.880 m^2
Area of Girder at the Support		=	1.495 m^2
Length of the normal section		=	30.400 m
Length of the tapered section		=	1.500 m
Length of the thickened section		=	1.500 m
Thickness of Intermediate X-Girder		=	0.300 m
Thickness of End X-Girder		=	0.400 m
Thickness of Deck slab		=	0.220 m
Thickness of Precast Deck slab placed on Girder top		=	0.000 m
Width of the Precast Deck slab placed on Girder top		=	0.000 m
Desity of Concrete Girder		=	25.0 KN/m^3
Desity of Concrete Top Slab		=	26.0 KN/m^3
Longitudinal Cantilever Length		=	0.750 m

Self Weight of the Girder during Stage-1 Loading

Weight of Girder at Center section	= 0.88 x 25	=	22.0 KN/m
Weight of Girder at End section	= 1.495 x 25	=	37.4 KN/m
Weight of Girder at Variable section	=(22 + 37.4) / 2	=	29.7 KN/m
Weight of End X-girder		=	2.3 KN
Weigt of Intermediate X-girder		=	4.1 KN

Weight of the Shuttering Slab during Stage-2 Loading

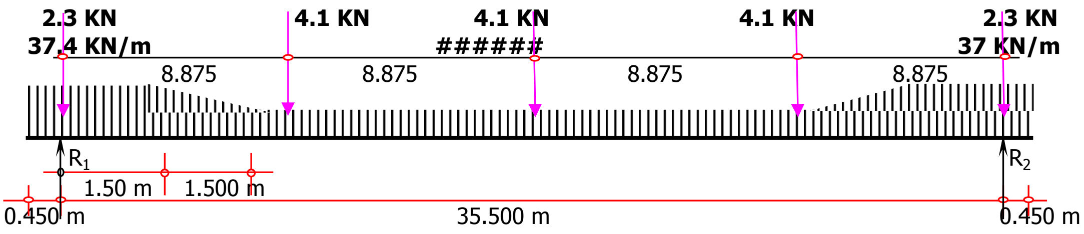
Weight of Shuttering		=	5.0 KN/m

Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	=2.8 x 0.22 x 26	=	15.7 KN/m
Wt. of end x-girder		=	7.7 KN
Wt. of intermediate x-girder		=	5.7 KN

Calculation of Moment & Shear Force at different section

Stage-1 loading **Self weight of girder**

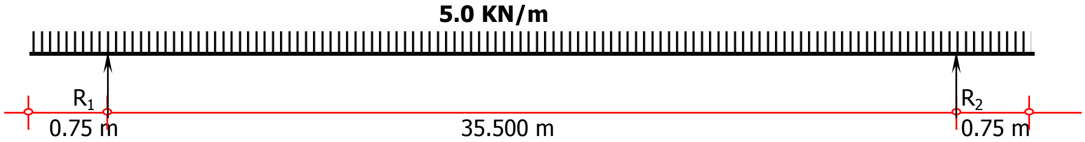


Support Reaction **R_1 = 450.3 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.450	4.888	9.325	13.763	18.200
Support load	450.35	450.35	450.35	450.35	450.35
End Load	16.82	72.90	72.90	72.90	72.90
Varing Load	0.00	44.54	44.54	44.54	44.54
Center Load	0.00	31.63	129.25	226.88	324.50
Point Load	2.27	2.27	6.37	6.37	10.47
Support C.G	0.00	4.44	8.88	13.31	17.75
End C.G	0.23	3.91	8.35	12.79	17.23
Varing C.G	0.00	2.25	6.69	11.13	15.56
Center C.G	0.00	0.72	2.94	5.16	7.38
Point C.g	0.00	4.44	3.16	7.60	7.32
Support Mom.	0.00	1998.43	3996.85	5995.28	7993.71
End Moment	3.78	285.21	608.68	932.16	1255.63
Varing Moment	0.00	100.31	297.94	495.57	693.21
Center Moment	0.00	22.73	379.67	1169.82	2393.19
Point Moment	0.00	10.06	20.12	48.37	76.62
BM (KNm)	-3.8	1580.1	2690.4	3349.4	3575.1
SF (KN)	431.3	299.0	197.3	99.7	-2.0

Stage-2 loading **Shuttering Load for Deck Slab**

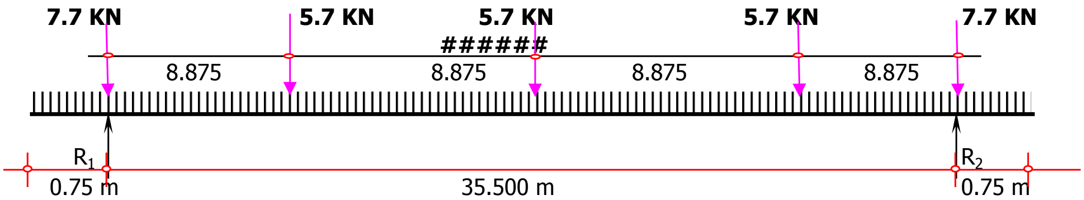
5.0 KN/m



Support Reaction **R_1 = 92.5 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.750	5.188	9.625	14.063	18.500
Support load	92.50	92.50	92.50	92.50	92.50
Center Load	3.75	25.94	48.13	70.31	92.50
Support C.G	0.00	4.44	8.88	13.31	17.75
Center C.G	0.38	2.59	4.81	7.03	9.25
Support Momer	0.00	410.47	820.94	1231.41	1641.88
Center Moment	1.41	67.28	231.60	494.38	855.63
BM (KNm)	-1.4	343.2	589.3	737.0	786.3
SF (KN)	88.8	66.6	44.4	22.2	0.0

Stage-3 loading **Deck Slab Load**



Support Reaction **R_1 = 307.3 KN**

Item	Sup	0.125L	0.250L	0.375L	0.500L
Frm. End	0.750	5.188	9.625	14.063	18.500
Support load	307.27	307.27	307.27	307.27	307.27
Center Load	11.80	81.60	151.40	221.20	291.01
Point Load	7.66	7.66	13.40	13.40	19.14
Support C.G	0.00	4.44	8.88	13.31	17.75
Center C.G	0.38	2.59	4.81	7.03	9.25
Point C.G	0.00	4.44	5.07	9.51	9.76
Support Momen	0.00	1363.53	2727.06	4090.59	5454.12
Center Moment	4.42	211.65	728.62	1555.33	2691.80
Point Moment	0.00	33.97	67.95	127.40	186.86
BM (KNm)	-4.4	1117.9	1930.5	2407.9	2575.5
SF (KN)	287.8	218.0	142.5	72.7	-2.9

1.5 Bending Moments & Shear Forces at Various Sections

A. MOMENT AT DIFFERENT SECTION (KNm) for OUTER GIRDER

Span Length (c/c of Brg.)	=	35.50	m
I.F for 70 R wheel	=	1.130	
I.F for Class A [1+4.5/(6+E_{ff} Span)]	=	1.108	
Warping Effect	=	1.000	

Note:- Value of Bending Moments are taken upto support section where as shear force is considered @ a distance 'd' effective from support

LOADINGS	c/L Brg.	L/8	L/4	3L/8	L/2	Factor SLS Rare
	0m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	0.0	1580.1	2690.4	3349.4	3575.1	1.0
2nd Stage DL	0.0	1117.9	1930.5	2407.9	2575.5	1.0
Shuttering Load	0.0	343.2	589.3	737.0	786.3	1.0
SIDL (Crash Barrier)	0.0	297.0	452.0	479.0	383.7	1.0
SIDL (Surfcaing/ Wearing coat	0.0	274.3	478.2	608.7	664.8	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	0.0	1231.2	2105.4	2653.2	2908.2	1.0
	0	4501	7657	9498	10107	

B. SHEAR AT DIFFERENT SECTION (KN) FOR OUTER GIRDER

LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	431.3	299.0	197.3	99.7	0.0	1.0
2nd Stage DL	287.8	218.0	142.5	72.7	0.0	1.0
SIDL (Crash Barrier)	94.0	76.3	43.8	7.3	26.7	1.0
SIDL (Surfcaing/ Wearing coat	76.3	67.8	47.2	26.3	5.1	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	339.9	336.6	289.9	223.7	128.3	1.0

1.6 Bending Moments & Shear Forces at Various Sections

A. MOMENT AT DIFFERENT SECTION (KNm) for INNER GIRDER

Span Length (c/c of Brg.) = 35.500 m

Note:- Value of Bending Moments are taken upto support section where as shear force is considered @ a distance 'd' effective from support

LOADINGS	c/L Brg.	L/8	L/4	3L/8	L/2	Factor SLS Rare
	0m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	0.0	1580.1	2690.4	3349.4	3575.1	1.0
2nd Stage DL	0.0	1117.9	1930.5	2407.9	2575.5	1.0
Shuttering Load	0.0	343.2	589.3	737.0	786.3	1.0
SIDL (Crash Barrier)	0.0	60.0	141.3	250.5	377.8	1.0
SIDL (Surfcaing/ Wearing coat	0.0	302.6	517.8	644.4	682.9	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	0.0	1397.6	2091.9	2373.9	2581.8	1.0
	0	4458	7372	9026	9793	

B. SHEAR AT DIFFERENT SECTION (KN) FOR INNER GIRDER

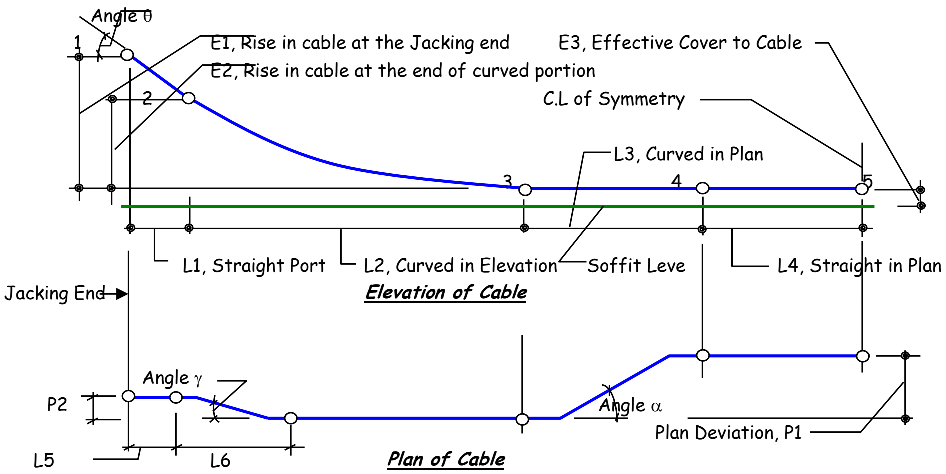
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	431.3	299.0	197.3	99.7	0.0	1.0
2nd Stage DL	287.8	218.0	142.5	72.7	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	0.0	0.0	0.0	0.0	0.0	1.0
Carriageway LL	383.1	366.3	290.2	197.7	275.1	1.0

3.0 Effect of Prestressing (Outer Girder):-

A. Basic Prestressing Data

1)	Nominal Diameter	Ds	12.7	mm	19	
2)	Nominal Area	A	98.7	sq.mm		
3)	Nominal Mass	Pu	0.775	Kg/m		
4)	Yield Strength	Fy	1670	MPa		
5)	Tensile Strength	Fu	1860	MPa		1395
6)	Minimum Breaking Load	Pn	183.7	KN		138
7)	Characteristic co-efficient value for Prestressing Force	γ_{sup}	1.0			
7a)	Characteristic co-efficient value for Prestressing Force	γ_{inf}	1.0			
8)	Multiplied by the co-efficient		183.7	KN		
9)	Young's Modulus of Elasticity	Eps	195	Gpa		
10)	Jacking Force at Transfer (% of Breaking Load)	Pj	75	%		
11)	Slip at Jacking end	s	6	mm		
12)	Coefficient of Friction	μ	0.17	per radian	Refer Table 7.1 of IRC:112	
13)	Wobble Friction Coefficient	k	0.002	per metre	Corrugated HDPE sheathing	
14)	Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		(Refer Table 6.2 , IRC:112)	
15)	Relaxation loss at 1000 hrs at 50% UTS (as % of initial stress)	Re2	0		(Refer Table 6.2 , IRC:112)	
16)	Age of concrete for 1st Stage prestressing	t_{d1}	14	days		
17)	Dia of Prestressing Duct	q_d	90	mm		
18)	Concrete Grade	Fcu_1	45	MPa		
19)	Modulus of Elasticity of Concrete for Girder (28 days)	Ec_1	34313	Mpa	(Refer Table 6.2 and Eq 6.2,	
20)	Concrete Grade at the time of 1st Stege stressing in 14 days	Fcj	40.6	MPa		
21)	Modulus of Elasticity of Concrete at 1st Stage stressing	Ecj	33463	Mpa		
22)	Modulus of Elasticity of Concrete for Girder (28 days)	Ec_2	34313	MPa		
23)	Concrete Grade for Deck Slab	Fcu_2	45	MPa		
24)	Overall depth of PSC composite girder	D	2.220	m		

B. Details of Prestressing Cables



Elevation of Cable

Plan of Cable

Cable No.	Strands per cable	Stage of Prestressing	L1 (L5+L6<=L1)	L2	L3	L4	P1	P2	L5	
1	18	1	2.200	3.200	0.000	12.800	0.000	0.000	0.000	2.200
2	18	1	2.200	3.200	0.000	12.800	0.000	0.000	0.000	2.200
3	18	1	4.700	3.200	0.000	10.300	0.000	0.000	0.000	4.700
4	17	1	5.200	4.500	0.000	8.500	0.000	0.000	0.000	5.200
5	17	1	5.700	6.000	0.000	6.500	0.000	0.000	0.000	5.700
6	0.0001	0	5.700	6.000	0.000	6.500	0.000	0.000	0.000	5.700
Cable No.	Strands per cable	Stage of Prestressing	E1	E2	Angle θ (degrees)	Angle α (degrees)	E3	Angle γ (degrees)	L6	E2+E3
1	18	1	0.265	0.111	3.992	0.000	0.135	0.000	0.000	0.246
2	18	1	0.265	0.111	3.992	0.000	0.135	0.000	0.000	0.246
3	18	1	0.485	0.123	4.404	0.000	0.315	0.000	0.000	0.438
4	17	1	0.705	0.212	5.411	0.000	0.495	0.000	0.000	0.707
5	17	1	0.925	0.318	6.077	0.000	0.675	0.000	0.000	0.993
6	0.0001	0	0.745	0.256	4.898	0.000	0.855	0.000	0.000	1.111

C. Force in Cables at nodal points after Friction & Slip Losses

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	1	0.000	2.205	5.407	5.407	18.207
Σθ (rad)		0.00000	0.00000	0.06967	0.06967	0.06967
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.99560	0.97760	0.97760	0.95289
$P_x = P_o * Z$ (KN)		2479.95	2469.04	2424.39	2424.39	2363.11
P_x^1		2231.58	2242.49	2287.14	2287.14	2348.41

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	2	0.000	2.205	5.407	5.407	18.207
Σθ (rad)		0.00000	0.00000	0.06967	0.06967	0.06967
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.99560	0.97760	0.97760	0.95289
$P_x = P_o * Z$ (KN)		2479.95	2469.04	2424.39	2424.39	2363.11
P_x^1		2231.58	2242.49	2287.14	2287.14	2348.41

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	3	0.000	4.714	7.916	7.916	18.216
Σθ (rad)		0.00000	0.00000	0.07687	0.07687	0.07687
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.99062	0.97151	0.97151	0.95170
$P_x = P_o * Z$ (KN)		2479.95	2456.68	2409.30	2409.30	2360.18
P_x^1		2235.64	2258.91	2306.29	2306.29	2355.41

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	4	0.000	5.223	9.728	9.728	18.228
Σθ (rad)		0.00000	0.00000	0.09443	0.09443	0.09443
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.98961	0.96511	0.96511	0.94884
$P_x = P_o * Z$ (KN)		2342.18	2317.83	2260.46	2260.46	2222.36
P_x^1		2107.39	2131.73	2189.10	2189.10	2222.36

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	5	0.000	5.732	11.741	11.741	18.241
Σθ (rad)		0.00000	0.00000	0.10606	0.10606	0.10606
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.98860	0.95934	0.95934	0.94695
$P_x = P_o * Z$ (KN)		2342.18	2315.48	2246.94	2246.94	2217.92
P_x^1		2108.45	2135.15	2203.69	2203.69	2217.92

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
Σx (m)	6	0.000	5.721	11.726	11.726	18.226
Σθ (rad)		0.00000	0.00000	0.08549	0.08549	0.08549
$Z = (\exp)^{-(\mu\Sigma\theta + k\Sigma x)}$		1.00000	0.98862	0.96273	0.96273	0.95029
$P_x = P_o * Z$ (KN)		0.01	0.01	0.01	0.01	0.01
P_x^1		-1.88	-1.88	-1.88	-3.76	-1.88

36.200

D. Force in Cable at chosen sections after Friction & Slip Losses

Cable No.	Notation	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
1	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	2.205	5.407	5.407	5.407
	x2	2.205	5.407	18.207	18.207	18.207
	y1	0.400	0.246	0.135	0.135	0.135
	y2	0.246	0.135	0.135	0.135	0.135
	y_{ord}	0.376	0.166	0.135	0.135	0.135
	θ	3.992	1.101	0.000	0.000	0.000
	p_{f1}	2479.950	2469.036	2424.390	2424.390	2424.390
	p_{f2}	2469.036	2424.390	2363.113	2363.113	2363.113
	p_{s1}	2231.575	2242.490	2287.136	2287.136	2287.136
	p_{s2}	2242.490	2287.136	2348.413	2348.413	2348.413
	$\mathbf{P_f}$	**2478.218**	**2436.692**	**2406.951**	**2385.289**	**2363.627**
	$\mathbf{P_s}$	**2233.308**	**2274.834**	**2304.574**	**2326.237**	**2347.899**
2	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	2.205	5.407	5.407	5.407
	x2	2.205	5.407	18.207	18.207	18.207
	y1	0.400	0.246	0.135	0.135	0.135
	y2	0.246	0.135	0.135	0.135	0.135
	y_{ord}	0.376	0.166	0.135	0.135	0.135
	θ	3.992	1.101	0.000	0.000	0.000
	p_{f1}	2479.950	2469.036	2424.390	2424.390	2424.390
	p_{f2}	2469.036	2424.390	2363.113	2363.113	2363.113
	p_{s1}	2231.575	2242.490	2287.136	2287.136	2287.136
	p_{s2}	2242.490	2287.136	2348.413	2348.413	2348.413
	$\mathbf{P_f}$	**2478.218**	**2436.692**	**2406.951**	**2385.289**	**2363.627**
	$\mathbf{P_s}$	**2233.308**	**2274.834**	**2304.574**	**2326.237**	**2347.899**
3	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	0.000	7.916	7.916	7.916
	x2	4.714	4.714	18.216	18.216	18.216
	y1	0.800	0.800	0.315	0.315	0.315
	y2	0.438	0.438	0.315	0.315	0.315
	y_{ord}	0.773	0.453	0.315	0.315	0.315
	θ	4.404	4.404	0.000	0.000	0.000
	p_{f1}	2479.950	2479.950	2409.304	2409.304	2409.304
	p_{f2}	2456.679	2456.679	2360.180	2360.180	2360.180
	p_{s1}	2235.643	2235.643	2306.289	2306.289	2306.289
	p_{s2}	2258.914	2258.914	2355.413	2355.413	2355.413
	$\mathbf{P_f}$	**2478.222**	**2457.612**	**2403.897**	**2382.316**	**2360.734**
	$\mathbf{P_s}$	**2237.371**	**2257.981**	**2311.696**	**2333.278**	**2354.859**

4	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	0.000	5.223	9.728	9.728
	x2	5.223	5.223	9.728	18.228	18.228
	y1	1.200	1.200	0.707	0.495	0.495
	y2	0.707	0.707	0.495	0.495	0.495
	y_{ord}	1.167	0.773	0.527	0.495	0.495
	θ	5.411	5.411	0.816	0.000	0.000
	p_{f1}	2342.175	2342.175	2317.835	2260.463	2260.463
	p_{f2}	2317.835	2317.835	2260.463	2222.360	2222.360
	p_{s1}	2107.391	2107.391	2131.731	2189.103	2189.103
	p_{s2}	2131.731	2131.731	2189.103	2222.360	2222.360
	P_f	**2340.544**	**2321.089**	**2269.101**	**2243.219**	**2222.935**
	P_s	**2109.022**	**2128.477**	**2180.465**	**2204.153**	**2221.858**

5	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	0.000	5.732	11.741	11.741
	x2	5.732	5.732	11.741	18.241	18.241
	y1	1.600	1.600	0.993	0.675	0.675
	y2	0.993	0.993	0.675	0.675	0.675
	y_{ord}	1.563	1.121	0.817	0.675	0.675
	θ	6.077	6.077	2.729	0.000	0.000
	p_{f1}	2342.175	2342.175	2315.477	2246.940	2246.940
	p_{f2}	2315.477	2315.477	2246.940	2217.918	2217.918
	p_{s1}	2108.452	2108.452	2135.150	2203.687	2203.687
	p_{s2}	2135.150	2135.150	2203.687	2217.918	2217.918
	P_f	**2340.545**	**2321.099**	**2277.631**	**2238.750**	**2218.546**
	P_s	**2110.082**	**2129.527**	**2172.995**	**2207.703**	**2217.610**

6	L_x^1	0.350	4.525	9.050	13.575	18.100
	x	**0.35**	**4.525**	**9.05**	**13.575**	**18.1**
	x1	0.000	0.000	5.721	11.726	11.726
	x2	5.721	5.721	11.726	18.226	18.226
	y1	1.600	1.600	1.111	0.855	0.855
	y2	1.111	1.111	0.855	0.855	0.855
	y_{ord}	1.570	1.214	0.969	0.855	0.855
	θ	4.898	4.898	2.714	0.000	0.000
	p_{f1}	0.014	0.014	0.014	0.013	0.013
	p_{f2}	0.014	0.014	0.013	0.013	0.013
	p_{s1}	-1.876	-1.876	-1.876	-3.765	-3.765
	p_{s2}	-1.876	-1.876	-1.876	-1.876	-1.876
	P_f	**0.014**	**0.014**	**0.013**	**0.013**	**0.013**
	P_s	**-1.876**	**-1.876**	**-1.876**	**-3.228**	**-1.913**

(*) Notations Used :

Σx (m) = Cumulative Length of Cable from jacking end in metres
$\Sigma\theta$ (rad) = Cumulative angle of deviation in radian from jacking end
P_o = Force at Jacking end before transfer (KN)
P_x = Force at the nodal points before transfer (KN)
P_x^1 = Force at the nodal points after transfer (KN)
L_x^1 = Cumulative Length of Cable from jacking end (in metres) at various sections (m)
P_f = Force at the critical sections before transfer (i,e before slip at anchorage) (KN)
P_s = Force at the critical sections after transfer (i,e after slip at anchorage)(KN)

E. Elongation Calculation Grip Length = **600** mm

Component (*)	Cable No.	Nodal Points of the Cable				
		1	2	3	4	5
P_x	1	2479.95	2469.04	2424.39	2424.39	2363.11
Σx (m)		0.000	2.205	5.407	5.407	18.207
Elongation (mm)		**129.0**				
P_x	2	2479.95	2469.04	2424.39	2424.39	2363.11
Σx (m)		0.000	2.205	5.407	5.407	18.207
Elongation (mm)		**129.0**				
P_x	3	2479.95	2456.68	2409.30	2409.30	2360.18
Σx (m)		0.000	4.714	7.916	7.916	18.216
Elongation (mm)		**129.1**				
P_x	4	2342.18	2317.83	2260.46	2260.46	2222.36
Σx (m)		0.000	5.223	9.728	9.728	18.228
Elongation (mm)		**129.1**				
P_x	5	2342.18	2315.48	2246.94	2246.94	2217.92
Σx (m)		0.000	5.732	11.741	11.741	18.241
Elongation (mm)		**129.2**				
P_x	6	0.01	0.01	0.01	0.01	0.01
Σx (m)		0.000	5.721	11.726	11.726	18.226
Elongation (mm)		**0.0**				

F. Horizontal & Vertical Component of Prestress Force

Section	Stage of Prestressing	Cable No.	No. of Cables	Y_{ord} (m)	θ (deg)	P_s.cosθ (KN)	P_s.cosθ. Y_{ord} (KN.m)	P_s.sinθ (KN)
SUPPORT SECTION	1	1	0.95	0.376	3.992	2110.6	792.8	147.3
	1	2	0.95	0.376	3.992	2110.6	792.8	147.3
	1	3	0.95	0.773	4.404	2113.4	1633.9	162.8
	1	4	0.89	1.167	5.411	1878.6	2192.3	177.9
	1	5	0.89	1.563	6.077	1877.4	2934.2	199.9
	0	6	0.00	1.570	4.898	0.0	0.0	0.0
	Total Stage 1					**10090.6**	**8346.1**	**835.1**
	Eff. Eccentricity of Cable			**0.2007**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
1/8th SPAN SECTION	1	1	0.95	0.166	1.101	2154.7	357.1	41.41499096
	1	2	0.95	0.166	1.101	2154.7	357.1	41.41499096
	1	3	0.95	0.453	4.404	2132.8	965.1	164.2755507
	1	4	0.89	0.773	5.411	1895.9	1466.2	179.5754301
	1	5	0.89	1.121	6.077	1894.7	2123.9	201.6979611
	0	6	0.00	1.121	6.077	0.0	0.0	-1.04541E-06
	Total Stage 1					**10232.8**	**5269.3**	**628.4**
	Eff. Eccentricity of Cable			**0.4814**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
1/4TH SPAN SECTION	1	1	0.95	0.135	0.000	2183.3	294.7	0.0
	1	2	0.95	0.135	0.000	2183.3	294.7	0.0
	1	3	0.95	0.315	0.000	2190.0	689.9	0.0
	1	4	0.89	0.526991	0.816	1950.7	1028.0	27.8
	1	5	0.89	0.817	2.729	1942.1	1587.6	92.6
	0	6	0.00	0.969	2.714	0.0	0.0	0.0
	Total Stage 1					**10449.4**	**3895.0**	**120.3**
	Eff. Eccentricity of Cable			**0.6236**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
3/8TH SPAN SECTION	1	1	0.95	0.135	0.000	2203.8	297.5	0.0
	1	2	0.95	0.135	0.000	2203.8	297.5	0.0
	1	3	0.95	0.315	0.000	2210.5	696.3	0.0
	1	4	0.89	0.495	0.816	1971.9	976.1	28.1
	1	5	0.89	0.675	0.000	1975.3	1333.3	0.0
	0	6	0.00	0.855	0.000	0.0	0.0	0.0
	Total Stage 1					**10565.3**	**3600.8**	**28.1**
	Eff. Eccentricity of Cable			**0.6555**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				
MID SPAN SECTION	1	1	0.95	0.135	0.000	2224.3	300.3	0
	1	2	0.95	0.135	0.000	2224.3	300.3	0
	1	3	0.95	0.315	0.000	2230.9	702.7	0
	1	4	0.89	0.495	0.000	1988.0	984.0	0
	1	5	0.89	0.675	0.000	1984.2	1339.3	0
	0	6	0.00	0.855	0.000	0.0	0.0	0
	Total Stage 1					**10651.7**	**3626.7**	**0.0**
	Eff. Eccentricity of Cable			**0.6559**				
	Total Stage 2					**0.0**	**0.0**	**0.0**
	Eff. Eccentricity of Cable			**0.0000**				

(*) Notations Used :

Y_{ord} (m) = Vertical Ordinate of Cable from soffit of deck

θ (deg) = Cumulative angle of deviation in radian from jacking end

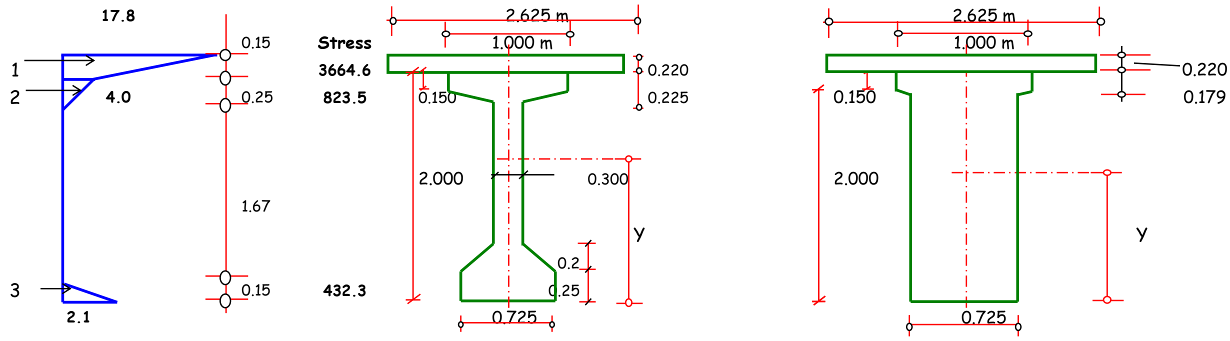
P_s = Force at the critical sections after transfer (i,e after slip at anchorage) (KN)

4.0 Effect of Temperature Gradient (As per Clause 218.3 of IRC:6 - 2010) - Outer Girder

			At Span	At Support	
Total Height of the girder	h	=	2.220	2.220	m
C.G. of Girder from bottom	Y_b	=	1.4376	1.3293	m
M.O.I. of the Section	I	=	0.8445	1.0107	m^4
Area of the Section	A	=	1.4575	2.0728	m^2
Modulus of Elasticity of Concrete	Ec	=	1.72E+07		KN/m^2
Coefficient of thermal expansion of concrete	α	=	1.20E-05		°C
Section Modulus at the top of Slab	Z_{TS}	=	1.0794	1.1347	m^3
Section Modulus at the top of Girder	Z_{TG}	=	1.5016	1.5069	m^3
Section Modulus at the bottom of Girder	Z_{BG}	=	0.5875	0.7603	m^3

T_1	=	17.8	°C	h_1	=	0.15	m
T_2	=	4.0	°C	h_2	=	0.25	m
T_3	=	2.1	°C	h_3	=	0.15	m

4.1 COMPUTATION OF STRESSES DUE TO RISE IN TEMPERATURE



Section of Girder at Span

Section of Girder at Support

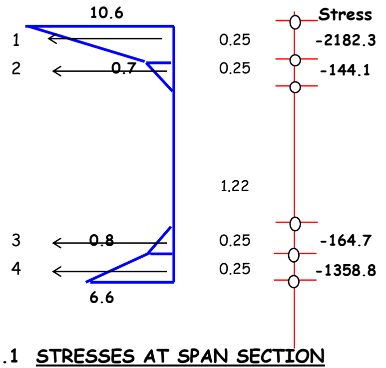
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	3664.6	2.625	883.60	0.059	0.723	639.05	3664.63	-748.27	-658.91	**2257.4**
2	0.070	823.5	2.625	130.14	0.183	0.599	77.99	823.51	-748.27	-473.64	**-398.4**
3	0.180	592.9	1.000	53.36	0.280	0.502	26.81	592.93	-748.27		
4	0.150	432.3	0.725	23.51	2.170	-1.388	-32.62	432.34	-748.27	1210.69	**894.8**
				1090.61			**711.23**				

4.1.2 STRESSES AT SUPPORT SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.150	3664.6	2.625	883.60	0.059	0.832	734.73	3664.63	-526.15	-745.78	**2392.7**
2	0.070	823.5	2.625	130.14	0.183	0.708	92.08	823.51	-526.15	-561.57	**-264.2**
3	0.180	592.9	1.000	53.36	0.280	0.611	32.59	592.93	-526.15		
4	0.150	432.3	0.725	23.51	1.452	-0.561	-13.19	432.34	-526.15	1113.03	**1019.2**
				1090.61			**846.22**				

4.2 COMPUTATION OF STRESSES DUE TO FALL IN TEMPERATURE



h	=	2.220	m				
h_1	=	0.250	m	T_1	=	10.6	°C
h_2	=	0.250	m	T_2	=	0.7	°C
h_3	=	0.250	m	T_3	=	0.8	°C
h_4	=	0.250	m	T_4	=	6.6	°C

4.2.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.220	-2182.3	2.625	-742.38	0.084	0.698	-518.17	-2182.31	628.74	331.01	**-1222.6**
2	0.030	-388.7	1.000	-7.99	0.233	0.550	-4.39	-388.70	628.74	237.94	**478.0**
3	0.195	-144.1	1.000	-17.14	0.327	0.456	-7.81	-144.11	628.74		
4	0.055	-31.7	0.300	-0.26	0.463	0.319	-0.08	-31.71	628.74		
5	0.250	-164.7	0.513	-10.55	1.887	-1.104	11.65	-164.70	628.74		
4	0.250	-1358.8	0.725	-138.07	1.952	-1.170	161.51	-1358.79	628.74	-608.21	**-1338.3**
				-916.39			**-357.29**				

2.2 STRESSES AT SUPPORT SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses: Assuming End Restrained	Stresses: Stress due to release of Axial Force	Stresses: Stress due to release of Moment	Stresses: Final Stress
	m	KN/m^2	m	KN	m	m	KNm	KN/m^2	KN/m^2	KN/m^2	KN/m^2
1	0.220	-2182.3	2.625	-742.38	0.084	0.806	-598.56	-2182.31	444.39	398.64	**-1339.3**
2	0.030	-388.7	1.000	-7.99	0.233	0.658	-5.26	-388.70	444.39	300.18	**355.9**
3	0.195	-144.1	1.000	-17.14	0.327	0.564	-9.67	-144.11	444.39		
4	0.055	-31.7	0.725	-0.63	0.463	0.427	-0.27	-31.71	444.39		
5	0.250	-164.7	0.725	-14.93	1.887	-0.996	14.87	-164.70	444.39		
4	0.250	-1358.8	0.725	-138.07	1.952	-1.062	146.56	-1358.79	444.39	-594.95	**-1509.3**
				-921.14			**-452.33**				

5.0 Check for Flexural Stresses at Various Sections (Outer Girder)

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.525**	**9.050**	**13.575**	**18.100**
A. Section property of Girder only						
Area of the section, A	m^2	1.4953	0.8800	0.8800	0.8800	0.8800
Depth of the section, d	m	2.000	2.000	2.000	2.000	2.000
CG of section from bottom, Y_b	m	1.0278	0.9963	0.9963	0.9963	0.9963
Inertia of section, I_{x-x}	m^4	0.5204	0.4097	0.4097	0.4097	0.4097
Top Section Modulus, Z_t	m^3	0.5353	0.4082	0.4082	0.4082	0.4082
Bottom Section Modulus, Z_b	m^3	0.5063	0.4112	0.4112	0.4112	0.4112
B. Dead Load Moments	KNm	0.0	1580.1	2690.4	3349.4	3575.1
Stress at Top Fibre, σ_t	KN/m^2	0.0	3870.5	6590.3	8204.3	8757.2
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-3842.2	-6542.1	-8144.3	-8693.1
C. Prestress after 7 days	KN	10090.6	10232.8	10449.4	10565.3	10651.7
Eff. Eccentricity of Cable, **e**	m	0.201	0.481	0.624	0.656	0.656
CG of Tendons from Bottom Y_{ord}	m	0.827	0.515	0.373	0.341	0.340
Prestressing Factor (Top) $(1/A-e/Z_t)$	m^{-2}	0.29386	-0.04281	-0.39112	-0.46935	-0.47016
Prestressing Factor (Bottom) $(1/A+e/Z_b)$	m^{-2}	1.06509	2.30691	2.65267	2.73033	2.73114
Stress at Top Fibre due to prestress	KN/m^2	2965.3	-438.0	-4086.9	-4958.8	-5008.0
Stress at Bottom Fibre due to prestress	KN/m^2	10747.4	23606.3	27718.8	28846.9	29091.4
Cumulative Stress at Top Fibre, σ_t	KN/m^2	**2965.3**	**3432.5**	**2503.4**	**3245.5**	**3749.1**
Cumulative Stress at Bottom Fibre, σ_b	KN/m^2	**10747.4**	**19764.0**	**21176.7**	**20702.6**	**20398.2**
D. Elastic Shortening Loss	KN	357.0	357.0	357.0	357.0	357.0
Stress after P-E loss						
Top	KN/m^2	2860.3	-422.8	-3947.3	-4791.2	-4840.2
Bottom	KN/m^2	10367.2	22782.6	26771.7	27872.1	28116.3
Cumulative Stress after E. S. loss						
Top Fibre, σ_t	KN/m^2	**2860.3**	**3447.8**	**2643.0**	**3413.1**	**3917.0**
Bottom Fibre, σ_b	KN/m^2	**10367.2**	**18940.4**	**20229.6**	**19727.8**	**19423.1**
Stress at CG of Tendon	KN/m^2	7262.7	14951.5	16951.9	16947.6	16783.4
Avg stress at CG of Tendon	KN/m^2	15218.5				
Check for loss due to elastic shortening	=	1/2 * 15218.5 * (195000/33463) * 98.7*82/1000000 KN				
	=	357.0				
E. Losses in prestress, 7-28days						
1) Relaxation loss (14-28 days)						
Prestressing force after E Loss	KN	9733.6	9875.8	10092.4	10208.3	10294.7
Initial Stress as % of UTS (f_p)	f_p	0.650	0.659	0.673	0.681	0.687
Relaxation loss for low relaxation steel at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.869	1.988	2.168	2.265	2.337
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	83.99				
Factor For time dependent loss		**1.0**				
Relaxation loss at mid-span for 7 **to 28**	KN	= 83.99/100 * 2.337/100 * (0.75*183.7*82) * 1				
days	KN	**176.4**	**187.6**	**204.7**	**213.8**	**220.6**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.525**	**9.050**	**13.575**	**18.100**
2) Shrinkage Loss (14-28 days)						
Residual Shrinkage Strain at	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Factor For time dependent loss		**1.0**				
Relaxation loss for 21 to 28 days	KN	=-(0.000034 - 0.000058) * 98.7 * 82 * 195000 * 1/1000				
	KN	**37.2**	**45.0**	**45.0**	**45.0**	**45.0**
3) Creep Loss (7-28 days)						
Creep Co-efficient at	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at	28 days	2.127	2.194	2.194	2.194	2.194
Factor For time dependent loss		**1.0**				
Creep Loss	KN	**164.2**	**169.5**	**169.6**	**169.8**	**169.8**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**377.9**	**402.1**	**419.3**	**428.6**	**435.5**
Stress after (R + S + C) loss						
Top	KN/m^2	-111.0	17.2	164.0	201.2	204.7
Bottom	KN/m^2	-402.5	-927.7	-1112.3	-1170.2	-1189.3
Cumulative Stress after R + S + C loss						
Top Fibre, σ_t	KN/m^2	**2749.3**	**3465.0**	**2807.0**	**3614.2**	**4121.7**
Bottom Fibre, σ_b	KN/m^2	**9964.7**	**18012.7**	**19117.3**	**18557.5**	**18233.8**
Stress at CG of Tendon	KN/m^2	6980.7	14267.1	16077.5	16011.1	15831.4
Avg stress at CG of Tendon	KN/m^2	14440.4				
Check for loss due to creep of concrete	=	(2.426245 - 2.1275) * 14440.4/1000 * 98.7 * 82 * 195000 * 1 /1000 KN				
	=	164.7	169.8	169.8	169.8	169.8
F. Shuttering Load Moments	KNm	0.0	343.2	589.3	737.0	786.3
Stress at Top Fibre, σ_t	KN/m^2	0.0	840.7	1443.6	1805.4	1925.9
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-834.5	-1433.0	-1792.1	-1911.9
G. 2nd Stage DL Moments **1) Sterss due to 2nd Stege DL**	KNm	0.0	1117.9	1930.5	2407.9	2575.5
Stress at Top Fibre, σ_t	KN/m^2	0.0	2738.3	4728.8	5898.1	6308.7
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	-2718.3	-4694.2	-5855.0	-6262.5
2) Effect in prestress due to 2nd Stage DL						
Stersses at CG of Tendon	KN/m^2	0.0	-1313.4	-2938.0	-3852.2	-4122.4
Gain due to dead load of Deck Slab	KN	0.0	-61.6	-137.9	-180.7	-193.4
Stress at Top Fibre, σ_t	KN/m^2	0.0	-2.6	-53.9	-84.8	-90.9
Stress at Bottom Fibre, σ_b	KN/m^2	0.0	142.2	365.7	493.5	528.3
3) Stersses after 2nd Stage DL						
Stress at Top Fibre, σ_t	KN/m^2	**2749.3**	**7041.4**	**8925.5**	**11232.8**	**12265.4**
Stress at Bottom Fibre, σ_b	KN/m^2	**9964.7**	**14602.0**	**13355.7**	**11403.9**	**10587.7**
I. Section property of Composite Section						
Area of the section, A	m^2	2.0728	1.4575	1.4575	1.4575	1.4575
Depth of the section, d	m	2.220	2.220	2.220	2.220	2.220
CG of section from bottom, Y_b	m	1.3293	1.4376	1.4376	1.4376	1.4376
Inertia of section, I_{x-x}	m^4	1.0107	0.8445	0.8445	0.8445	0.8445
Top Section Modulus, Z_{tS}	m^3	1.1347	1.0794	1.0794	1.0794	1.0794
Top Section Modulus, Z_{tG}	m^3	1.5069	1.5016	1.5016	1.5016	1.5016
Bottom Section Modulus, Z_b	m^3	0.7603	0.5875	0.5875	0.5875	0.5875

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.525**	**9.050**	**13.575**	**18.100**
CG of Tendons from Bottom Y_{ord}	m	0.827	0.515	0.373	0.341	0.340
Revised Eff. Eccentricity of Cable, **e**	m	0.502	0.923	1.065	1.097	1.097
Prestressing Factor (Top) $(1/A-e/Z_{tS})$	m^{-2}	0.03986	-0.16868	-0.30042	-0.33001	-0.33031
Prestressing Factor (Top) $(1/A-e/Z_{tG})$	m^{-2}	0.14917	0.07167	-0.02302	-0.04429	-0.04451
Prestressing Factor (Bottom) $(1/A+e/Z_b)$	m^{-2}	1.14297	2.25670	2.49875	2.55312	2.55368
J. Stress due to release of shuttering load						
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	-317.9	-546.0	-682.8	-728.4
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	-228.5	-392.5	-490.8	-523.6
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	584.2	1003.2	1254.6	1338.4
K. Stress after release of shuttering load						
Stress at Top of Deck, σ_{tS}	KN/m^2	**0.0**	**-317.9**	**-546.0**	**-682.8**	**-728.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**2749.3**	**6812.8**	**8533.0**	**10742.0**	**11741.8**
Stress at Bottom of Girder, σ_b	KN/m^2	**9964.7**	**15186.2**	**14358.9**	**12658.5**	**11926.1**
L. Losses in prestress, 28-49days						
1) Relaxation loss (28-49 days)						
Prestressing force after E Loss	KN	9733.6	9875.8	10092.4	10208.3	10294.7
Initial Stress as % of UTS (f_p)	f_p	0.650	0.659	0.673	0.681	0.687
Relaxation loss for low relaxation steel at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.869	1.988	2.168	2.265	2.337
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.3, IRC:112 - 2011)	%	12.81				
Factor For time dependent loss		**1.0**				
Relaxation loss at mid-span for **28 to 49**	KN	= 12.81/100 * 2.337/100 * (0.75*183.7*82) * 1				
days	KN	**26.9**	**28.6**	**31.2**	**32.6**	**33.7**
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at	49 days	0.00009	0.00010	0.00010	0.00010	0.00010
Factor For time dependent loss		**1.0**				
Relaxation loss for **28 to 49 days**	KN	=- (0.000058 - 0.000093) * 98.7 * 82 * 195000 * 1/1000				
	KN	**55.1**	**51.7**	**51.7**	**51.7**	**51.7**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.525**	**9.050**	**13.575**	**18.100**
3) Creep Loss (28-49 days)						
Creep Co-efficient at	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at	49 days	1.912	1.972	1.972	1.972	1.972
Factor For time dependent loss		**1.0**				
Creep Loss	KN	**95.9**	**99.0**	**99.0**	**99.1**	**99.1**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**177.9**	**179.3**	**181.9**	**183.4**	**184.5**
Stress after (R + S + C) loss						
Top of Deck	KN/m^2	-7.1	30.2	54.7	60.5	60.9
Top of Girder	KN/m^2	-26.5	-12.8	4.2	8.1	8.2
Bottom of Girder	KN/m^2	-203.3	-404.6	-454.6	-468.2	-471.1
Cumulative Stress after R + S + C loss						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-7.1**	**-287.7**	**-491.3**	**-622.3**	**-667.5**
Stress at Top of Girder, σ_{tG}	KN/m^2	**2722.8**	**6800.0**	**8537.2**	**10750.1**	**11750.0**
Stress at Bottom of Girder, σ_b	KN/m^2	**9761.4**	**14781.7**	**13904.3**	**12190.3**	**11455.0**
Stress at CG of Tendon	KN/m^2	6850.5	12726.6	12904.0	11944.9	11505.2
Avg stress at CG of Tendon	KN/m^2	11688.4				
Check for loss due to creep of concrete	=	(2.1275 - 1.912101) * 11688.4/1000 * 98.7 * 82 * 195000 * 1 /1000 KN				
	=	96.1	99.1	99.1	99.1	99.1
M. SIDL+Crash Barrier DL Moments	KNm	0.0	626.2	1025.8	1209.4	1181.5
1) Sterss due to SIDL						
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	580.1	950.4	1120.5	1094.6
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	417.0	683.2	805.4	786.8
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	-1065.9	-1746.2	-2058.8	-2011.1
2) Effect in prestress due to SIDL						
Stersses at CG of Tendon	KN/m^2	0.0	-684.1	-1293.5	-1570.7	-1534.8
Gain due to SIDL	KN	0.0	-32.1	-60.7	-73.7	-72.0
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	-5.4	-18.2	-24.3	-23.8
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	2.3	-1.4	-3.3	-3.2
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	72.4	151.6	188.2	183.9
3) Stersses after SIDL						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-7.1**	**287.0**	**440.8**	**473.9**	**403.3**
Stress at Top of Girder, σ_{tG}	KN/m^2	**2722.8**	**7219.3**	**9219.0**	**11552.3**	**12533.6**
Stress at Bottom of Girder, σ_b	KN/m^2	**9761.4**	**13788.2**	**12309.7**	**10319.7**	**9627.8**
O. Losses in prestress, 49days onwards						
1) Relaxation loss (49 days onwards)						
Prestressing force after E Loss	KN	9733.6	9875.8	10092.4	10208.3	10294.7
Initial Stress as % of UTS (f_p)	f_p	0.650	0.659	0.673	0.681	0.687
Relaxation loss at 1000 hrs. (Ref. Table 6.2, IRC:112 - 2011)	%	1.869	1.988	2.168	2.265	2.337
Time after prestressing	hr.	Infinity				
Residual Relaxation loss as % of loss at 1000 hrs. (Ref. Table 6.3, IRC:112 - 2011)	%	203.20				
Factor For time dependent loss		1				
Relaxation loss at mid-span for **49 days**	KN	= 203.2/100 * 2.337/100 * (0.75*183.7*82) * 1				
to Infinity	KN	**426.8**	**453.9**	**495.2**	**517.3**	**533.7**

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	**0.350**	**4.525**	**9.050**	**13.575**	**18.100**
2) Shrinkage Loss (49 days onwards)						
Residual Shrinkage Strain at	49 days	0.00009	0.00010	0.00010	0.00010	0.00010
Residual Shrinkage Strain at	infinity	0.00025	0.00027	0.00027	0.00027	0.00027
Factor For time dependent loss		1.0				
Relaxation loss for **49 days to Infinity**	KN	=- (0.000093 - 0.000247) * 98.7 * 82 * 195000 * 1/1000				
	KN	**241.4**	**273.9**	**273.9**	**273.9**	**273.9**
3) Creep Loss (49 days onwards)						
Creep Co-efficient at	49 days	1.912	1.972	1.972	1.972	1.972
Creep Co-efficient at	infinity	1.298	1.339	1.339	1.339	1.339
Factor For time dependent loss		1				
Creep Loss	KN	**207.9**	**214.8**	**215.2**	**215.6**	**215.8**
Total Loss (Relaxation + Shrinkage + Creep)	KN	**876.1**	**942.6**	**984.3**	**1006.8**	**1023.5**
Stress after (R + S + C) loss						
Top of Deck	KN/m^2	-34.9	159.0	295.7	332.3	338.1
Top of Girder	KN/m^2	-130.7	-67.6	22.7	44.6	45.6
Bottom of Girder	KN/m^2	-1001.3	-2127.2	-2459.5	-2570.5	-2613.6
Cumulative Stress after R + S + C loss i.e. at service without LL						
Top of Deck, σ_{tS}	KN/m^2	**-42.0**	**446.0**	**736.5**	**806.1**	**741.4**
Top of Girder, σ_{tG}	KN/m^2	**2592.1**	**7151.7**	**9241.6**	**11596.9**	**12579.2**
Bottom of Girder, σ_b	KN/m^2	**8760.0**	**11661.0**	**9850.2**	**7749.2**	**7014.1**
Stress at CG of Tendon	KN/m^2	6209.2	10500.0	9736.8	8404.8	7961.5
Avg stress at CG of Tendon	KN/m^2	8931.8				
Check for loss due to creep of concrete	=	(1.912101 - 1.298361)/10 * 8931.8/1000 * 98.7 * 82 * 195000 * 1 /1000 KN				
	=	209.3	215.8	215.8	215.8	215.8
P. Live Load Moments	KNm	0.0	1231.2	2105.4	2653.2	2908.2
Stress at Top of Deck, σ_{tS}	KN/m^2	0.0	1140.7	1950.6	2458.1	2694.3
Stress at Top of Girder, σ_{tG}	KN/m^2	0.0	819.9	1402.1	1766.9	1936.7
Stress at Bottom of Girder, σ_b	KN/m^2	0.0	-2095.9	-3584.0	-4516.5	-4950.5
Cumulative Stress at Service with LL						
Top of Deck, σ_{tS}	KN/m^2	**-42.0**	**1586.7**	**2687.1**	**3264.2**	**3435.6**
Top of Girder, σ_{tG}	KN/m^2	**2592.1**	**7971.6**	**10643.7**	**13363.8**	**14515.8**
Bottom of Girder, σ_b	KN/m^2	**8760.0**	**9565.1**	**6266.3**	**3232.7**	**2063.7**
Q. Total time dependent Losses + Elastic Shortening Loss in cables						
	KN	**1788.9**	**1787.3**	**1744.0**	**1721.4**	**1735.0**
% Loss	%	**17.7**	**17.5**	**16.7**	**16.3**	**16.3**
R. Stress due to Differential Shrinkage & Creep						
Strain due to differential shrinkage and creep		**2.00E-04**				
Reduction factor due to creep		**0.43**				
Axial force **P**	KN	1704.2	1704.2	1704.2	1704.2	1704.2
Eccentricity **e**	m	0.781	0.672	0.672	0.672	0.672
Stress due to Shrinkage & Creep	KN/m^2	-956.3	-720.1	-720.1	-720.1	-720.1
	KN/m^2	1705.1	1932.3	1932.3	1932.3	1932.3
	KN/m^2	-927.7	-781.4	-781.4	-781.4	-781.4

Item	Unit	Support Section	1/8th span section	1/4th span section	3/8th span section	Mid span
	m	0.350	4.525	9.050	13.575	18.100
1) Stress at service with Differential Shrinkage and Differential Creep effect (without Live Load)						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-998.3**	**-274.1**	**16.4**	**86.0**	**21.3**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4297.1**	**9084.0**	**11173.9**	**13529.2**	**14511.5**
Stress at Bottom of Girder, σ_b	KN/m^2	**7832.3**	**10879.7**	**9068.9**	**6967.8**	**6232.8**
2) Stress at service with Differential Shrinkage and Differential Creep effect (with Live Load)						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-998.3**	**866.6**	**1967.0**	**2544.1**	**2715.5**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4297.1**	**9904.0**	**12576.0**	**15296.1**	**16448.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**7832.3**	**8783.8**	**5484.9**	**2451.3**	**1282.3**
S. Sterss due to Temperature Rise and Fall						
Stress due to Temperature Rise Case	KN/m^2	2392.7	2257.4	2257.4	2257.4	2257.4
	KN/m^2	-264.2	-398.4	-398.4	-398.4	-398.4
	KN/m^2	1019.2	894.8	894.8	894.8	894.8
Stress due to Temperature Fall Case	KN/m^2	-1339.3	-1222.6	-1222.6	-1222.6	-1222.6
	KN/m^2	355.9	478.0	478.0	478.0	478.0
	KN/m^2	-1509.3	-1338.3	-1338.3	-1338.3	-1338.3
1) Stress at Service without Live Load + Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**1394.4**	**1983.3**	**2273.9**	**2343.5**	**2278.7**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4032.9**	**8685.6**	**10775.5**	**13130.8**	**14113.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**8851.5**	**11774.4**	**9963.7**	**7862.6**	**7127.5**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-2337.5**	**-1496.7**	**-1206.1**	**-1136.5**	**-1201.3**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4653.0**	**9562.0**	**11651.9**	**14007.2**	**14989.5**
Stress at Bottom of Girder, σ_b	KN/m^2	**6323.0**	**9541.4**	**7730.6**	**5629.6**	**4894.5**
2) Stress at Service with 75% Live Load + Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**1394.4**	**2838.8**	**3736.8**	**4187.0**	**4299.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4032.9**	**9300.6**	**11827.1**	**14456.0**	**15565.6**
Stress at Bottom of Girder, σ_b	KN/m^2	**8851.5**	**10202.5**	**7275.7**	**4475.2**	**3414.7**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-2337.5**	**-641.1**	**256.8**	**707.0**	**819.4**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4653.0**	**10177.0**	**12703.5**	**15332.4**	**16442.0**
Stress at Bottom of Girder, σ_b	KN/m^2	**6323.0**	**7969.5**	**5042.7**	**2242.2**	**1181.7**
2) Stress at Service with 100% Live Load + 60%Temperature Gradient						
Temperature Rise Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**437.4**	**2221.0**	**3321.4**	**3898.6**	**4070.0**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4138.6**	**9664.9**	**12337.0**	**15057.1**	**16209.1**
Stress at Bottom of Girder, σ_b	KN/m^2	**8443.8**	**9320.6**	**6021.8**	**2988.2**	**1819.2**
Temperature Fall Case						
Stress at Top of Deck, σ_{tS}	KN/m^2	**-1801.8**	**133.0**	**1233.5**	**1810.6**	**1982.0**
Stress at Top of Girder, σ_{tG}	KN/m^2	**4510.7**	**10190.7**	**12862.8**	**15582.9**	**16734.9**
Stress at Bottom of Girder, σ_b	KN/m^2	**6926.7**	**7980.8**	**4682.0**	**1648.4**	**479.4**

6.0 Summary Of Extreme Fiber Stress (Outer Girder)

STAGES	Stress at top of Deck		Stress at top of Girder		Stress at bottom of Girder		Allowable Stress in Slab		Allowable Stress in Girder		Status	
	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve	max +ve	max -ve		
Stress after 28 days before 2nd Stage DL			4.12	2.75	19.12	9.96			19.5	-2.98	OK	OK
Stress after 49 days before (SIDL i.e Crash Barrier & Surfacing)	-0.01	-0.67	11.75	2.72	14.78	9.76	21.60	-2.70	21.60	-2.98	OK	OK
Stress at service without LL	0.81	-0.04	12.58	2.59	11.66	7.01	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with LL	3.44	-0.04	14.52	2.59	9.57	2.06	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with Diff. Shr.& Creep (without LL)	0.09	-1.00	14.51	4.30	10.88	6.23	21.60	-2.70	21.60	0.00	OK	OK
Stress at service with Diff. Shr.& Creep (with LL)	2.72	-1.00	16.45	4.30	8.78	1.28	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service without LL + Diff. Shr.& Creep + Temp rise	2.34	1.39	14.11	4.03	11.77	7.13	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service without LL + Diff. Shr.& Creep + Temp fall	-1.14	-2.34	14.99	4.65	9.54	4.89	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with75% LL + Diff. Shr.& Creep + Temp rise	4.30	1.39	15.57	4.03	10.20	3.41	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 75% LL + Diff. Shr.& Creep + Temp fall	0.82	-2.34	16.44	4.65	7.97	1.18	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 100% LL + Diff. Shr.& Creep + Temp rise with 60% temp. gradient	4.07	0.44	16.21	4.14	9.32	1.82	21.60	-2.70	21.60	0.00	OK	OK
Stress at Service with 100% LL + Diff. Shr.& Creep + Temp fall with 60% temp. gradient	1.98	-1.80	16.73	4.51	7.98	0.48	21.60	-2.70	21.60	0.00	OK	OK

8.0 ULTIMATE MOMENT CHECK (ULS) FOR PSC T-GIRDER (OUTER) AS PER IRC : 112 - 2011 :-

Span Length (c/c of Brg.)		=	35.500	m

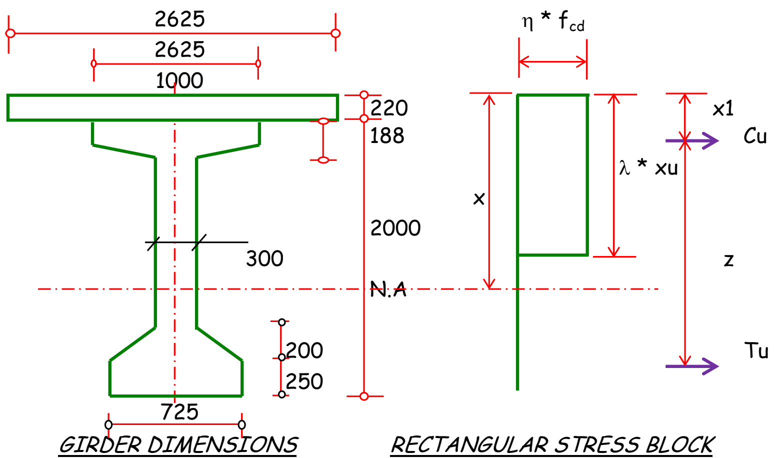
Design Parameters

Grade of of Concrete of Longitudinal PSC Girder		=	M 45	MPa
Modulus of Elasticity of Concrete of PSC Girder	$E_{cm/g}$	=	34	GPa
Grade of of Concrete of RCC Deck Slab		=	M 45	Mpa
Modulus of Elasticity of Concrete of RCC Deck Slab	$E_{cm/d}$	=	34	GPa
Partial factor for Concrete	γ_m	=	1.5	
Longterm Strength factor for Concrete	α	=	0.67	
Characteristic yield strength of Reinforcement		=	Fe 500	Mpa
Modulus of Elasticity of Reinforcement Steel	E_s	=	200	GPa
Partial factor for Reinforcement Steel	γ_s	=	1.15	
Nomonal Diameter of Prestressing Strands		=	12.7	mm
Nominal Area of each Strands		=	98.7	mm^2
Nos of Strands		=	88	
Modulus of Elasticity of Prestressing Steel	E_p	=	195	GPa
Partial factor for Prestressing Steel	γ_p	=	1.15	
Charecteristic tensile strength of Prestressing steel	f_{pk}	=	1860	Mpa
Dia. of Untensioned Reinforcement	ϕ	=	12	mm
c.g of UntensionedReinforcement from soffit girder	c	=	58	mm

Using Rectangular Stress block,

Effective height factor	λ	=	0.8
Compression zone factor	η	=	1.0

Dimension of Inner Girder



GIRDER DIMENSIONS   RECTANGULAR STRESS BLOCK

Bending Moment at Different Section (KN-m) for Inner Girder

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor
	0m	4.4375m	8.875m	13.3125m	17.75m	ULS
Moment due to 1st Stage Dead Load (DL)	0.0	1580.1	2690.4	3349.4	3575.1	1.35
Moment due to 2nd Stage Dead Load (DL)	0.0	1117.9	1930.5	2407.9	2575.5	1.35
Moment due to SIDL (Crash Barrier)	0.0	297.0	452.0	479.0	383.7	1.35
Moment due to SIDL (Surfcaing/ Wearing coat)	0.0	274.3	478.2	608.7	664.8	1.75
Moment due to Pedestrian Live Load (LL)	0.0	0.0	0.0	0.0	0.0	1.5
Moment due to Live Load (LL)	0.0	1231.2	2105.4	2653.2	2908.2	1.5
Total Moment at ULS	**0.0**	**6370.2**	**10843.4**	**13464.0**	**14346.9**	

Prestressing Forces & Losess

Initial Prestress applied (KN)	10091	10233	10449	10565	10652
Total / Final Loses (KN)	1789	1787	1744	1721	1735
Final Prestress after all Loses (KN)	8302	8446	8705	8844	8917
c.g of Strands from Girder soffit (mm)	827	515	373	341	340

Material Data

Grade of Reinforcement Steel	Fe 500	Fe 500	Fe 500	Fe 500	Fe 500	
Charecteristic strength of Reinforcement, f_{yk} (Mpa)	500	500	500	500	500	Table 18.1, IRC 112
Partial factor for Reinforcement Steel, γ_s	1.15	1.15	1.15	1.15	1.15	Cl 6.2.2, IRC 112
Design value for Tensile Strength, $f_{yd} = f_{yk}/\gamma_s$ (Mpa)	434.8	434.8	434.8	434.8	434.78	Cl 6.2.2, IRC 112
Modulus of Elasticity of Reinforcement, Es (Gpa)	200	200	200	200	200	Cl 6.2.2, IRC 112
Grade of Concrete	M 45	M 45	M 45	M 45	M 45	
Charecteristics compressive strength, fck (Mpa)	45	45	45	45	45	Table 6.5, IRC 112
α	0.67	0.67	0.67	0.67	0.67	Cl 6.4.2.8, IRC 112
Concrete material factor, γ_m	1.5	1.5	1.5	1.5	1.5	Cl 6.4.2.8, IRC 112
Design value for concrete compressive strength, $f_{cd} = \alpha f_{ck}/\gamma_m$ (MPa)	20.1	20.1	20.1	20.1	20.1	Cl 6.4.2.8, IRC 112
Modulus of Elasticity of Concrete, E_{cm} (GPa)	34	34	34	34	34	Table 6.5, IRC 112
Modulus of Elasticity of Prestressing Steel, Ep (GPa)	195	195	195	195	195	
Charecteristic tensile strength of Prestressing steel, f_{pk} (Mpa)	1860	1860	1860	1860	1860	Cl 6.3.5, IRC 112
Partial factor for Prestressing Steel, γ_s	1.15	1.15	1.15	1.15	1.15	Cl 6.3.5, IRC 112
Design tensile strength of Prestressing steel, f_{pd} (MPa)	1407.1	1407.1	1407.1	1407.1	1407.1	Cl 6.3.5, IRC 112
Initial prestrain of prestressing steel	0.00490	0.00499	0.00514	0.00522	0.00526	

Geometric Data

Width of Bottom Flange of Girder, b (mm)	725	725	725	725	725
Total Depth of Girder, D (mm)	2220	2220	2220	2220	2220
c.g of Reinforcement from soffit girder, c (mm)	58	58	58	58	58
Dia. of Untensioned Reinforcement, ϕ (mm)	12	12	12	12	12
Effective depth of Girder, d (mm)	2162	2162	2162	2162	2162
Spacing of Reinforcement, s (mm) or	0	0	0	0	0
Number of Reinforcement Bar, n	6	6	6	6	6
Area of Untensioned Reinforcement, A_t (mm^2)	679	679	679	679	679
Number of Strands	88	88	88	88	88
Area of Single Strand (mm^2)	98.7	98.70	98.70	98.70	98.70
Total area of Strand (mm^2)	8686	8686	8686	8686	8686
c.g of Strands from soffit of Girder (mm)	827	515	373	341	340
Reinforcement (%)	0.0422	0.0422	0.0422	0.0422	0.0422

Ultimate Moment check

Position of N.A from Compression face, x (mm)	332	332	332	332	332
Total Compressive Force, C (N)	12516823	12516823	12516823	12516823	12516823
Total Tensile Force, T (N)	12516823	12516823	12516823	12516823	12516823
Difference, C-T	0	0	0	0	0
Strain in Concrete	0.0035	0.0035	0.0035	0.0035	0.0035
Strain in Reinforcement	0.01932	0.01932	0.01932	0.01932	0.01932
Strain in Prestressing Steel	0.01611	0.01949	0.02114	0.02156	0.02161
So, Stress in Reinforcement (MPa)	434.8	434.8	434.8	434.8	434.8
So, Stress in Prestressing Steel (MPa)	1407.1	1407.1	1407.1	1407.1	1407.1
Lever Arm from c/g of Reinforcement (mm)	2029.4	2029.4	2029.4	2029.4	2029.4
Lever Arm from c/g of Prestressing Steel (mm)	1260.3	1572.4	1714.6	1746.6	1746.9
Tensile Force in Reinforcement (KN)	295.0	295.0	295.0	295.0	295.0
Tensile Force in Prestressing Steel (KN)	12221.8	12221.8	12221.8	12221.8	12221.8
Moment of Resistance of the Section (KN-m)	16001	19817	21555	21945	21949
Status	**OK**	**OK**	**OK**	**OK**	**OK**

9.0 CHECK FOR SHEAR (ULS) FOR PSC T-GIRDER (OUTER) AS PER IRC : 112 - 2011 :-

Span Length (c/c of Bearing)		=	35.500	m		
Grade of Concrete of Girder	f_{ck}	=	45	Mpa		
Axial Tensile Stress of Concrete	$f_{ctk,0.05}$	=	2.5	Mpa		(Table 6.5 of IRC: 112 - 2011)
Longterm Strength factor for Concrete	α	=	0.67			
Partial factor for Concrete	γ_m	=	1.5			
Characteristic strength of Steel	f_{yk}	=	500	Mpa		
Partial factor for Reinforcement Steel	γ_s		1.15			
Nomonal Diameter of Prestressing Strands		=	12.7	mm		
Nominal Area of each Strands		=	98.7	mm^2		
Nos of Strands		=	88		8685.61	

Shear at Different Section (KN) for Inner Girder

Location	d' away from Support	L/8	L/4	3L/8	L/2	Factor ULS	As per table 3.2 of IRC 6-201
Distance from Support in (m)	2.00	4.438	8.875	13.313	17.750		
Loadings							
1st Stage DL (KN)	431.3	299.0	197.3	99.7	0.0	1.35	
2nd Stage DL (KN)	287.8	218.0	142.5	72.7	0.0	1.35	
SIDL (Crash Barrier) (KN)	94.0	76.3	43.8	7.3	26.7	1.35	
SIDL (Surfcaing/ Wearing coat) (KN)	76.3	67.8	47.2	26.3	5.1	1.75	
Carriageway LL (KN)	0.0	0.0	0.0	0.0	0.0	1.5	
Carriageway LL (KN)	339.9	336.6	289.9	223.7	128.3	1.5	

Total loss in Prestress	1788.9	1787.3	1744.0	1721.4	1735.0

Shear Resistance (refer Cl. 10.3.2 of IRC:112-2011)

Location	d' away from Support	L/8	L/4	3L/8	L/2	Remarks
Ultimate Shear, V_u (KN)	1741.0	1424.6	1035.3	624.2	237.4	
1. Elements not requiring Design Shear Reinforcement						
a) Effective Width, b_{wc} (m)	0.68	0.255	0.255	0.255	0.255	= web thk. - 0.5 x dia of duct.
b) Overall Depth, D (m)	2.220	2.220	2.220	2.220	2.220	= Depth of Girder + Depth of Deck Slab
c) CG of Tendons from Bottom Y_{ord} (m)	0.827	0.515	0.373	0.341	0.340	
d) Depth , d_b (m)	1.393	1.705	1.847	1.879	1.880	d_b = D - Y_{ord} of cable from soffit of girder
e) Axial Tensile Strength of Concrete, $f_{ctk,005}$ (Mpa)	2.50	2.50	2.50	2.50	2.50	Ref. Cl 10.3.2, IRC:112
f) Partial Safety factor fo Concrete, γ_m	1.50	1.50	1.50	1.50	1.50	
g) Partial Safety factor fo Steel, γ_m	1.15	1.15	1.15	1.15	1.15	
h) Design Tensile Strength of Concrete, f_{ctd} (Mpa)	1.67	1.67	1.67	1.67	1.67	= f_{ctd}/γ_m, where γ_m = 1.5
i) Applied Longitudinal Force,N_{ED} (KN)	8301.7	8445.5	8705.4	8843.9	8916.7	= Axial Prestressing Force - Total loss due to prestress
j) Compressive Stress due to prestress, σ_{cp} (Mpa)	4.01	4.02	4.02	4.02	4.02	Min. value of N_{ED}/Ac or 0.2 x fcd
k) Second Moment of Area, I (m^4)	1.0107	0.8445	0.8445	0.8445	0.8445	Calculated from section property
l) First Moment of Area, S (m^3)	0.6406	0.4971	0.4971	0.4971	0.4971	Considered from section property
m) Value of k_1	1.0	1.0	1.0	1.0	1.0	Ref. Cl 10.3.1, IRC:112
n) Effect of Vertical Prestress, V_{pd} (KN)	655.2	495.39	96.24	22.62	0.00	= Vertical Prestressing Force x (1 - Ratio of loss due to prestress with Applied lobgitudinal Force)
o) Area of Prestressing Steel, A_s(mm^2)	8686	8686	8686	8686	8686	
p) Value of K	1.38	1.34	1.33	1.33	1.33	Ref. Cl 10.3.1, IRC:112
q) $v_{min} = 0.031\ K^{3/2} fck^{1/2}$	0.337	0.323	0.319	0.318	0.318	Eq. 10.3
r) % of Prestressing Steel, ρ_1	0.009	0.020	0.018	0.018	0.018	= Area of Prestressing Steel/(b_{wc} x d)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk,0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk,0.5}/\gamma_m$	-1.67	-1.67	-1.67	-1.67	-1.67	- denotes for "tensile stress"
u) Flexural tensile strength (Mpa)	6.927	7.981	4.682	1.648	0.479	Flexural Tensile strength in service condition
v) Shear Zone	B	B	C	C	D	Applicable for Shear Zone as per fig. 10.1(a) of IRC:111
w) Is flexural tensile strength under max. BM is smaller than $f_{ctk0.5}/\gamma_m$	No	No	No	No	No	
x) Shear Resistance, V_{Rdc} = $(I*b_{wc}/s)*((f_{ctd})^2+k1*\sigma_{cp}*f_{ctd})^{0.5}$	3298.6	1333.7	1333.7	1333.7	1333.7	Ref. Eq. 10.4 of, IRC:113
y) Shear Resistance, $V_{Rdc} = [0.12*K*(80*\rho_1*f_{ck})^{0.33} + 0.15*\sigma_{cp}]\ b_w * d$	1066.0	549.3	584.0	591.7	591.8	Ref. Cl 10.3.2, IRC:112
z) Design Shear resistance	1066.0	549.3	584.0	591.7	591.8	
aa) Min. Conc. Shear capacity $v_{Rd.c}\ min = (v_{min} + 0.15 * \sigma_{cp})\ b_w * d$	888.0	402.8	434.1	441.2	441.2	Ref. Eq. 10.1 of IRC:112
Governing Values of, V_{Rdc}	**1066.0**	**549.3**	**584.0**	**591.7**	**591.8**	

2. Elements requiring Design Shear Reinforcement

a) **Net Design Shear Force, V_{NS}(KN)**	**1085.9**	**929.2**	**939.1**	**601.5**	**237.4**	**V_{NS} = Vu - Vpd**
b) Design yeild strength of Shear Reinforcement	347.8	347.8	347.8	347.8	347.8	
c) $\cot\theta$	2.50	2.50	2.50	2.50	2.50	Assumed value
d) Lever arm, z	1.39	1.71	1.85	1.88	1.88	= Difference between cg of tension force and compression force
e) Steel Shear capacity VRd.s = (Asw / s) z fywd cot θ, KN	3247.0	3974.7	4306.2	2464.1	2464.6	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity VRd.max = $\alpha cw*b*z*v1*[fcd/(\cot\theta + \tan\theta)]$, KN	4723.7	2169.8	2350.7	2391.3	2391.8	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of VRd.s & VRd.max	3247.0	2169.8	2350.7	2391.3	2391.8	
h) Design value for concrete compressive strength ($fcd = \alpha fck/\gamma m$), Mpa	20.1	20.1	20.1	20.1	20.1	
i) v1 =	0.60	0.60	0.60	0.60	0.60	as fck is less than 80 Mpa
j) αcw =	1.20	1.20	1.20	1.20	1.20	
m) Effective Width, b_{wc} (m)	0.68	0.255	0.255	0.255	0.255	
n) Depth , d (m)	1.393	1.705	1.847	1.879	1.880	
o) Dia. of Shear reinforcement (mm)	16	16	16	12	12	
p) No. of Links	2	2	2	2	2	
t) Spacing of Reinforcement	150	150	150	150	150	
r) Minimum shear reinf. ρ_{min} (mm^2/m)	915	420	455	463	463	Ref. Cl 10.3.3.5, IRC:112
t) **Shear Reinforcement, Asw**=	**402**	**402**	**402**	**226**	**226**	in mm2
u) V_{RDC} + V_{RDS}	**4313**	**2719**	**2935**	**2983**	**2984**	
v) Is (V_{RDC} + V_{RDS}) > V_{NS}	***OK***	***OK***	***OK***	***OK***	***OK***	
w) Actual shear reinf. provided(mm^2/m)	2681	2681	2681	1508	1508	
x) Is Shear reinf. Pro. > Min. shear reinf. ρ_{min}	***OK***	***OK***	***OK***	***OK***	***OK***	
	4.0	2.9	3.1	5.0	12.6	
	2.5	1.9	2.8	4.8	12.6	

10.0 DESIGN OF SHEAR CONNECTOR FOR OUTER GIRDER

(Refer Clause 10.3.4 of IRC : 112 - 2011)

Span Length (c/c of Bearing)		=	35.500	m
Grade of Concrete	f_{ck}	=	45	Mpa
Characteristic Strength of Steel	f_{yk}	=	500	Mpa

Shear at Different Section (KN) for Inner Girder

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Factor ULS
Distance from Support in (m)	2.000	4.438	8.875	13.313	17.750	
Loadings						
1st Stage DL (KN)	431.3	299.0	197.3	99.7	0.0	1.35
2nd Stage DL (KN)	287.8	218.0	142.5	72.7	0.0	1.35
SIDL (Crash Barrier) (KN)	94.0	76.3	43.8	7.3	26.7	1.35
SIDL (Surfcaing/ Wearing coat) (KN)	76.3	67.8	47.2	26.3	5.1	1.75
Carriageway LL (KN)	0.0	0.0	0.0	0.0	0.0	1.5
Carriageway LL (KN)	339.9	336.6	289.9	223.7	128.3	1.5

A. Check for Limiting Shear

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Remarks
1. Ultimate Shear, V_{ED} (KN)	1741.0	1424.6	1035.3	624.2	237.4	
2. Thk. Of Deck Slab (mm)	220	220	220	220	220	
3. Depth, d_b (m)	1.393	1.705	1.847	1.879	1.880	

B. Check for Interface Shear Reinforcement

Location	'd' away from Support	L/8	L/4	3L/8	L/2	Remarks
1. Ultimate Shear Capacity of Section uncracked in Bending (refer Cl. 10.3.2 of IRC:112-2011):-						
a) Width of Interface, b_i (mm)	1000	1000	1000	1000	1000	
b) Lever Arm, z (mm)	1283	1595	1737	1769	1770	
c) β	1.0	1.0	1.0	1.0	1.0	
d) Interface Shear Stress, V_{EDi} (Mpa)	1.357	0.893	0.596	0.353	0.134	
e) Surface Factor, μ	0.6	0.6	0.6	0.6	0.6	
f) Angle of Reinf. To the Interface, α	90	90	90	90	90	
g) $f_{yd,}$ (MPa)	400	400	400	400	400	
h) Max Co-existing Normal Stress, σ_n (Mpa)	0.000	0.000	0.000	0.000	0.000	
i) Required Reinforcement, ρ (%)	0.00565	0.00372	0.00248	0.00147	0.00056	
j) Minimum Reinforcement, ρ (%)	0.0015	0.0015	0.0015	0.0015	0.0015	
k) Required Area of Interface Reinforcement (mm^2/m)	5655	3721	2483	1500	1500	
l) Dia. of Shear reinforcement (mm)	16	16	16	12	12	Provide at web of Girder
m) No. of Legs	2	2	2	2	2	
n) Addl.Shear reinforcement dia (mm)	16	16	12	12	12	Provide at top of Girder
o) No. of Legs	4	2	2	2	2	
p) Spacing required (mm)	213	216	253	302	302	
q) Spacing provided (mm)	150	150	150	150	150	
r) Check, Spacing pro. > Spacing req.	**OK**	**OK**	**OK**	**OK**	**OK**	

11.0 Check For Stress Behind Anchorage & Design Of End Block

A. Check for Bearing Stress behind Anchorage (As per Cl. 7.3 of IRC : 18 - 2000)

$f_b = 0.48 * f_{cj}*(A2/A1)^{0.5}$ or 0.8*fcj whichever is lesser

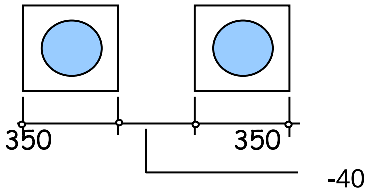
Where :

f_b = Allowable stress behind anchorages

fcj = Concrete Strength at the time of stressing (in Mpa)

A1 = Bearing area of anchorage (Eq. Square of dimension "a")

A2 = As defined in Clause 7.3 of IRC : 18 (Eq. Square of dimension "b")



fcj =	**40.59**	Mpa
a =	**350**	mm
b =	**300**	mm
f_b =	**16.7**	Mpa

B. Force acting at the anchorage

a) Force per cable at Jacking end after friction & Sli	=	2235.6	KN
b) Elastic Shortening Loss per cable	=	77.1	KN
c) Relaxation Loss per cable	=	136.1	KN
d) Net force after instanteneous losses , F_n	=	2022.5	KN
e) Stress behind anchorage, $\sigma_b=(F_n/a^2)$	=	16.5	Mpa

Status **OK**

C. Design of End Block & Calculation of Bursting Reinforcement (Ref. Clause 13.5.1 of IRC:112 - 2011

$2*Y_o$ = Side of End Block	=	300	mm
$2*Y_{po}$ = Side of loaded area	=	350	mm
P_k = Load in the Tendon	=	3307	KN
Y_{po}/Y_o =	=	**1.17**	
F_{bst}/P_k (from Table 13.1, IRC:112-2011)	=	**0.12**	
F_{bst} =	=	**397**	KN

Reinforcement required for this force is distributed from $0.2*Y_o$ to $2.0*Y_o$

$0.2*Y_o$ = 30 mm $2.0*Y_o$ = 300 mm

Area of Stirrups required = $F_{bst}/0.87*f_y$ =	=	**912**	mm^2
Provided, 6-legged T-16 in 2 layers, in the form of spiral, A_s =		**2412.0**	mm^2

Status **OK**

DESIGN BOX TYPE MINOR BRIDGE
2CELL - 6.0M X 3.5M

Input Data : -

Dimension Detail:-

1	Nos of cells of RCC multicell box structure		=	2	
2	Clear Span		=	6.000	m
3	Opening height		=	3.500	m
4	Angle of skew		=	0.0	deg.
5	Width of carriageway		=	12.500	m
6	Width of structure		=	13.500	m
7	Width of crash barrier		=	0.500	m
8	Width of footpath / raised safety kerb		=	0.000	m
9	Wearing coat (WC) thickness		=	100	mm
10	Filling on top / deck slab		=	0	mm
11	Filling on bottom / base slab		=	0	mm
12	Thickness of top / deck slab		=	0.575	m
13	Thickness of external / outer vertical wall		=	0.475	m
14	Thickness of internal vertical wall		=	0.375	m
15	Thickness of bottom / base slab		=	0.600	m
16	Horizontal thickness of haunch		=	0.300	m
17	Vertical thickness of haunch		=	0.300	m
18	Effective Clear Span		=	6.000	m
19	Design of road as		=	3	Lane

Design Parameter:-

20	Grade of concrete			=	M 35	
21	Grade of steel / reinforcement			=	Fe 500	
22	Clear cover to reinforcement	top / deck slab		=	50	mm
		outer wall		=	75	mm
		internal wall		=	50	mm
		bottom / base slab		=	75	mm
23	Unit weight of concrete			=	2.5	T/m^3
24	Unit weight of profile corrective course / filling			=	2.5	T/m^3
25	Unit weight of wearing coat (WC)			=	2.2	T/m^3
26	Unit weight of backfill soil			=	2.0	T/m^3
27	Angle of repose of backfill soil		ϕ	=	30	deg.
28	Angle of friction bet^w wall and earthfill		δ	=	20	deg.
29	Coefficient of earth pressure at rest		K_O	=	0.500	
30	Allowable safe bearing capacity at founding level			=	15.0	T/m^2
31	Allowable settlement at founding statra			=	25.0	mm
32	Spring Constant at founding statra			=	6000	KN/m

b. Load calculations for RCC multicell box structure"-

Dead Load calculation:-

Self weight of the structure has been calculated directly in STAAD file by the comment "SELFWEIGHT -1".

Super Imposed Dead Load calculattion:-

(a) Top Slab

Thickness of wearing coat		=	0.1	m
Load (UDL) due to WC	0.1 x 22	=	2.2	kN/m width
Thickness of earth/profile corrective course fill		=	0.0	m
Load (UDL) due to earth fill	0 x 20	=	0.0	KN/m
Total Load on top slab		=	2.2	kN/m width

Earth Pressure calculation:-

Total filling above top slab	=	0.100	m
Thickness of top slab	=	0.575	m
Height of haunch	=	0.300	m
Clear height between top & bottom slab	=	3.500	m
Height of haunch	=	0.300	m
Thickness of bottom slab	=	0.600	m
Free Board below Soffit level	=	0.600	m

Height from top (m)		Intensity of Earth pressure (KN/m^2)				
		Dry Earth Pressure			Submerged Pressure	Water Pressure
0.388	0.000	0.5 * 20 * 0.3875	=	3.9	0.0	0.0
0.288	0.675	0.5 * 20 * 0.675	=	6.8	2.9	0.0
0.300	0.975	0.5 * 20 * 0.975	=	9.8	4.4	0.0
2.900	3.875	0.5 * 20 * 3.875	=	38.8	18.9	26.0
0.300	4.175	0.5 * 20 * 4.175	=	41.8	20.4	29.0
0.300	4.475	0.5 * 20 * 4.475	=	44.8	21.9	32.0

Live Load Surcharge calculation:-

Equivalent height		=	1.2	m
Uniform Intensity of loading	1.2 x 0.5 x 20	=	12.0	KN/m^2

Braking Load calculation:-

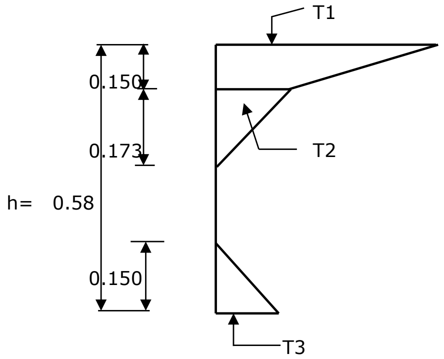
Carriageway Live Load

(a) 70 R wheel load	=	1000	KN
(b) Class A 1lane wheel load	=	554	KN
Effective Width of the box from Live load	=	13.5	m
Braking Load	=	16.9	KN/m

c. Calculation of Temperature gradient effect:-

Temperature Rise case:-

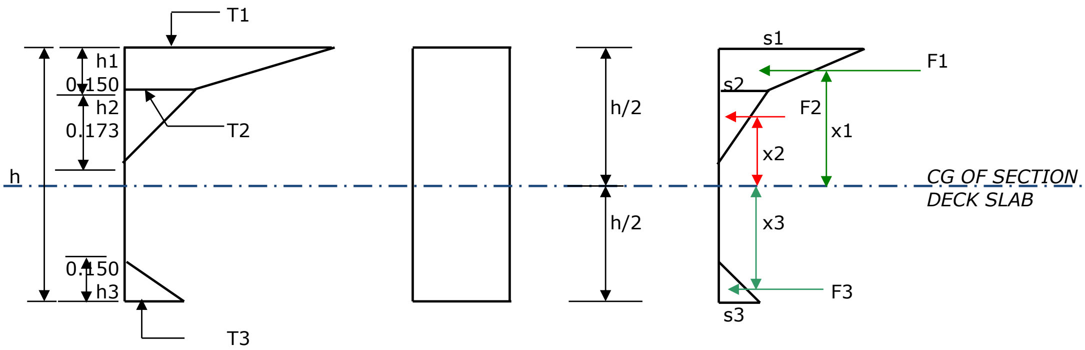
The top slab is designed for the effects of the distribution of the temperature across the deck depth as given in the sketch below.



Parameters:

Depth of Superstructure	h =	0.575	m
Grade of Concrete	M =	35	Mpa
Elasticity for Concrete	E_i =	3.23E+06	T / m^2
Modified Elasticity of Concrete	E_c = E_i	1.62E+06	T / m^2
Coeff. of Thermal Expansion	α =	0.000012	/ °C
Stress	σ =	E α t	
Force	F =	F1 + F2 + F3	
Moment	M =	F1x1 + F2x2 -F3x3	

Generalised Temperature & corresponding Force Diagram



Force & Moment calculation:-

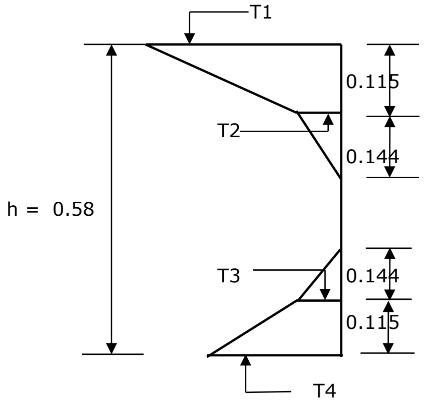
Member	h	h1	h2	h3	T1	T2	T3
	(m)	(m)	(m)	(m)	°C	°C	°C
Top slab	0.575	0.150	0.173	0.150	17.80	4.00	2.10

Member	s 1	s 2	s 3	F1	F2	F3	Force
	T / m2	T / m2	T / m2	T	T	T	T
Top slab	345.1	77.5	40.71	31.69	6.69	3.05	**41.4**

Member	cg from Top	cg from Bottom	cg of Top Block from Top	cg of Mid Block from Top	cg of Bottom Block from Bottom	x1	x2	x3	Moment
						m	m	m	Tm
Top slab	0.2875	0.2875	0.0592	0.2075	0.0500	0.2283	0.0800	0.2375	**7.0**

Temperature Fall case:-

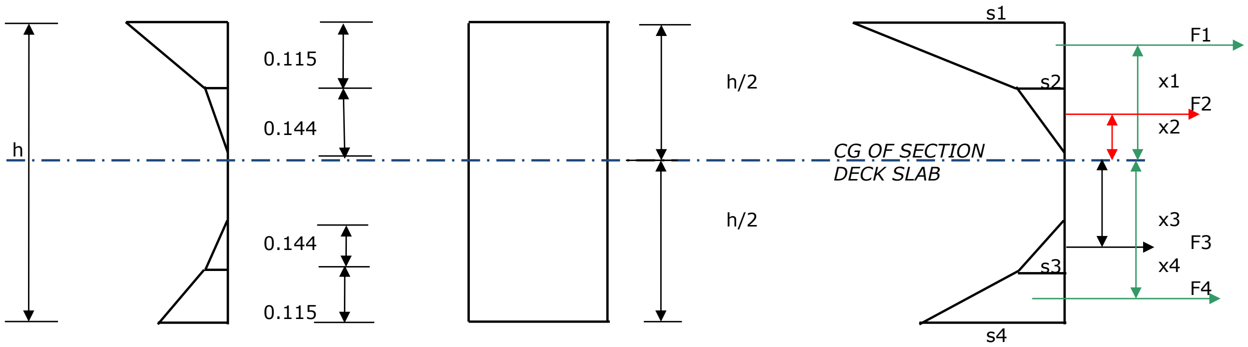
The top slab is designed for the effects of the distribution of the temperature across the deck depth as given in the sketch below.



Parameters:

Depth of Superstructure	h =	0.575	m
Grade of Concrete	M =	35	Mpa
Elasticity for Concrete	E_i =	3.23E+06	T / m^2
Modified Elasticity of Concrete	$E_c = E_i$	1.62E+06	T / m^3
Coeff. of Thermal Expansion	α =	1.2E-05	/ °C
Stress	σ =	E α t	
Force	F =	F1 + F2 + F3 + F4	
Moment	M =	F1x1 + F2x2 - F3x3 - F4x4	

Generalised Temperature & corresponding Force Diagram



Force & Moment calculation:-

Member	h	h1	h2	h3	h4	T1	T2	T3	T4
	(m)	(m)	(m)	(m)	(m)	°C	°C	°C	°C
Top slab	0.575	0.115	0.144	0.144	0.115	10.60	0.70	0.80	6.60

Member	s 1	s 2	s 3	s 4	F1	F2	F3	F4	Force
	T / m2	T / m2	T / m2	T / m3	T	T	T	T	T
Top slab	205.5	13.6	15.5	127.9	12.60	0.98	1.11	8.25	**22.9**

Member	cg from Top	cg from Bottom	cg of Top Block from Top	cg of Mid Block from Top	cg of Mid Block from Bottom	cg of Bottom Block from Bottom	x1	x2	x3	x4	Moment
							m	m	m	m	Tm
Top slab	0.288	0.288	0.041	0.163	0.163	0.073	0.247	0.125	0.125	0.215	**1.3**

Calculation Of Effective Width Of Live Load:-

Effective Live Load for 70R-Wheel Load

We find the effective live load & its position in a tabular as per IRC:-21 - 2000.

X-cord (start)	X-cord (end)	Member Number
0.000	0.238	36
0.238	0.538	37
0.538	0.813	38
0.813	3.238	39
3.238	5.663	40
5.663	5.938	41
5.938	6.238	42
6.238	6.425	43
6.425	6.613	44
6.613	6.913	45
6.913	7.188	46
7.188	9.613	47
9.613	12.038	48
12.038	12.313	49
12.313	12.613	50
12.613	12.850	51

0.22

Total load on 70 R-Wheel = 1220 kN with I.F = 1.22

Wearing coat thickness = 0.100 m

Load	Dimension of tyre		Load	Dist. Betwn.	Final Load
	Length	Width	kN	each load	kN
1st	0.187	0.86	170	0.00	207.4
2nd	0.187	0.86	170	1.37	207.4
3rd	0.187	0.86	170	3.05	207.4
4th	0.187	0.86	170	1.37	207.4
5th	0.132	0.86	120	2.13	146.4
6th	0.132	0.86	120	1.52	146.4
7th	0.088	0.86	80	3.96	97.6
			1000		1220

	Total Span	=	12.85	m
Lo	= Effective Span	=	6.00	m
Lo	= C/C span	=	6.43	m
b	= Width Of Slab	=	13.50	m
b /L o		=	2.25	
α =	a constant, depends on value of b /L o	=	2.60	
b1		=	1.06	m

	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Case 7		Case 8		Case 9	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	78.82	0.10	46.12	0.65	37.81	1.20	33.44	1.75	32.24	2.30	31.74	2.85	31.71	3.40	32.00	3.95	32.97	4.50
P2							52.97	0.38	41.21	0.93	35.22	1.48	32.73	2.03	31.90	2.58	31.73	3.13
P3																	81.96	0.08
P4											69.13							
P5																		
P6																		
P7																		

	Case 10		Case 11		Case 12		Case 13		Case 14		Case 15		Case 16		Case 17		Case 18	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	36.10	5.05	42.85	5.60	59.52	6.15	59.52	6.70	42.85	7.25	36.10	7.80	32.97	8.35	32.00	8.90	31.71	9.45
P2	31.78	3.68	32.41	4.23	34.03	4.78	39.00	5.33	48.48	5.88	96.65	6.43	48.24	6.98	38.88	7.53	33.97	8.08
P3	46.54	0.63	38.02	1.18	33.50	1.73	32.27	2.28	31.74	2.83	31.71	3.38	31.98	3.93	32.93	4.48	35.92	5.03
P4					53.59	0.36	41.50	0.91	35.38	1.46	32.78	2.01	31.92	2.56	31.73	3.11	31.77	3.66
P5													36.35	0.43	28.58	0.98	24.59	1.53
P6																	67.41	0.01
P7																		

	Case 19		Case 20		Case 21		Case 22		Case 23		Case 24		Case 25		Case 26		Case 27	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	31.74	10.00	32.24	10.55	33.44	11.10	37.81	11.65	46.12	12.20	78.82	12.75						
P2	32.39	8.63	31.78	9.18	31.73	9.73	31.91	10.28	32.75	10.83	35.30	11.38	41.35	11.93	53.28	12.48		
P3	42.52	5.58	57.96	6.13	61.19	6.68	43.19	7.23	36.28	7.78	33.02	8.33	32.03	8.88	31.71	9.43	31.73	9.98
P4	32.37	4.21	33.90	4.76	38.76	5.31	48.00	5.86	94.37	6.41	48.72	6.96	39.13	7.51	34.10	8.06	32.43	8.61
P5	23.03	2.08	22.49	2.63	22.41	3.18	22.45	3.73	22.94	4.28	24.26	4.83	27.97	5.38	35.10	5.93	60.91	6.48
P6	33.97	0.56	27.41	1.11	23.95	1.66	22.86	2.21	22.43	2.76	22.40	3.31	22.53	3.86	23.13	4.41	24.94	4.96
P7															23.97	0.45	18.93	1.00

	Case 28		Case 29		Case 30		Case 31		Case 32		Case 33		Case 34		Case 35		Case 36	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3	32.21	10.53	33.39	11.08	37.59	11.63	45.71	12.18	75.93	12.73								
P4	31.79	9.16	31.73	9.71	31.89	10.26	32.71	10.81	35.14	11.36	41.06	11.91	52.67	12.46				
P5	33.24	7.03	27.04	7.58	23.75	8.13	22.80	8.68	22.41	9.23	22.39	9.78	22.56	10.33	23.20	10.88	25.21	11.43
P6	29.24	5.51	37.72	6.06	47.96	6.61	31.37	7.16	26.07	7.71	23.44	8.26	22.67	8.81	22.39	9.36	22.39	9.91
P7	16.32	1.55	15.34	2.10	14.98	2.65	14.94	3.20	14.98	3.75	15.31	4.30	16.24	4.85	18.77	5.40	23.65	5.95

	Case 37		Case 38		Case 39		Case 40		Case 41		Case 42		Case 43		Case 44		Case 45	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5	29.73	11.98	39.62	12.53														
P6	22.66	10.46	23.43	11.01	26.04	11.56	31.30	12.11	47.58	12.66								
P7	38.96	6.50	21.95	7.05	17.92	7.60	15.78	8.15	15.19	8.70	14.94	9.25	14.92	9.80	15.05	10.35	15.49	10.90

	Case 46		Case 47		Case 48		Case 49		Case 50		Case 51		Case 52		Case 53		Case 54	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5																		
P6																		
P7	16.88	11.45	19.97	12.00	27.10	12.55												

Calculation Of Effective Width Of Live Load:-

Effective Live Load for 70R-Bogie Load

We find the effective live load & its position in a tabular as per IRC:-21 - 2000.

X-cord (start)	X-cord (end)	Member Number
0.000	0.238	36
0.238	0.538	37
0.538	0.813	38
0.813	3.238	39
3.238	5.663	40
5.663	5.938	41
5.938	6.238	42
6.238	6.425	43
6.425	6.613	44
6.613	6.913	45
6.913	7.188	46
7.188	9.613	47
9.613	12.038	48
12.038	12.313	49
12.313	12.613	50
12.613	12.850	51

0.22

Total load on 70 R-Wheel = 488 kN with I.F = 1.22

Wearing coat thickness = 0.100 m

Load	Dimension of tyre		Load	Dist. Betwn	Final Load
	Length	Width	kN	each load	kN
1st	0.221	0.86	200	0.00	244
2nd	0.221	0.86	200	1.22	244
			400		488

	Total Span	=	12.85	m
Lo	= Effective Span	=	6.00	m
Lo	= C/C span	=	6.43	m
b	= Width Of Slab	=	13.50	m
b /L o		=	2.25	
α =	a constant, depends on value of b /L o	=	2.60	
b1		=	1.06	m

	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Case 7		Case 8		Case 9	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	92.73	0.10	59.92	0.45	50.91	0.80	45.13	1.15	41.25	1.50	39.02	1.85	38.12	2.20	37.56	2.55	37.32	2.90
P2									87.73	0.13	58.96	0.48	50.31	0.83	44.73	1.18	40.98	1.53
	Case 10		Case 11		Case 12		Case 13		Case 14		Case 15		Case 16		Case 17		Case 18	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	37.35	3.25	37.35	3.60	37.65	3.95	38.28	4.30	39.26	4.65	41.96	5.00	46.18	5.35	52.50	5.70	62.50	6.05
P2	38.93	1.88	38.06	2.23	37.53	2.58	37.31	2.93	37.33	3.28	37.36	3.63	37.69	3.98	38.35	4.33	39.36	4.68
	Case 19		Case 20		Case 21		Case 22		Case 23		Case 24		Case 25		Case 26		Case 27	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	108.47	6.40	65.62	6.75	53.66	7.10	46.93	7.45	42.47	7.80	39.45	8.15	38.40	8.50	37.72	8.85	37.37	9.20
P2	42.26	5.03	46.62	5.38	53.19	5.73	64.03	6.08	113.70	6.43	63.27	6.78	52.96	7.13	46.47	7.48	42.16	7.83
	Case 28		Case 29		Case 30		Case 31		Case 32		Case 33		Case 34		Case 35		Case 36	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	37.33	9.55	37.31	9.90	37.51	10.25	38.02	10.60	38.87	10.95	40.81	11.30	44.48	11.65	49.93	12.00	58.35	12.35
P2	39.33	8.18	38.33	8.53	37.68	8.88	37.36	9.23	37.34	9.58	37.32	9.93	37.54	10.28	38.08	10.63	38.96	10.98
	Case 37		Case 38		Case 39		Case 40		Case 41		Case 42		Case 43		Case 44		Case 45	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	84.71	12.70																
P2	41.07	11.33	44.86	11.68	50.51	12.03	59.28	12.38	89.33	12.73								
	Case 46		Case 47		Case 48		Case 49		Case 50		Case 51		Case 52		Case 53		Case 54	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		

Effective Live Load for 70R-Track Load

We find the effective live load & its position in a tabular as per IRC:-21 - 2000.

X-cord (start)	X-cord (end)	Member Number
0.000	0.238	36
0.238	0.538	37
0.538	0.813	38
0.813	3.238	39
3.238	5.663	40
5.663	5.938	41
5.938	6.238	42
6.238	6.425	43
6.425	6.613	44
6.613	6.913	45
6.913	7.188	46
7.188	9.613	47
9.613	12.038	48
12.038	12.313	49
12.313	12.613	50
12.613	12.850	51

0.10

Total load on 70 R-Track = 770 kN with I.F = 1.10

Wearing coat thickness = 0.100 m

Load	Dimension of tyre Length	Width	Load kN	Dist. Betw each load	Final Load kN
1st	0.381	0.85	58.333	0	64.2
2nd	0.762	0.85	116.667	0.762	128.3
3rd	0.762	0.85	116.667	0.762	128.3
4th	0.762	0.85	116.667	0.762	128.3
5th	0.762	0.85	116.667	0.762	128.3
6th	0.762	0.85	116.667	0.762	128.3
7th	0.381	0.85	58.333	0.762	64.2
			700	4.57	770

Total Span = 12.9 m

Lo = Effective Span = 6.00 m

Lo = C/C span = 6.43 m

b = Width Of Slab = 13.50 m

b /L o = 2.25

α = a constant, depends on value of b /L o = 2.60

b1 = 1.05 m

	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Case 7		Case 8		Case 9	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	24.57	0.10	14.59	0.55	12.18	1.00	10.77	1.45	9.91	1.90	9.65	2.35	9.53	2.80	9.53	3.25	9.54	3.70
P2					39.02	0.24	27.40	0.69	23.34	1.14	20.91	1.59	19.62	2.04	19.20	2.49	19.04	2.94
P3									32.63	0.38	25.92	0.83	22.47	1.28	20.38	1.73	19.47	2.18
P4											42.78	0.06	29.69	0.51	24.66	0.96	21.71	1.41
P5															41.20	0.20	27.83	0.65
P6																		
P7																		

	Case 10		Case 11		Case 12		Case 13		Case 14		Case 15		Case 16		Case 17		Case 18	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	9.68	4.15	10.02	4.60	10.96	5.05	12.50	5.50	15.14	5.95	28.78	6.40	15.55	6.85	12.72	7.30	11.09	7.75
P2	19.04	3.39	19.15	3.84	19.51	4.29	20.52	4.74	22.70	5.19	26.31	5.64	34.19	6.09	47.95	6.54	28.99	6.99
P3	19.13	2.63	19.04	3.08	19.05	3.53	19.23	3.98	19.67	4.43	21.08	4.88	23.62	5.33	27.87	5.78	41.39	6.23
P4	19.93	1.86	19.33	2.31	19.07	2.76	19.07	3.21	19.08	3.66	19.34	4.11	19.93	4.56	21.73	5.01	24.69	5.46
P5	23.60	1.10	21.07	1.55	19.67	2.00	19.23	2.45	19.05	2.90	19.04	3.35	19.13	3.80	19.47	4.25	20.39	4.70
P6	34.06	0.34	26.28	0.79	22.68	1.24	20.51	1.69	19.50	2.14	19.14	2.59	19.04	3.04	19.04	3.49	19.20	3.94
P7			28.58	0.03	15.12	0.48	12.48	0.93	10.95	1.38	10.02	1.83	9.68	2.28	9.54	2.73	9.53	3.18

	Case 19		Case 20		Case 21		Case 22		Case 23		Case 24		Case 25		Case 26		Case 27	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	10.11	8.20	9.71	8.65	9.55	9.10	9.53	9.55	9.53	10.00	9.63	10.45	9.87	10.90	10.65	11.35	11.99	11.80
P2	24.26	7.44	21.47	7.89	19.79	8.34	19.29	8.79	19.06	9.24	19.06	9.69	19.09	10.14	19.38	10.59	20.09	11.04
P3	38.30	6.68	27.25	7.13	23.26	7.58	20.86	8.03	19.61	8.48	19.19	8.93	19.04	9.38	19.04	9.83	19.15	10.28
P4	29.74	5.91	53.16	6.36	32.15	6.81	25.79	7.26	22.39	7.71	20.34	8.16	19.45	8.61	19.12	9.06	19.05	9.51
P5	22.48	5.15	25.95	5.60	32.75	6.05	51.43	6.50	29.50	6.95	24.55	7.40	21.65	7.85	19.89	8.30	19.32	8.75
P6	19.63	4.39	20.93	4.84	23.36	5.29	27.44	5.74	39.20	6.19	40.38	6.64	27.67	7.09	23.50	7.54	21.01	7.99
P7	9.53	3.63	9.65	4.08	9.91	4.53	10.78	4.98	12.20	5.43	14.61	5.88	24.72	6.33	16.76	6.78	13.07	7.23

	Case 28		Case 29		Case 30		Case 31		Case 32		Case 33		Case 34		Case 35		Case 36	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	14.25	12.25	22.43	12.70														
P2	21.98	11.49	25.11	11.94	30.49	12.39	59.35	12.84										
P3	19.52	10.73	20.57	11.18	22.78	11.63	26.45	12.08	34.74	12.53								
P4	19.05	9.96	19.24	10.41	19.69	10.86	21.14	11.31	23.71	11.76	28.03	12.21	42.25	12.66				
P5	19.07	9.20	19.06	9.65	19.08	10.10	19.35	10.55	19.97	11.00	21.80	11.45	24.80	11.90	29.93	12.35	54.67	12.80
P6	19.65	8.44	19.22	8.89	19.04	9.34	19.04	9.79	19.13	10.24	19.48	10.69	20.44	11.14	22.56	11.59	26.08	12.04
P7	11.30	7.68	10.23	8.13	9.75	8.58	9.57	9.03	9.52	9.48	9.52	9.93	9.61	10.38	9.82	10.83	10.49	11.28

	Case 37		Case 38		Case 39		Case 40		Case 41		Case 42		Case 43		Case 44		Case 45	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5																		
P6	33.25	12.49																
P7	11.73	11.73	13.79	12.18	19.98	12.63												

	Case 46		Case 47		Case 48		Case 49		Case 50		Case 51		Case 52		Case 53		Case 54	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5																		
P6																		
P7																		

Calculation Of Effective Width Of Live Load:-

Effective Live Load for Class-A Wheel Load

We find the effective live load & its position in a tabular as per IRC:-21 - 2000.

X-cord (start)	X-cord (end)	Member Number
0.000	0.238	36
0.238	0.538	37
0.538	0.813	38
0.813	3.238	39
3.238	5.663	40
5.663	5.938	41
5.938	6.238	42
6.238	6.425	43
6.425	6.613	44
6.613	6.913	45
6.913	7.188	46
7.188	9.613	47
9.613	12.038	48
12.038	12.313	49
12.313	12.613	50
12.613	12.850	51

0.38

Total load on Class -A Wheel = 761.75 kN with I.F = 1.38

Wearing coat thickness = 0.100 m

Load	Dimension of tyre		Load	Dist. Betwn.	Final Load
	Length	Width	kN	each load	kN
1st	0.150	0.2	27.000	0.0	37.1
2nd	0.150	0.2	27.000	1.1	37.1
3rd	0.250	0.5	114.000	3.2	156.8
4th	0.250	0.5	114.000	1.2	156.8
5th	0.200	0.38	68.000	4.3	93.5
6th	0.200	0.38	68.000	3.0	93.5
7th	0.200	0.38	68.000	3.0	93.5
8th	0.200	0.38	68.000	3.0	93.5
			554		761.75

Total Span = 12.85 m

Lo = Effective Span = 6.00 m

Lo = C/C span = 6.43 m

b = Width Of Slab = 13.50 m

b /L o = 2.25

α = a constant, depends on value of b /L o = 2.60

b1 for 27 kN = 0.40 m

b1 for 68 kN = 0.58 m

b1 for 114 kN = 0.70 m

	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Case 7		Case 8		Case 9	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	15.12	0.10	9.65	0.85	8.39	1.60	7.80	2.35	7.66	3.10	7.72	3.85	8.16	4.60	9.17	5.35	12.38	6.10
P2					10.95	0.50	8.87	1.25	8.01	2.00	7.68	2.75	7.66	3.50	7.89	4.25	8.61	5.00
P3													48.36	0.30	37.53	1.05	33.44	1.80
P4																	41.78	0.60
P5																		
P6																		
P7																		
P8																		

	Case 10		Case 11		Case 12		Case 13		Case 14		Case 15		Case 16		Case 17		Case 18	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1	11.51	6.85	8.99	7.60	8.07	8.35	7.69	9.10	7.65	9.85	7.85	10.60	8.51	11.35	9.89	12.10	16.88	12.85
P2	10.09	5.75	15.52	6.50	9.71	7.25	8.42	8.00	7.81	8.75	7.66	9.50	7.71	10.25	8.14	11.00	9.13	11.75
P3	31.63	2.55	31.37	3.30	31.89	4.05	34.15	4.80	38.98	5.55	55.62	6.30	41.49	7.05	35.40	7.80	32.43	8.55
P4	35.54	1.35	32.49	2.10	31.38	2.85	31.39	3.60	32.56	4.35	35.69	5.10	42.07	5.85	53.28	6.60	38.54	7.35
P5									37.27	0.05	24.03	0.80	20.85	1.55	19.34	2.30	18.93	3.05
P6																	37.27	0.05
P7																		
P8																		

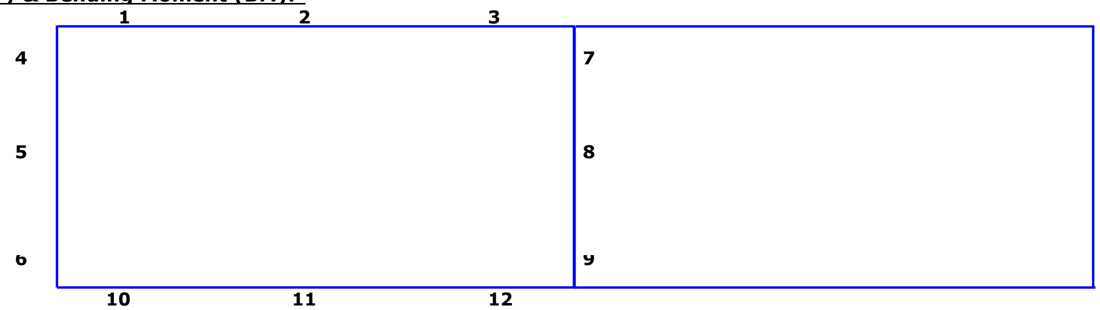
	Case 19		Case 20		Case 21		Case 22		Case 23		Case 24		Case 25		Case 26		Case 27	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2	12.14	12.50																
P3	31.37	9.30	31.40	10.05	32.63	10.80	35.84	11.55	42.37	12.30								
P4	33.93	8.10	31.81	8.85	31.39	9.60	31.69	10.35	33.63	11.10	37.92	11.85	50.19	12.60				
P5	19.04	3.80	20.04	4.55	22.38	5.30	28.38	6.05	28.38	6.80	22.38	7.55	20.04	8.30	19.04	9.05	18.93	9.80
P6	24.03	0.80	20.85	1.55	19.34	2.30	18.93	3.05	19.04	3.80	20.04	4.55	22.38	5.30	28.38	6.05	28.38	6.80
P7							37.27	0.05	24.03	0.80	20.85	1.55	19.34	2.30	18.93	3.05	19.04	3.80
P8															37.27	0.05	24.03	0.80

	Case 28		Case 29		Case 30		Case 31		Case 32		Case 33		Case 34		Case 35		Case 36	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5	19.34	10.55	20.85	11.30	24.03	12.05	37.27	12.80										
P6	22.38	7.55	20.04	8.30	19.04	9.05	18.93	9.80	19.34	10.55	20.85	11.30	24.03	12.05	37.27	12.80		
P7	20.04	4.55	22.38	5.30	28.38	6.05	28.38	6.80	22.38	7.55	20.04	8.30	19.04	9.05	18.93	9.80	19.34	10.55
P8	20.85	1.55	19.34	2.30	18.93	3.05	19.04	3.80	20.04	4.55	22.38	5.30	28.38	6.05	28.38	6.80	22.38	7.55

	Case 37		Case 38		Case 39		Case 40		Case 41		Case 42		Case 43		Case 44		Case 45	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5																		
P6																		
P7	20.85	11.30	24.03	12.05	37.27	12.80												
P8	20.04	8.30	19.04	9.05	18.93	9.80	19.34	10.55	20.85	11.30	24.03	12.05	37.27	12.80				

	Case 46		Case 47		Case 48		Case 49		Case 50		Case 51		Case 52		Case 53		Case 54	
	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position	Load	Position
P1																		
P2																		
P3																		
P4																		
P5																		
P6																		
P7																		
P8																		

Summary of Shear force (SF) & Bending Moment (BM):-



Summary of Bending Moment & Shear Force:-

Design Component	Section	Face	ULS/STRUCTURAL STRENGTH CHECK LOAD COMBINATION								SLS/RARE LOAD COMBINATION		SLS/QUASI-PERMANENT LOAD COMBINATION	
			BM (KN-m)				SF (KN)				BM (KN-m)		BM (KN-m)	
			DL+SIDL+EP*+Surcharge+Breaking	DL+SIDL+EP+Surcharge+Breaking	Design BM (EP leading load) M_{ULS}	Design BM (LL leading load) M_{ULS}	DL+SIDL+EP*+Surcharge+Breaking	DL+SIDL+EP+Surcharge+Breaking	Design SF (EP leading load) SF_{ULS}	Design SF (LL leading load) SF_{ULS}	DL+SIDL+EP+Surcharge+Breaking+Temp	Design BM (LL leading load) M_{RARE}	DL+SIDL+EP+Temp	Design BM M_{QP}
Top/Deck slab	1	Top	57.4	41.5	115.3	117.0	47.9	42.0	47.9	42.0	52.5	102.8	21.4	21.4
		Bottom	31.5	44.5	70.8	95.9	57.1	47.5	57.1	47.5	30.3	64.6	14.5	14.5
	2	Top	0.0	0.0	20.3	26.5	18.4	19.5	18.4	19.5	0.0	17.6	0.0	0.0
		Bottom	41.8	44.1	120.4	146.7	32.6	36.0	32.6	36.0	31.7	100.1	31.2	31.2
	3	Top	176.8	185.1	279.0	318.6	83.2	84.7	83.2	84.7	129.8	218.8	115.0	115.0
		Bottom	0.0	0.0	3.5	4.6	97.4	101.0	97.4	101.0	0.0	3.1	0.0	0.0
Outer wall	4	Outer	60.7	50.2	127.7	137.7	0.0	0.0	20.8	27.2	43.7	102.0	15.3	15.3
		Inner	25.4	36.7	67.1	91.0	71.0	48.7	87.8	70.6	43.4	79.6	19.6	19.6
	5	Outer	48.5	35.8	102.4	106.2	40.0	43.9	60.8	71.0	20.4	67.3	16.4	16.4
		Inner	42.1	24.6	54.9	41.2	0.0	0.0	16.8	21.9	36.9	48.0	15.9	15.9
	6	Outer	199.9	191.9	280.6	297.1	174.1	142.6	194.9	169.8	150.6	220.8	120.5	120.5
		Inner	0.0	0.0	26.4	34.5	0.0	0.0	16.8	21.9	0.0	23.0	0.0	0.0
Inner wall	7	Outer	48.2	53.0	79.6	94.0	0.0	0.0	8.4	10.9	35.3	62.6	0.0	0.0
		Inner	0.0	0.0	31.4	41.0	23.2	25.4	31.5	36.3	0.0	27.3	0.0	0.0
	8	Outer	0.0	0.0	13.6	17.7	0.0	0.0	8.4	10.9	0.0	11.8	0.0	0.0
		Inner	3.9	4.2	17.8	22.2	23.2	25.4	31.5	36.3	2.8	14.8	0.0	0.0
	9	Outer	0.0	0.0	23.8	31.1	0.0	0.0	8.4	10.9	0.0	20.7	0.0	0.0
		Inner	49.1	61.7	73.2	93.2	23.2	25.4	31.5	36.3	40.8	61.8	0.0	0.0
Bottom/Base slab	10	Top	0.0	0.0	34.8	45.4	124.0	116.8	203.3	220.3	0.0	30.3	0.0	0.0
		Bottom	221.4	202.1	280.0	278.4	123.5	118.1	219.0	242.7	164.7	215.6	123.2	123.2
	11	Top	127.5	132.9	213.6	245.2	40.9	34.9	58.3	57.6	93.4	168.2	75.4	75.4
		Bottom	0.0	0.0	20.6	26.8	53.4	49.3	70.9	72.1	0.0	17.9	0.0	0.0
	12	Top	0.0	0.0	22.7	29.7	95.0	95.2	178.8	204.5	0.0	19.8	0.0	0.0
		Bottom	99.9	148.2	231.0	319.1	121.3	125.3	208.7	239.3	104.6	218.6	87.4	87.4

Summary of Bending Moment & Shear Force due to Live Load:-

Design Component	Beam No	Face	70 R Wheel Load		70 R Track Load		Class A 3 lane Load		70 RW Bogie Load		BM (lf=1)	SF (lf=1)	lf=1.15 (LL as accompanying load)	Desgin BM	Desgin SF	lf=1.5 (LL as leading load)	Desgin BM	Desgin SF
			BM(KN-m)	SF(KN)	BM(KN-m)	SF(KN)	BM(KN-m)	SF(KN)	BM(KN-m)	SF(KN)								
Top/Deck slab	1	Top	39.2	67.8	50.3	68.7	34.7	67.5	36.8	67.8	50.3	68.7	1.15	57.9	79.0	1.5	75.5	103.1
		Bottom	34.2	64.1	32.9	63.5	20.3	62.3	31.2	63.5	34.2	64.1	1.15	39.4	73.7	1.5	51.3	96.2
	2	Top	15.6	24.9	17.6	29.0	11.6	39.5	14.0	23.8	17.6	39.5	1.15	20.3	45.4	1.5	26.5	59.2
		Bottom	53.7	27.4	68.4	25.7	59.5	31.8	54.0	26.7	68.4	31.8	1.15	78.6	36.6	1.5	102.6	47.7
	3	Top	88.9	82.6	71.7	75.4	73.7	67.2	87.0	80.6	88.9	82.6	1.15	102.3	94.9	1.5	133.4	123.8
		Bottom	0.0	80.2	3.1	72.3	0.0	73.6	0.0	79.5	3.1	80.2	1.15	3.5	92.3	1.5	4.6	120.4
Outer wall	4	Outer	49.9	17.7	58.3	18.1	42.0	7.7	50.0	16.3	58.3	18.1	1.15	67.1	20.8	1.5	87.5	27.2
		Inner	35.5	8.9	33.2	14.6	21.3	8.0	36.2	8.0	36.2	14.6	1.15	41.6	16.8	1.5	54.3	21.9
	5	Outer	41.0	17.7	46.9	18.1	34.4	7.7	38.7	17.7	46.9	18.1	1.15	54.0	20.8	1.5	70.4	27.2
		Inner	11.1	8.9	10.9	14.6	4.3	8.0	10.7	14.6	11.1	14.6	1.15	12.7	16.8	1.5	16.6	21.9
	6	Outer	70.2	17.7	66.6	18.1	48.0	7.7	68.6	18.1	70.2	18.1	1.15	80.7	20.8	1.5	105.3	27.2
		Inner	14.2	8.9	23.0	14.6	9.7	8.0	12.4	9.7	23.0	14.6	1.15	26.4	16.8	1.5	34.5	21.9
Inner wall	7	Outer	18.6	7.1	27.3	7.3	17.7	5.0	16.9	6.4	27.3	7.3	1.15	31.4	8.4	1.5	41.0	10.9
		Inner	15.8	4.2	27.3	7.3	13.7	4.7	13.8	3.6	27.3	7.3	1.15	31.4	8.4	1.5	41.0	10.9
	8	Outer	11.8	7.1	11.5	7.3	7.9	5.0	10.6	6.9	11.8	7.3	1.15	13.6	8.4	1.5	17.7	10.9
		Inner	9.2	4.2	12.0	7.3	5.5	4.7	9.2	3.7	12.0	7.3	1.15	13.8	8.4	1.5	18.1	10.9
	9	Outer	20.7	7.1	10.9	7.3	9.3	5.0	18.7	6.3	20.7	7.3	1.15	23.8	8.4	1.5	31.1	10.9
		Inner	17.1	4.2	21.0	7.3	8.0	4.7	15.9	4.0	21.0	7.3	1.15	24.1	8.4	1.5	31.4	10.9
Bottom/Base slab	10	Top	17.0	69.0	30.3	68.0	11.2	52.6	16.6	68.0	30.3	69.0	1.15	34.8	79.4	1.5	45.4	103.5
		Bottom	48.5	83.1	50.9	57.7	32.4	56.7	46.4	80.4	50.9	83.1	1.15	58.5	95.5	1.5	76.3	124.6
	11	Top	74.8	9.4	67.6	15.2	51.1	7.2	70.8	8.1	74.8	15.2	1.15	86.1	17.4	1.5	112.3	22.7
		Bottom	17.9	9.4	15.2	15.2	10.4	7.2	16.8	9.0	17.9	15.2	1.15	20.6	17.5	1.5	26.8	22.8
	12	Top	19.8	72.9	18.4	62.9	6.3	46.4	18.8	72.7	19.8	72.9	1.15	22.7	83.8	1.5	29.7	109.3
		Bottom	104.4	76.0	113.9	62.6	68.8	43.4	104.4	73.0	113.9	76.0	1.15	131.0	87.4	1.5	170.9	114.0

Design of RCC Box structures Component:-

Material Properties:-

Grade of Concrete			=	M 35	MPa	
Characteristic compressive cube strength of Concrete at 28 days		f_{ck}	=	35	MPa	
Mean value of Concrete cube compressive strength		f_{cm}	=	45	MPa	Page:235 of IRC:112-2011 & Table 6.5 of IRC:112-2011
Grade of Steel Reinforcement			=	Fe 500	MPa	
Characteristic yield strength of Reinforcement		f_{yk}	=	500	MPa	
Design yield of strength of shear Reinforcement		f_{ywd}	=	400	MPa	Page:86 of IRC:112-2011
Modulus of Elasticity of Steel		E_s	=	200000	MPa	Page:29 of IRC:112-2011
Modulus of Elasticity of Concrete,	for Short Term loading	E_{cm}	=	32308	MPa	Page:236 of IRC:112-2011 & Table 6.5 of IRC:112-2011
	for Long Term loading	E_{cm}'	=	13842	MPa	for Top/Deck slab
				13726	MPa	for Outer wall
				13576	MPa	for Inner wall
				16154	MPa	for Bottom/Base slab
Mean value of axial tensile strength of Concrete		f_{ctm}	=	2.8	MPa	Page:235 of IRC:112-2011 & Table 6.5 of IRC:112-2011
Mean value of tensile strength of Concrete at time of cracks		$f_{ct.eff}$	=	2.9	MPa	
Design compressive strength of concrete		f_{cd}	=	15.6	MPa	
Ultimate compressive strain in concrete		ε_{cu2}	=	0.0035		
Characteristic strain of Steel at maximum load		ε_{uk}	=	0.0045		
Limiting design strain of Steel		ε_{ud}	=	0.00405		
Limiting value of depth of N.A to effective depth (d)		xu_{max}/d	=	0.464		
Age of concrete at the time of loading		t_0	=	90	days	
t_∞ considered		t_∞	=	25550	days	
Creep factor		ϕ	=	1.33	for Top/Deck slab	Refer:- Creep coefficient calculation
				1.35	for Outer wall	
				1.38	for Inner wall	
				1.00	for Bottom/Base slab	

Permissible Stresses:-

Permissible concrete compressive stress i.e	$0.48*f_{ck}$	=	16.8	MPa	at SLS Rare load combination
Permissible concrete compressive stress i.e	$0.36*f_{ck}$	=	12.6	MPa	at SLS QP load combination
Permissible tensile stress in steel i.e	$0.8*f_{yk}$	=	400	MPa	
Permissible Crack width (for Moderate Exposure)	w_k	=	0.3	mm	at SLS QP load combination

Minimum Reinforcement:-

As,min = max [$0.26*f_{ctm}/f_{yk}*b_t*d$, $0.0013*b_t*d$] Refer IRC:112, cl 16.5.1.1)

where,

b_t = mean width of tension zone

Maximum Reinforcement:-

Tension Reinf.	≤	$0.025*A_c$	At section other than lap
Tension + Compression Reinf.	≤	$0.04*A_c$	At any section

where, A_c = Gross cross sectional area of concrete

A) Ultimate Limit State (ULS) :-

1) ULS Capacity Check:-

Main tensile Reinforcement calculation

Design Component	Section	Face	M_{Ed}	b	Overall depth, D	d	Area of steel provided: Dia.	Spacing	Dia.	Spacing	$A_{st, pro}$	xu_{max}	$xu=0.87f_{yk}*A_{st}/0.362f_{ck}*b$	Check	$z=(d-0.416xu)$	$A_{st,cal}=M_{Ed}/0.87 f_{yk}*z$	$A_{st,max}$	Check $A_{st,cal}<A_{st,pro}<A_{st,max}$	$A_{st,min}$	Check $A_{st,pro}>A_{st,min}$	$\Delta F_d = 0.5V_{Ed}$	$M_{Ed}/z+\Delta F_d$	$M_{Rd}=0.87 * f_{yk} * A_{st} * z$	M_{Rd}/z	Check $M_{Rd}/z > M_{Ed}/z+\Delta F_d$
			Tm	mm	mm	mm	mm	mm	mm	mm	mm^2	mm	mm		mm	mm^2	mm^2		mm^2		T	T	Tm	Tm/m	
Top/Deck slab	1	Top	11.7	1000	575	519.0	ts1-12	100	w1-12	100	2262	240.6	77.7	**UR,OK**	486.7	552	12975	**OK**	748	**OK**	2.4	26.4	47.9	98.4	**OK**
		Bottom	9.6	1000	575	519.0	ts2-12	100		100	1131	240.6	38.8	**UR,OK**	502.8	438	12975	**OK**	748	**OK**	2.9	21.9	24.7	49.2	**OK**
	2	Top	2.6	1000	575	519.0	ts1-12	100		100	1131	240.6	38.8	**UR,OK**	502.8	121	12975	**OK**	748	**OK**	1.0	6.2	24.7	49.2	**OK**
		Bottom	14.7	1000	575	519.0	ts2-12	100	ts8-00	100	1131	240.6	38.8	**UR,OK**	502.8	670	12975	**OK**	748	**OK**	1.8	31.0	24.7	49.2	**OK**
	3	Top	31.9	1000	575	519.0	ts1-12	100	ts3-12	100	2262	240.6	77.7	**UR,OK**	486.7	1505	12975	**OK**	748	**OK**	4.2	69.7	47.9	98.4	**OK**
		Bottom	0.5	1000	575	519.0	ts2-12	100		100	1131	240.6	38.8	**UR,OK**	502.8	21	12975	**OK**	748	**OK**	5.0	6.0	24.7	49.2	**OK**
Outer wall	4	Outer	13.8	1000	475	394.0	w1-12	100	ts1-12	100	2262	182.6	77.7	**UR,OK**	361.7	875	9850	**OK**	568	**OK**	1.4	39.4	35.6	98.4	**OK**
		Inner	9.1	1000	475	394.0	w2-12	100		100	1131	182.6	38.8	**UR,OK**	377.8	554	9850	**OK**	568	**OK**	4.4	28.5	18.6	49.2	**OK**
	5	Outer	10.6	1000	475	394.0	w1-12	100		100	1131	182.6	38.8	**UR,OK**	377.8	646	9850	**OK**	568	**OK**	3.6	31.7	18.6	49.2	**OK**
		Inner	5.5	1000	475	394.0	w2-12	100		100	1131	182.6	38.8	**UR,OK**	377.8	334	9850	**OK**	568	**OK**	1.1	15.6	18.6	49.2	**OK**
	6	Outer	29.7	1000	475	394.0	w1-12	100	bs1-16	100	3142	182.6	107.9	**UR,OK**	349.1	1957	9850	**OK**	568	**OK**	9.7	94.9	47.7	136.7	**OK**
		Inner	3.4	1000	475	394.0	w2-12	100		100	1131	182.6	38.8	**UR,OK**	377.8	210	9850	**OK**	568	**OK**	1.1	10.2	18.6	49.2	**OK**
Inner wall	7	Outer	9.4	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	713	7975	**OK**	460	**OK**	0.5	31.6	14.9	49.2	**OK**
		Inner	4.1	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	311	7975	**OK**	460	**OK**	1.8	15.3	14.9	49.2	**OK**
	8	Outer	1.8	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	135	7975	**OK**	460	**OK**	0.5	6.4	14.9	49.2	**OK**
		Inner	2.2	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	169	7975	**OK**	460	**OK**	1.8	9.1	14.9	49.2	**OK**
	9	Outer	3.1	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	236	7975	**OK**	460	**OK**	0.5	10.8	14.9	49.2	**OK**
		Inner	9.3	1000	375	319.0	w3-12	100		100	1131	147.9	38.8	**UR,OK**	302.8	707	7975	**OK**	460	**OK**	1.8	32.6	14.9	49.2	**OK**
Bottom/Base slab	10	Top	4.5	1000	600	517.0	bs3-16	100		100	2011	239.7	69.0	**UR,OK**	488.3	214	12925	**OK**	745	**OK**	11.0	20.3	42.7	87.5	**OK**
		Bottom	28.0	1000	600	517.0	bs1-16	100	w1-12	100	3142	239.7	107.9	**UR,OK**	472.1	1363	12925	**OK**	745	**OK**	12.1	71.4	64.5	136.7	**OK**
	11	Top	24.5	1000	600	517.0	bs3-16	100	bs6-00	100	2011	239.7	69.0	**UR,OK**	488.3	1154	12925	**OK**	745	**OK**	2.9	53.1	42.7	87.5	**OK**
		Bottom	2.7	1000	600	517.0	bs1-16	100		100	2011	239.7	69.0	**UR,OK**	488.3	126	12925	**OK**	745	**OK**	3.6	9.1	42.7	87.5	**OK**
	12	Top	3.0	1000	600	517.0	bs3-16	100		100	2011	239.7	69.0	**UR,OK**	488.3	140	12925	**OK**	745	**OK**	10.2	16.3	42.7	87.5	**OK**
		Bottom	31.9	1000	600	517.0	bs1-16	100	bs2-12	100	3142	239.7	107.9	**UR,OK**	472.1	1554	12925	**OK**	745	**OK**	12.0	79.6	64.5	136.7	**OK**

ΔF_d = Additional tensile force, to be accounted in longitudinal reinforcement

Distribution Reinforcement calculation:- (Refer IRC:112-2011 clause 16.1.1)

Distribution Reinforcement shall be at least 20% of main reinforcement.

Design Component	Section	Face	$A_{st, req}$	Distribution Reinforcement: Dia.	Spacing	$A_{st, pro}$	Check
			mm^2/m	mm	mm	mm^2/m	
Top/Deck slab	1,2,3	Top	452	ts4-10	100	785	**OK**
		Bottom	226	ts5-10	100	785	**OK**
Outer wall	4,5,6	Outer	628	w4-10	100	785	**OK**
		Inner	226	w5-10	100	785	**OK**
Inner wall	7,8,9	Outer	226	w6-10	150	524	**OK**
		Inner	226	w6-10	150	524	**OK**
Bottom/Base	10,11,12	Top	402	bs4-10	100	785	**OK**
		Bottom	628	bs5-10	100	785	**OK**

2) ULS Shear Check:-

Check for Maximum & Minimum Shear Capacity

Design Component	Section	Face	V_{Ed}	b_w	d	z=0.9d	α_{cw}	$\nu 1$	$V_{Rd.max}=\alpha_{cw}*b_w*z*\nu 1*f_{cd}/2$	Check	$\nu=0.6[1-f_{ck}/310)$	$0.5*b_w*d*\nu*f_{cd}$	$V_{Ed} \leq 0.5*b_w*d*\nu*f_{cd}$	θ	θ adopted	$\Delta F_d=0.5V_{Ed}*\cot\theta$
			T	mm	mm	mm			T				Check	deg	deg	T
Top/Deck slab	1	Top	4.8	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	0.63	45	2.4
		Bottom	5.7	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	0.75	45	2.9
	2	Top	1.9	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	0.25	45	1.0
		Bottom	3.6	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	0.47	45	1.8
	3	Top	8.5	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	1.11	45	4.2
		Bottom	10.1	1000	519.0	467.1	1	0.6	219.1	**OK**	0.532	215.9	**OK**	1.32	45	5.0
Outer wall	4	Outer	2.7	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	0.47	45	1.4
		Inner	8.8	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	1.51	45	4.4
	5	Outer	7.1	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	1.22	45	3.6
		Inner	2.2	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	0.38	45	1.1
	6	Outer	19.5	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	3.37	45	9.7
		Inner	2.2	1000	394.0	354.6	1	0.6	166.3	**OK**	0.532	163.9	**OK**	0.38	45	1.1
Inner wall	7	Outer	1.1	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.23	45	0.5
		Inner	3.6	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.77	45	1.8
	8	Outer	1.1	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.23	45	0.5
		Inner	3.6	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.77	45	1.8
	9	Outer	1.1	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.23	45	0.5
		Inner	3.6	1000	319.0	287.1	1	0.6	134.6	**OK**	0.532	132.7	**OK**	0.77	45	1.8
Bottom/Base slab	10	Top	22.0	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	2.90	45	11.0
		Bottom	24.3	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	3.19	45	12.1
	11	Top	5.8	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	0.77	45	2.9
		Bottom	7.2	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	0.95	45	3.6
	12	Top	20.4	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	2.69	45	10.2
		Bottom	23.9	1000	517.0	465.3	1	0.6	218.2	**OK**	0.532	215.1	**OK**	3.15	45	12.0

Check for Shear Reinforcement requirement

Design Component	Section	Face	V_{Ed}	b_w	d	Asl	$\beta=a_v/2d$	βV_{Ed}	$K=Min[1+(200/d)^{0.5},2]$	$\rho 1=Min[Asl/b_w d,0.02]$	$\nu_{min}=0.031 K^{3/2} fck^{1/2}$	σ_{cp}	$V_{Rd.c1}=(\nu_{min}+0.15\sigma_{cp})b_w d$	$V_{Rd.c2}=[0.12K(80\rho 1 f_{ck})^{0.33}+0.15\sigma_{cp}]b_w d$	$V_{Rd.c}=Max[V_{Rd.c1} \& V_{Rd.c2}]$	Check
			T	mm	mm	mm^2		T				Mpa	T	T	T	
Top/Deck slab	1	Top	4.8	1000	519.0	2262	1.0	4.8	1.62	0.004	0.378	0	19.6	23.0	23.0	**SR Not Required**
		Bottom	5.7	1000	519.0	1131	1.0	5.7	1.62	0.002	0.378	0	19.6	18.3	19.6	**SR Not Required**
	2	Top	1.9	1000	519.0	1131	1.0	1.9	1.62	0.002	0.378	0	19.6	18.3	19.6	**SR Not Required**
		Bottom	3.6	1000	519.0	1131	1.0	3.6	1.62	0.002	0.378	0	19.6	18.3	19.6	**SR Not Required**
	3	Top	8.5	1000	519.0	2262	1.0	8.5	1.62	0.004	0.378	0	19.6	23.0	23.0	**SR Not Required**
		Bottom	10.1	1000	519.0	1131	1.0	10.1	1.62	0.002	0.378	0	19.6	18.3	19.6	**SR Not Required**
Outer wall	4	Outer	2.7	1000	394.0	2262	1.0	2.7	1.71	0.006	0.411	0	16.2	20.2	20.2	**SR Not Required**
		Inner	8.8	1000	394.0	1131	1.0	8.8	1.71	0.003	0.411	0	16.2	16.1	16.2	**SR Not Required**
	5	Outer	7.1	1000	394.0	1131	1.0	7.1	1.71	0.003	0.411	0	16.2	16.1	16.2	**SR Not Required**
		Inner	2.2	1000	394.0	1131	1.0	2.2	1.71	0.003	0.411	0	16.2	16.1	16.2	**SR Not Required**
	6	Outer	19.5	1000	394.0	3142	1.0	19.5	1.71	0.008	0.411	0	16.2	22.6	22.6	**SR Not Required**
		Inner	2.2	1000	394.0	1131	1.0	2.2	1.71	0.003	0.411	0	16.2	16.1	16.2	**SR Not Required**
Inner wall	7	Outer	1.1	1000	319.0	1131	1.0	1.1	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
		Inner	3.6	1000	319.0	1131	1.0	3.6	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
	8	Outer	1.1	1000	319.0	1131	1.0	1.1	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
		Inner	3.6	1000	319.0	1131	1.0	3.6	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
	9	Outer	1.1	1000	319.0	1131	1.0	1.1	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
		Inner	3.6	1000	319.0	1131	1.0	3.6	1.79	0.004	0.440	0	14.0	14.6	14.6	**SR Not Required**
Bottom/Base slab	10	Top	22.0	1000	517.0	2011	1.0	22.0	1.62	0.004	0.379	0	19.6	22.1	22.1	**SR Not Required**
		Bottom	24.3	1000	517.0	3142	1.0	24.3	1.62	0.006	0.379	0	19.6	25.6	25.6	**SR Not Required**
	11	Top	5.8	1000	517.0	2011	1.0	5.8	1.62	0.004	0.379	0	19.6	22.1	22.1	**SR Not Required**
		Bottom	7.2	1000	517.0	2011	1.0	7.2	1.62	0.004	0.379	0	19.6	22.1	22.1	**SR Not Required**
	12	Top	20.4	1000	517.0	2011	1.0	20.4	1.62	0.004	0.379	0	19.6	22.1	22.1	**SR Not Required**
		Bottom	23.9	1000	517.0	3142	1.0	23.9	1.62	0.006	0.379	0	19.6	25.6	25.6	**SR Not Required**

* SR stands for Shear Reinforcement

B) Serviceability Limit State (SLS) :-

1) SLS Stress Check:-

Formula used for calculation of stress:-			
	M_{ST}	=	$M_{RARE} - M_{Q-P}$
	$E_{c,eq}$	=	$\frac{E_{cm}*(M_{Q-P}+M_{ST})}{M_{ST} + (1+\phi) * M_{Q-P}}$
Modular ratio	m		$E_s / E_{c,eq}$ for Rare load combination
Modular ratio	m	=	$E_s / E_{cm'}$ for Quasi-Permanent load combination
Depth of neutral axis (N.A)	x	=	$[-m * A_s+(m^2 * A_s^2 + 2 * m * A_s * b * d)^{1/2}] / b$
Transformed Moment of Inertia	I_{NA}	=	$b * x^3 /3 + m * A_s * (d-x)^2$
Compressive stress in concrete	σ_r	=	$M * x / I_{NA}$
Tensile stress in steel	σ_s	=	$m * M * (d-x) / I_{NA}$

Mean flexural tensile strength of solid beam $f_{ctm,fl}$ = max [{1.6-(h/1000)} f_{ctm} ; f_{ctm}]

where, h = total depth of member in mm

a) For Rare Load Combination:-

Design Component	Section	Face	M_{Rare} (Tm)	b (mm)	Overall depth, D (mm)	d (mm)	$A_{st, pro}$ (mm^2)	I (mm^4)	Y/2 (mm)	Stress 'σ_c' (Mpa)	$f_{ctm,fl}$ (Mpa)	Cracked/ Uncracked		Modular ratio 'm'	N.A depth 'x' (mm)	I_{NA} (mm^4)	Comp. stress (Mpa)	Per. stress (Mpa)	Check	Tensile stress (Mpa)	Per. stress (Mpa)	Check
Top/Deck slab	1	Top	10.3	1000	575	519.0	2262	1E+10	287.5	2.5	2.8	Uncracked		7.9	119.5	3.4E+09	3.6	16.8	**OK**	94.9	400	**OK**
		Bottom	6.5	1000	575	519.0	1131	1E+10	287.5	1.6	2.8	Uncracked		8.0	88.5	1.9E+09	3.0	16.8	**OK**	116.6	400	**OK**
	2	Top	1.8	1000	575	519.0	1131	1E+10	287.5	0.4	2.8	Uncracked		6.2	78.5	1.5E+09	0.9	16.8	**OK**	31.6	400	**OK**
		Bottom	10.0	1000	575	519.0	1131	1E+10	287.5	2.5	2.8	Uncracked		8.8	92.0	2.1E+09	4.5	16.8	**OK**	181.2	400	**OK**
	3	Top	21.9	1000	575	519.0	2262	1E+10	287.5	5.4	2.8	Cracked		10.5	135.2	4.3E+09	6.8	16.8	**OK**	204.1	400	**OK**
		Bottom	0.3	1000	575	519.0	1131	1E+10	287.5	0.1	2.8	Uncracked		6.2	78.5	1.5E+09	0.2	16.8	**OK**	5.5	400	**OK**
Outer wall	4	Outer	10.2	1000	475	394.0	2262	5E+09	237.5	4.8	3.1	Cracked		7.4	99.6	1.8E+09	5.7	16.8	**OK**	125.0	400	**OK**
		Inner	8.0	1000	475	394.0	1131	5E+09	237.5	3.7	3.1	Cracked		8.3	76.9	1.1E+09	5.6	16.8	**OK**	191.0	400	**OK**
	5	Outer	6.7	1000	475	394.0	1131	5E+09	237.5	3.1	3.1	Cracked		8.2	76.9	1.1E+09	4.8	16.8	**OK**	161.6	400	**OK**
		Inner	4.8	1000	475	394.0	1131	5E+09	237.5	2.2	3.1	Uncracked		9.0	79.8	1.2E+09	3.3	16.8	**OK**	115.5	400	**OK**
	6	Outer	22.1	1000	475	394.0	3142	5E+09	237.5	10.3	3.1	Cracked		10.8	132.9	3.1E+09	9.5	16.8	**OK**	201.0	400	**OK**
		Inner	2.3	1000	475	394.0	1131	5E+09	237.5	1.1	3.1	Uncracked		6.2	67.6	8.5E+08	1.8	16.8	**OK**	54.7	400	**OK**
Inner wall	7	Outer	6.3	1000	375	319.0	1131	3E+09	187.5	4.3	3.4	Cracked		6.2	60.2	5.4E+08	7.0	16.8	**OK**	185.3	400	**OK**
		Inner	2.7	1000	375	319.0	1131	3E+09	187.5	1.9	3.4	Uncracked		6.2	60.2	5.4E+08	3.0	16.8	**OK**	80.8	400	**OK**
	8	Outer	1.2	1000	375	319.0	1131	3E+09	187.5	0.8	3.4	Uncracked		6.2	60.2	5.4E+08	1.3	16.8	**OK**	35.0	400	**OK**
		Inner	1.5	1000	375	319.0	1131	3E+09	187.5	1.0	3.4	Uncracked		6.2	60.2	5.4E+08	1.6	16.8	**OK**	43.8	400	**OK**
	9	Outer	2.1	1000	375	319.0	1131	3E+09	187.5	1.4	3.4	Uncracked		6.2	60.2	5.4E+08	2.3	16.8	**OK**	61.3	400	**OK**
		Inner	6.2	1000	375	319.0	1131	3E+09	187.5	4.3	3.4	Cracked		6.2	60.2	5.4E+08	6.9	16.8	**OK**	182.8	400	**OK**
Bottom/Base slab	10	Top	3.0	1000	600	517.0	2011	1E+10	300	0.8	2.8	Uncracked		6.2	101.7	2.5E+09	1.2	16.8	**OK**	31.2	400	**OK**
		Bottom	21.6	1000	600	517.0	3142	1E+10	300	5.6	2.8	Cracked		9.7	149.8	5.2E+09	6.2	16.8	**OK**	146.9	400	**OK**
	11	Top	16.8	1000	600	517.0	2011	1E+10	300	4.4	2.8	Cracked		9.0	119.7	3.4E+09	5.9	16.8	**OK**	175.4	400	**OK**
		Bottom	1.8	1000	600	517.0	2011	1E+10	300	0.5	2.8	Uncracked		6.2	101.7	2.5E+09	0.7	16.8	**OK**	18.4	400	**OK**
	12	Top	2.0	1000	600	517.0	2011	1E+10	300	0.5	2.8	Uncracked		6.2	101.7	2.5E+09	0.8	16.8	**OK**	20.4	400	**OK**
		Bottom	21.9	1000	600	517.0	3142	1E+10	300	5.7	2.8	Cracked		8.7	142.8	4.8E+09	6.5	16.8	**OK**	148.2	400	**OK**

b) For Quasi-Permanent Load Combination:-

Design Component	Section	Face	M_{QP} (Tm)	b (mm)	Overall depth, D (mm)	d (mm)	$A_{st, pro}$ (mm^2)	I (mm^4)	Y/2 (mm)	Stress 'σ_c' (Mpa)	$f_{ctm,fl}$ (Mpa)	Cracked/ Uncracked	Modular ratio 'm'	N.A depth 'x' (mm)	I_{NA} (mm^4)	Comp. stress (Mpa)	Per. stress (Mpa)	Check	Tensile stress (Mpa)	Per. stress (Mpa)	Check
Top/Deck slab	1	Top	2.1	1000	575	519.0	2262	1E+10	287.5	0.5	2.8	Uncracked	14.4	154.4	5.6E+09	0.6	12.6	**OK**	20.2	400	**OK**
		Bottom	1.5	1000	575	519.0	1131	1E+10	287.5	0.4	2.8	Uncracked	14.4	114.9	3.2E+09	0.5	12.6	**OK**	26.7	400	**OK**
	2	Top	0.0	1000	575	519.0	1131	1E+10	287.5	0.0	2.8	Uncracked	14.4	114.9	3.2E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Bottom	3.1	1000	575	519.0	1131	1E+10	287.5	0.8	2.8	Uncracked	14.4	114.9	3.2E+09	1.1	12.6	**OK**	57.4	400	**OK**
	3	Top	11.5	1000	575	519.0	2262	1E+10	287.5	2.8	2.8	Uncracked	14.4	154.4	5.6E+09	3.2	12.6	**OK**	108.7	400	**OK**
		Bottom	0.0	1000	575	519.0	1131	1E+10	287.5	0.0	2.8	Uncracked	14.4	114.9	3.2E+09	0.0	12.6	**OK**	0.0	400	**OK**
Outer wall	4	Outer	1.5	1000	475	394.0	2262	5E+09	237.5	0.7	3.1	Uncracked	14.6	131.5	3.0E+09	0.7	12.6	**OK**	19.3	400	**OK**
		Inner	2.0	1000	475	394.0	1131	5E+09	237.5	0.9	3.1	Uncracked	14.6	98.7	1.8E+09	1.1	12.6	**OK**	48.0	400	**OK**
	5	Outer	1.6	1000	475	394.0	1131	5E+09	237.5	0.8	3.1	Uncracked	14.6	98.7	1.8E+09	0.9	12.6	**OK**	40.2	400	**OK**
		Inner	1.6	1000	475	394.0	1131	5E+09	237.5	0.7	3.1	Uncracked	14.6	98.7	1.8E+09	0.9	12.6	**OK**	38.9	400	**OK**
	6	Outer	12.1	1000	475	394.0	3142	5E+09	237.5	5.6	3.1	Cracked	14.6	149.6	3.9E+09	4.7	12.6	**OK**	111.5	400	**OK**
		Inner	0.0	1000	475	394.0	1131	5E+09	237.5	0.0	3.1	Uncracked	14.6	98.7	1.8E+09	0.0	12.6	**OK**	0.0	400	**OK**
Inner wall	7	Outer	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Inner	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
	8	Outer	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Inner	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
	9	Outer	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Inner	0.0	1000	375	319.0	1131	3E+09	187.5	0.0	3.4	Uncracked	14.7	87.8	1.1E+09	0.0	12.6	**OK**	0.0	400	**OK**
Bottom/Base slab	10	Top	0.0	1000	600	517.0	2011	1E+10	300	0.0	2.8	Uncracked	12.4	137.5	4.5E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Bottom	12.3	1000	600	517.0	3142	1E+10	300	3.2	2.8	Cracked	12.4	165.4	6.3E+09	3.2	12.6	**OK**	84.9	400	**OK**
	11	Top	7.5	1000	600	517.0	2011	1E+10	300	2.0	2.8	Uncracked	12.4	137.5	4.5E+09	2.3	12.6	**OK**	79.6	400	**OK**
		Bottom	0.0	1000	600	517.0	2011	1E+10	300	0.0	2.8	Uncracked	12.4	137.5	4.5E+09	0.0	12.6	**OK**	0.0	400	**OK**
	12	Top	0.0	1000	600	517.0	2011	1E+10	300	0.0	2.8	Uncracked	12.4	137.5	4.5E+09	0.0	12.6	**OK**	0.0	400	**OK**
		Bottom	8.7	1000	600	517.0	3142	1E+10	300	2.3	2.8	Uncracked	12.4	165.4	6.3E+09	2.3	12.6	**OK**	60.2	400	**OK**

2) SLS Crack Width Check:- (Quasi-Permanent load combination)

Check $A_{st,min}$ for Crack control & Crack Width:-

Design Component	Section	Face	b	Overall depth, D/h	d	A_{ct} = bh/2	$\sigma_c = f_{yk}$	k	kc	$A_{s,min} = kc*k*f_{ct,eff}*A_{ct}/\sigma_c$	$A_{st, pro}$ or $A_{s pro}$	Check	Bar dia ϕeq	Clear cover 'c'	$h_{c,eff}$=min[2.5 (h-d), (h-x/3), h/2]	$A_{c,eff} = h_{c,eff}$ * b	$\rho_{peff} = A_s / A_{c,eff}$	S_{max} = 3.4c+0.17ϕ /$\rho_{p,eff}$	σ_{sc}	N.A depth 'x'	k_t	$\alpha_e = E_s/E_{cm}$'	$\varepsilon_{sm} - \varepsilon_{cm}$	w_k	Check
			mm	mm	mm	mm^2	Mpa			mm^2	mm^2		mm	mm	mm^2	mm^2		mm	Mpa	mm				mm	
Top/Deck slab	1	Top	1000	575	519.0	287500	500	0.808	0.4	528	2262	**OK**	12	50	140.0	140000	0.016	296.3	20.2	154.4	0.5	14.4	0.0001	0.018	**OK**
		Bottom	1000	575	519.0	287500	500	0.8075	0.4	528	1131	**OK**	12	50	140.0	140000	0.008	422.5	26.7	114.9	0.5	14.4	0.0001	0.034	**OK**
	2	Top	1000	575	519.0	287500	500	0.8075	0.4	528	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	114.9	0.5	14.4	0.0000	0.000	**OK**
		Bottom	1000	575	519.0	287500	500	0.8075	0.4	528	1131	**OK**	12	50	140.0	140000	0.008	422.5	57.4	114.9	0.5	14.4	0.0002	0.073	**OK**
	3	Top	1000	575	519.0	287500	500	0.8075	0.4	528	2262	**OK**	12	50	140.0	140000	0.016	296.3	108.7	154.4	0.5	14.4	0.0003	0.097	**OK**
		Bottom	1000	575	519.0	287500	500	0.8075	0.4	528	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	114.9	0.5	14.4	0.0000	0.000	**OK**
Outer wall	4	Outer	1000	475	394.0	237500	500	0.878	0.4	520	2262	**OK**	12	75	197.0	197000	0.011	432.7	19.3	131.5	0.5	14.6	0.0001	0.025	**OK**
		Inner	1000	475	394.0	237500	500	0.8775	0.4	520	1131	**OK**	12	75	197.0	197000	0.006	610.3	48.0	98.7	0.5	14.6	0.0001	0.088	**OK**
	5	Outer	1000	475	394.0	237500	500	0.8775	0.4	520	1131	**OK**	12	75	197.0	197000	0.006	610.3	40.2	98.7	0.5	14.6	0.0001	0.074	**OK**
		Inner	1000	475	394.0	237500	500	0.8775	0.4	520	1131	**OK**	12	75	197.0	197000	0.006	610.3	38.9	98.7	0.5	14.6	0.0001	0.071	**OK**
	6	Outer	1000	475	394.0	237500	500	0.8775	0.4	520	3142	**OK**	14	75	197.0	197000	0.016	407.3	111.5	149.6	0.5	14.6	0.0003	0.136	**OK**
		Inner	1000	475	394.0	237500	500	0.8775	0.4	520	1131	**OK**	12	75	197.0	197000	0.006	610.3	0.0	98.7	0.5	14.6	0.0000	0.000	**OK**
Inner wall	7	Outer	1000	375	319.0	187500	500	0.948	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
		Inner	1000	375	319.0	187500	500	0.9475	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
	8	Outer	1000	375	319.0	187500	500	0.9475	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
		Inner	1000	375	319.0	187500	500	0.9475	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
	9	Outer	1000	375	319.0	187500	500	0.9475	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
		Inner	1000	375	319.0	187500	500	0.9475	0.4	482	1131	**OK**	12	50	140.0	140000	0.008	422.5	0.0	87.8	0.5	14.7	0.0000	0.000	**OK**
Bottom/Base slab	10	Top	1000	600	517.0	300000	500	0.790	0.4	525	2011	**OK**	16	75	207.5	207500	0.010	535.7	0.0	137.5	0.5	12.4	0.0000	0.000	**OK**
		Bottom	1000	600	517.0	300000	500	0.79	0.4	525	3142	**OK**	14	75	207.5	207500	0.015	415.4	84.9	165.4	0.5	12.4	0.0003	0.106	**OK**
	11	Top	1000	600	517.0	300000	500	0.79	0.4	525	2011	**OK**	16	75	207.5	207500	0.010	535.7	79.6	137.5	0.5	12.4	0.0002	0.128	**OK**
		Bottom	1000	600	517.0	300000	500	0.79	0.4	525	2011	**OK**	16	75	207.5	207500	0.010	535.7	0.0	137.5	0.5	12.4	0.0000	0.000	**OK**
	12	Top	1000	600	517.0	300000	500	0.79	0.4	525	2011	**OK**	16	75	207.5	207500	0.010	535.7	0.0	137.5	0.5	12.4	0.0000	0.000	**OK**
		Bottom	1000	600	517.0	300000	500	0.79	0.4	525	3142	**OK**	14	75	207.5	207500	0.015	415.4	60.2	165.4	0.5	12.4	0.0002	0.075	**OK**

STAAD INPUT FILE FOR DL, SIDL, FPLL ETC

```
STAAD PLANE MULTICELL RCC BOX [2 X 6 X 3.5]
START JOB INFORMATION
ENGINEER NAME DEB SANKAR ROY
ENGINEER DATE 27-12-2019
CHECKER NAME SUPHAL GHOSH
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
 1   0.000   0.000   0.000  ;
 2   0.238   0.000   0.000  ;
 3   0.538   0.000   0.000  ;
 4   1.438   0.000   0.000  ;
 5   2.338   0.000   0.000  ;
 6   3.238   0.000   0.000  ;
 7   4.138   0.000   0.000  ;
 8   5.038   0.000   0.000  ;
 9   5.938   0.000   0.000  ;
 10  6.238   0.000   0.000  ;
 11  6.425   0.000   0.000  ;
 12  6.613   0.000   0.000  ;
 13  6.913   0.000   0.000  ;
 14  7.813   0.000   0.000  ;
 15  8.713   0.000   0.000  ;
 16  9.613   0.000   0.000  ;
 17  10.513  0.000   0.000  ;
 18  11.413  0.000   0.000  ;
 19  12.313  0.000   0.000  ;
 20  12.613  0.000   0.000  ;
 21  12.850  0.000   0.000  ;
 22  0.000   0.300   0.000  ;
 23  0.000   0.600   0.000  ;
 24  0.000   3.500   0.000  ;
 25  0.000   3.800   0.000  ;
 26  6.425   0.300   0.000  ;
 27  6.425   0.600   0.000  ;
 28  6.425   3.500   0.000  ;
 29  6.425   3.800   0.000  ;
 30  12.850  0.300   0.000  ;
 31  12.850  0.600   0.000  ;
 32  12.850  3.500   0.000  ;
 33  12.850  3.800   0.000  ;
 34  0.000   4.088   0.000  ;
 35  0.238   4.088   0.000  ;
 36  0.538   4.088   0.000  ;
 37  0.813   4.088   0.000  ;
 38  3.238   4.088   0.000  ;
 39  5.663   4.088   0.000  ;
 40  5.938   4.088   0.000  ;
 41  6.238   4.088   0.000  ;
 42  6.425   4.088   0.000  ;
 43  6.613   4.088   0.000  ;
 44  6.913   4.088   0.000  ;
 45  7.188   4.088   0.000  ;
 46  9.613   4.088   0.000  ;
 47  12.038  4.088   0.000  ;
 48  12.313  4.088   0.000  ;
 49  12.613  4.088   0.000  ;
 50  12.850  4.088   0.000  ;
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MEMBER INCEDENCE

1	1	2	;
2	2	3	;
3	3	4	;
4	4	5	;
5	5	6	;
6	6	7	;
7	7	8	;
8	8	9	;
9	9	10	;
10	10	11	;
11	11	12	;
12	12	13	;
13	13	14	;
14	14	15	;
15	15	16	;
16	16	17	;
17	17	18	;
18	18	19	;
19	19	20	;
20	20	21	;
21	1	22	;
22	22	23	;
23	23	24	;
24	24	25	;
25	25	34	;
26	11	26	;
27	26	27	;
28	27	28	;
29	28	29	;
30	29	42	;
31	21	30	;
32	30	31	;
33	31	32	;
34	32	33	;
35	33	50	;
36	34	35	;
37	35	36	;
38	36	37	;
39	37	38	;
40	38	39	;
41	39	40	;
42	40	41	;
43	41	42	;
44	42	43	;
45	43	44	;
46	44	45	;
47	45	46	;
48	46	47	;
49	47	48	;
50	48	49	;
51	49	50	;

MEMBER PROPERTY

1	PRIS	YD	0.90	ZD	1.000
2	PRIS	YD	0.75	ZD	1.000
3	PRIS	YD	0.60	ZD	1.000
4	PRIS	YD	0.60	ZD	1.000
5	PRIS	YD	0.60	ZD	1.000
6	PRIS	YD	0.60	ZD	1.000
7	PRIS	YD	0.60	ZD	1.000
8	PRIS	YD	0.60	ZD	1.000
9	PRIS	YD	0.75	ZD	1.000
10	PRIS	YD	0.90	ZD	1.000
11	PRIS	YD	0.90	ZD	1.000
12	PRIS	YD	0.75	ZD	1.000
13	PRIS	YD	0.60	ZD	1.000
14	PRIS	YD	0.60	ZD	1.000
15	PRIS	YD	0.60	ZD	1.000
16	PRIS	YD	0.60	ZD	1.000
17	PRIS	YD	0.60	ZD	1.000
18	PRIS	YD	0.60	ZD	1.000
19	PRIS	YD	0.75	ZD	1.000
20	PRIS	YD	0.90	ZD	1.000
21	PRIS	YD	0.78	ZD	1.000
22	PRIS	YD	0.63	ZD	1.000
23	PRIS	YD	0.48	ZD	1.000
24	PRIS	YD	0.63	ZD	1.000
25	PRIS	YD	0.78	ZD	1.000
26	PRIS	YD	0.68	ZD	1.000
27	PRIS	YD	0.53	ZD	1.000
28	PRIS	YD	0.38	ZD	1.000
29	PRIS	YD	0.53	ZD	1.000
30	PRIS	YD	0.68	ZD	1.000
31	PRIS	YD	0.78	ZD	1.000
32	PRIS	YD	0.63	ZD	1.000
33	PRIS	YD	0.48	ZD	1.000
34	PRIS	YD	0.63	ZD	1.000
35	PRIS	YD	0.78	ZD	1.000
36	PRIS	YD	0.88	ZD	1.000
37	PRIS	YD	0.73	ZD	1.000
38	PRIS	YD	0.58	ZD	1.000
39	PRIS	YD	0.58	ZD	1.000
40	PRIS	YD	0.58	ZD	1.000
41	PRIS	YD	0.58	ZD	1.000
42	PRIS	YD	0.73	ZD	1.000
43	PRIS	YD	0.88	ZD	1.000
44	PRIS	YD	0.88	ZD	1.000
45	PRIS	YD	0.73	ZD	1.000
46	PRIS	YD	0.58	ZD	1.000
47	PRIS	YD	0.58	ZD	1.000
48	PRIS	YD	0.58	ZD	1.000
49	PRIS	YD	0.58	ZD	1.000
50	PRIS	YD	0.73	ZD	1.000
51	PRIS	YD	0.88	ZD	1.000

```
DEFINE MATERIAL START
ISOTROPIC                 MATERIAL1
E 32308250
DENSITY  25
POISSON 0.15
ALPHA 1.17E-5
END DEFINE MATERIAL
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 51
SUPPORTS
    1      FIXED BUT  FZ MX MY MZ KFY 713
    2      FIXED BUT  FZ MX MY MZ KFY 1613
    3      FIXED BUT  FZ MX MY MZ KFY 3600
    4      FIXED BUT  FZ MX MY MZ KFY 5400
    5      FIXED BUT  FZ MX MY MZ KFY 5400
    6      FIXED BUT  FZ MX MY MZ KFY 5400
    7      FIXED BUT  FZ MX MY MZ KFY 5400
    8      FIXED BUT  FZ MX MY MZ KFY 5400
    9      FIXED BUT  FZ MX MY MZ KFY 3600
   10      FIXED BUT  FZ MX MY MZ KFY 1463
   11      FIXED BUT  FZ MX MY MZ KFY 1125
   12      FIXED BUT  FZ MX MY MZ KFY 1463
   13      FIXED BUT  FZ MX MY MZ KFY 3600
   14      FIXED BUT  FZ MX MY MZ KFY 5400
   15      FIXED BUT  FZ MX MY MZ KFY 5400
   16      FIXED BUT  FZ MX MY MZ KFY 5400
   17      FIXED BUT  FZ MX MY MZ KFY 5400
   18      FIXED BUT  FZ MX MY MZ KFY 5400
   19      FIXED BUT  FZ MX MY MZ KFY 3600
   20      FIXED BUT  FZ MX MY MZ KFY 1613
   21      FIXED BUT  FZ MX MY MZ KFY 713

           LOAD 1  SELF WEIGHT
           SELFWEIGHT Y -1
           LOAD 2 SIDL
           MEMBER  LOAD
           36 to 51 UNI                GY -2.200
           LOAD 3  ACTIVE EARTH PRESSURE ( BOTH SIDE ) IN DRY CONDITION
           MEMBER LOAD
           21 TRAP GX                   44.750     41.750
           22 TRAP GX                   41.750     38.750
           23 TRAP GX                   38.750      9.750
           24 TRAP GX                    9.750      6.750
           25 TRAP GX                    6.750      3.875
           31 TRAP GX                  -44.750    -41.750
           32 TRAP GX                  -41.750    -38.750
           33 TRAP GX                  -38.750     -9.750
           34 TRAP GX                   -9.750     -6.750
           35 TRAP GX                   -6.750     -3.875
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LOAD 4  ACTIVE EARTH PRESSURE ( BOTH SIDE ) IN SUBMERGED CONDITION
MEMBER LOAD
*** for Submerged Earth Pressure only***
21 TRAP GX                              21.875      20.375
22 TRAP GX                              20.375      18.875
23 TRAP GX                              18.875       4.375
24 TRAP GX                               4.375       2.875
25 TRAP GX                               2.875       0.000
31 TRAP GX                             -21.875     -20.375
32 TRAP GX                             -20.375     -18.875
33 TRAP GX                             -18.875      -4.375
34 TRAP GX                              -4.375      -2.875
35 TRAP GX                              -2.875      -0.000
LOAD 5 LL SURCHARGE ON BOTH SIDE
MEMBER LOAD
21 TO 25 UNI GX                          12.00
31 TO 35 UNI GX                         -12.00
LOAD 6 LL SURCHARGE ON LEFT SIDE
MEMBER LOAD
21 TO 25 UNI GX                          12.00
LOAD 7 LL SURCHARGE ON RIGHT SIDE
MEMBER LOAD
31 TO 35 UNI GX                         -12.00
LOAD 8 BRAKING LOAD ON LEFT SIDE
JOINT LOAD
34  FX                                   16.87
LOAD 9 BRAKING LOAD ON RIGHT SIDE
JOINT LOAD
50  FX                                  -16.87
LOAD 10 TEMPERATURE LOAD (GRADIENT RISE)
JOINT LOAD
34 FX                                  -414.35
34 MZ                                    70.47
50 FX                                   414.35
50 MZ                                   -70.47
LOAD 11 TEMPERATURE GRADIENT (GRADIENT FALL )
JOINT LOAD
34 FX                                   229.34
34 MZ                                   -13.18
50 FX                                  -229.34
50 MZ                                    13.18
*** ULS / STRUCTURAL STRENGTH CHECK LOAD COMBINATION ***
*** EARTH PRESSURE AS LEADING LOAD ***
LOAD COMB 101
1 1.35 2 1.75 3 1.5 5 1.2 8 1.15
LOAD COMB 102
1 1.35 2 1.75 4 1.5 5 1.2 8 1.15
LOAD COMB 103
1 1.35 2 1.75 3 1.5 6 1.2 8 1.15
LOAD COMB 104
1 1.35 2 1.75 4 1.5 6 1.2 8 1.15
*** LIVE LOAD AS LEADING LOAD ***
LOAD COMB 111
1 1.35 2 1.75 3 1.0 5 1.2 8 1.5
LOAD COMB 112
1 1.35 2 1.75 4 1.0 5 1.2 8 1.5
LOAD COMB 113
1 1.35 2 1.75 3 1.0 6 1.2 8 1.5
LOAD COMB 114
1 1.35 2 1.75 4 1.0 6 1.2 8 1.5
```

```
*** SLS / RARE LOAD COMBINATION ***
*** LIVE LOAD AS LEADING LOAD ***
LOAD COMB 201
1 1 2 1 3 1 5 0.8 8 1 10 0.6
LOAD COMB 202
1 1 2 1 4 1 5 0.8 8 1 10 0.6
LOAD COMB 203
1 1 2 1 3 1 6 0.8 8 1 10 0.6
LOAD COMB 204
1 1 2 1 4 1 6 0.8 8 1 10 0.6
LOAD COMB 205
1 1 2 1 3 1 5 0.8 8 1 11 0.6
LOAD COMB 206
1 1 2 1 4 1 5 0.8 8 1 11 0.6
LOAD COMB 207
1 1 2 1 3 1 6 0.8 8 1 11 0.6
LOAD COMB 208
1 1 2 1 4 1 6 0.8 8 1 11 0.6

*** SLS / QUASI-PERMANENT LOAD COMBINATION ***
LOAD COMB 211
1 1 2 1 3 1 10 0.5
LOAD COMB 212
1 1 2 1 4 1 10 0.5
LOAD COMB 213
1 1 2 1 3 1 11 0.5
LOAD COMB 214
1 1 2 1 4 1 11 0.5

*** MAX. BASE PRESSURE CHECK ***
LOAD COMB 500
1 1.0 2 1.0 3 1.0
LOAD COMB 501
1 1.0 2 1.0 3 1.0 5 1.0 8 1.0
LOAD COMB 502
1 1.0 2 1.0 4 1.0
LOAD COMB 503
1 1.0 2 1.0 4 1.0 5 1.0 8 1.0
LOAD COMB 504
1 1.0 2 1.0 3 1.0 6 1.0 8 1.0
LOAD COMB 505
1 1.0 2 1.0 4 1.0 6 1.0 8 1.0

PERFORM ANALYSIS
 DEFINE ENVELOPE
 101 TO 104 ENVELOPE 1 ULS EP
 111 TO 114 ENVELOPE 2 ULS LL
 201 TO 208 ENVELOPE 3 SLS RARE
 211 TO 214 ENVELOPE 4 SLS Q-P
 500 TO 505 ENVELOPE 5 BASE PRESSURE
 END DEFINE ENVELOPE
PRINT SUPPORT REACTION
FINISH
```

Check for Safe Bearing Capacity of Soil:-

Summary of Support Reactions (from STAAD Output)

Support/ Node No.	Max. Support Reaction (KN)	Min. Support Reaction (KN)
1	7.1	5.9
2	14.8	12.4
3	32.2	27.4
4	47.6	42.3
5	46.2	42.2
6	45.2	42.3
7	44.9	42.9
8	45.1	43.8
9	29.8	29.4
10	11.8	11.7
11	9.4	9.4
12	11.8	11.8
13	30.1	29.8
14	46.3	45.1
15	46.9	44.8
16	48.1	45.1
17	50.2	46.0
18	52.9	47.5
19	36.7	32.0
20	17.0	14.7
21	8.3	7.0
Total Load	**682.3**	**633.5**

Net Allowable Bearing Capacity	=	150.0	KN/m^2	
Design width of Box barrel ' B '	=	1.0	m	
Design Length of Box barrel ' L '	=	13.325	m	
Area of Box barrel ' A '	=	13.325	m^2	
Max. Bearing capacity/ Stress below Box i.e σ_{max}	=	51.2	KN/m^2	**OK**
Min. Bearing capacity/ Stress below Box i.e σ_{min}	=	47.5	KN/m^2	**OK**

Hydrological Calculation for Major Bridge : Shiloni River Major Bridge (3x30m) at design Ch.119+535KM

Hydrological Calculation for Major Bridge : Shiloni River

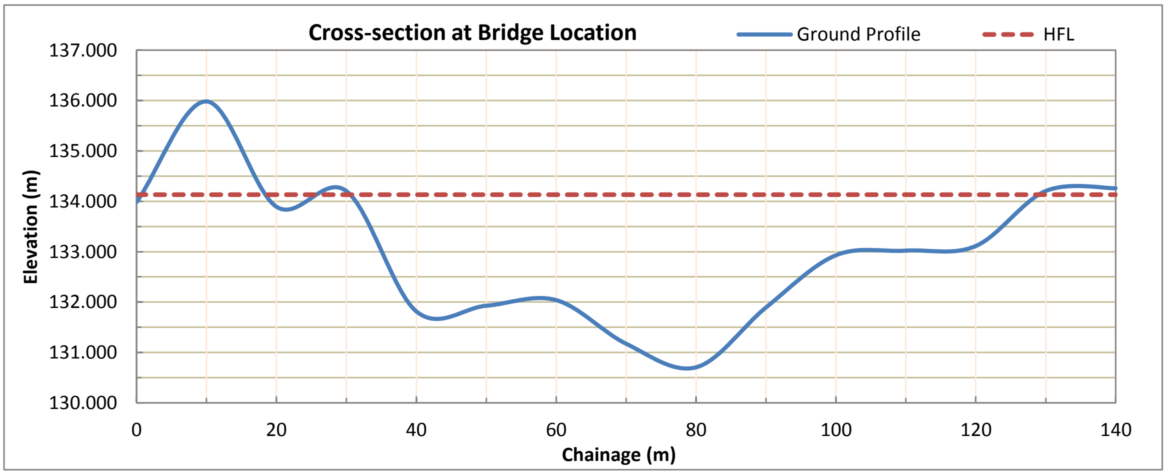
Bridge Data:-

Road Name	=	Daboka-Manja
Bridge Name/ID No.	=	-
Name of Nallah / Stream / River	=	Shiloni River
Existing Ch.	=	-
Design Ch.	=	119+535 KM
Type of Proposed bridge	=	PSC I-Girder
Span Arrangement of proposed bridge	=	3x30m
Existing FRL	=	-
Design FRL	=	141.000 m
HFL at bridge location	=	134.132 m
Rugosity Co-efficient,n	=	0.04
(Reff: IRC: SP: 13 - 2004 , Table 5.1)		
Bed Slope of Stream,S	=	0.011
G.T Sheet No.	=	-
Scale	=	-
Catchment Area	=	170.000 Sq. Km

1.0 Discharge Calculation By Area Velocity Method

Cross- section at Bridge Location

S.No.	Distance (m)	Bed Level (m)	Average Scour Bed Level (m)	HFL (m)	Depth of Water (m)	Average Depth (m)	Area (Sq. m)	Perimeter (m)	Top width of Flow (m)
1	0	133.982	133.982	134.1316	0.149555				
2	10	135.982	135.982	134.1316		0.075	0.374	5.002	5.000
3	20	133.892	133.892	134.1316	0.239555	0.120	0.599	5.006	5.000
5	30	134.203	134.203	134.1316		0.120	0.599	5.006	5.000
7	40	131.815	131.815	134.1316	2.316555	1.158	5.791	5.511	5.000
9	50	131.927	131.927	134.1316	2.204555	2.261	22.606	10.001	10.000
11	60	132.039	132.039	134.1316	2.092555	2.149	21.486	10.001	10.000
13	70	131.173	131.173	134.1316	2.958555	2.526	25.256	10.037	10.000
15	80	130.705	130.705	134.1316	3.426555	3.193	31.926	10.011	10.000
17	90	131.893	131.893	134.1316	2.238555	2.833	28.326	10.070	10.000
19	100	132.929	132.929	134.1316	1.202555	1.721	17.206	10.054	10.000
21	110	133.021	133.021	134.1316	1.110555	1.157	11.566	10.000	10.000
23	120	133.113	133.113	134.1316	1.018555	1.065	10.646	10.000	10.000
25	130	134.205	134.205	134.1316		0.509	2.546	5.103	5.000
27	140	134.259	134.259	134.1316					
				TOTAL			**178.924**	**105.801**	**105.000**



Hydraulic Mean Radius (R)	A / P	=	1.691 m
Velocity of Water (V)	$(1/n)(R)^{2/3}(S)^{1/2}$	=	3.684 m/sec
Discharge (Q)	A x V	=	**659.120 Cumecs**

Summary of Discharge By Area Velocity Method

1	Discharge at bridge location	=	659.120 Cumecs
	Maximum Discharge By Area velocity method	=	**659.120 Cumecs**

Hydrological Calculation for Major Bridge : Shiloni River

Name of Nallah / Stream / River	:	Shiloni River
Location	:	119+535 KM
Catchment Area	:	170 Sq. Km

2.0 Discharge by Dicken's Formula : (Refer IRC - SP : 13 - 2004, Clause : 4.2)

Discharge as per Dicken's Formula, **$Q = C M^{3/4}$**

C = 14 - 19 where annual rainfall is more than 120 cm
= 11 - 14 where annual rainfall is 60-120 cm
= 22 in Western Ghats

Value of " C " adopted in the present case	C	=	14
Catchment Area,	M	=	170.000 Sq. Km
Discharge,	**Q**	=	**659.120 Cumecs**

3.0 Discharge by Ryve's Formula : (Refer IRC - SP : 13 - 2004, Clause : 4.3)

Discharge as per Ryve's Formula **$Q = C M^{2/3}$**

C = 6.8 for areas within 25 km of the coast
= 8.5 for areas within 25 km and 160 km of the coast
= 10 for limited areas near hills

Value of " C " adopted in the present case	C	=	10
Catchment Area,	M	=	170.000 Sq. Km
Discharge,	**Q**	=	**306.878 Cumecs**

4.0 Discharge by Rational Formula : (Refer IRC - SP : 13 - 2004, Clause : 4.7)

Catchment Area,	A	=	170.000 Sq. Km
		=	17000.00 Hectares
Length of longest path from Toposheet,	L	=	38.666 Km
Difference in levels from Toposheet,	H	=	457 m
(Ref: Index Map / G.T.Sheet)			
Maximum Rainfall	F	=	170 mm
Duration of Storm	T	=	24 Hrs
One Hour Rainfall	$I_o = (F / T) \times (T + 1)/(1 + 1) =$		88.54 mm / Hr
Time of Concentration	$t_c = (0.87 \times L^3 / H)^{0.385}$	=	6.11 Hrs
Critical Rainfall Intensity	$I_c = I_o \times (2 / (1 + t_c))$	=	24.91 mm / Hr
Discharge,			
Coefficient of Runoff, P (ref. Table-4.1 of IRC:SP 13 2004)		=	0.800
Fraction of maximum point intensity at centre of storm, depends on area, f (Ref. fig-4.2 of IRC:SP 13 2004)		=	0.670
Critical Intensity of Rainfall, I_c		=	2.491 cm / Hr
Maximum Discharge	**$Q = 0.028 \times P \times f \times A \times I_c$**	=	**635.484 Cumecs**

5.0 Design Discharge : (Refer IRC - SP : 13 - 2004, Clause : 6.2.1)

Discharge by Area Velocity Method	=	659.120 Cumecs
Discharge by Dicken's Formula	=	659.120 Cumecs
Discharge by Rational Formula	=	635.484 Cumecs
Design Discharge	=	**659.120 Cumecs**
Design Discharge with 10% safty factor	=	**725.032 Cumecs**

6.0 Afflux Calculations : (Refer IRC:5 2015, Clause : 106.6)

Design Discharge	Q	=	725.032 cumecs
Unobstructed area of Cross Section of river	A	=	178.92 m^2
Velocity of Water,	V	=	4.052 m/sec
Observed HFL	HFL	=	134.13155 m
BED Level	OGL	=	130.705 m
Depth of Water	d	=	3.427 m
Available Water way	a	=	148.59196 m^2
Using Molesworth's Formula,	$(V^2/17.9)+0.015) \times \{(A/a)^2-1)\}$	=	0.419479 ~ 0.420m

The +ve afflux value obtained by above formula reflects that there is heading up of water due to obstruction & the value is rounded up to nearest decimal place.

7.0 Vertical Clearance : (Refer IRC : 5 - 1998, Clause : 106.2.1)

Design Discharge,	Q	=	725.032 Cumecs
Vertical Clearance (adopted),		=	1.20 m

Hydrological Calculation for Major Bridge : Shiloni River

Name of Nallah / Stream / River : Shiloni River

Minimum Soffit Level of Deck Slab = HFL + Afflux + Vertical Clearance = 135.752 m

8.0 Linear Waterway :

As per Lacy's formula required linear water way , 4.8 √(Q)	=	**129.247**	m
HFL Spread including afflux at Proposed Bridge Location	=	**105.000**	m
Adopted obstructed linear waterway	=	87.200	m
Restriction to the linear waterway	=	32.5 %	**< 33%, O.K.**

9.0 Scour Depth : (Refer IRC : 78 - 2000, Clause : 703)

% Increase in Design Discharge = **30 %**

So, Increased Design Discharge = **942.542 Cumecs**

Mean Depth of Scour, $\mathbf{d_{sm} = 1.34 \times (D_b^{\,2} / K_{sf})^{1/3}}$

D_b = Design discharge per metre width = 8.977 Cumecs / m

K_{sf} = Silt factor = 1.500

$\mathbf{d_{sm}}$ **=** = **5.056 m**

Maximum Scour Depth:

For Piers, $\mathbf{2.0 \times d_{sm}}$ = 10.112 m

For Abutments, $\mathbf{1.27 \times d_{sm}}$ **=** 6.421 m

Maximum Scour Level :

Maximum Scour Level = HFL - Maximum Scour Depth

For Piers, = 124.439 m

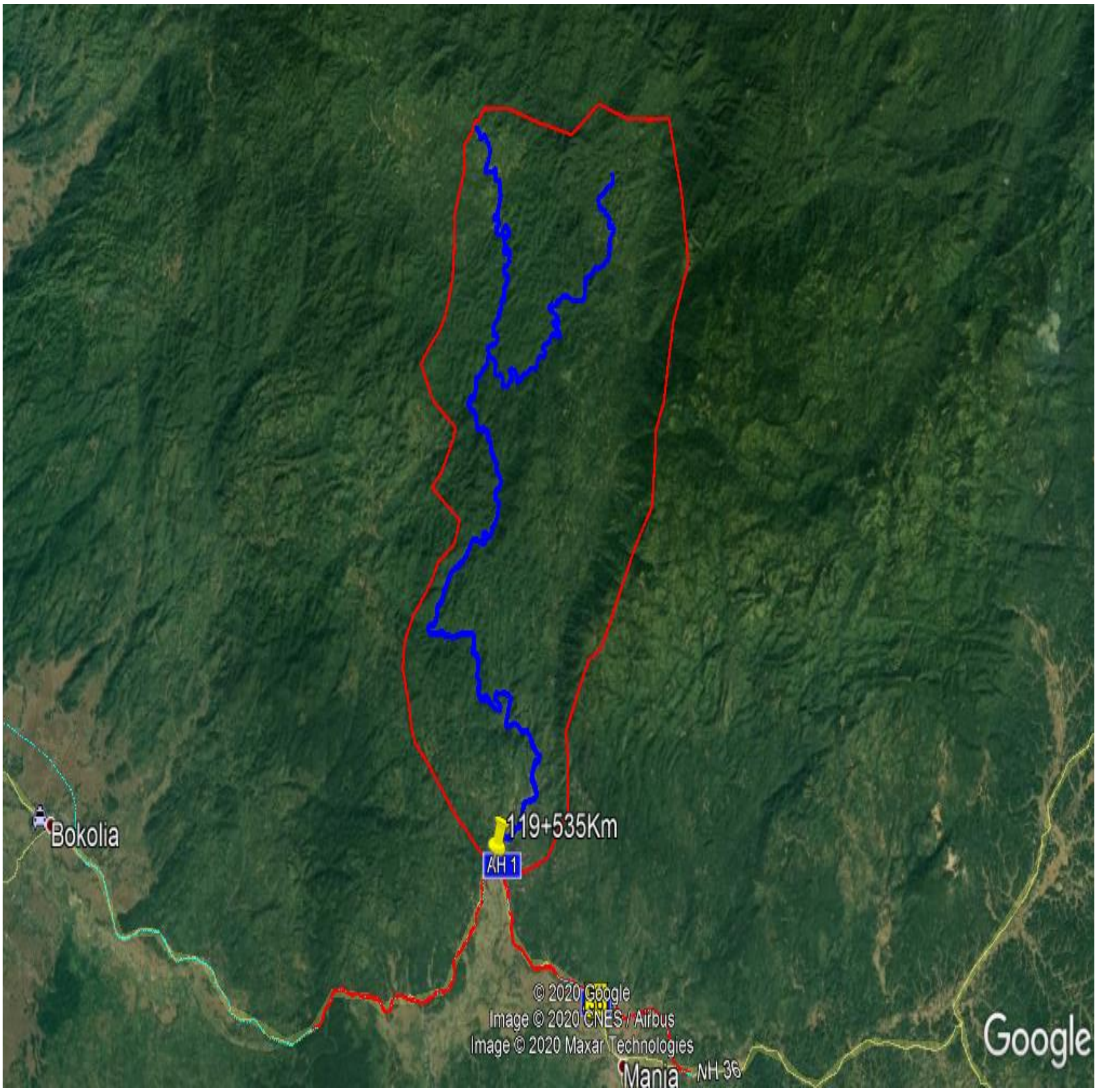
For Abutments, = 128.130 m

Proposed Bridge Span Arrangement :

Total Length of Proposed Bridge	=	3x30m	m
Span Length for each	=	30	m
Number of Spans	=	3	
Proposed Depth of Super Structure at Centre	=	2.025	m
Thickness of Wearing Coat	=	0.065	m
Overall depth of superstructure (considering 150mm for camber)	=	2.240	m

10.0 Check for Proposed Level at Important Locations :

S.No	Location	RL as per Calculation (m)	Proposed RL	Safe/ Unsafe
1	HFL at Proposed location with Afflux	134.552	134.552	
2	Lowest Bed Level at Proposed Location	130.705	130.705	
3	Minimum Soffit Level at proposed location	135.752	138.760	**Safe**
4	Proposed Formation Level	137.992	141.000	**Safe**



CATCHMENT AREA= 170 Sq.Km

Hydrological Calculation for Major Bridge : Yammuna River Major Bridge (3x22m) at design Ch.129+000KM

Hydrological Calculation for Major Bridge : Yammuna River

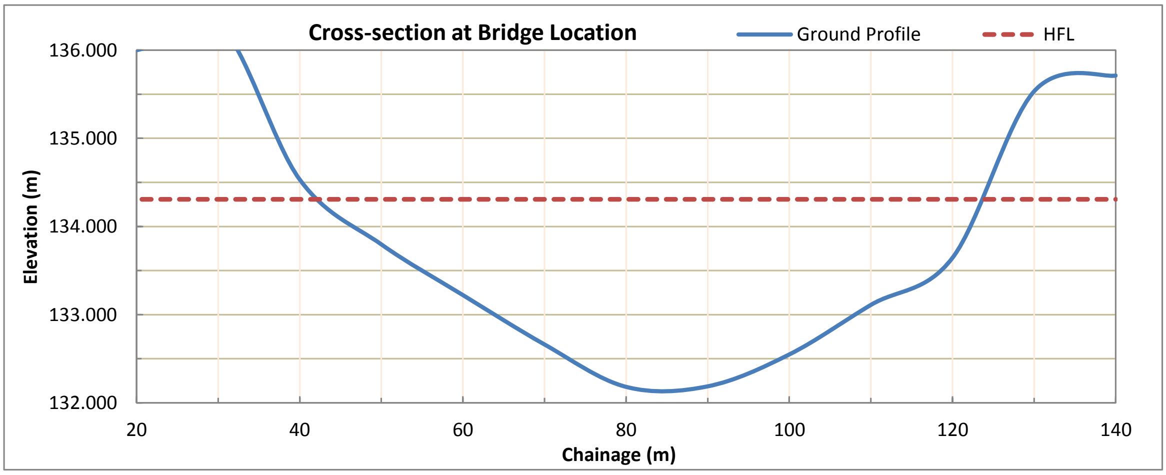
Bridge Data:-

Road Name	=	Daboka-Manja
Bridge Name/ID No.	=	-
Name of Nallah / Stream / River	=	Yammuna River
Existing Ch.	=	-
Design Ch.	=	129+000 KM
Type of Proposed bridge	=	RCC I-Girder
Span Arrangement of proposed bridge	=	3x22m
Existing FRL	=	-
Design FRL	=	142.000 m
HFL at bridge location	=	134.308 m
Rugosity Co-efficient,n	=	0.04
(Reff: IRC: SP: 13 - 2004 , Table 5.1)		
Bed Slope of Stream,S	=	0.011
G.T Sheet No.	=	-
Scale	=	-
Catchment Area	=	63.400 Sq. Km

1.0 Discharge Calculation By Area Velocity Method

Cross- section at Bridge Location

S.No.	Distance (m)	Bed Level (m)	Average Scour Bed Level (m)	HFL (m)	Depth of Water (m)	Average Depth (m)	Area (Sq. m)	Perimeter (m)	Top width of Flow (m)
1	0	136.103	136.103	134.3082					
2	10	136.000	136.000	134.3082					
3	20	136.000	136.000	134.3082					
5	30	136.304	136.304	134.3082					
7	40	134.532	134.532	134.3082					
9	50	133.797	133.797	134.3082	0.511249	0.256	1.278	5.026	5.000
11	60	133.222	133.222	134.3082	1.086249	0.799	7.987	10.017	10.000
13	70	132.662	132.662	134.3082	1.646249	1.366	13.662	10.016	10.000
15	80	132.181	132.181	134.3082	2.127249	1.887	18.867	10.012	10.000
17	90	132.187	132.187	134.3082	2.121249	2.124	21.242	10.000	10.000
19	100	132.549	132.549	134.3082	1.759249	1.940	19.402	10.007	10.000
21	110	133.111	133.111	134.3082	1.197249	1.478	14.782	10.016	10.000
23	120	133.645	133.645	134.3082	0.663249	0.930	9.302	10.014	10.000
25	130	135.533	135.533	134.3082		0.332	1.658	5.044	5.000
27	140	135.712	135.712	134.3082					
			TOTAL				**108.184**	**80.150**	**80.000**



Hydraulic Mean Radius (R)	A / P	=	1.350 m
Velocity of Water (V)	$(1/n)(R)^{2/3}(S)^{1/2}$	=	3.170 m/sec
Discharge (Q)	A x V	=	**342.909 Cumecs**

Summary of Discharge By Area Velocity Method

1	Discharge at bridge location	=	342.909 Cumecs
	Maximum Discharge By Area velocity method	=	**342.909 Cumecs**

Hydrological Calculation for Major Bridge : Yammuna River

Name of Nallah / Stream / River	:	Yammuna River
Location	:	129+000 KM
Catchment Area	:	63.4 Sq. Km

2.0 **Discharge by Dicken's Formula :** (Refer IRC - SP : 13 - 2004, Clause : 4.2)

Discharge as per Dicken's Formula, $\mathbf{Q = C\ M^{3/4}}$

C = 14 - 19 where annual rainfall is more than 120 cm
= 11 - 14 where annual rainfall is 60-120 cm
= 22 in Western Ghats

Value of " C " adopted in the present case	C	=	14
Catchment Area,	M	=	63.400 Sq. Km
Discharge,	**Q**	=	**314.554 Cumecs**

3.0 **Discharge by Ryve's Formula :** (Refer IRC - SP : 13 - 2004, Clause : 4.3)

Discharge as per Ryve's Formula $\mathbf{Q = C\ M^{2/3}}$

C = 6.8 for areas within 25 km of the coast
= 8.5 for areas within 25 km and 160 km of the coast
= 10 for limited areas near hills

Value of " C " adopted in the present case	C	=	10
Catchment Area,	M	=	63.400 Sq. Km
Discharge,	**Q**	=	**158.998 Cumecs**

4.0 **Discharge by Rational Formula :** (Refer IRC - SP : 13 - 2004, Clause : 4.7)

Catchment Area,	A	=	63.400 Sq. Km
		=	6340.00 Hectares
Length of longest path from Toposheet,	L	=	24.196 Km
Difference in levels from Toposheet,	H	=	270 m
Maximum Rainfall	F	=	170 mm
Duration of Storm	T	=	24 Hrs
One Hour Rainfall	$\mathbf{I_o = (F / T) \times (T + 1)/(1 + 1) =}$		88.54 mm / Hr
Time of Concentration	$\mathbf{t_c = (0.87 \times L^3 / H)^{0.385}}$	=	4.35 Hrs
Critical Rainfall Intensity	$\mathbf{I_c = I_o \times (2 / (1 + t_c)}$	=	33.08 mm / Hr
Discharge,			
Coefficient of Runoff, P (ref. Table-4.1 of IRC:SP 13 2004)		=	0.800
Fraction of maximum point intensity at centre of storm, depends on area, f (Ref. fig-4.2 of IRC:SP 13 2004)		=	0.730
Critical Intensity of Rainfall, I_c		=	3.308 cm / Hr
Maximum Discharge	$\mathbf{Q = 0.028 \times P \times f \times A \times I_c}$	=	**342.910 Cumecs**

5.0 **Design Discharge :** (Refer IRC - SP : 13 - 2004, Clause : 6.2.1)

Discharge by Area Velocity Method	=	342.909 Cumecs
Discharge by Dicken's Formula	=	314.554 Cumecs
Discharge by Rational Formula	=	342.910 Cumecs
Design Discharge	=	**342.910 Cumecs**
Design Discharge with 10% safty factor	=	**377.200 Cumecs**

6.0 **Afflux Calculations :** (Refer IRC:5 2015, Clause : 106.6)

Design Discharge	Q	=	377.200 cumecs
Unobstructed area of Cross Section of river	A	=	108.18 m^2
Velocity of Water,	V	=	3.487 m/sec
Observed HFL	HFL	=	134.308 m
BED Level	OGL	=	132.181 m
Depth of Water	d	=	2.127 m
Available Water way	a	=	86.141 m^2
Using Molesworth's Formula,	$\{(V^2/17.9)+0.015\} \times \{(A/a)^2-1)\}$	=	0.400701 ~ 0.401m

The +ve afflux value obtained by above formula reflects that there is heading up of water due to obstruction & the value is rounded up to nearest decimal place.

7.0 **Vertical Clearance :** (Refer IRC : 5 - 1998, Clause : 106.2.1)

Design Discharge,	Q	=	377.200 Cumecs
Vertical Clearance (adopted),		=	1.20 m

Hydrological Calculation for Major Bridge : Yammuna River

Name of Nallah / Stream / River : Yammuna River

Minimum Soffit Level of Deck Slab = HFL + Afflux + Vertical Clearance = 135.909 m

8.0 Linear Waterway :

As per Lacy's formula required linear water way , 4.8 √(Q)	=	**93.224**	m
HFL Spread including afflux at Proposed Bridge Location	=	**80.000**	m
Adopted obstructed linear waterway	=	63.700	m
Restriction to the linear waterway	=	31.7 %	**< 33%, O.K.**

9.0 Scour Depth : (Refer IRC : 78 - 2000, Clause : 703)

% Increase in Design Discharge = **30 %**

So, Increased Design Discharge = **490.361 Cumecs**

Mean Depth of Scour, $\mathbf{d_{sm} = 1.34 \times (D_b^{\,2} / K_{sf})^{1/3}}$

D_b = Design discharge per metre width = 6.130 Cumecs / m

K_{sf} = Silt factor = 1.500

$\mathbf{d_{sm}}$ = = **3.921 m**

Maximum Scour Depth:

For Piers, $\mathbf{2.0 \times d_{sm}}$ = 7.841 m

For Abutments, $\mathbf{1.27 \times d_{sm}}$ **=** 4.979 m

Maximum Scour Level :

Maximum Scour Level = HFL - Maximum Scour Depth

For Piers, = 126.868 m

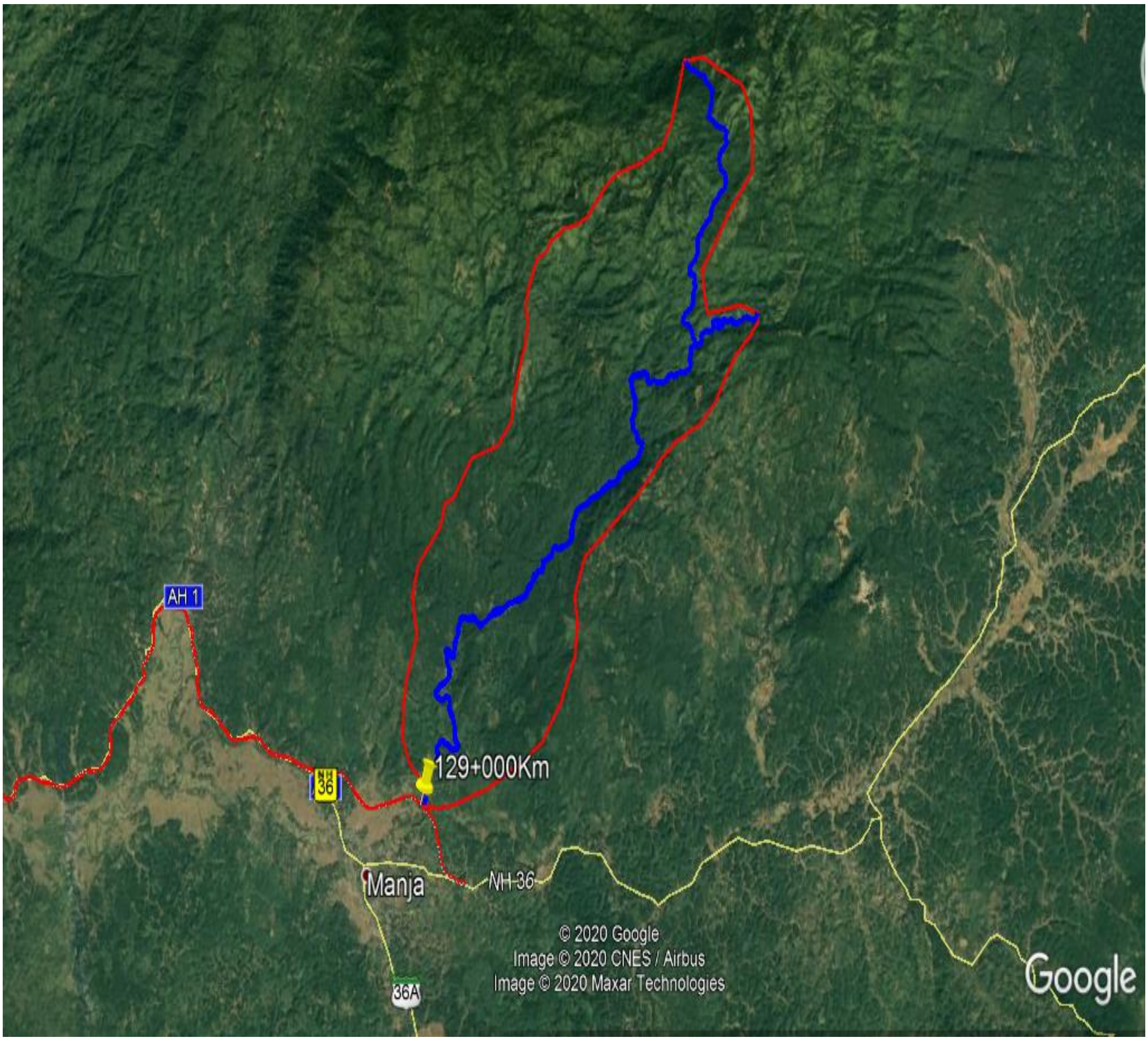
For Abutments, = 129.730 m

Proposed Bridge Span Arrangement :

Total Length of Proposed Bridge	=	3x22m	m
Span Length for each	=	22	m
Number of Spans	=	3	
Proposed Depth of Super Structure at Centre	=	1.975	m
Thickness of Wearing Coat	=	0.065	m
Overall depth of superstructure (considering 150mm for camber conservatively)	=	2.190	m

10.0 Check for Proposed Level at Important Locations :

S.No	Location	RL as per Calculation (m)	Proposed RL	Safe/ Unsafe
1	HFL at Proposed location with Afflux	134.709	134.709	
2	Lowest Bed Level at Proposed Location	132.181	132.181	
3	Minimum Soffit Level at proposed location	135.909	139.810	**Safe**
4	Proposed Formation Level	138.099	142.000	**Safe**



CATCHMENT AREA= 63.4 Sq.Km

PART-III

GEOTECHNICAL REPORT

Soil Investigation Report of Major Bridge at Chainage: 119+535

TABLE OF CONTENTS

1.0 INTRODUCTION

A subsoil investigation was necessary for the purpose of the foundation design and construction of the proposed project. Accordingly the subsoil exploration work of boreholes at structure location having maximum required depth, as proposed by the project authority. During borehole exploration, undisturbed/disturbed/SPT samples were collected.

The present report deals with the geotechnical investigation findings at the location and the discussion on both the aspects regarding bearing capacity of open foundations and pile capacity for deep foundation in the form of bored cast in-situ pile depending on the field and laboratory test results. However, the Foundation Designer may modulate the type of foundations and other values regarding foundation geometry and soil design parameter to meet any specific design criteria.

2.0 FIELD INVESTIGATION

Boreholes were sunk within the premises of the proposed project, the depth of borehole was measured from the existing ground level and hence the depth of borehole indicates depth below ground level (BGL).

Schedule of boreholes is tabulated form is given below:

Bore Hole No.	**Terminating Depth (m)**	**Water Table below EGL (m)**
BH-1 (A1)	35.45	1.50
BH-2 (P2)	35.45	1.70

3.0 Exploratory BORING

The provision laid down in BIS 1892: 1979 was followed in sinking the exploratory boreholes. Borehole was advanced into the soil by shell and augur boring to sink 150 mm diameter bore hole by using manually operated equipment. The boring was carried out by boring up to maximum depth of 35.45 m. Adequate care as per specification and

Indian standard practice was taken to prevent any possible side collapse in bore hole. The details of the bore hole including field tests of Standard Penetration tests and also collection of undisturbed/disturbed/SPT soil samples are given in Bore Log enclosed in Annexure. Disturbed representative samples of sub-surface deposits were collected from bore hole, labeled depth wise and placed in polythene bags. Reference Numbers and depth of these samples are shown in Bore Log Data Sheet.

Field and Laboratory Works

Field and laboratory works associated with this investigation has been conducted as per the following specifications of the Bureau of Indian Standards (BIS):

FIELD WORK		
Sl No.	**Description**	**Relevant IS Codes**
1	Boring, Drilling work and Collections of samples.	IS: 1892-1979 IS: 2131-1981 IS: 2132-1981
2	Labeling and Packing	IS: 1892-1979
3	Standard Penetration Test (SPT)	IS: 9640-1980 IS: 2131-1981
LABORATORY TEST		
Sl No.	**Description**	**Relevant IS Codes**
1	Natural Moisture Content	IS: 1892(Part-2)-1973
2	Bulk Density	IS: 2720(Part-28)-1974
3	Dry Density	IS: 2720(Part-2)-1973
4	Grain-Size Analysis	IS: 2720(Part-4)-1985
5	Liquid Limit (LL) and Plastic Limit (PL)	IS: 2720(Part-5)-1976
6	Shrinkage Limit (SL)	IS: 2720(Part-6)-1976
7	Free Swell Index	IS: 2720 (Part 40)-1977
8	Specific Gravity	IS: 2720(Part-3)-1980
9	Consolidation Test	IS: 2720(Part-15)-1986
10	Tri-axial Shear Test	IS: 2720(Part-11)-1971

4.0 SAMPLING

Disturbed and undisturbed samples were collected and standard penetration test were done during boring. In addition to this, study in change of strata, ground water level, visual identification of soil such as colour, nature and stiffness were recorded during boring.

- Disturbed Sample: Disturbed/SPT samples were collected at different depths and were properly packed after collection.
- Undisturbed Sample: Undisturbed sample were collected from soil layers which are cohesive in nature.

5.0 STANDARD PENETRATION TEST (SPT)

These tests were conducted in the boreholes at regular intervals or the change of strata; it was carried out by standard sampler (a split-spoon sampler) of standard design and dimension (50 mm OD and 35 mm ID, with minimum length of 450 mm). The sampler was driven by a 63.5 kg drive weight (monkey) as per guidelines laid in IS: 2131. As per the IS code of practice for this test, the monkey was allowed to fall on the top of the drill rod from a height of 750 mm several times until the sample penetrates about 150 mm into the soil as a seating drive. The numbers of blows required to drive the spoon from 150 mm to 450 mm i.e., beyond the seating drive, were recorded and this number of blows is called 'N' value or Standard Penetration Test (SPT) value of the sub-soil at that particular depth. Where the test has been carried outon completion of a test, the split spoon sampler was brought out of the borehole and opened the same. The collected soil sample from the split spoon sampler was preserved in air tight polythene packets for classification purpose. The samples were labeled properly with the project name, borehole and the depth of sampling.

Followings are the corrections on SPT values in cohesion-less soil:

1. Due to overburden: N value for cohesion-less soil shall be corrected for overburden as per Fig. 1 of IS: 2131 (N').
2. Due to Dilatancy: The values corrected for overburden shall be corrected for dilatancy if the stratum consists of fine sand and silt below water table for values of N' greater than 15, as under (N"): N" = 15+0.5*(N'-15)

Typical calculation for N value correction:

BH No.-1. Depth: 4.50 m to 4.95 m. Field N = 13. Water table: 1.50 m below EGL.

Effective OVP at the average depth: = 5.75 t/m^2.

From Fig 1.of IS: 2131, correction factor = 1.19

So corrected SPT value for overburden, N' = 1.19x13 = 15.47.

Corrected SPT for dilatancy N" = 15+{0.5x(15.47-15)} = 15.24

Same calculation will be valid for other N values in sandy layer. All the correction in detail is presented in a tabular form below in Table-1.

BH No.	Start Depth (m)	End Depth (m)	Avg Depth (m)	Field N	Correction		Corrected N	Stratum No
					Overburden	Dilatancy		
1	1.50	1.95	1.73	4	--	--	4	I
	4.50	4.95	4.73	13	15.47	15.24	15	II
	6.00	6.45	6.23	16	17.86	16.43	16	II
	7.50	7.95	7.73	15	15.87	15.44	15	II
	9.00	9.45	9.23	18	18.15	16.58	17	II
	10.50	10.95	10.73	20	19.31	17.15	17	II
	12.00	12.45	12.23	24	22.25	18.63	19	II
	13.50	13.95	13.73	26	23.21	19.11	19	II
	15.00	15.45	15.23	28	24.13	19.56	20	II
	16.50	16.95	16.73	32	26.66	20.83	21	III
	18.00	18.45	18.23	35	28.25	21.62	22	III
	19.50	19.95	19.73	38	29.74	22.37	22	III
	21.00	21.45	21.23	42	31.92	23.46	23	III
	22.50	22.95	22.73	46	33.99	24.49	24	III
	24.00	24.45	24.23	52	37.38	26.19	26	III
	25.50	25.95	25.73	56	39.20	27.10	27	III
	27.00	27.45	27.23	60	40.93	27.97	28	III
	28.50	28.95	28.73	61	40.58	27.79	28	III
	30.00	30.45	30.23	63	40.90	27.95	28	III
	31.50	31.95	31.73	66	41.83	28.42	28	III
	33.00	33.45	33.23	68	42.10	28.55	29	III
	35.00	35.45	35.23	70	42.03	28.52	29	III
2	2.00	2.45	2.23	5	--	--	5	I
	3.00	3.45	3.23	7	8.83	--	9	II
	4.50	4.95	4.73	11	12.93	--	13	II
	6.00	6.45	6.23	14	15.50	15.25	15	II
	7.50	7.95	7.73	21	22.05	18.53	19	II
	9.00	9.45	9.23	16	16.03	15.51	16	II
	10.50	10.95	10.73	18	17.27	16.13	16	II
	12.00	12.45	12.23	26	23.97	19.48	19	II
	13.50	13.95	13.73	26	23.09	19.04	19	II
	15.00	15.45	15.23	25	21.43	18.22	18	II
	16.50	16.95	16.73	33	27.37	21.18	21	III
	18.00	18.45	18.23	32	25.71	20.35	20	III
	19.50	19.95	19.73	39	30.39	22.70	23	III
	21.00	21.45	21.23	48	36.33	25.66	26	III
	22.50	22.95	22.73	42	30.90	22.95	23	III
	24.00	24.45	24.23	58	41.53	28.27	28	III
	25.50	25.95	25.73	63	43.93	29.47	29	III
	27.00	27.45	27.23	37	25.15	20.07	20	III
	28.50	28.95	28.73	51	33.81	24.40	24	III
	30.00	30.45	30.23	61	39.46	27.23	27	III
	31.50	31.95	31.73	67	42.32	28.66	29	III
	33.00	33.45	33.23	70	43.19	29.10	29	III
	35.00	35.45	35.23	74	44.29	29.64	30	III

Table-1: Correction of field N values.

Stratum wise “N” values are presented in tabular form given below:

Stratum No.	Stratum Description	“N” Values		
		Average	Maximum	Minimum
I	Soft to medium, silty clay/clayey silt	5	5	4
II	Loose to medium dense, silty sand	17	20	9
III	Dense to very dense, silty sand.	26	30	20

Note: N Value means Standard Penetration Test (SPT) Values.

6.0 GROUND WATER TABLE (GWT)

Ground water observations were made during boring and the depth at which it was encountered and the standing water level was recorded in the respective bore log sheet. It was noticed that the ground water table was found within a depth of 1.50 m to 1.70 m below EGL.

7.0 LABORATORY TEST

Relevant laboratory tests were conducted on selected disturbed/SPT soil samples collected during the field investigation for proper identification, classification and for determining the various engineering properties including the shear strength parameters of these sub-soils deposits. Some of the routine tests were also carried out using the soil samples. In general, the following tests were carried out on representative soil samples collected from exploratory boreholes at different depth/ strata**:**

On Soil Sample:

1. Atterberg limits (Liquid limit, Plastic limit).
2. Grain size analysis (Sieve and Hydrometer).
3. Triaxial Test (UU).
4. Consolidation Test.
5. Specific Gravity.
6. Free Swell Index.
7. Chemical Test.

The above mentioned laboratory tests were conducted as per the relevant Indian Standard Codes of practice and the results of these tests are furnished in the Annexure of this report. Results have been presented in the form of tables and graphs.

8.0 SOIL PROFILE

The average subsoil stratification has been considered for the design. The soil stratification may, in general, has been summarized as shown in Fig 1.

8.1Stratum-I:

The soil in this layer consists of soft to medium, greyish brown, silty clay/clayey silt with sand. Average “N” value of this layer is 5. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	--	Bulk Density (gm/cc)	1.849
Sand (%)	7.50	NMC (%)	32.10
Silt (%)	62.50	Dry Density (gm/cc)	1.400
Clay (%)	30.00	Plasticity Index (%)	17.60
Specific Gravity	2.60	Free Swell Index (%)	9.60
Liquid Limit (%)	44.50	E value (kg/cm^2)	60.00
Plastic Limit (%)	26.90	Swelling pressure (kg/cm^2)	0.25
Triaxial Shear Test (UU)			
Cohesion (kg/cm^2)	0.24		
Φ (degree)	3.00		

Note: Average properties are based on laboratory test results only.

IS Classification: CI

8.2Stratum-II:

The soil in this layer consists of loose to medium dense, light grey, silty fine to medium sand with mica. Average corrected “N” value of this layer is 17. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	--
Sand (%)	65.94
Silt+Clay (%)	34.06
Specific Gravity	2.63
Free Swell Index (%)	NIL

Note: Average properties are based on laboratory test results only.

IS Classification: SM

8.3Stratum-III:

The soil in this layer consists of dense to very dense, light grey, silty fine to medium sand with mica and gravels. Average corrected “N” value of this layer is 26. The soil samples that could be collected from this layer, shows the following average properties of the layer.

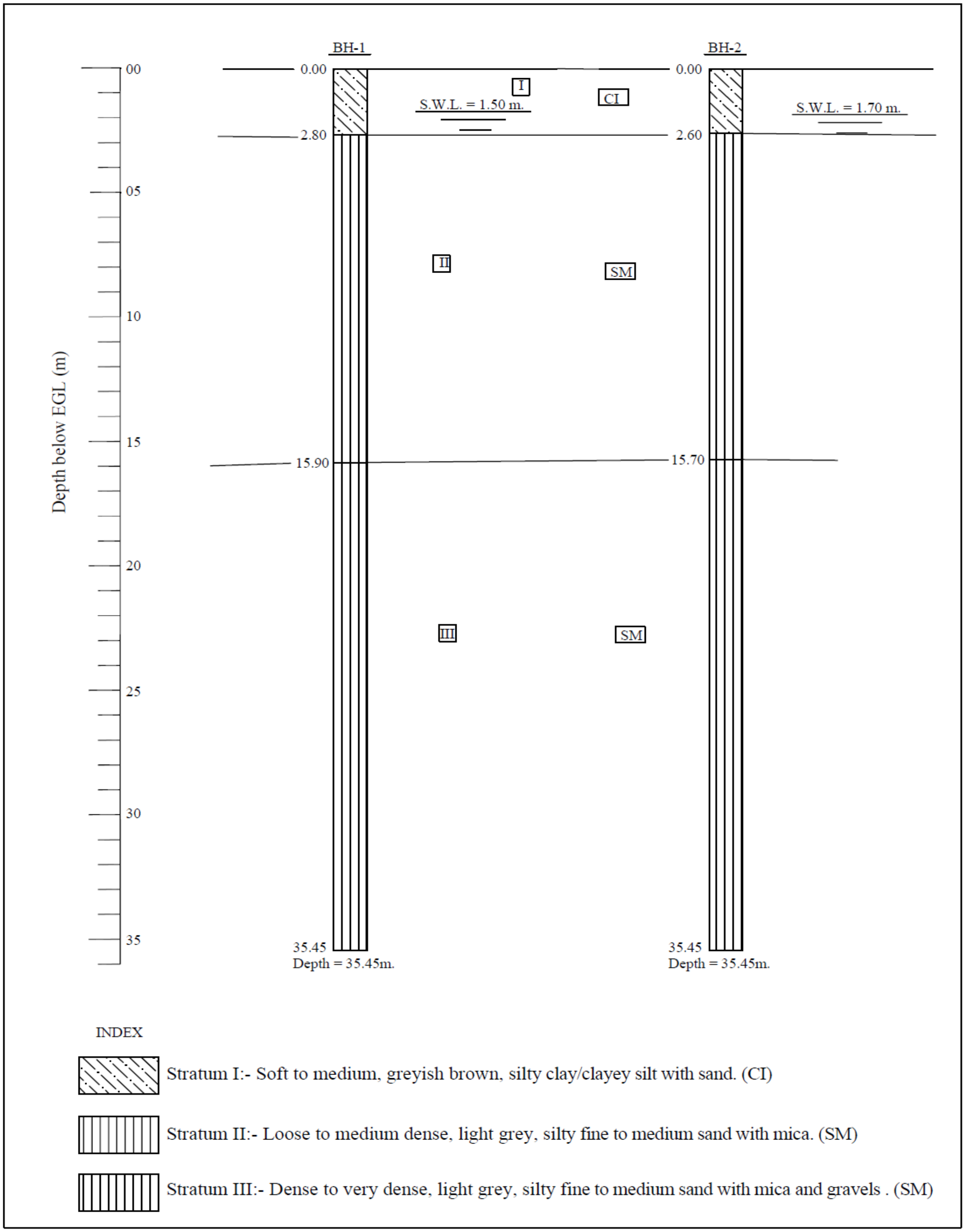


Fig.1 Generalized Sub Soil Profile

Gravel (%)	--
Sand (%)	87.32
Silt+Clay (%)	12.68
Specific Gravity	2.65
Free Swell Index (%)	NIL

Note: Average properties are based on laboratory test results only.

IS Classification: SM

9.0 FOUNDATIONS

In views of the above sub-soil conditions in mind and the type of structure both the aspects regarding bearing capacity of open foundationsand pile capacity for deep foundation in the form of bored cast in-situ pile have been discussedin the following paragraphs. Designer can choose the requirement depending upon the loading & geometry of the structure envisaged.

10.0 SUSCEPTIBILITY OF SUBSOIL TO LIQUEFACTION *[Ref: IS 1893 (Part 1)]*

The present site is under seismic zone-V.The liquefaction potential of subsoil is evaluated as per provision laid down in Indian Standarad and *"Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" by Dr. Gonzalo Castro et al. published in "Journal of Geotechnical and Geo-environmental Engineering", October' 2001*. Based on the SPT values, the liquefaction resistance of the borehole was evaluated for zone – V as per IS:1893 Part I and presented below. The ratio of CRR/CSR ≤ 1.0 indicates that the soil is prone to liquefaction whereas CRR/CSR > 1.0 or corrected N1 > 30 indicates the soil is non liquefiable. Based on the above, the liquefaction resistance of the subsoil is determined and presented below.

As per H. B. Seed and I.M. Idriss (1982) a clayey soil is said to be non-liquefiable if any one of the following three criteria is satisfied.

a) the soil contains fine grained soils with clay contents greater than 15%,
b) liquid limit greater than 35% or

c) moisturecontents less than 90% of the liquid limit.

In the present case, Stratum-I is having more than 15 % of clay content and liquid limit greater than 35%. So this layer is non-liquefiable. For Stratum-II/III, the liquefaction potential has been evaluated as follows.

Zone:V			a_{max}/g	0.36	Earthquake magnitude =							6.00			Bulk density =			1.85		
Structure Location	Avg Depth	Field N	Total OVP (t/sqm)	Effective OVP (t/sqm)	C_N	C_{HT}	C_{HW}	C_{BD}	C_{SS}	C_{RD}	$(N_1)_{60}$	FC	α	β	$(N_1)_{60CS}$	r_d	CSR	$CRR_{7.5}$	FOS	REMARKS
	1.73	4	3.19	1.47	Silty clay/clayey silt															Non Liquifiable
	4.73	13	8.74	4.02	1.58	0.75	0.98	1.05	1.10	0.85	14.9	34	4.93	1.19	22.6	0.97	0.49	0.25	0.896	Liquifiable
	6.23	16	11.52	5.29	1.37	0.75	0.98	1.05	1.10	0.95	17.8	34	4.93	1.19	26.1	0.96	0.49	0.32	1.141	Non Liquifiable
	7.73	15	14.29	6.57	1.23	0.75	0.98	1.05	1.10	0.95	15.0	34	4.93	1.19	22.7	0.94	0.48	0.25	0.930	Liquifiable
	9.23	18	17.07	7.84	1.13	0.75	0.98	1.05	1.10	0.95	16.5	34	4.93	1.19	24.5	0.92	0.47	0.28	1.061	Non Liquifiable
	10.73	20	19.84	9.12	1.05	0.75	0.98	1.05	1.10	1.00	17.9	34	4.93	1.19	26.2	0.89	0.45	0.32	1.231	Non Liquifiable
	12.23	24	22.62	10.39	0.98	0.75	0.98	1.05	1.10	1.00	20.1	34	4.93	1.19	28.8	0.85	0.43	0.40	1.628	Non Liquifiable
	13.73	26	25.39	11.67	0.93	0.75	0.98	1.05	1.10	1.00	20.5	34	4.93	1.19	29.3	0.80	0.41	0.43	1.834	Non Liquifiable
	15.23	28	28.17	12.94	0.88	0.75	0.98	1.05	1.10	1.00	21.3	34	4.93	1.19	30.3	0.75	0.38	0.49	2.236	Non Liquifiable
	16.73	32	30.94	14.22	0.84	0.75	0.98	1.05	1.10	1.00	23.2	13	1.89	1.04	26.0	0.70	0.36	0.31	1.535	Non Liquifiable
BH-1	18.23	35	33.72	15.49	0.80	0.75	0.98	1.05	1.10	1.00	24.4	13	1.89	1.04	27.1	0.66	0.34	0.34	1.791	Non Liquifiable
	19.73	38	36.49	16.77	0.77	0.75	0.98	1.05	1.20	1.00	27.7	13	1.89	1.04	30.6					Non Liquifiable
	21.23	42	39.27	18.04	0.74	0.75	0.98	1.05	1.20	1.00	29.5	13	1.89	1.04	32.5					Non Liquifiable
	22.73	46	42.04	19.32	0.72	0.75	0.98	1.05	1.20	1.00	31.3	13	1.89	1.04	34.3					Non Liquifiable
	24.23	52	44.82	20.59	0.70	0.75	0.98	1.05	1.20	1.00	34.2	13	1.89	1.04	37.4					Non Liquifiable
	25.73	56	47.59	21.87	0.68	0.75	0.98	1.05	1.20	1.00	35.8	13	1.89	1.04	39.0					Non Liquifiable
	27.23	60	50.37	23.14	0.66	0.75	0.98	1.05	1.20	1.00	37.3	13	1.89	1.04	40.5		$(N_1)_{60}$>30			Non Liquifiable
	28.73	61	53.14	24.42	0.64	1.00	0.98	1.05	1.10	1.00	45.1	13	1.89	1.04	48.6					Non Liquifiable
	30.23	63	55.92	25.69	0.62	1.00	0.98	1.05	1.10	1.00	45.4	13	1.89	1.04	49.0					Non Liquifiable
	31.73	66	58.69	26.97	0.61	1.00	0.98	1.05	1.10	1.00	46.4	13	1.89	1.04	50.0					Non Liquifiable
	33.23	68	61.47	28.24	0.60	1.00	0.98	1.05	1.10	1.00	46.7	13	1.89	1.04	50.3					Non Liquifiable
	35.23	70	65.17	29.94	0.58	1.00	0.98	1.05	1.10	1.00	46.7	13	1.89	1.04	50.3					Non Liquifiable
	2.23	5	4.12	1.89	Silty clay/clayey silt															Non Liquifiable
	3.23	7	5.97	2.74	1.70	1.00	0.98	1.05	1.10	0.80	11.0	34	4.93	1.19	18.0	0.98	0.50	0.19	0.678	Liquifiable
	4.73	11	8.74	4.02	1.58	1.00	0.98	1.05	1.10	0.85	17.0	34	4.93	1.19	25.2	0.97	0.49	0.30	1.055	Non Liquifiable
	6.23	14	11.52	5.29	1.37	1.00	0.98	1.05	1.10	0.95	21.1	34	4.93	1.19	30.0	0.96	0.49	0.47	1.697	Non Liquifiable
	7.73	21	14.29	6.57	1.23	1.00	0.98	1.05	1.10	0.95	28.4	34	4.93	1.19	38.7	0.94	0.48	0.07	0.258	Non Liquifiable
	9.23	16	17.07	7.84	1.13	1.00	0.98	1.05	1.10	0.95	19.8	34	4.93	1.19	28.5	0.92	0.47	0.39	1.459	Non Liquifiable
	10.73	18	19.84	9.12	1.05	1.00	0.98	1.05	1.10	1.00	21.8	34	4.93	1.19	30.8					Non Liquifiable
	12.23	26	22.62	10.39	0.98	1.00	0.98	1.05	1.10	1.00	29.5	34	4.93	1.19	39.9					Non Liquifiable
	13.73	26	25.39	11.67	0.93	1.00	0.98	1.05	1.10	1.00	27.8	34	4.93	1.19	38.0					Non Liquifiable
	15.23	25	28.17	12.94	0.88	1.00	0.98	1.05	1.10	1.00	25.4	34	4.93	1.19	35.1					Non Liquifiable
	16.73	33	30.94	14.22	0.84	1.00	0.98	1.05	1.10	1.00	32.0	13	1.89	1.04	35.0					Non Liquifiable
BH-2	18.23	32	33.72	15.49	0.80	1.00	0.98	1.05	1.10	1.00	29.7	13	1.89	1.04	32.7					Non Liquifiable
	19.73	39	36.49	16.77	0.77	1.00	0.98	1.05	1.10	1.00	34.8	13	1.89	1.04	38.0					Non Liquifiable
	21.23	48	39.27	18.04	0.74	1.00	0.98	1.05	1.10	1.00	41.3	13	1.89	1.04	44.7					Non Liquifiable
	22.73	42	42.04	19.32	0.72	1.00	0.98	1.05	1.10	1.00	34.9	13	1.89	1.04	38.1		$(N_1)_{60}$>30			Non Liquifiable
	24.23	58	44.82	20.59	0.70	1.00	0.98	1.05	1.10	1.00	46.7	13	1.89	1.04	50.3					Non Liquifiable
	25.73	63	47.59	21.87	0.68	1.00	0.98	1.05	1.10	1.00	49.2	13	1.89	1.04	52.9					Non Liquifiable
	27.23	37	50.37	23.14	0.66	1.00	0.98	1.05	1.10	1.00	28.1	13	1.89	1.04	31.0					Non Liquifiable
	28.73	51	53.14	24.42	0.64	1.00	0.98	1.05	1.10	1.00	37.7	13	1.89	1.04	41.0					Non Liquifiable
	30.23	61	55.92	25.69	0.62	1.00	0.98	1.05	1.10	1.00	44.0	13	1.89	1.04	47.5					Non Liquifiable
	31.73	67	58.69	26.97	0.61	1.00	0.98	1.05	1.10	1.00	47.1	13	1.89	1.04	50.8					Non Liquifiable
	33.23	70	61.47	28.24	0.60	1.00	0.98	1.05	1.10	1.00	48.1	13	1.89	1.04	51.8					Non Liquifiable
	35.23	74	65.17	29.94	0.58	1.00	0.98	1.05	1.10	1.00	49.4	13	1.89	1.04	53.1					Non Liquifiable

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 8.00 m depth below EGL.

Liquefaction Measures

Liquefaction potential can be improved by following ground improvement techniques.

(a) **Compaction of loose sand**. Loose saturated sands are more prone to liquefaction. The liquefaction potential can be reduced by compacting the loose sand deposits. This may be achieved by rolling with rubber tyre rollers, with vibratory rollers, with driven piles, with vibro-floatation or with blasting.

(b) **Grouting and chemical stabilization**. Grouting with mixtures of cement and chemical stabilization in the form of cement, lime and fly-ash can reduce liquefaction potential of the ground.

(c) **Application of surcharge.** Application of surcharge over the soil deposits can be used as effective measures against liquefaction.

(d) **Blankets and Drains.** Blankets and drains of sand or geo-synthetic material with high permeability speed up the drainage and reduce the liquefaction hazard.

11.0 CAPACITY CALCULATION:

11.1 USE OF DEEP FOUNDATION:

Alternatively, bored cast in-situ piles are preferred due to typical geological formation, availability of construction agencies; ease of construction and less sound pollution.
While calculating the pile capacity, let us assume that,

a) Pile capacities shall be ascertained by approved method (IS 2911 (Part 1/Sec 2):2010).
b) Assumed Grade of Concrete = M35.
c) Diameter of pile used = 1200 mm.
d) Depth of liquefaction below EL = 8.00 m.
e) Pile termination depth below existing ground level = 30.00 m.
f) Cut-off depth = 2.00 m below existing ground level.

Sample Calculation of Safe Vertical Pile Capacity around BH-1 [As per IS 2911 (Part 1/Sec 2) : 2010]

Design Strength Parameters:

STRATUM-II:

Design corrected “N” = 15. Corresponding $\Phi = 31^{o}$ *[Ref: Fig. No. 1.of IS 6403]*

Use, $\Phi = 28+15xD_r$. [In absence of any codal reference]

[Ref: "Foundation Analysis and Design", Fifth Edition, by J.E.Bowles, Table: 3-4, PP-162.]

Consistency of the layer = Medium dense. Corresponding $D_r = 0.35$. $\Phi = 33.25^o$.

As the pile is bored cast-in-situ type, appreciable loosening of soil is anticipated due to boring of hole and this may lead to reduction in the angle of internal friction of soil around the wall of borehole.

Considering the subsoil condition,

Use, design C = 0.00kg/cm^2 and $\Phi = 29^o$.

$\gamma_{sat} = 1.90$ t/m^3. $\gamma_{sub} = 0.90$ t/m^3 *[Effective density]*

STRATUM-III:

Design corrected "N" = 26. Corresponding $\Phi = 36^o$ *[Ref: Fig. No. 1.of IS 6403]*

Use, $\Phi = 28+15xD_r$. [In absence of any codal reference]

[Ref: "Foundation Analysis and Design", Fifth Edition, by J.E.Bowles, Table: 3-4, PP-162.]

Consistency of the layer = Dense. Corresponding $D_r = 0.65$. $\Phi = 37.75^o$.

As the pile is bored cast-in-situ type, appreciable loosening of soil is anticipated due to boring of hole and this may lead to reduction in the angle of internal friction of soil around the wall of borehole.

Considering the subsoil condition,

Use, design C = 0.00kg/cm^2 and $\Phi = 33^o$.

$\gamma_{sat} = 1.95$ t/m^3. $\gamma_{sub} = 0.95$ t/m^3 *[Effective density]*

The ultimate vertical pile capacity of bored cast in situ RCC pile in soil may be estimated using the formula as given below:

$$Q_u = (A_p \times P_D \times N_q) + (A_p \times N_c \times c_p) + \sum [K_i \times P_{Di} \times \tan\delta_i \times A_{si}] + \sum [\alpha_i \times c_i \times A_{si}]$$

where, Q_u = Ultimate vertical load carrying capacity of RCC bored pile,

A_p = Cross sectional area of pile tip. = $\pi/4 \times (D)^2$

D = Diameter of pile.

P_D = Effective overburden pressure at pile tip.

N_q = Bearing capacity factor for bored pile depending on Φ.

N_c = Bearing capacity factor, may be taken as 9.

c_p = Average cohesion at pile tip.

K_i = Co-efficient of earth pressure in i^{th} layer.

P_{Di} =Mean Effective overburden pressure of i^{th} layer.

δ_i = Angle of wall friction between pile and soil for the i^{th} layer.

A_{si}= Surface area of pile shaft in i^{th} layer = π x D x L_i

L_i = Length of pile in respective stratum,

Φ = Angle of internal friction of soil.

α_i= Adhesion factor for the i^{th} layer.

c_i= Average cohesion for the i^{th} layer.

Pile terminating depth below existing ground level = 30.00 m

Cut-off depth = 2.00 m below existing ground level.

Liquefaction depth = 8.00 m below EGL.

Consider the Scour Depth =8.00m below EGL.

Shaft length contributing pile capacity = 28.00 m.

Diameter of pile = 1200 mm.

Sample Calculation for Reinforced Concrete Circular Pile as per IS:2911:2010**

Pile Dia : 1.200m
Existing Ground level : 0.000m
Existing GWT level : -0.500m
Pile Cut-Off level : -2.000m
Pile Termination level : -30.000m

For maximum overburden pressure at Pile tip 18 x dia length of pile has been taken

****ULTIMATE END BEARING CAPACITY****

For Granular Soils

Qeg = Ap(0.5 * D * W * Nr + Pd * Nq)
where Ap = Cross sectional Area = 1.1304m^2
D = Pile Stem Dia = 1.20m
W = Effective unit wt. of soil at Pile tip = 9.69kN/m^3
Nr = Bearing Capacity factor = 35.19
Pd = Effective overburden pressure at pile tip = 54.26kN/m^2
Nq = Bearing Capacity factor = 35.60

Ultimate End Bearing Capacity Qeg =1.130*(0.5*1.200*9.693*35.19+54.265*35.60) =2415.07kN

For Cohesive Soils

Qec = Ap * Nc * Cp

where Ap = As defined above = 1.1304m^2

Nc = Bearing Capacity factor = 9

Cp = Average Cohesion at pile tip = 0.00kN/m^2

Ultimate End Bearing Capacity Qec =1.130*0.000*9 =0.00kN/m^2

Total Ultimate End Bearing Capacity Qu = Qeg + Qec = 2415.07kN

Pd Level for this pile= -21.600m

Layer No.1

Effective overburden pressure due to this layer =0.500x0.000+7.500x(0.000-9.807346327) =-73.555 kN/m^2

Layer No.2

Effective overburden pressure due to this layer =8.000x(19.000-9.807346327) =73.541 kN/m^2

Layer No.3

Effective overburden pressure due to this layer =5.600x(19.500-9.807346327) =54.279 kN/m^2

Layer No.4

Effective overburden pressure due to this layer =0.000x(19.500-9.807346327) =0.000 kN/m^2

Total effective overburden pressure up to -21.600M level from EGL=54.265 kN/m^2

****ULTIMATE SKIN FRICTION CAPACITY****

For Granular Soils

Qsg = Sum[K * Pdi * tan(d) * Asi] for all layers

where K = Earth Pressure Coeff.

Pdi= Effective Overburden presure for ith layer

d = Angle of wall friction for ith layer

Asi = Surface area of pile stem for ith layer

Negative Skin Friction Qsg(_ve) = Sum[0.5 * K * W * Ln * tan(d) * Asi] for all layers

W = Bulk Unit Wt. of soil

W = Bulk Unit Wt. of soil

Ln = Thk. of Compressible layer
K = As defined above
d = As defined above
Asi = As defined above but with Ln

For Cohesive Soils
Qsc = Sum[a * C * As] for all layers
where a = Adhesion factor
C = Average Cohesion
Asi = Surface area of pile stem for ith layer
Negative Skin Friction Qsc(_ve) = Sum[S * Asi] for all layers
S = Shear strength
Asi = As defined above but with Ln

Layer no. 1:
Layer Thickness : 8.000
Pdi : -36.78kN/m^2
K : 0.00
Pdi : -36.78kN/m^2
tan(d) : 0.00
Asi : 22.62m^2
Qsg = K*Pdi*tan(d)*Asi =0.00*-36.78*0.00*22.62 = -0.00kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00
Average Cohesion(C): 0.00kN/m^2
Qsc = a*C*Asi =1.00*0.00*22.62 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]
=[-0.00-0.00]+[0.00-0.00]=0.00kN/m^2

Layer no. 2:
Layer Thickness : 8.000
Pdi : -36.78kN/m^2
K : 1.00
Pdi : -36.78kN/m^2
tan(d) : 0.55
Asi : 30.16m^2
Qsg = K*Pdi*tan(d)*Asi =1.00*-36.78*0.55*30.16 = -614.95kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00
Average Cohesion(C): 0.00kN/m^2
Qsc = a*C*Asi =1.00*0.00*30.16 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]
=[-614.95-0.00]+[0.00-0.00]=-614.95kN/m^2

Layer no. 3:

Layer Thickness : 5.600
Pdi : 27.13kN/m^2
K : 1.15
Pdi : 27.13kN/m^2
tan(d) : 0.65
Asi : 21.11m^2
Qsg = K*Pdi*tan(d)*Asi =1.15*27.13*0.65*21.11 = 427.67kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00
Average Cohesion(C): 0.00kN/m^2
Qsc = a*C*Asi =1.00*0.00*21.11 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]
=[427.67-0.00]+[0.00-0.00]=427.67kN/m^2

Layer no. 4:

Layer Thickness : 8.400
Pdi : 54.26kN/m^2
K : 1.15
Pdi : 54.26kN/m^2
tan(d) : 0.65
Asi : 31.67m^2
Qsg = K*Pdi*tan(d)*Asi =1.15*54.26*0.65*31.67 = 1283.35kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00
Average Cohesion(C): 0.00kN/m^2
Qsc = a*C*Asi =1.00*0.00*31.67 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]
=[1283.35-0.00]+[0.00-0.00]=1283.35kN/m^2

Total Skin Friction Capacity Qus = Qsg + Qsc = 1096.08kN

Total Ultimate Pile Capacity Qu = Qus + Que = 3511.14kN

****Safe Pile Capacity Qus/FOS + Que/FOS = 1404.46kN****

****Computation of Tension capacity****

Tension capacity is computed based upon skin friction of pile only

Total ultimate skin friction Qus= 1096.08 kN
Average Void Factor = 12.00 %
Total ultimate tension capacity Qut= 1096.08kN
F.O.S. in tension = 2.50

****Safe tension capacity = Qut/FOS = 438.43kN

Calculation of lateral pile capacity (Fixed head pile)

Lateral capacity is computed based up on
Allowable top deflection Y = 12.0mm

Free Length L1 = 6.000m.
For lateral capacity soil is considered as GRANULAR
Fixed Head Pile
Mod. of Subgrade Reaction for Granular Soil(nb)= 0.800kN/m^3 X 1e3 (From IS Code)
E = 29580000.00 kN/m^2 I = 0.101788m^4.
T =(EI/nb)^0.2=(29580000.00x0.101788/0.800x1e3)^0.2= 5.189m
L1/T = 1.156
Depth of Fixity Lf = 10.322m

Total Ultimate Horizontal Load Capacity H[As per IS Code] =12EIY/(L1+Lf)^3 =99.707kN

Based on soil design parameters, safe capacities for pile are presented below

Recommended Allowable bearing capacity values are as follows:

Termination Depth of pile below existing ground level (m)	Vertical Shaft Length (m)	Dia of Pile (mm)	Recommended vertical pile capacity (t)	Recommended uplift pile capacity (t)	Recommended lateral load capacity(t)
					Recommended lateral load capacity(t)
30.00	28.00	1200	140	40	9.0

Note: The above load should confirmed by Load Test of pile as per IS: 2911(Part 4)

13.0 CONCLUSION AND GENERAL RECOMMENDATIONS

Based on the field tests and the foregoing discussion the following are summarized:

a)The subsoil is characterized by a soft to medium, silty clay/clayey silt layer at top followed by a layer of loose to medium dense, silty sand. After that a dense to very dense silty sand layer is observed and that continues up to terminating depth of both the boreholes.

b)The present report deals with the geotechnical investigation findings at the location and the discussion on both the aspects regarding allowable bearing capacity of open foundationsand safe pile capacity for deep foundation in the form of bored cast in-situ pile depending on the field and laboratory test results. However, the Foundation Designer may modulate the type of foundations and other values regarding foundation geometry and soil design parameter to meet any specific design criteria.

c) Standing water level was found within a depth of 1.50 m to 1.70 m below EGL. However, ground water level is considered at ground level for design purpose.

d) Susceptibility of liquefaction of the sub soil has been discussed in section 11. The site is liquefiable up to 8.00 m below EGL.

e)The foundation designer may consider adoption of suitable diameter and length of pile foundation depending on the type size and other considerations of the proposed bridge structure. For other diameter and length of pile, the load capacity may be estimated, using the sample calculation as provided in this report. The type of foundations shall be adopted based on possible construction conditions.

f)***<u>Considering the site condition and to be on a safer side pile foundation is strongly recommended.</u>***

g) Precaution in all respect should be taken for nearby existing structures, if any. The final decision regarding the foundation will depend on the judgment of the engineer concerned.

h) **Comment on the chemical nature of soil & ground water:**
Chemical test was carried out on a soil and water samples to determine the pH value, Sulphate and Chloride. It is seen that the values are within permissible limits (as per IS:456-2000) and so no special cement will be required for foundation concrete. **Either**

Ordinary Portland cement or Portland slag cement or Portland Pozzolana cement can be used for concreting.

i) No Expansive soil has been found on the site.

j) Seismic co-efficient as per IS specification should be considered during design.

ANNEXURE -1
BORE LOG DATA SHEETS

BORE / DRILL LOG

Bore Hole No. : BH 1(A1)

Site: Assum NH-29		**Method of Boring / Drilling : Wash**
Standing Water Level : 1.50 m b.g.l.		Dia.of Boring / Drilling : 150mm
Casing Lowered **: 3.00m**	N - 538920.53	
R.L -	E - 2878156.108	

Date	Depth (m)		Length (m)	Nature of Sampling	SPT : No. of blows				Description
	From	To			00-15 cm	15-30 cm	30-45 cm	N' Value	
0	0.50			D					Soft, greyish brown, silty clay/clayey silt
	1.50	1.95	0.45	P	1	2	2	4	
									2.80m
	3.00	3.45	0.45	U	* Not - Received				
	3.50	-		D					
	4.50	4.95	0.45	P	4	5	8	13	
	6.00	6.45	0.45	P	4	7	9	16	
	7.50	7.95	0.45	P	5	6	9	15	Medium dense, light grey, silty fine to medium sand with mica
	9.00	9.45	0.45	P	6	8	10	18	
	10.50	10.95	0.45	P	6	9	11	20	
	12.00	12.45	0.45	P	8	10	14	24	
	13.50	13.95	0.45	P	9	11	15	26	
	15.00	15.45	0.45	P	9	12	16	28	
									15.90m
	16.50	16.95	0.45	P	11	14	18	32	
	18.00	18.45	0.45	P	12	15	20	35	
	19.50	19.95	0.45	P	14	17	21	38	

	21.00	21.45	0.45	P	15	19	23	42	
	22.50	22.95	0.45	P	18	22	24	46	
	24.00	24.45	0.45	P	21	24	28	52	
	25.50	25.95	0.45	P	21	27	29	56	Dense to very dense, light grey, silty fine to medium sand with mica .
	27.00	27.45	0.45	P	22	29	31	60	
	28.50	28.95	0.45	P	21	28	33	61	
	30.00	30.45	0.45	P	22	30	33	63	
	31.50	31.95	0.45	P	22	32	34	66	
	33.00	33.45	0.45	P	23	31	37	68	
	35.00	35.45	0.45	P	23	32	38	70	
00-01	35.45	Termination Depth							

Abbreviations: U-Undisturbed Sample D-Disturbed Sample P-Standard Penetration Test

BORE / DRILL LOG

Bore Hole No. : BH 2 (P2)

Site: Assum NH-29		**Method of Boring / Drilling : Wash**
Standing Water Level : 1.70 m b.g.l.		Dia.of Boring / Drilling : 150mm
Casing Lowered **: 3.00m**	N - 538978.74	
R.L -	E - 2878141.564	

Date	Depth (m) From	Depth (m) To	Length (m)	Nature of Sampling	SPT : No. of blows 00-15 cm	15-30 cm	30-45 cm	N' Value	Description
0	0.50			D					
	1.50	1.95	0.45	U					Soft to medium, greyish brown, silty clay/clayey silt with sand
	2.00	2.45	0.45	P	1	2	3	5	2.60m
	3.00	3.45	0.45	P	2	3	4	7	
	4.50	4.95	0.45	P	3	5	6	11	
	5.50	-		D					
	6.00	6.45	0.45	P	4	7	7	14	
	7.50	7.95	0.45	P	6	9	12	21	Loose to medium dense, light grey, silty fine to medium sand with mica
	8.50	-		D					
	9.00	9.45	0.45	P	5	7	9	16	
	10.50	10.95	0.45	P	6	9	9	18	
	11.50	-		D					
	12.00	12.45	0.45	P	8	12	14	26	
	13.50	13.95	0.45	P	7	11	15	26	
	14.50	-		D					
	15.00	15.45	0.45	P	10	11	14	25	15.70m
	16.50	16.95	0.45	P	11	13	20	33	

	17.50	-		D					
	18.00	18.45	0.45	P	11	14	18	32	
	19.50	19.95	0.45	P	13	18	21	39	
	20.50	-		D					
	21.00	21.45	0.45	P	14	21	27	48	
	22.50	22.95	0.45	P	15	14	28	42	
	23.50	-		D					
	24.00	24.45	0.45	P	16	28	30	58	
	25.50	25.95	0.45	P	16	30	33	63	Dense to very dense, light grey, silty fine to medium sand with mica and gravels.
	26.50	-		D					
	27.00	27.45	0.45	P	12	17	20	37	
	28.50	28.95	0.45	P	13	22	29	51	
	29.50			D					
	30.00	30.45	0.45	P	20	28	33	61	
	31.50	31.95	0.45	P	23	30	37	67	
	32.00	-		D					
	33.00	33.45	0.45	P	26	33	37	70	
	35.00	35.45	0.45	P	28	34	40	74	
00-01	35.45	Termination Depth							

Abbreviations: U-Undisturbed Sample D-Disturbed Sample P-Standard Penetration Test

ANNEXURE-2

LABORATORY TEST RESULTS
(SOIL SAMPLE)

BH 1(A1)

	Type	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Natural Moisture Content (%)	Bulk Density (gm/cc)	Dry Density (gm/cc)	Liquid Limit(%)	Plastic Limit (%)	Plasticity Index (%)	IS Classification	Swell pressure (kg/cm^2)	Free Swelling Index (%)	Type of Test	Cohesion (kg/cm^2)	Angle of Friction (degree)	E Value (kg/cm^2)	Sp.Gravity	Pressure Range (kg/cm^2)	m_v (cm^2/kg)
BH - 1	D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompresive undrained test DS= Direct shear test	3.50	-	57.80	42.20					Non-Plastic			SM		NIL					2.62		
	P	7.50	-	62.37	37.63					Non-Plastic			SM									
	P	10.50	-	69.50	30.50					Non-Plastic			SM							2.63		
	P	15.00	-	72.60	27.40					Non-Plastic			SM									
	P	19.50	-	78.90	21.10					Non-Plastic			SM							2.64		
	P	24.00	-	81.20	18.80					Non-Plastic			SM									
	P	27.00	-	86.90	13.10					Non-Plastic			SM							2.65		
	P	31.50	-	92.30	7.70					Non-Plastic			SP-SM									
	P	35.00	-	96.57	3.43					Non-Plastic			SP		NIL					2.66		
	D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompresive undrained test DS= Direct shear test																					

BH 2(P2)

	Type	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Natural Moisture Content (%)	Bulk Density (gm/cc)	Dry Density (gm/cc)	Liquid Limit(%)	Plastic Limit (%)	Plasticity Index (%)	IS Classification	Swell pressure (kg/cm^2)	Free Swelling Index (%)	Type of Test	Cohesion (kg/cm^2)	Angle of Friction (degree)	E Value (kg/cm^2)	Sp.Gravity	Pressure Range (kg/cm^2)	m_v (cm^2/kg)
BH - 2	U	1.50	-	7.50	62.50	30.00	32.10	1.849	1.400	44.50	26.90	17.60	CI	0.25	15.66	UU	0.24	3.00	60.00	2.60	0.25-0.5	0.0465
																					0.5-1.0	0.0367
																					1.0-2.0	0.0254
																					2.0-4.0	0.0197
																					4.0-8.0	0.0120
	D	5.50	-	57.89	42.11					Non-Plastic			SM							2.63		
	P	9.00	-	66.50	33.50					Non-Plastic			SM									
	P	13.50	-	74.90	25.10					Non-Plastic			SM									
	D	17.50	-	79.10	20.90					Non-Plastic			SM							2.64		
	P	21.00	-	83.50	16.50					Non-Plastic			SM									
	P	25.50	-	88.90	11.10					Non-Plastic			SM							2.65		
	D	29.50	-	91.70	8.30					Non-Plastic			SP-SM							2.66		
	P	35.00	-	94.10	5.90					Non-Plastic			SP-SM							2.66		

D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompresive undrained test DS= Direct shear test

ANNEXURE-3

CHEMICAL TEST RESULTS

CHEMICAL TESTS

Chemical tests were performed on soil and water samples for determining the pH value, Sulphate, Chloride content etc. The results are given in a tabular form below:

CHEMICAL TEST RESULTS ON SOIL SAMPLES:-

BH No.	Depth (m)	pH value	Sulphate as SO_3 (%)	Chloride as Cl (%)
BH-1	1.50	7.12	0.012	0.044
BH-2	4.50	7.15	0.010	0.025

CHEMICAL TEST RESULTS ON WATER SAMPLE:-

BH/Sample No.	Depth (m)	pH value	Sulphate as SO_3 (mg/ltr)	Chloride as Cl (mg/ltr)
1 / WS-01	1.50	7.36	48.52	23.66
2 / WS-01	1.70	7.18	29.09	32.59
Permissible Limit		>6	400	500

Chemical test was carried out on a soil and water samples to determine the pH value, Sulphate and Chloride. It is seen that the values are within permissible limits (as per IS:456-2000) and so no special cement will be required for foundation concrete. **Either Ordinary Portland cement or Portland slag cement or Portland Pozzolana cement can be used for concreting.**

Soil Investigation Report of Major Bridge at Chainage: 129+905

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1.0 INTRODUCTION

A subsoil investigation was necessary for the purpose of the foundation design and construction of the proposed project. Accordingly the subsoil exploration work of two boreholes at structure location having maximum depth 38.45 m, as proposed by the project authority. During borehole exploration, undisturbed/disturbed/SPT samples were collected.

The present report deals with the geotechnical investigation findings at the location and the discussion on both the aspects regarding bearing capacity of open foundationsand pile capacity for deep foundation in the form of bored cast in-situ pile depending on the field and laboratory test results. However, the Foundation Designer may modulate the type of foundations and other values regarding foundation geometry and soil design parameter to meet any specific design criteria.

2.0 FIELD INVESTIGATION

Boreholes were sunk within the premises of the proposed project, the depth of borehole was measured from the existing ground level and hence the depth of borehole indicates depth below ground level (BGL).

Schedule of boreholes is tabulated form is given below:

Bore Hole No.	Terminating Depth (m)	Water Table below EGL (m)
BH-1 (A1)	38.25	1.50
BH-2 (P2)	38.45	2.00

3.0 EXPLORATORY BORING

The provision laid down in BIS 1892: 1979 was followed in sinking the exploratory boreholes. Borehole was advanced into the soil by shell and augur boring to sink 150 mm diameter bore hole by using manually operated equipment. The boring was carried out by boring up to maximum depth of 35.45 m. Adequate care as per specification and Indian standard practice was taken to prevent any possible side collapse in bore hole.

The details of the bore hole including field tests of Standard Penetration tests and also collection of undisturbed/disturbed/SPT soil samples are given in Bore Log enclosed in Annexure. Disturbed representative samples of sub-surface deposits were collected from bore hole, labeled depth wise and placed in polythene bags. Reference Numbers and depth of these samples are shown in Bore Log Data Sheet.

Field and Laboratory Works

Field and laboratory works associated with this investigation has been conducted as per the following specifications of the Bureau of Indian Standards (BIS):

FIELD WORK		
Sl No.	**Description**	**Relevant IS Codes**
1	Boring, Drilling work and Collections of samples.	IS: 1892-1979 IS: 2131-1981 IS: 2132-1981
2	Labeling and Packing	IS: 1892-1979
3	Standard Penetration Test (SPT)	IS: 9640-1980 IS: 2131-1981
LABORATORY TEST		
Sl No.	**Description**	**Relevant IS Codes**
1	Natural Moisture Content	IS: 1892(Part-2)-1973
2	Bulk Density	IS: 2720(Part-28)-1974
3	Dry Density	IS: 2720(Part-2)-1973
4	Grain-Size Analysis (sieve & hydrometer)	IS: 2720(Part-4)-1985
5	Liquid Limit (LL) and Plastic Limit (PL)	IS: 2720(Part-5)-1976
6	Free Swell Index	IS: 2720 (Part 40)-1977
7	Specific Gravity	IS: 2720(Part-3)-1980
8	Consolidation Test	IS: 2720(Part-15)-1986
9	Tri-axial Shear Test	IS: 2720(Part-11)-1971

4.0 SAMPLING

Disturbed and undisturbed samples were collected and standard penetration test were done during boring. In addition to this, study in change of strata, ground water level, visual identification of soil such as colour, nature and stiffness were recorded during boring.

- Disturbed Sample: Disturbed/SPT samples were collected at different depths and were properly packed after collection.
- Undisturbed Sample: Undisturbedsample could not be collected from soil layers.

5.0 STANDARD PENETRATION TEST (SPT)

These tests were conducted in the boreholes at regular intervals or the change of strata; it was carried out by standard sampler (a split-spoon sampler) of standard design and dimension (50 mm OD and 35 mm ID, with minimum length of 450 mm). The sampler was driven by a 63.5 kg drive weight (monkey) as per guidelines laid in IS: 2131. As per the IS code of practice for this test, the monkey was allowed to fall on the top of the drill rod from a height of 750 mm several times until the sample penetrates about 150 mm into the soil as a seating drive. The numbers of blows required to drive the spoon from 150 mm to 450 mm i.e., beyond the seating drive, were recorded and this number of blows is called 'N' value or Standard Penetration Test (SPT) value of the sub-soil at that particular depth. Where the test has been carried outon completion of a test, the split spoon sampler was brought out of the borehole and opened the same. The collected soil sample from the split spoon sampler was preserved in air tight polythene packets for classification purpose. The samples were labeled properly with the project name, borehole and the depth of sampling.

Followings are the corrections on SPT values in cohesion-less soil:

1. Due to overburden: N value for cohesion-less soil shall be corrected for overburden as per Fig. 1 of IS: 2131 (N').
2. Due to Dilatancy: The values corrected for overburden shall be corrected for dilatancy if the stratum consists of fine sand and silt below water table for values of N' greater than 15, as under (N"): N" = 15+0.5*(N'-15)

<u>Typical calculation for N value correction:</u>

BH No.-1. Depth: 13.50 m to 13.95 m. Field N = 36. Water table: 1.50 m below EGL.

Effective OVP at the average depth: = 12.35 t/m^2.

From Fig 1.of IS: 2131, correction factor = 0.93

So corrected SPT value for overburden, N' = 0.93x36 = 33.48.

Corrected SPT for dilatancy N" = 15+{0.5x(33.48-15)} = 24.24

Same calculation will be valid for other N values in sandy layer. All the correction in detail is presented in a tabular form below in Table-1.

Stratum wise “N” values are presented in tabular form given below:

Stratum No.	**Stratum Description**	**“N” Values**		
		Average	**Maximum**	**Minimum**
II	Loose to medium dense, silty sand	16	23	8
III	Dense to very dense, silty sand.	29	38	23

Note: N Value means Standard Penetration Test (SPT) Values.

BH No.	Start Depth (m)	End Depth (m)	Avg Depth (m)	Field N	Correction		Corrected N	Stratum No
					Overburden	Dilatancy		
1	2.00	2.45	2.23	8	12.32	--	12	II
	3.00	3.45	3.23	7	9.91	--	10	II
	4.50	4.95	4.73	13	16.74	15.87	16	II
	6.00	6.45	6.23	16	19.13	17.06	17	II
	7.50	7.95	7.73	18	20.22	17.61	18	II
	9.00	9.45	9.23	27	28.73	21.86	22	II
	10.50	10.95	10.73	29	29.39	22.20	22	II
	12.00	12.45	12.23	35	33.94	24.47	24	III
	13.50	13.95	13.73	36	33.48	24.24	24	III
	15.00	15.45	15.23	40	35.86	25.43	25	III
	16.50	16.95	16.73	45	38.93	26.96	27	III
	18.00	18.45	18.23	52	43.49	29.24	29	III
	19.50	19.95	19.73	57	46.16	30.58	31	III
	21.00	21.45	21.23	60	47.12	31.06	31	III
	22.50	22.95	22.73	60	45.75	30.38	30	III
	24.00	24.45	24.23	64	47.43	31.22	31	III
	25.50	25.95	25.73	62	44.71	29.85	30	III
	27.00	27.45	27.23	60	42.13	28.56	29	III
	28.50	28.95	28.73	61	41.73	28.37	28	III
	30.00	30.45	30.23	90	60.04	37.52	38	III
	31.50	31.95	31.73	69	44.92	29.96	30	III
	33.00	33.45	33.23	96	61.01	38.00	38	III
	35.00	35.45	35.23	100	61.60	38.30	38	III
	36.50	36.95	36.73	87	52.37	33.69	34	III
	38.00	38.45	38.23	100	58.86	36.93	37	III
2	2.00	2.45	2.23	7	10.78	--	11	II
	3.00	3.45	3.23	6	8.49	--	8	II
	4.50	4.95	4.73	11	14.17	--	14	II
	6.00	6.45	6.23	18	21.52	18.26	18	II
	7.50	7.95	7.73	20	22.47	18.73	19	II
	9.00	9.45	9.23	29	30.86	22.93	23	II
	10.50	10.95	10.73	33	33.45	24.22	24	III
	12.00	12.45	12.23	39	37.82	26.41	26	III
	13.50	13.95	13.73	33	30.73	22.86	23	III
	15.00	15.45	15.23	39	34.96	24.98	25	III
	16.50	16.95	16.73	43	37.20	26.10	26	III
	18.00	18.45	18.23	53	44.32	29.66	30	III
	19.50	19.95	19.73	50	40.49	27.75	28	III
	21.00	21.45	21.23	48	37.70	26.35	26	III
	22.50	22.95	22.73	41	31.26	23.13	23	III
	24.00	24.45	24.23	49	36.32	25.66	26	III
	25.50	25.95	25.73	44	31.73	23.36	23	III
	27.00	27.45	27.23	52	36.51	25.75	26	III
	28.50	28.95	28.73	58	39.68	27.34	27	III
	30.00	30.45	30.23	65	43.36	29.18	29	III
	31.50	31.95	31.73	73	47.52	31.26	31	III
	33.00	33.45	33.23	82	52.11	33.56	34	III
	35.00	35.45	35.23	92	56.67	35.83	36	III
	36.50	36.95	36.73	78	46.96	30.98	31	III
	38.00	38.45	38.23	85	50.03	32.52	33	III

Table-1: Correction of field N values.

6.0 GROUND WATER TABLE (GWT)

Ground water observations were made during boring and the depth at which it was encountered and the standing water level was recorded in the respective bore log sheet. It was noticed that the ground water table was found within a depth of 1.50 m to 2.00 m below EGL.

7.0 LABORATORY TEST

Relevant laboratory tests were conducted on selected disturbed/SPT soil samples collected during the field investigation for proper identification, classification and for determining the various engineering properties including the shear strength parameters of these sub-soils deposits. Some of the routine tests were also carried out using the soil samples. In general, the following tests were carried out on representative soil samples collected from exploratory boreholes at different depth/ strata**:**

On Soil Sample:

1. Atterberg limits (Liquid limit, Plastic limit).
2. Grain size analysis (Sieve and Hydrometer).
3. Specific Gravity.
4. Free Swell Index.
5. Chemical Test.

The above mentioned laboratory tests were conducted as per the relevant Indian Standard Codes of practice and the results of these tests are furnished in the Annexure of this report. Results have been presented in the form of tables and graphs.

8.0 SOIL PROFILE

The average subsoil stratification has been considered for the design. The soil stratification may, in general, has been summarized as shown in Fig 1.

8.1Stratum-I:

The soil in this layer consists of Dark grey, clayey silt with sand mixture. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Sand (%)	14.56	Liquid Limit (%)	44.25
Silt (%)	48.30	Plastic Limit (%)	20.31
Clay (%)	37.20	Plasticity Index (%)	23.95
Specific Gravity	2.71	Free Swell Index (%)	12.30

Note: Average properties are based on laboratory test results only.

IS Classification: CI

8.2Stratum-II:

The soil in this layer consists of loose to medium dense, silver grey, silty fine to medium sand with mica. Average corrected “N” value of this layer is 16. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	--
Sand (%)	61.86
Silt+Clay (%)	38.14
Specific Gravity	2.63
Free Swell Index (%)	NIL

Note: Average properties are based on laboratory test results only.

IS Classification: SM

8.3Stratum-III:

The soil in this layer consists of dense to very dense, silver grey, silty fine to medium/ coarse sand with mica & gravels. Average corrected “N” value of this layer is 29. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	5.13
Sand (%)	83.89
Silt+Clay (%)	10.98
Specific Gravity	2.65
Free Swell Index (%)	NIL

Note: Average properties are based on laboratory test results only.

IS Classification: SM

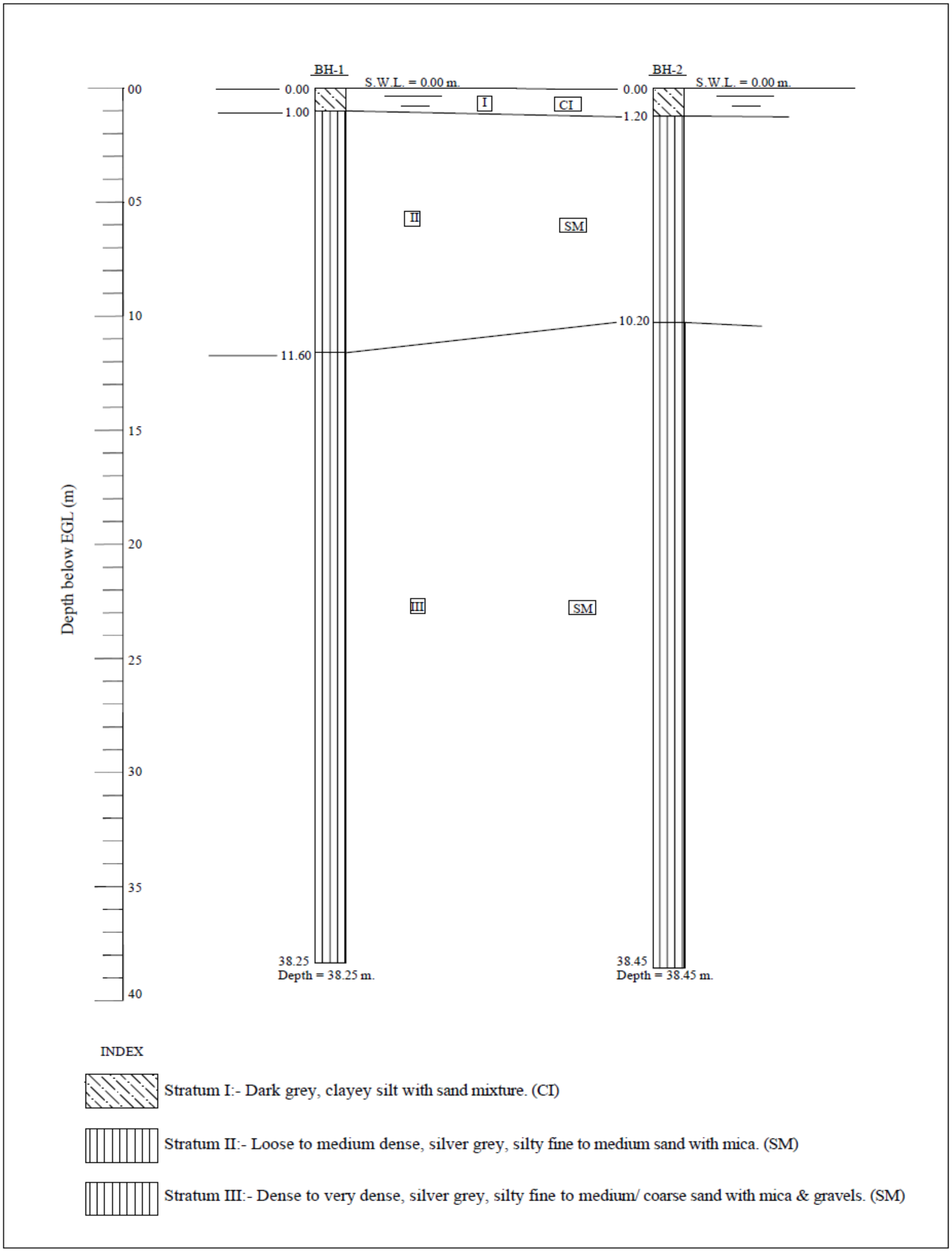

Fig.1 Generalized Sub Soil Profile

9.0 FOUNDATIONS

In views of the above sub-soil conditions in mind and the type of structure both the aspects regarding bearing capacity of open foundationsand pile capacity for deep foundation in the form of bored cast in-situ pile have been discussedin the following paragraphs. Designer can choose the requirement depending upon the loading & geometry of the structure envisaged.

10.0 SUSCEPTIBILITY OF SUBSOIL TO LIQUEFACTION *[Ref: IS 1893 (Part 1)]*

The present site is under seismic zone-V.The liquefaction potential of subsoil is evaluated as per provision laid down in Indian Standarad and *"Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" by Dr. Gonzalo Castro et al. published in "Journal of Geotechnical and Geo-environmental Engineering", October' 2001*. Based on the SPT values, the liquefaction resistance of the borehole was evaluated for zone – V as per IS:1893 Part I and presented below. The ratio of CRR/CSR ≤ 1.0 indicates that the soil is prone to liquefaction whereas CRR/CSR > 1.0 or corrected N1 > 30 indicates the soil is non liquefiable. Based on the above, the liquefaction resistance of the subsoil is determined and presented below.

As per H. B. Seed and I.M. Idriss (1982) a clayey soil is said to be non-liquefiable if any one of the following three criteria is satisfied.

a) the soil contains fine grained soils with clay contents greater than 15%,
b) liquid limit greater than 35% or
c) moisturecontents less than 90% of the liquid limit.

In the present case, Stratum-I is having more than 15 % of clay content and liquid limit greater than 35%. So this layer is non-liquefiable. For Stratum-II/III, the liquefaction potential has been evaluated as follows.

Zone:V			a_{max}/g	0.36	Earthquake magnitude =							6.00			Bulk density =			1.85		
Structure Location	**Avg Depth**	**Field N**	**Total OVP (t/sqm)**	**Effective OVP (t/sqm)**	C_N	C_{HT}	C_{HW}	C_{BD}	C_{SS}	C_{RD}	$(N_1)_{60}$	**FC**	α	β	$(N_1)_{60CS}$	r_d	**CSR**	$CRR_{7.5}$	**FOS**	**REMARKS**
BH-1	2.23	8	4.12	1.89	1.70	0.75	0.98	1.05	1.10	0.75	8.7	38	5.00	1.20	15.4	0.99	0.50	0.16	0.577	Liquifiable
	3.23	7	5.97	2.74	1.70	0.75	0.98	1.05	1.10	0.80	8.1	38	5.00	1.20	14.7	0.98	0.50	0.16	0.556	Liquifiable
	4.73	13	8.74	4.02	1.58	0.75	0.98	1.05	1.10	0.85	14.9	38	5.00	1.20	22.8	0.97	0.49	0.25	0.909	Liquifiable
	6.23	16	11.52	5.29	1.37	0.75	0.98	1.05	1.10	0.95	17.8	38	5.00	1.20	26.4	0.96	0.49	0.32	1.165	Non Liquifiable
	7.73	18	14.29	6.57	1.23	0.75	0.98	1.05	1.10	0.95	18.0	38	5.00	1.20	26.6	0.94	0.48	0.33	1.203	Non Liquifiable
	9.23	27	17.07	7.84	1.13	0.75	0.98	1.05	1.10	0.95	24.7	38	5.00	1.20	34.6	$(N_1)_{60}$>30				Non Liquifiable
	10.73	29	19.84	9.12	1.05	0.75	0.98	1.05	1.10	1.00	25.9	38	5.00	1.20	36.1					Non Liquifiable
	12.23	35	22.62	10.39	0.98	0.75	0.98	1.05	1.10	1.00	29.3	11	1.21	1.03	31.3					Non Liquifiable
	13.73	36	25.39	11.67	0.93	0.75	0.98	1.05	1.10	1.00	28.9	11	1.21	1.03	30.8					Non Liquifiable
	15.23	40	28.17	12.94	0.88	0.75	0.98	1.05	1.10	1.00	30.5	11	1.21	1.03	32.5					Non Liquifiable
	16.73	45	30.94	14.22	0.84	0.75	0.98	1.05	1.10	1.00	32.7	11	1.21	1.03	34.8					Non Liquifiable
	18.23	52	33.72	15.49	0.80	0.75	0.98	1.05	1.20	1.00	39.5	11	1.21	1.03	41.7					Non Liquifiable
	19.73	57	36.49	16.77	0.77	0.75	0.98	1.05	1.20	1.00	41.6	11	1.21	1.03	43.9					Non Liquifiable
	21.23	60	39.27	18.04	0.74	0.75	0.98	1.05	1.20	1.00	42.2	11	1.21	1.03	44.5					Non Liquifiable
	22.73	60	42.04	19.32	0.72	0.75	0.98	1.05	1.20	1.00	40.8	11	1.21	1.03	43.1					Non Liquifiable
	24.23	64	44.82	20.59	0.70	0.75	0.98	1.05	1.20	1.00	42.1	11	1.21	1.03	44.5					Non Liquifiable
	25.73	62	47.59	21.87	0.68	0.75	0.98	1.05	1.20	1.00	39.6	11	1.21	1.03	41.9					Non Liquifiable
	27.23	60	50.37	23.14	0.66	1.00	0.98	1.05	1.10	1.00	45.6	11	1.21	1.03	48.0					Non Liquifiable
	28.73	61	53.14	24.42	0.64	1.00	0.98	1.05	1.10	1.00	45.1	11	1.21	1.03	47.5					Non Liquifiable
	30.23	90	55.92	25.69	0.62	1.00	0.98	1.05	1.10	1.00	64.9	11	1.21	1.03	67.8					Non Liquifiable
	31.73	69	58.69	26.97	0.61	1.00	0.98	1.05	1.10	1.00	48.5	11	1.21	1.03	51.0					Non Liquifiable
	33.23	96	61.47	28.24	0.60	2.00	0.98	1.05	1.10	1.00	132.0	11	1.21	1.03	136.7					Non Liquifiable
	35.23	100	65.17	29.94	0.58	3.00	0.98	1.05	1.10	1.00	200.2	11	1.21	1.03	206.8					Non Liquifiable
	36.73	87	67.94	31.22	0.57	4.00	0.98	1.05	1.10	1.00	227.5	11	1.21	1.03	234.7					Non Liquifiable
	38.23	100	70.72	32.49	0.55	5.00	0.98	1.05	1.10	1.00	320.4	11	1.21	1.03	330.1					Non Liquifiable
BH-2	2.23	7	4.12	1.89	1.70	1.00	0.98	1.05	1.10	0.75	10.3	38	5.00	1.20	17.4	0.99	0.50	0.18	0.649	Liquifiable
	3.23	6	5.97	2.74	1.70	1.00	0.98	1.05	1.10	0.80	9.4	38	5.00	1.20	16.3	0.98	0.50	0.17	0.613	Liquifiable
	4.73	11	8.74	4.02	1.58	1.00	0.98	1.05	1.10	0.85	17.0	38	5.00	1.20	25.4	0.97	0.49	0.30	1.075	Non Liquifiable
	6.23	18	11.52	5.29	1.37	1.00	0.98	1.05	1.10	0.95	27.2	38	5.00	1.20	37.6	$(N_1)_{60}$>30				Non Liquifiable
	7.73	20	14.29	6.57	1.23	1.00	0.98	1.05	1.10	0.95	27.1	38	5.00	1.20	37.5					Non Liquifiable
	9.23	29	17.07	7.84	1.13	1.00	0.98	1.05	1.10	0.95	35.9	38	5.00	1.20	48.1					Non Liquifiable
	10.73	33	19.84	9.12	1.05	1.00	0.98	1.05	1.10	1.00	39.9	11	1.21	1.03	42.2					Non Liquifiable
	12.23	39	22.62	10.39	0.98	1.00	0.98	1.05	1.10	1.00	44.2	11	1.21	1.03	46.6					Non Liquifiable
	13.73	33	25.39	11.67	0.93	1.00	0.98	1.05	1.10	1.00	35.3	11	1.21	1.03	37.4					Non Liquifiable
	15.23	39	28.17	12.94	0.88	1.00	0.98	1.05	1.10	1.00	39.6	11	1.21	1.03	41.9					Non Liquifiable
	16.73	43	30.94	14.22	0.84	1.00	0.98	1.05	1.10	1.00	41.7	11	1.21	1.03	44.0					Non Liquifiable
	18.23	53	33.72	15.49	0.80	1.00	0.98	1.05	1.10	1.00	49.2	11	1.21	1.03	51.7					Non Liquifiable
	19.73	50	36.49	16.77	0.77	1.00	0.98	1.05	1.10	1.00	44.6	11	1.21	1.03	47.0					Non Liquifiable
	21.23	48	39.27	18.04	0.74	1.00	0.98	1.05	1.10	1.00	41.3	11	1.21	1.03	43.6					Non Liquifiable
	22.73	41	42.04	19.32	0.72	1.00	0.98	1.05	1.10	1.00	34.1	11	1.21	1.03	36.2					Non Liquifiable
	24.23	49	44.82	20.59	0.70	1.00	0.98	1.05	1.10	1.00	39.4	11	1.21	1.03	41.7					Non Liquifiable
	25.73	44	47.59	21.87	0.68	1.00	0.98	1.05	1.10	1.00	34.4	11	1.21	1.03	36.5					Non Liquifiable
	27.23	52	50.37	23.14	0.66	1.00	0.98	1.05	1.10	1.00	39.5	11	1.21	1.03	41.7					Non Liquifiable
	28.73	58	53.14	24.42	0.64	1.00	0.98	1.05	1.10	1.00	42.9	11	1.21	1.03	45.2					Non Liquifiable
	30.23	65	55.92	25.69	0.62	1.00	0.98	1.05	1.10	1.00	46.8	11	1.21	1.03	49.3					Non Liquifiable
	31.73	73	58.69	26.97	0.61	2.00	0.98	1.05	1.10	1.00	102.7	11	1.21	1.03	106.6					Non Liquifiable
	33.23	82	61.47	28.24	0.60	3.00	0.98	1.05	1.10	1.00	169.1	11	1.21	1.03	174.8					Non Liquifiable
	35.23	92	65.17	29.94	0.58	4.00	0.98	1.05	1.10	1.00	245.6	11	1.21	1.03	253.4					Non Liquifiable
	36.73	78	67.94	31.22	0.57	5.00	0.98	1.05	1.10	1.00	255.0	11	1.21	1.03	262.9					Non Liquifiable
	38.23	85	70.72	32.49	0.55	6.00	0.98	1.05	1.10	1.00	326.8	11	1.21	1.03	336.7					Non Liquifiable

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 6.00 m depth below EGL.

<u>Liquefaction Measures</u>

Liquefaction potential can be improved by following ground improvement techniques.

(a) **Compaction of loose sand**. Loose saturated sands are more prone to liquefaction. The liquefaction potential can be reduced by compacting the loose sand deposits. This may be achieved by rolling with rubber tyre rollers, with vibratory rollers, with driven piles, with vibro-floatation or with blasting.

(b) **Grouting and chemical stabilization**. Grouting with mixtures of cement and chemical stabilization in the form of cement, lime and fly-ash can reduce liquefaction potential of the ground.

(c) **Application of surcharge.** Application of surcharge over the soil deposits can be used as effective measures against liquefaction.

(d) **Blankets and Drains.** Blankets and drains of sand or geo-synthetic material with high permeability speed up the drainage and reduce the liquefaction hazard.

11.0 CAPACITY CALCULATION:

11.1 USE OF DEEP FOUNDATION:

Alternatively, bored cast in-situ piles are preferred due to typical geological formation, availability of construction agencies; ease of construction and less sound pollution.

While calculating the pile capacity, let us assume that,

a) Pile capacities shall be ascertained by approved method (IS 2911 (Part 1/Sec 2):2010).
b) Assumed Grade of Concrete = M35.
c) Diameter of pile used = 1200 mm.
d) Depth of liquefaction below EL = 6.00 m.
e) Pile termination depth below existing ground level = 30.00 m.
f) Cut-off depth = 2.00 m below existing ground level.

Sample Calculation of Safe Vertical Pile Capacity around BH-1 [As per IS 2911 (Part 1/Sec 2) : 2010]

Design Strength Parameters:

STRATUM-II:

Design corrected "N" = 15. Corresponding $\Phi = 31^o$ *[Ref: Fig. No. 1.of IS 6403]*

Use, $\Phi = 28+15xD_r$. [In absence of any codal reference]

[Ref: "Foundation Analysis and Design", Fifth Edition, by J.E.Bowles, Table: 3-4, PP-162.]

Consistency of the layer = Medium dense. Corresponding $D_r = 0.35$. $\Phi = 33.25^o$.

As the pile is bored cast-in-situ type, appreciable loosening of soil is anticipated due to boring of hole and this may lead to reduction in the angle of internal friction of soil around the wall of borehole.

Considering the subsoil condition,

Use, design C = 0.00kg/cm^2 and Φ = 29^o.

γ_{sat} = 1.90 t/m^3. γ_{sub} = 0.90 t/m^3*[Effective density]*

STRATUM-III:

Design corrected "N" = 26. Corresponding Φ = 36^o*[Ref: Fig. No. 1.of IS 6403]*

Use, Φ = 28+15xD_r. [In absence of any codal reference]

[Ref: "Foundation Analysis and Design", Fifth Edition, by J.E.Bowles, Table: 3-4, PP-162.]

Consistency of the layer = Dense. Corresponding D_r = 0.65. Φ = 37.75^o.

As the pile is bored cast-in-situ type, appreciable loosening of soil is anticipated due to boring of hole and this may lead to reduction in the angle of internal friction of soil around the wall of borehole.

Considering the subsoil condition,

Use, design C = 0.00kg/cm^2 and Φ = 33^o.

γ_{sat} = 1.95 t/m^3. γ_{sub} = 0.95 t/m^3*[Effective density]*

The ultimate vertical pile capacity of bored cast in situ RCC pile in soil may be estimated using the formula as given below:

$$Q_u = (A_p \times P_D \times N_q) + (A_p \times N_c \times c_p) + \sum [K_i \times P_{Di} \times \tan\delta_i \times A_{si}] + \sum [\alpha_i \times c_i \times A_{si}]$$

where, Q_u = Ultimate vertical load carrying capacity of RCC bored pile,

A_p = Cross sectional area of pile tip. = π/4 x (D)^2

D = Diameter of pile.

P_D = Effective overburden pressure at pile tip.

N_q = Bearing capacity factor for bored pile depending on Φ.

N_c = Bearing capacity factor, may be taken as 9.

c_p = Average cohesion at pile tip.

K_i = Co-efficient of earth pressure in ith layer.

P_{Di} =Mean Effective overburden pressure of ith layer.

δ_i = Angle of wall friction between pile and soil for the ith layer.

A_{si}= Surface area of pile shaft in i^{th} layer = π x D x L_i

L_i = Length of pile in respective stratum,

Φ = Angle of internal friction of soil.

α_i= Adhesion factor for the i^{th} layer.

c_i= Average cohesion for the i^{th} layer.

Pile terminating depth below existing ground level = 30.00 m

Cut-off depth = 2.00 m below existing ground level.

Liquefaction depth = 6.00 m below EGL.

Consider the Scour Depth =8.00m below EGL.

Shaft length contributing pile capacity = 28.00 m.

Diameter of pile = 1200 mm.

Sample Calculation for Reinforced Concrete Circular Pile as per IS:2911:2010

Pile Dia : 1.200m

Existing Ground level : 0.000m

Existing GWT level : -0.500m

Pile Cut-Off level : -2.000m

Pile Termination level : -30.000m

For maximum overburden pressure at Pile tip 18 x dia length of pile has been taken

****ULTIMATE END BEARING CAPACITY****

For Granular Soils

Qeg = Ap(0.5 * D * W * Nr + Pd * Nq)

where Ap = Cross sectional Area = 1.1304m^2

D = Pile Stem Dia = 1.20m

W = Effective unit wt. of soil at Pile tip = 9.69kN/m^3

Nr = Bearing Capacity factor = 35.19

Pd = Effective overburden pressure at pile tip = 56.46kN/m^2

Nq = Bearing Capacity factor = 35.60

Ultimate End Bearing Capacity Qeg

=1.130*(0.5*1.200*9.693*35.19+56.465*35.60)=2503.60kN

For Cohesive Soils

Qec = Ap * Nc * Cp

where Ap = As defined above = 1.1304m^2

Nc = Bearing Capacity factor = 9

Cp = Average Cohesion at pile tip = 0.00kN/m^2

Ultimate End Bearing Capacity Qec =1.130*0.000*9 =0.00kN/m^2

Total Ultimate End Bearing Capacity Qu = Qeg + Qec = 2503.60kN

Pd Level for this pile= -21.600m

Layer No.1

Effective overburden pressure due to this layer =0.500x0.000+7.500x(0.000-9.807346327) =-73.555 kN/m^2

Layer No.2

Effective overburden pressure due to this layer =3.600x(19.000-9.807346327) =33.094 kN/m^2

Layer No.3

Effective overburden pressure due to this layer =10.000x(19.500-9.807346327) =96.927 kN/m^2

Layer No.4

Effective overburden pressure due to this layer =0.000x(19.500-9.807346327) =0.000 kN/m^2

Total effective overburden pressure up to -21.600M level from EGL=56.465 kN/m^2

****ULTIMATE SKIN FRICTION CAPACITY****

For Granular Soils

Qsg = Sum[K * Pdi * tan(d) * Asi] for all layers

where K = Earth Pressure Coeff.

Pdi= Effective Overburden presure for ith layer

d = Angle of wall friction for ith layer

Asi = Surface area of pile stem for ith layer

Negative Skin Friction Qsg(_ve) = Sum[0.5 * K * W * Ln * tan(d) * Asi] for all layers

W = Bulk Unit Wt. of soil

W = Bulk Unit Wt. of soil

Ln = Thk. of Compressible layer

K = As defined above

d = As defined above

Asi = As defined above but with Ln

For Cohesive Soils

Qsc = Sum[a * C * As] for all layers

where a = Adhesion factor

C = Average Cohesion

Asi = Surface area of pile stem for ith layer

Negative Skin Friction Qsc(_ve) = Sum[S * Asi] for all layers

S = Shear strength

Asi = As defined above but with Ln

Layer no. 1:

Layer Thickness : 8.000

Pdi : -36.78kN/m^2

K : 0.00

Pdi : -36.78kN/m^2

tan(d) : 0.00

Asi : 22.62m^2

Qsg = K*Pdi*tan(d)*Asi =0.00*-36.78*0.00*22.62 = -0.00kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m^2

Qsc = a*C*Asi =1.00*0.00*22.62 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]

=[-0.00-0.00]+[0.00-0.00]=0.00kN/m^2

Layer no. 2:

Layer Thickness : 3.600

Pdi : -57.01kN/m^2

K : 1.00

Pdi : -57.01kN/m^2

tan(d) : 0.55

Asi : 13.57m^2

Qsg = K*Pdi*tan(d)*Asi =1.00*-57.01*0.55*13.57 = -428.87kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m^2

Qsc = a*C*Asi =1.00*0.00*13.57 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]

=[-428.87-0.00]+[0.00-0.00]=-428.87kN/m^2

Layer no. 3:

Layer Thickness : 10.000

Pdi : 8.00kN/m^2

K : 1.15

Pdi : 8.00kN/m^2

tan(d) : 0.65

Asi : 37.70m^2

Qsg = K*Pdi*tan(d)*Asi =1.15*8.00*0.65*37.70 = 225.28kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m^2

Qsc = a*C*Asi =1.00*0.00*37.70 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]

=[225.28-0.00]+[0.00-0.00]=225.28kN/m^2

Layer no. 4:

Layer Thickness : 8.400

Pdi : 56.46kN/m^2

K : 1.15

Pdi : 56.46kN/m^2

tan(d) : 0.65

Asi : 31.67m^2

Qsg = K*Pdi*tan(d)*Asi =1.15*56.46*0.65*31.67 = 1335.38kN

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m^2

Qsc = a*C*Asi =1.00*0.00*31.67 = 0.00kN

Total net skin friction of this layer =[Qsg-Qsg(_ve)]+[Qsc-Qsc(_ve)]

=[1335.38-0.00]+[0.00-0.00]=1335.38kN/m^2

Total Skin Friction Capacity Qus = Qsg + Qsc = 2127.61kN

Total Ultimate Pile Capacity Qu = Qus + Que = 4631.20kN

****Safe Pile Capacity Qus/FOS + Que/FOS = 1852.48kN****

****Computation of Tension capacity****

Tension capacity is computed based upon skin friction of pile only

Total ultimate skin friction Qus= 2127.61 kN

Average Void Factor = 12.00 %

Total ultimate tension capacity Qut= 2127.61kN

F.O.S. in tension = 2.50

****Safe tension capacity = Qut/FOS = 851.04kN

<u>Calculation of lateral pile capacity (Fixed head pile)</u>

Computation of Lateral Capacity****

Lateral capacity is computed based up on

Allowable top deflection Y = 12.0mm

Free Length L1 = 6.000m.

For lateral capacity soil is considered as GRANULAR

Fixed Head Pile

Mod. of Subgrade Reaction for Granular Soil(nb)= 0.800kN/m^3 X 1e3(From IS Code)

E = 29580000.00 kN/m^2 I = 0.101788m^4.

T =(EI/nb)^0.2=(29580000.00x0.101788/0.800x1e3)^0.2= 5.189m

L1/T = 1.156

Depth of Fixity Lf = 10.322m

Total Ultimate Horizontal Load Capacity H[As per IS Code] =12EIY/(L1+Lf)^3 =99.707kN

Based on soil design parameters, safe capacities for pile are presented below

Recommended Allowable bearing capacity values are as follows:

Termination Depth of pile below existing ground level (m)	Vertical Shaft Length (m)	Dia of Pile (mm)	Recommended vertical pile capacity (t)	Recommended uplift pile capacity (t)	Recommended lateral load capacity(t)
					Recommended lateral load capacity(t)
30.00	28.00	1200	170	60	9.0

Note: The above load should confirmed by Load Test of pile as per IS: 2911(Part 4)

12.0 CONCLUSION AND GENERAL RECOMMENDATIONS

Based on the field tests and the foregoing discussion the following are summarized:

a)The subsoil is characterized by a, silty clay/clayey silt layer at top followed by a layerof loose to medium dense, silty sand. After that adense to very dense silty sand layer is observed and that continues up to terminating depth of both the boreholes.

b)The present report deals with the geotechnical investigation findings at the location and the discussion on both the aspects regarding allowable bearing capacity of open foundationsand safe pile capacity for deep foundation in the form of bored cast in-situ pile depending on the field and laboratory test results. However, the Foundation Designer may modulate the type of foundations and other values regarding foundation geometry and soil design parameter to meet any specific design criteria.

c) Standing water level was found within a depth of 1.50 m to 2.00 m below EGL. However, ground water level is considered at ground level for design purpose.

d) Susceptibility of liquefaction of the sub soil has been discussed in section 11. The site is liquefiableupto 6.00 m below EGL.

e)Proper liquefaction mitigation measures have to be adopted to improve minimum 6.00 m below EGL before placing the open foundation.

g)The foundation designer may consider adoption of suitable diameter and length of pile foundation depending on the type size and other considerations of the proposed bridge structure. For other diameter and length of pile, the load capacity may be estimated, using the sample calculation as provided in this report. The type of foundations shall be adopted based on possible construction conditions.

h)***<u>Considering the site condition and to be on a safer side pile foundation is strongly recommended.</u>***

i) Precaution in all respect should be taken for nearby existing structures, if any. The final decision regarding the foundation will depend on the judgment of the engineer concerned.

j) **Comment on the chemical nature of soil & ground water:**
Chemical test was carried out on a soil and water samples to determine the pH value, Sulphate and Chloride. It is seen that the values are within permissible limits (as per IS:456-2000) and so no special cement will be required for foundation concrete. **Either Ordinary Portland cement or Portland slag cement or Portland Pozzolana cement can be used for concreting.**

k) No Expansive soil has been found on the site.

l) Seismic co-efficient as per IS specification should be considered during design.

ANNEXURE -1
BORE LOG DATA SHEETS

BORE / DRILL LOG

Bore Hole No. : BH-1 (A1)

Site: Assum NH-29		**Method of Boring / Drilling : Rotary**
Standing Water Level : In Water (2.00 m)		Dia.of Boring / Drilling : 150mm
Casing Lowered **: 3.00m**	N - 545573.578	
R.L -	E - 2873356.652	

Date	Depth (m) From	Depth (m) To	Length (m)	Nature of Sampling	SPT : No. of blows 00-15 cm	15-30 cm	30-45 cm	N' Value	Description
0									Dark grey, clayey silt with sand mixture.
	0.50	-		D					1.00m
	1.50	1.95	0.45	U	Not received				
	2.00	2.45	0.45	P	3	4	4	8	
	3.00	3.45	0.45	P	2	3	4	7	
	4.00	-		D					Loose to medium dense, silver grey, silty fine to medium sand with mica.
	4.50	4.95	0.45	P	5	6	7	13	
	6.00	6.45	0.45	P	4	7	9	16	
	7.00	-		D					
	7.50	7.95	0.45	P	6	8	10	18	
	9.00	9.45	0.45	P	8	11	16	27	
	10.00	-		D					
	10.50	10.95	0.45	P	7	12	17	29	
									11.60m
	12.00	12.45	0.45	P	9	16	19	35	
	13.00	-		D					
	13.50	13.95	0.45	P	10	14	22	36	

	15.00	15.45	0.45	P	13	18	22	40	
	16.00	-		D					
	16.50	16.95	0.45	P	10	18	27	45	
	18.00	18.45	0.45	P	13	22	30	52	
	19.00	-		D					
	19.50	19.95	0.45	P	14	23	34	57	
	21.00	21.45	0.45	P	16	26	34	60	Dense to very dense, silver grey, silty fine to medium/ coarse sand with mica & gravels.
	22.00	-		D					
	22.50	22.95	0.45	P	15	28	32	60	
	24.00	24.45	0.45	P	18	30	34	64	
	25.00	-		D					
	25.50	25.95	0.45	P	17	27	35	62	
	27.00	27.45	0.45	P	14	28	32	60	
	28.00	-		D					
	28.50	28.95	0.45	P	16	25	36	61	
	30.00	30.45	0.45	P	25	40	50	90	
	31.00	-		D					
	31.50	31.95	0.45	P	18	30	39	69	
	33.00	33.45	0.45	P	23	43	53	96	
	34.00	-		D					
	35.00	35.45	0.45	P	60	75 blows for 5 cm>100			
	36.50	36.95	0.45	P	20	35	52	87	
	37.00	-		D					
	38.00	38.25	0.25	P	49	89 blows	for 10 cm N > 100		

00-01	38.25	Termination Depth					
Abbreviations: U-Undisturbed Sample D-Disturbed Sample P-Standard Penetration Test							

BORE / DRILL LOG

Bore Hole No. : BH-2 (P2)

Site: Assum NH-29		**Method of Boring / Drilling : Rotary**
Standing Water Level : In Water (1.50 m)		Dia.of Boring / Drilling : 150mm
Casing Lowered **: 3.00m**	N - 545605.042	
R.L -	E -2873325.895	

Date	Depth (m)		Length (m)	Nature of Sampling	SPT : No. of blows				Description
	From	To			00-15 cm	15-30 cm	30-45 cm	N' Value	
0									Dark grey, clayey silt.
	0.50	-		D					1.20m
	1.50	1.95	0.45	U	Not received				
	2.00	2.45	0.45	P	2	3	4	7	
	3.00	3.45	0.45	P	2	3	3	6	
	4.00	-		D					Loose to medium dense, silver grey, silty fine to medium sand with mica .
	4.50	4.95	0.45	P	3	5	6	11	
	6.00	6.45	0.45	P	5	7	11	18	
	7.00	-		D					
	7.50	7.95	0.45	P	6	8	12	20	
	9.00	9.45	0.45	P	9	13	16	29	
	10.00	-		D					10.20m
	10.50	10.95	0.45	P	10	14	19	33	
	12.00	12.45	0.45	P	12	16	23	39	
	13.00	-		D					
	13.50	13.95	0.45	P	9	13	20	33	

	15.00	15.45	0.45	P	10	17	22	39	
	16.00	-		D					
	16.50	16.95	0.45	P	11	19	24	43	
	18.00	18.45	0.45	P	13	23	30	53	
	19.00	-		D					
	19.50	19.95	0.45	P	10	20	30	50	
	21.00	21.45	0.45	P	12	19	29	48	
	22.00	-		D					
	22.50	22.95	0.45	P	10	17	24	41	Dense to very dense, silver grey, silty fine to medium /coarse sand with mica & gravels.
	24.00	24.45	0.45	P	13	20	29	49	
	25.00	-		D					
	25.50	25.95	0.45	P	11	18	26	44	
	27.00	27.45	0.45	P	14	22	30	52	
	28.00	-		D					
	28.50	28.95	0.45	P	12	24	34	58	
	30.00	30.45	0.45	P	15	26	39	65	
	31.00	-		D					
	31.50	31.95	0.45	P	18	30	43	73	
							.		
	33.00	33.45	0.45	P	20	35	47	82	
	34.00	-		D					
	35.00	35.45	0.45	P	26	40	52	92	
	36.50	36.95	0.45	P	22	32	46	78	
	37.00	-		D					
	38.00	38.45	0.45	P	20	36	49	85	
00-01	38.45	Termination Depth							

Abbreviations: U-Undisturbed Sample D-Disturbed Sample P-Standard Penetration Test

ANNEXURE-2

LABORATORY TEST RESULTS
(SOIL SAMPLE)

BH 1(A1)

BH. No.	Type	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Natural Moisture Content (%)	Bulk Density (gm/cc)	Dry Density (gm/cc)	Liquid Limit(%)	Plastic Limit (%)	Plasticity Index (%)	IS Classification	Swell pressure (kg/cm^2)	Free Swelling Index (%)	Type of Test	Cohesion (kg/cm^2)	Angle of Friction (degree)	E Value (kg/cm^2)	Sp.Gravity	Pressure Range (kg/cm^2)	m_v (cm^2/kg)
BH - 01	D	0.50	-	14.56	48.3	37.2				44.25	20.31	23.95	CI		12.30					2.71		
	P	2.00	-	54.70	45.30					Non-Plastic			SM		NIL							
	P	4.50	-	62.30	37.70					Non-Plastic			SM							2.63		
	P	7.50	-	68.20	31.80					Non-Plastic			SM									
	P	12.00	-	74.10	25.90					Non-Plastic			SM							2.64		
	D	16.50	-	84.50	15.50					Non-Plastic			SM									
	P	21.00	3.20	89.50	7.30					Non-Plastic			SP							2.66		
	P	27.00	6.50	85.10	8.40					Non-Plastic			SP		NIL							
	P	31.50	4.67	90.10	5.23					Non-Plastic			SP							2.66		
	P	35.00	12.30	86.50	1.20					Non-Plastic			SP									

D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompresive undrained test DS= Direct shear test

BH 2(P2)

BH. No.	Type	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Natural Moisture Content (%)	Bulk Density (gm/cc)	Dry Density (gm/cc)	Liquid Limit(%)	Plastic Limit (%)	Plasticity Index (%)	IS Classification	Swell pressure (kg/cm^2)	Free Swelling Index (%)	Type of Test	Cohesion (kg/cm^2)	Angle of Friction (degree)	E Value (kg/cm^2)	Sp.Gravity	Pressure Range (kg/cm^2)	m_v (cm^2/kg)
BH - 02	P	2.00	-	58.20	41.80					Non-Plastic			SM							2.62		
	P	6.00	-	65.90	34.10					Non-Plastic			SM									
	P	10.50	-	72.10	27.90					Non-Plastic			SM							2.63		
	P	16.50	-	76.90	23.10					Non-Plastic			SM									
	D	22.00	3.50	84.60	11.90					Non-Plastic			SM							2.64		
	P	27.00	7.00	86.90	6.10					Non-Plastic			SP									
	P	31.00	4.90	91.20	3.90					Non-Plastic			SP							2.66		
	D	34.00	10.50	86.20	3.30					Non-Plastic			SP									
	P	38.00	14.10	82.90	3.00					Non-Plastic			SP							2.67		

D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompresive undrained test DS= Direct shear test

ANNEXURE-3

CHEMICAL TEST RESULTS

CHEMICAL TESTS

Chemical tests were performed on soil and water samples for determining the pH value, Sulphate, Chloride content etc. The results are given in a tabular form below:

CHEMICAL TEST RESULTS ON SOIL SAMPLES:-

BH No.	Depth (m)	pH value	Sulphate as SO_3 (%)	Chloride as Cl (%)
BH-1	3.00	7.15	0.008	0.041
BH-2	4.50	7.02	0.010	0.065

CHEMICAL TEST RESULTS ON WATER SAMPLE:-

BH/Sample No.	Depth (m)	pH value	Sulphate as SO_3 (mg/ltr)	Chloride as Cl (mg/ltr)
1 / WS-01	0.00	7.22	35.22	36.99
2 / WS-01	0.00	7.15	44.01	32.12
Permissible Limit		>6	400	500

Chemical test was carried out on a soil and water samples to determine the pH value, Sulphate and Chloride. It is seen that the values are within permissible limits (as per IS:456-2000) and so no special cement will be required for foundation concrete. **Either Ordinary Portland cement or Portland slag cement or Portland Pozzolana cement can be used for concreting.**