



**NATIONAL HIGHWAYS AND INFRASTRUCTURE DEVELOPMENT CORPORATION 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+7(combined): Km
113+830 – Km 145+712**



**Final
Detailed Project Report**

**Volume - II
Design Report**



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Voyants Solutions Pvt. Ltd

Corporate Office:

403, 4th Floor, Park Centra, Sector-30, NH-8, Gurugram-122001, Haryana, India

CIN- U74140HR2004PTC046918

Ph: 0124-4598200, Fax: 0124-4019051, E-mail: info@voyants.in, www.voyants.in

Regional Office:

Jindal Towers, Block-A, 4th Floor, 21/1A/3, Darga Road,
Kolkata – 700 017

Tele: 033- 40519300/ 40063240/ 300070350

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PART-I
DESIGN OF ROAD WORKS

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, land use, 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, land use, 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. There are 3 nos. of major intersections have been proposed. Besides, there are 35 nos. of minor intersections along the project road are 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 (AASHTO, 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.

SI 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	Howraghat Tinali junction (Km 85+400 of NH-29)
		31.07.18 to 01.08.18 (8:00 AM) (8:00 AM)	1 day x 24 hrs	Manja junction (Km 128+300 of NH-29)
4	Speed Delay Survey	30.07.18-31.07.18	-	project road stretch

SI No.	Type of Survey	Date of Survey	Period of Survey	Location
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.
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- 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	<ul style="list-style-type: none"> Geometric design standards Road Inventory Type of area, rural or urban including available ROW and roadside developments Suitability of location for new bridges 	<ul style="list-style-type: none"> Location of widening carriageway Improvement to sub-standard curves and steep grade sections
2	Intersections/ Junctions	<ul style="list-style-type: none"> Peak-hour traffic intensities and turning movement data 	<ul style="list-style-type: none"> Design of at-grade intersections Installation of traffic control measures
3	Pavement design strengthening of existing pavement	<ul style="list-style-type: none"> 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	<ul style="list-style-type: none"> Traffic loading in terms of cumulative standard axles for design lane Soaked laboratory CBR of soil samples from prospective borrow areas Initial design life and stage development strategy 	<ul style="list-style-type: none"> Thickness and composition of various pavement courses Design life determination
5	Road furniture and safety measures	<ul style="list-style-type: none"> Road inventory Alignment plans Locations of intersections on urban areas 	<ul style="list-style-type: none"> Identification of different types of signs on linear plans

SI No.	Project Component	Basis	Outcome
			<ul style="list-style-type: none"> Identification of locations for installation of crash barriers and pedestrian guard rails Pavement marking details
6	Roadside Drains	Results from drainage study	<ul style="list-style-type: none"> Location, type and size of roadside drains to be provided
7	Wayside Amenities	Inventory of existing wayside amenities by location, type and quality grade	<ul style="list-style-type: none"> 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

SI 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%
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

Sl No.	Attributes	Geometric Design Standards
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. 42.0 m PROW is also considered for cut section. However, 42 m PROW has been considered for section 6.

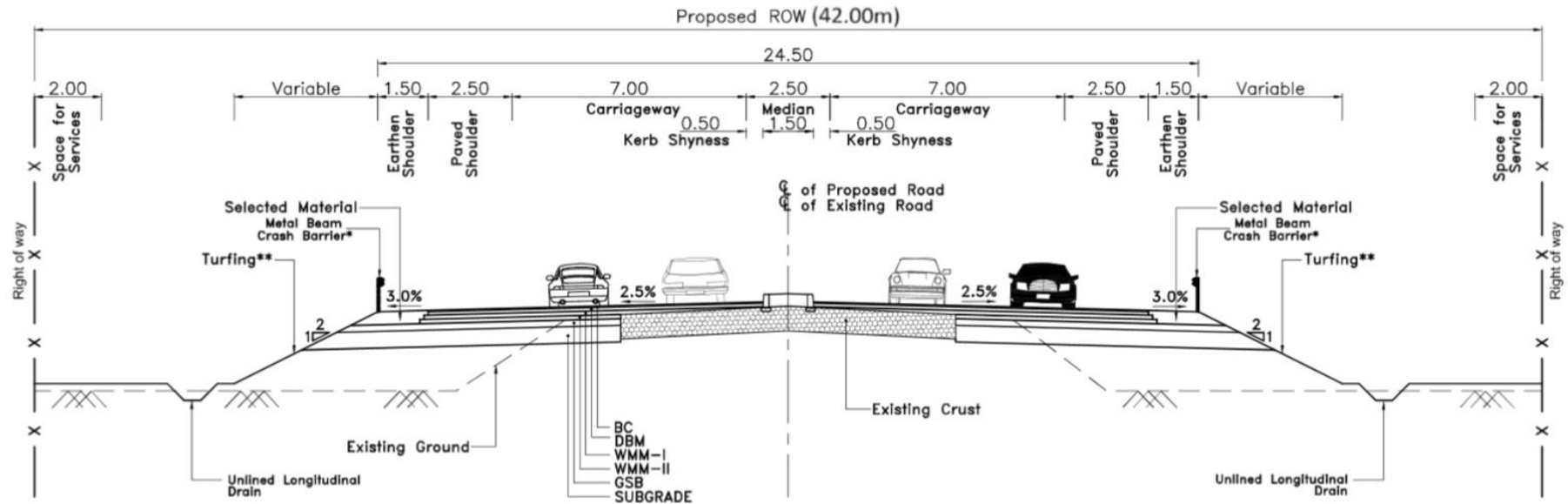
As per assessment at this stage tentative land acquisition is assessed as below:

- **For Section 6+7: 84.20 Ha**

2.6 Typical Cross Sections

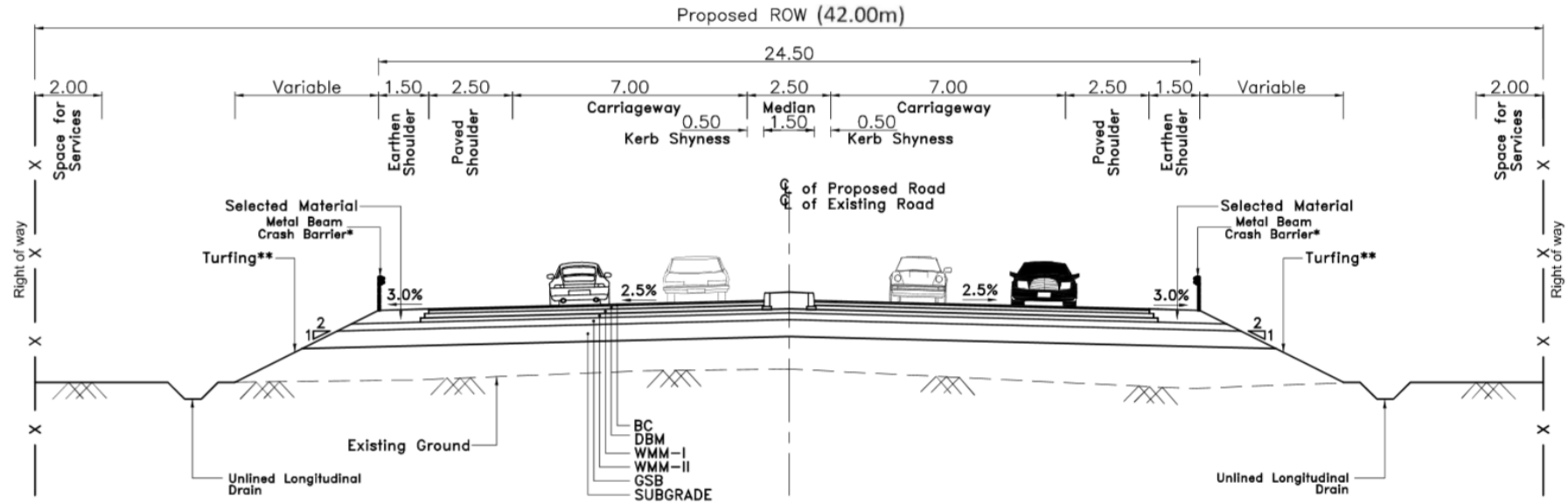
Selection of cross-section has been governed by the widening scheme adopted and by other considerations like land-use, drainage condition, traffic characteristics etc. The following typical cross sections are proposed for the widening/new construction of project highway. These depict generalized or critical/important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



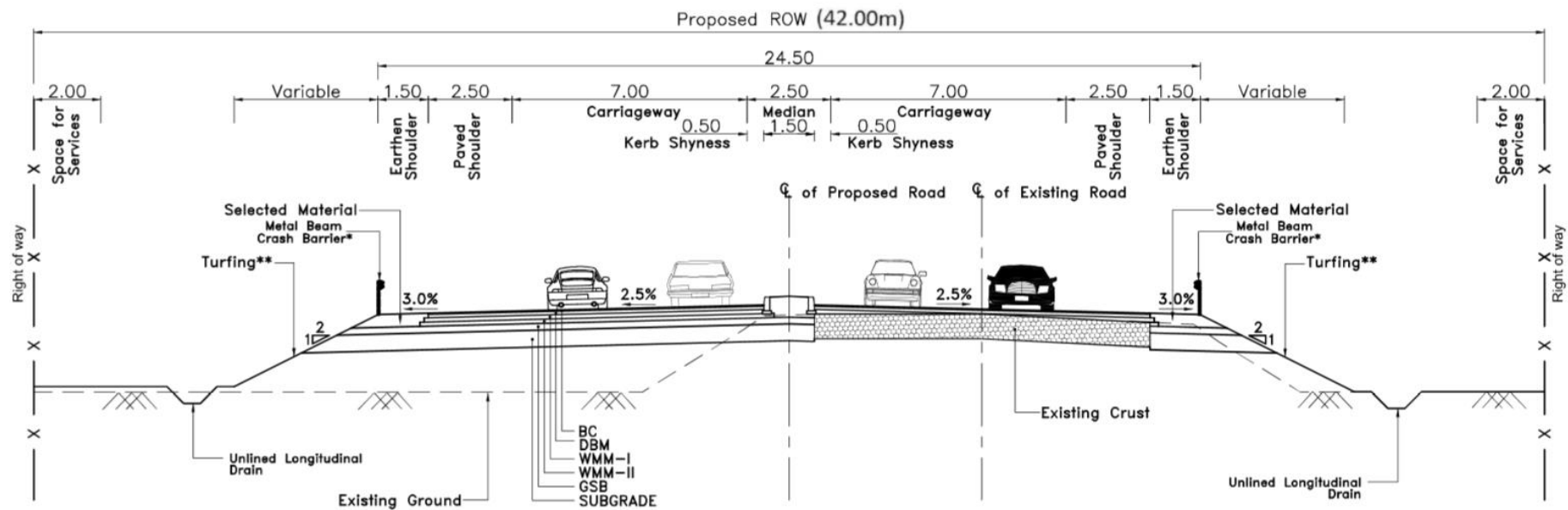
TYPE-1 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN RURAL AREA (CONCENTRIC WIDENING)

Fig 2.1



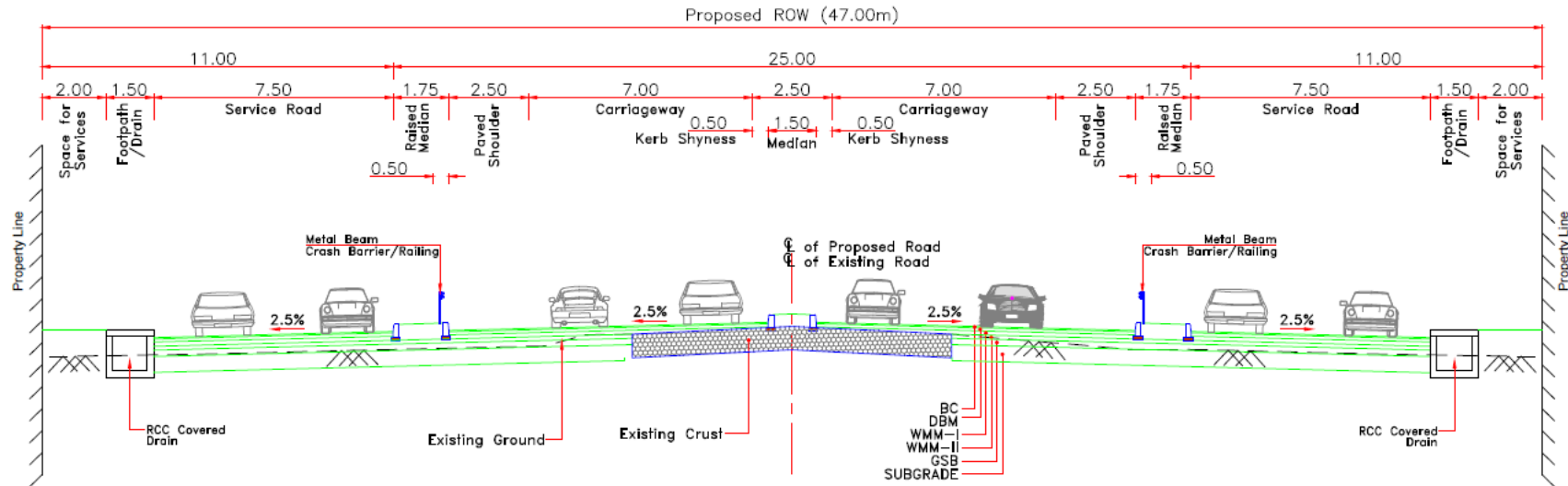
TYPE-1A : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN BYPASS/REALIGNMENT STRETCHES

Fig 2.2



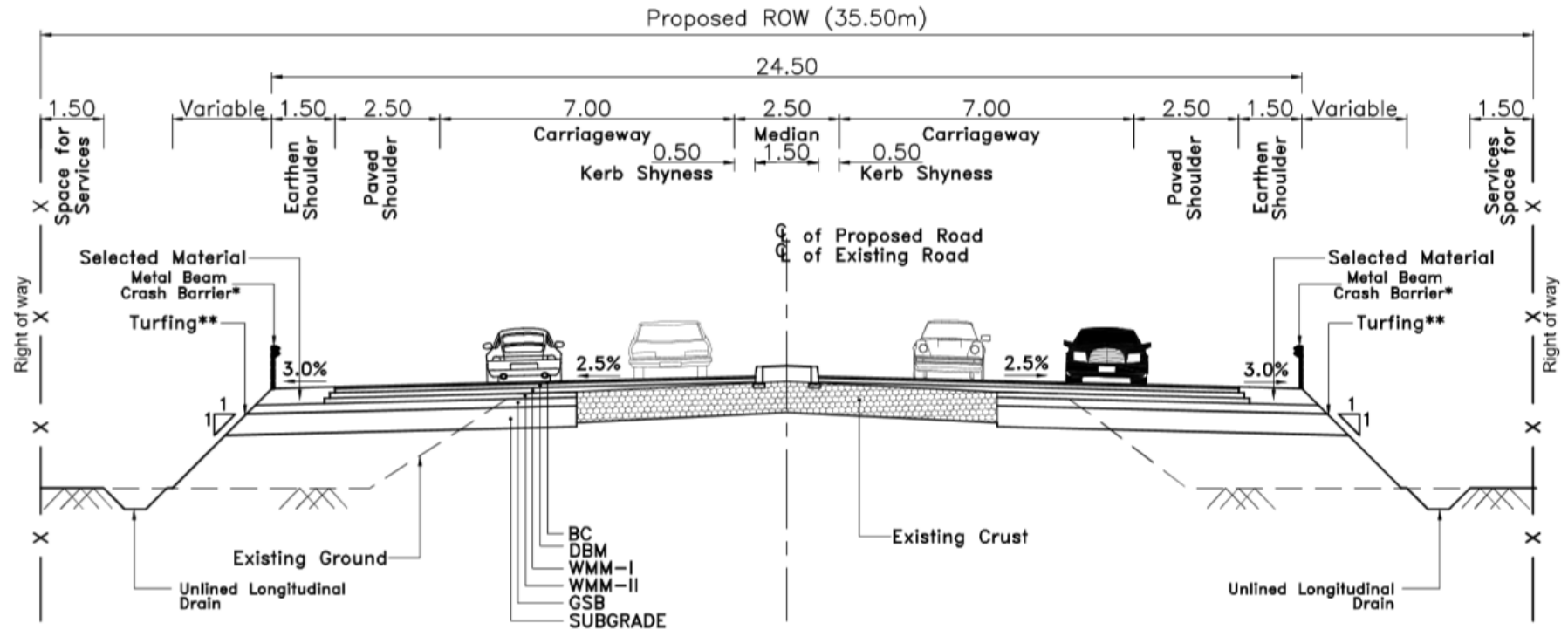
TYPE-2 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN IN RURAL AREA (ECCENTRIC WIDENING)

Fig 2.3



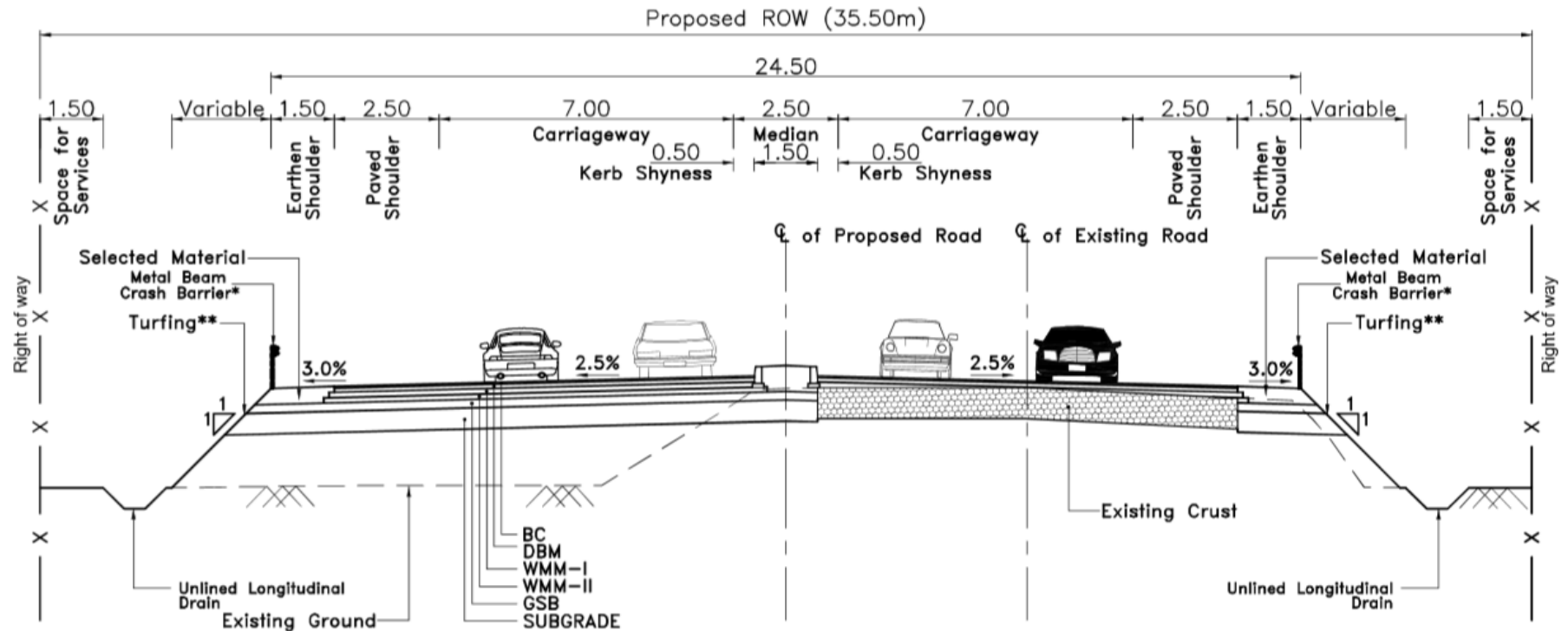
TYPE-3 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN
 AND WITH SERVICE ROAD ON BOTH SIDES IN BUILT UP AREA

Fig 2.4



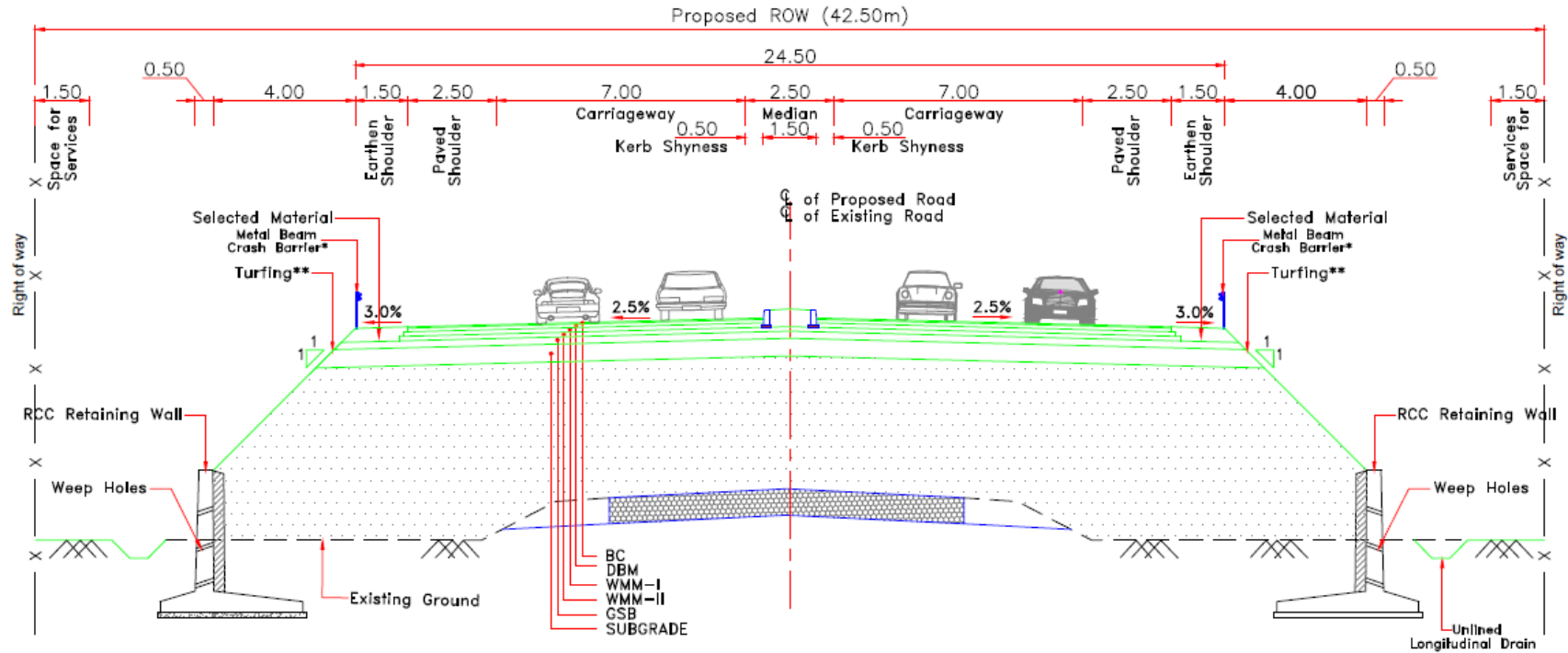
TYPE-4 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (CONCENTRIC WIDENING)

Fig 2.5



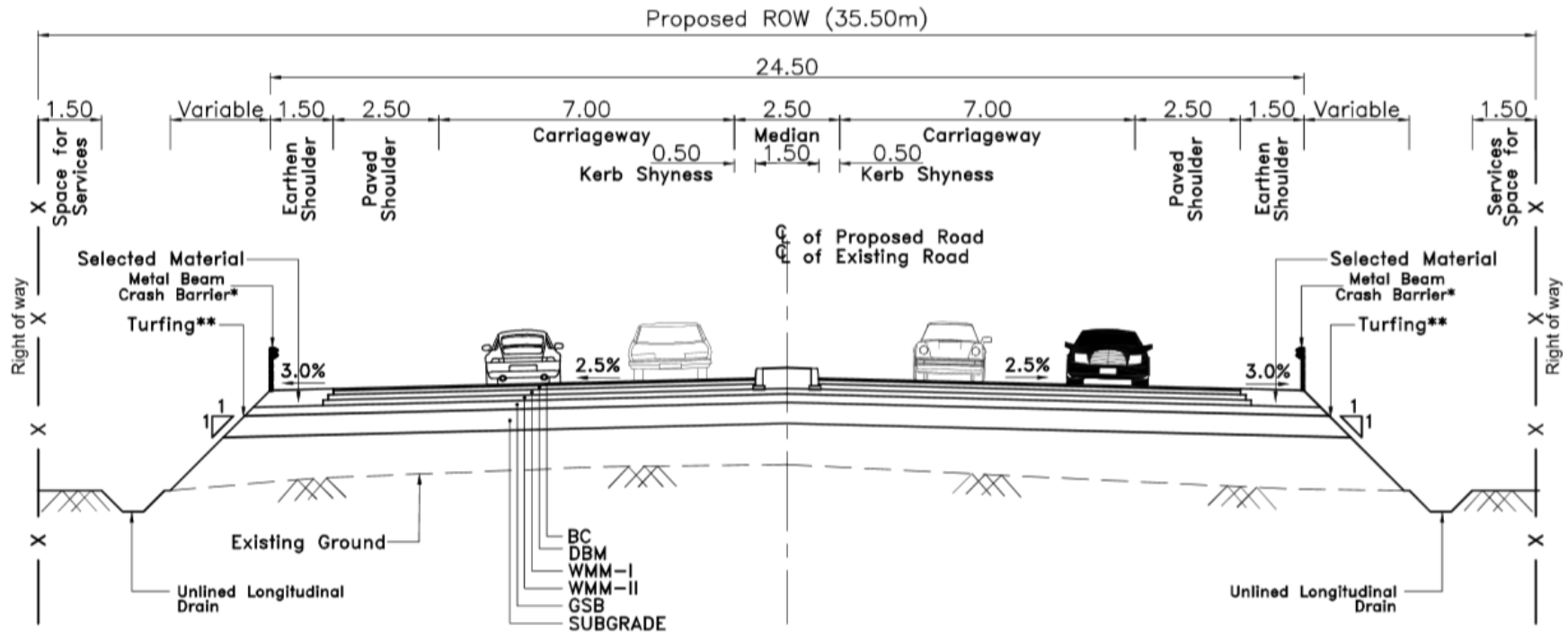
TYPE-4A : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (ECCENTRIC WIDENING)

Fig 2.6



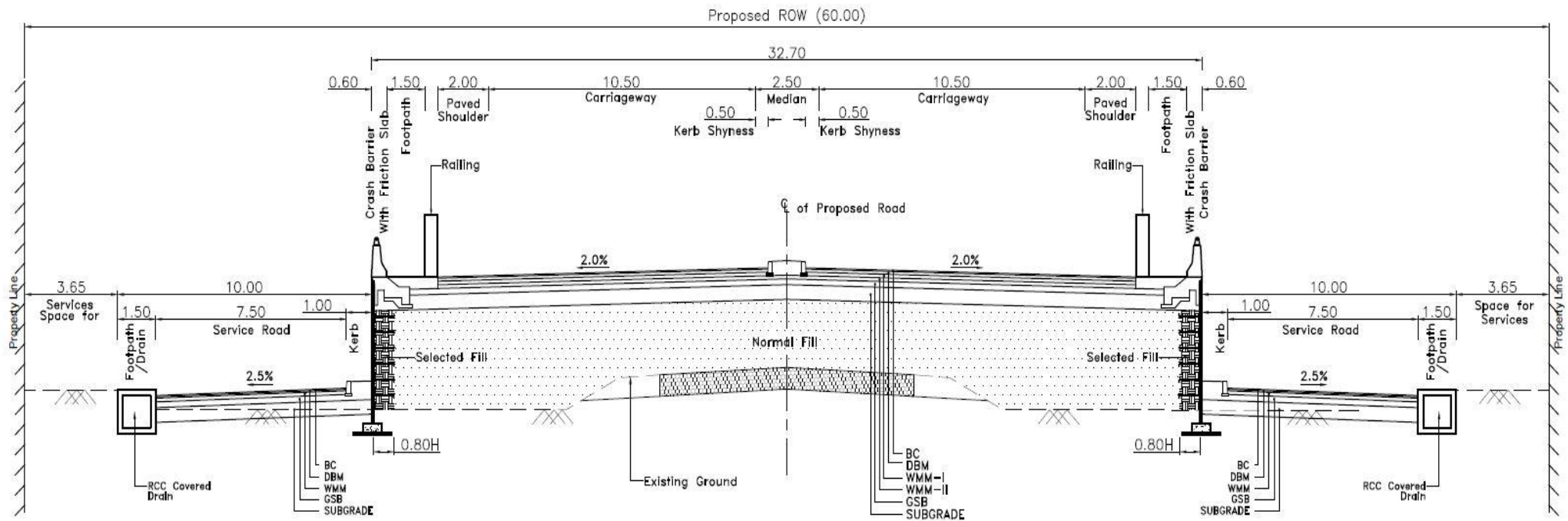
TYPE-5 : TYPICAL CROSS SECTION OF APPROACHES OF ELEPHANT UNDERPASSES (FOREST AREA)

Fig 2.7



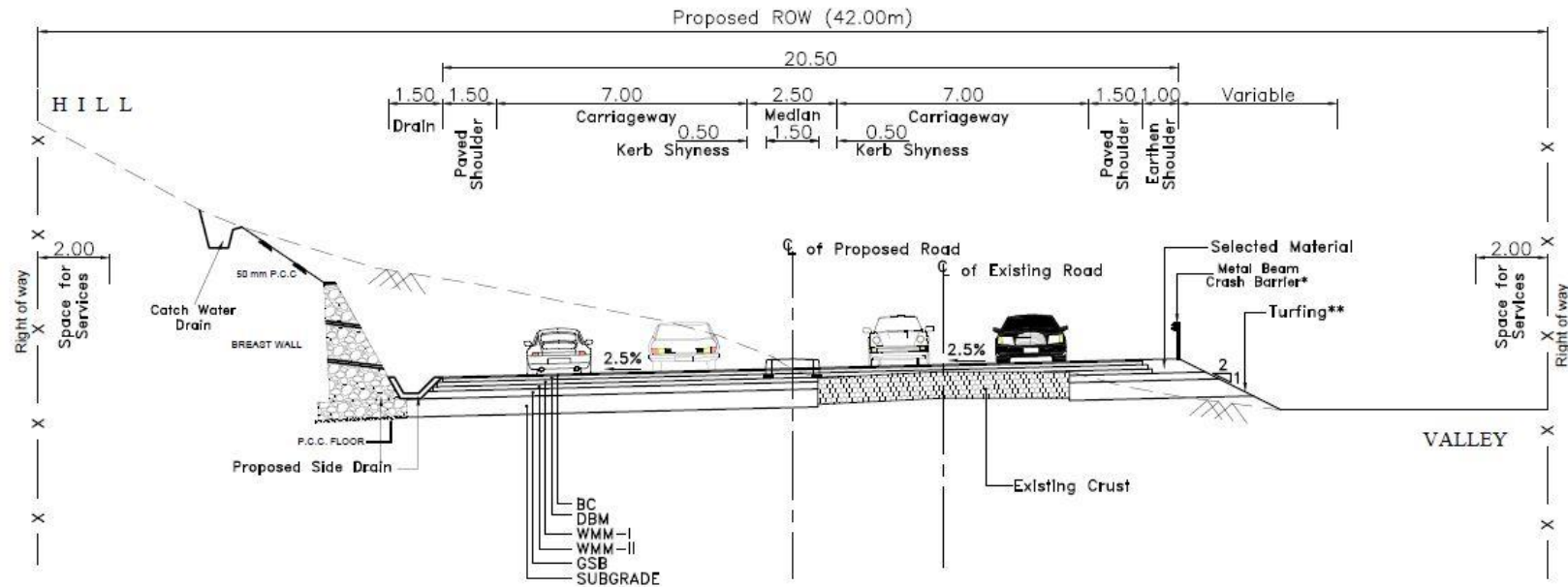
TYPE-6 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN WITH MINIMUM LAND ACQUISITION (NEW CONSTRUCTION)

Fig 2.8



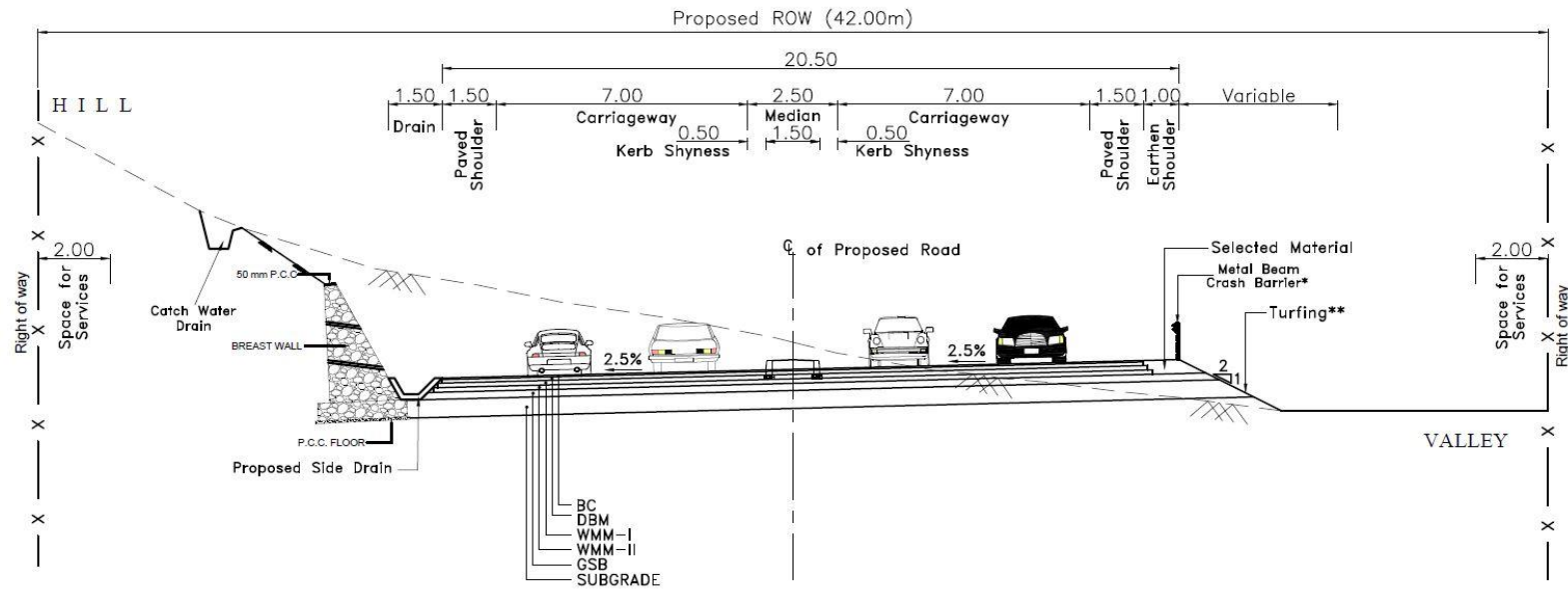
**TYPE-07 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY AT GRADE SEPARATOR
APPROACHES WITH SERVICE ROAD AND RE WALL ON BOTH SIDES**

Fig 2.9



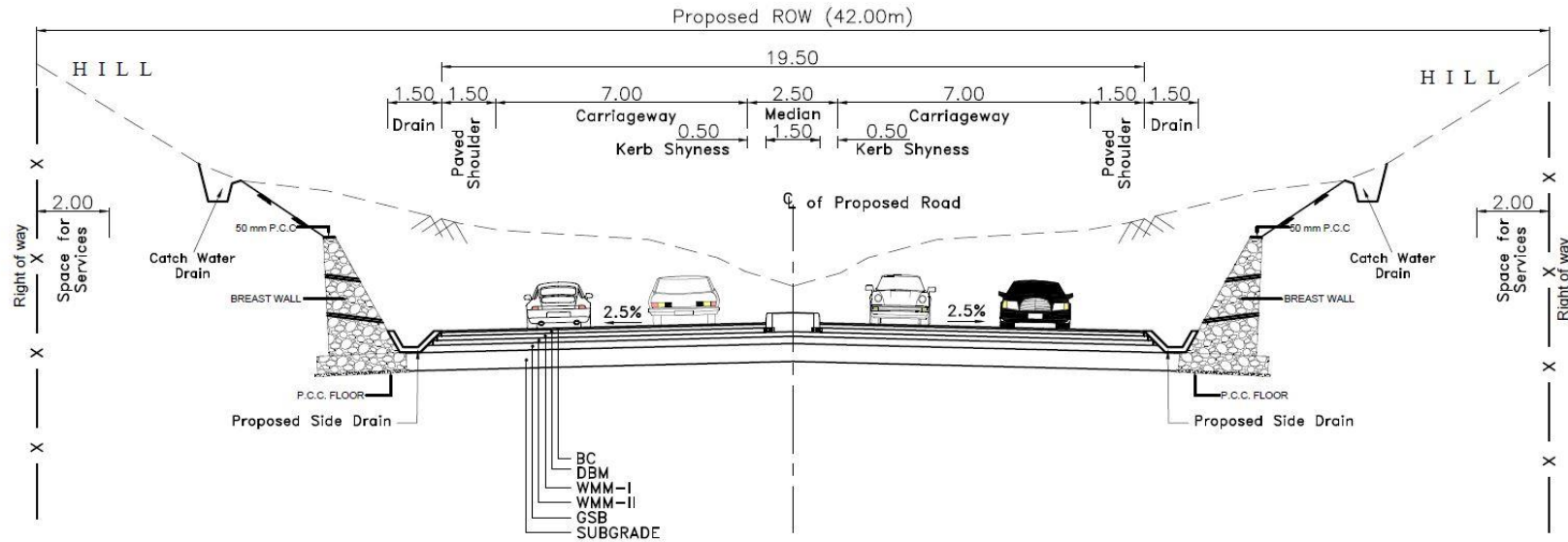
TYPE-08 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN (NEW CONSTRUCTION) IN CUT SECTION ONE SIDE(Mountainous Terrain One Side - Hill and One Side - Valley)

Fig 2.10



TYPE-08A : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN (NEW CONSTRUCTION) IN CUT SECTION ONE SIDE(Mountainous Terrain One Side - Hill and One Side - Valley)

Fig 2.11



**TYPE-09 : TYPICAL CROSS SECTION OF 4-LANE DIVIDED CARRIAGEWAY WITH 1.5M WIDE RAISED MEDIAN
(NEW CONSTRUCTION) IN CUT SECTION BOTH SIDE**

Fig 2.12

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 carriageways. 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

drain	Design Chainage (km)		Length (m)	TCS	Remarks
	From	To			
Daboka – Lahorijan Road (NH 29)					
Section 6+7					
1	114+470	114+590	120	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
2	114+700	114+890	190	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
3	114+940	115+170	230	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
4	115+850	115+980	130	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
5	116+180	116+500	320	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
6	117+120	117+260	140	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
7	117+570	117+710	140	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment

drain	Design Chainage (km)		Length (m)	TCS	Remarks
	From	To			
8	117+810	118+150	340	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
9	118+250	118+420	170	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
10	118+830	119+840	1010	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
11	120+750	121+100	350	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
12	121+330	121+470	140	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
13	123+150	123+650	500	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
14	130+840	130+990	150	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
15	132900	133200	300	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
16	134291.8	134430	138.2	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
17	134430	134550	120	8A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in Cut Section (Realignment)
18	134700	134900	200	8A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in Cut Section (Realignment)
19	136900	136950	50	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
20	136950	137220	270	9	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in Cut Section (New Construction)

drain	Design Chainage (km)		Length (m)	TCS	Remarks
	From	To			
21	137220	137500	280	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
22	139000	139150	150	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
23	139150	139200	50	8A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
24	139430	139550	120	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
25	139800	139904.3	104.3	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
26	139926.8	140100	173.2	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
27	140500	140600	100	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
28	140600	140750	150	8A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
29	141030	141200	170	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
30	143500	143616	116	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
31	143624	143800	176	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
32	144400	144530	130	8A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
33	144530	144700	170	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment

drain	Design Chainage (km)		Length (m)	TCS	Remarks
	From	To			
34	145150	145200	50	9	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
35	145200	145350	150	9	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment
36	145350	145550	200	1A	Typical Cross Section of 4-Lane Divided Carriageway with 1.5 m Wide Raised Median in bypasses and realignment

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, toll plaza etc. 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+7			
Unlined Trapezoidal Drains	Both	49052.6	TCS – 1,1A,2,4,4A,5,6
Lined Trapezoidal Drains	Hill	3502	TCS – 8,8A
Lined Trapezoidal Drains	Both	5280	TCS – 8, 8A, 9
RCC covered drain	Both	1400	Toll plaza location

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 3 nos. major intersections have been proposed. Besides, there are 35 nos. 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)	Type of Intersection	Type	Side	Improvement Proposals
1	125+500	Major	3 - legged	Right	At Grade
2	130+500	Major	3 - legged	Right	At Grade
3	144+940	Major	3 legged	Left	At Grade

Table 2.4B: List of Minor Junctions

Sl. No.	Design Chainage (km)	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	4 legged	Both	Grade Separated
18	131+480	Minor	3 legged	Right	At Grade
19	131+940	Minor	3 legged	Right	At Grade
20	132+170	Minor	3 legged	Left	At Grade
21	132+880	Minor	3 legged	Left	At Grade
22	133+370	Minor	3 legged	Left	At Grade
23	135+100	Minor	3 legged	Left	At Grade
24	135+640	Minor	3 legged	Left	At Grade
25	135+770	Minor	3 legged	Left	At Grade
26	139+650	Minor	3 legged	Left	At Grade
27	139+760	Minor	3 legged	Right	At Grade
28	140+680	Minor	3 legged	Left	At Grade
29	140+950	Minor	4 Legged	Both	At Grade
30	141+650	Minor	3 legged	Left	At Grade

Sl. No.	Design Chainage (km)	Type of Intersection	Type	Side	Improvement Proposals
31	142+120	Minor	3 legged	Right	At Grade
32	143+480	Minor	4 Legged	Both	At Grade
33	143+950	Minor	3 legged	Left	At Grade
34	144+680	Minor	3 legged	Left	At Grade
35	144+770	Minor	3 legged	Left	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. However, there is no major built up stretch exist in section -6.

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+7				
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 6 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

SI No.	Type of Underpasses	Design Chainage (km)	Span Arrangement (Nos. x Length in m)	Total Length (m)	Overall Width (m)	Structure Type
Section 6+7						
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 Protection Work for Hilly Terrain

Breast Wall

Breast wall has been proposed in hilly terrain where cut height is more than 6m. Total length of 4762m breast wall has been proposed with a height of 2.5m to 5m

2.19 Protection Work for High Embankment Zone

Toe / Retaining Wall

Toe wall has been proposed in high embankment zone such to restrict the embankment toe within the PROW. Total length of 3240m toe wall has been proposed with a height of 2m to 3m

2.20 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

2.20.1 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.

2.20.2 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.

2.20.3 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.

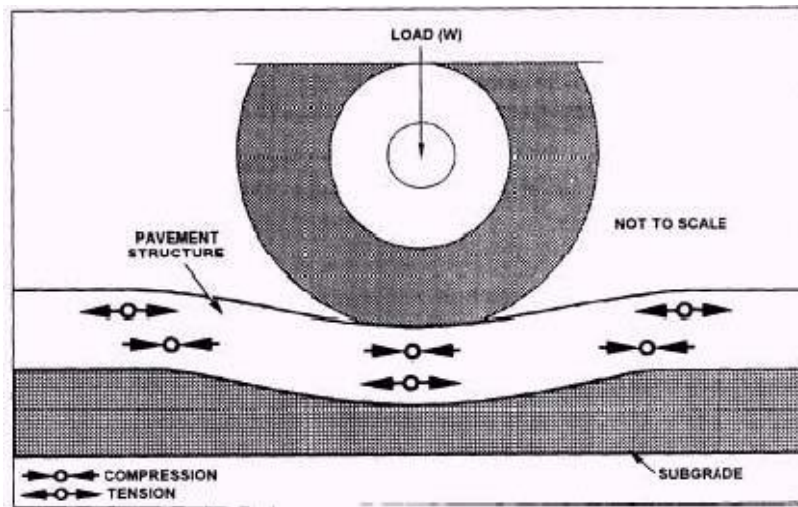


Figure 3.1 : Pavement Deflection Results in Tensile and Compressive Stresses in Pavement Structure

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:

$$N = [365 \times \{(1+r)^n - 1\} / r] \times A \times D \times F$$

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.

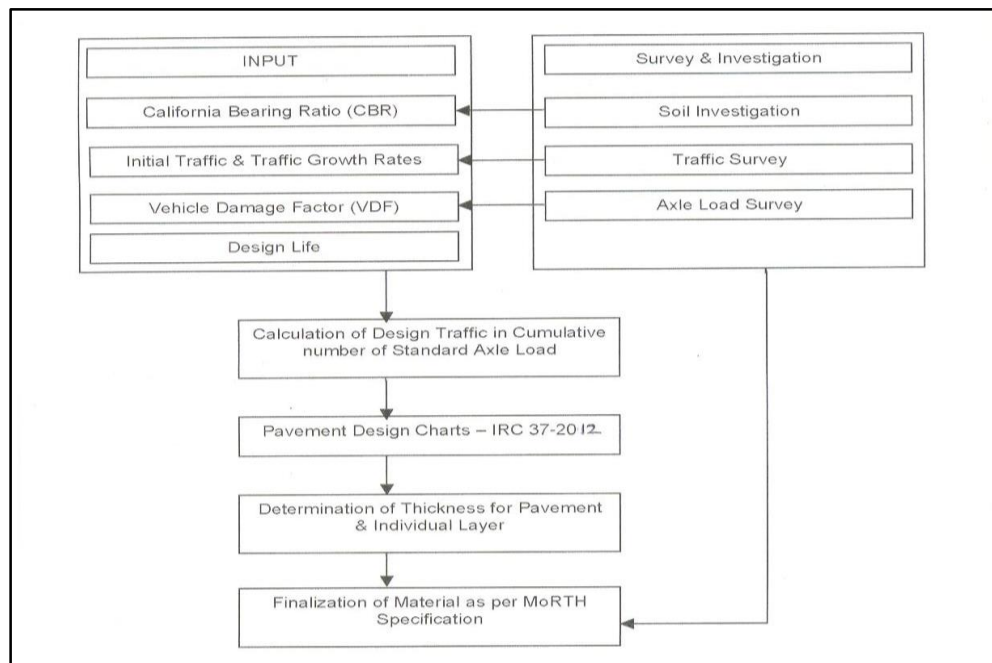


Figure 3.2 : Methodology Flow Chart for Design of New Flexible Pavement

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	Granular	Subgrade	Bituminous	Granular
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 2.2: Horizontal Alignment Report																					
Element ID	ELEMENT DETAILS								Transition Details				Horizontal Intersection Point (HIP)			Deflectin Angle			Speed (Kmph)	Superelevation	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	130	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+125.868	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+542.037	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%	-
42	131+213.808	131+220.690	546730.767	2871659.681	546736.648	2871656.108	350	Right	131+163.808	50	50	131+270.690	131217.249	546733.725	2871657.923	1	7	35.319	65	5.0%	-
43	131+334.446	131+432.339	546831.571	2871593.686	546927.247	2871583.991	150	Left	131+274.446	60	60	131+492.339	131385.207	546877.769	2871572.649	37	23	33.242	60	7.0%	0.6
44	131+776.621	131+910.146	547244.012	2871717.925	547376.016	2871717.508	255	Right	131+691.621	85	85	131+995.146	131844.952	547310.07	2871735.403	30	0	5.855	80	7.0%	0.6
45	132+183.931	132+209.360	547628.183	2871611.821	547653.299	2871608.001	170	Left	132+113.931	70	70	132+279.360	132196.67	547640.598	2871608.97	8	34				

Element ID	ELEMENT DETAILS								Transition Details				Horizontal Intersection Point (HIP)			Deflectin Angle			Speed (Kmph)	Superelevation	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			
53	133+790.786	133+861.632	549154.859	2871590.703	549207.107	2871638.46	500	Left	133+695.786	95	95	133+956.632	133826.269	549182.678	2871612.727	8	7	5.965	100	5.0%	-
54	134+190.247	134+206.712	549410.935	2871896.075	549419.592	2871910.079	300	Left	134+140.247	50	50	134+256.712	134198.481	549415.456	2871902.958	3	8	40.823	65	5.0%	0.6
55	134+330.494	134+409.334	549476.46	2872019.848	549544.605	2872054.266	90	Right	134+290.494	40	40	134+449.334	134372.645	549502.473	2872053.015	50	11	28.799	50	7.0%	0.9
56	134+525.443	134+543.940	549659.43	2872038.213	549677.921	2872037.926	250	Left	134+455.443	70	70	134+613.940	134534.696	549668.67	2872037.727	4	14	21.689	65	7.0%	0.6
57	134+695.322	134+917.144	549828.109	2872055.92	550032.985	2871985.001	300	Right	134+620.322	75	75	134+992.144	134811.579	549944.288	2872060.157	42	21	53.72	80	5.0%	0.6
58	135+077.271	135+198.870	550146.369	2871872.171	550258.888	2871831.232	200	Left	135+007.271	70	70	135+268.870	135140.015	550196.206	2871834.052	34	50	8.318	65	7.0%	0.6
59	135+406.745	135+623.724	550465.701	2871849.418	550654.119	2871755.581	255	Right	135+321.745	85	85	135+708.724	135522.292	550581.17	2871845.187	48	45	9.777	80	7.0%	0.6
60	136+207.866	136+386.196	550947.713	2871250.849	551075.598	2871128.686	400	Left	136+152.866	55	55	136+441.196	136298.538	550997.81	2871175.273	25	32	37.942	80	5.0%	-
61	136+639.875	136+709.979	551301.375	2871013.099	551367.036	2870988.702	500	Left	136+594.875	45	45	136+754.979	136674.985	551333.349	2870998.595	8	2	0.17	80	5.0%	-
62	136+831.212	136+883.853	551484.437	2870958.51	551533.733	2870940.154	400	Right	136+791.212	40	40	136+923.853	136857.57	551509.69	2870950.956	7	32	24.879	80	5.0%	-
63	137+243.055	137+300.594	551852.79	2870775.298	551898.973	2870741.061	400	Right	137+168.055	75	75	137+375.594	137271.874	551877.115	2870759.843	8	14	30.735	80	5.0%	-
64	137+462.399	137+698.064	552015.47	2870628.957	552238.697	2870604.55	220	Left	137+392.399	70	70	137+768.064	137592.962	552119.841	2870550.515	61	22	31.377	65	7.0%	0.6
65	137+987.164	138+057.957	552483.128	2870758.529	552549.647	2870782.269	300	Right	137+912.164	75	75	138+132.957	138022.726	552514.98	2870774.342	13	31	13.378	80	5.0%	0.6
66	138+335.327	138+364.337	552824.996	2870814.767	552853.478	2870820.264	600	Left	138+255.327	80	80	138+444.337	138349.834	552839.303	2870817.171	2	46	12.798	100	5.0%	-
67	138+706.213	138+766.789	553180.743	2870918.035	553228.14	2870955.242	170	Left	138+636.213	70	70	138+836.789	138736.826	553207.792	2870932.371	20	24	58.09	65	7.0%	0.6
68	138+923.442	138+935.407	553312.459	2871087.016	553319.773	2871096.483	300	Right	138+853.442	70	70	139+005.407	138929.425	553316.022	2871091.823	2	17	6.386	80	5.0%	0.6
69	139+219.883	139+279.448	553523.907	2871294.063	553578.598	2871316.884	170	Right	139+149.883	70	70	139+349.448	139249.974	553549.233	2871310.314	20	4	31.914	65	7.0%	0.6
70	139+435.598	139+477.929	553734.096	2871327.96	553774.523	2871340.247	200	Left	139+365.598	70	70	139+547.929	139456.843	553754.962	2871331.956	12	7	37.882	65	7.0%	0.6
71	139+655.138	139+696.829	553927.549	2871429.038	553967.869	2871439.221	170	Right	139+585.138	70	70	139+766.829	139676.089	553947.082	2871436.614	14	3	4.103	65	7.0%	0.6
72	139+832.372	139+857.069	554103.15	2871435.116	554127.694	2871437.718	200	Left	139+782.372	50	50	139+907.069	139844.736	554115.502	2871435.658	7	4	30.186	65	7.0%	0.6
73	140+102.104	140+106.811	554361.844	2871509.28	554366.132	2871511.222	300	Left	140+027.104	75	75	140+181.811	140104.457	554363.996	2871510.234	0	53	56.469	80	5.0%	0.6
74	140+275.860	140+329.674	554513.373	2871593.81	554565.959	2871604.136	170	Right	140+205.860	70	70	140+399.674	140302.994	554538.842	2871603.169	18	8	13.565	65	7.0%	0.6
75	140+497.032	140+511.371	554730.313	2871575.878	554743.411	2871570.055	170	Right	140+427.032	70	70	140+581.371	140504.205	554736.985	2871573.243	4	49	57.683	65	7.0%	0.6
76	140+723.130	140+808.182	554914.433	2871445.632	554992.522	2871412.651	300	Left	140+648.130	75	75	140+883.182	140765.943	554951.124	2871423.57	16	14	38.012	80	5.0%	0.6
77	140+984.543	141+011.356	555167.127	2871390.276	555193.76	2871393.195	200	Left	140+914.543	70	70	141+081.356	140997.97	555180.541	2871390.842	7	40	53.012	65	7.0%	0.6
78	141+169.483	141+388.484	555344.276	2871441.153	555559.212	2871426.342	350	Right	141+094.483	75	75	141+463.484	141282.702	555454.139	2871468.511	35	51	3.102	80	5.0%	-
79	141+902.039	142+047.328	556014.117	2871188.196	556152.492	2871145.616	500	Left	141+857.039	45	45	142+092.328	141975.199	556080.19	2871156.782	16	38	55.898	80	5.0%	-
80	142+306.503	142+329.966	556410.292	2871121.745	556433.411	2871125.634	170	Left	142+236.503	70	70	142+399.966	142318.253	556421.986	2871122.891	7	54	27.55	65	7.0%	0.6
81	142+496.737	142+603.650	556587.745	2871187.99	556692.579	2871179.714	170	Right	142+426.737	70	70	142+673.650	142552.028	556641.508	2871200.9	36	2	0.444	65	7.0%	0.6
82	142+734.094	142+755.049	556803.515	2871111.443	556822.089	2871101.753	300	Left	142+684.094	50	50	142+805.049	142744.576	556812.633	2871106.274	4	0	7.041	65	5.0%	0.6
83	143+027.738	143+144.369	557076.921	2871004.84	557190.368	2870978.362	700	Left	142+957.738	70	70	143+214.369	143086.189	557132.539	2870986.865	9	32	46.932	100	5.0%	-
84	143+353.324	143+365.369	557397.955	2870955.113	557409.62	2870952.121	200	Right	143+303.324	50	50	143+415.369	143359.349	557403.833	2870953.793	3	27	2.169	65	7.0%	0.6
85	143+506.630	143+562.190	557541.004	2870900.538	557596.064	2870895.251	170	Left	143+456.630	50	50	143+612.190	143534.66	557568.098	2870893.355	18	43	32.462	65	7.0%	0.6
86	143+807.504	143+819.426	557835.251	2870948.699	557846.494	2870952.665	450	Left	143+707.504	100	100	143+919.426	143813.466	557840.899	2870950.608	1	31	4.542	100	5.0%	-
87	144+222.472	144+300.376	558207.068	2871132.216	558265.56	2871183.34	300	Left	144+147.472	75	75	144+375.376	144261.644	558239.652	2871153.959	14	52	42.719	80	5.0%	0.6
88	144+643.372	144+748.771	558465.353	2871461.659	558560.904	2871500.765	150	Right	144+573.372	70	70	144+818.771	144698.352	558505.962	2871498.723	40	15	33.139	60	7.0%	0.6
89	144+887.865	144+986.275	558697.339	2871476.467	558756.939	2871403.148	100	Right	144+847.865	40	40	145+026.275	144941.467	558746.789	2871455.781	56	23	5.131	50	7.0%	0.9
90	145+067.674	145+107.525	558756.78	2871322.001	558744.42	2871284.279	130	Right	145+027.674	40	40	145+147.525	145087.757	558753.514	2871302.185	17	33	49.91	60	7.0%	0.6
91	145+242.848	145+412.747	558669.431	2871171.811	558686.983	2871011.809	150	Left	145+192.848	50	50	145+462.747	145338.218	558627.342	2871086.231	64	53	48.393	60	7.0%	0.6
92	145+515.693	145+548.732	558759.898	2870939.318	558777.565	2870911.478	150	Right	145+465.693	50	50	145+598.732	145532.279	558770.271	2870926.375	12	37	11.358	60	7.0%	0.6

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

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		
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
69	131+182.493	146.745	60	0.623	3.994	3.371	131+152.493	131+212.493	146.558	147.943	Sag	17.797
70	131+406.592	155.696	150	3.994	-3.972	-7.966	131+331.592	131+481.592	152.701	152.717	Hog	18.829
71	131+550.000	150	70	-3.972	0	3.972	131+515.000	131+585.000	151.39	150	Sag	17.623
72	131+673.858	150	90	0	3.272	3.272	131+628.858	131+718.858	150	151.473	Sag	27.503
73	131+879.002	156.713	100	3.272	0.4	-2.872	131+829.002	131+929.002	155.077	156.913	Hog	34.82
74	131+992.387	157.167	60	0.4	2.473	2.073	131+962.387	132+022.387	157.047	157.909	Sag	28.949
75	132+090.000	159.581	60	2.473	0.812	-1.661	132+060.000	132+120.000	158.839	159.825	Hog	36.125
76	132+165.604	160.195	60	0.812	3.2	2.388	132+135.604	132+195.604	159.951	161.155	Sag	25.126
77	132+312.224	164.887	215	3.2	-3.237	-6.437	132+204.724	132+419.724	161.447	161.407	Hog	33.4
78	132+462.361	160.027	65	-3.237	-0.711	2.526	132+429.861	132+494.861	161.079	159.796	Sag	25.732
79	132+610.608	158.973	85	-0.711	-3.297	-2.586	132+568.108	132+653.108	159.275	157.572	Hog	32.875
80	132+771.595	153.666	90	-3.297	-0.322	2.975	132+726.595	132+816.595	155.149	153.521	Sag	30.256
81	132+972.587	153.019	150	-0.322	-3.235	-2.913	132+897.587	133+047.587	153.26	150.592	Hog	51.484

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		
82	133+143.167	147.5	85	-3.235	0	3.235	133+100.667	133+185.667	148.875	147.5	Sag	26.272
83	133+270.045	147.5	60	0	-0.436	-0.436	133+240.045	133+300.045	147.5	147.369	Hog	137.738
84	133+442.677	146.748	40	-0.436	1.544	1.980	133+422.677	133+462.677	146.835	147.057	Sag	20.203
85	133+498.365	147.608	40	1.544	0.031	-1.513	133+478.365	133+518.365	147.299	147.614	Hog	26.44
86	133+617.321	147.645	130	0.031	1.297	1.266	133+552.321	133+682.321	147.625	148.488	Sag	102.749
87	133+795.288	149.953	95	1.297	0.195	-1.102	133+747.788	133+842.788	149.337	150.046	Hog	86.213
88	133+950.339	150.255	80	0.195	1.973	1.778	133+910.339	133+990.339	150.177	151.044	Sag	44.998
89	134+083.411	152.88	120	1.973	-1.645	-3.618	134+023.411	134+143.411	151.696	151.893	Hog	33.174
90	134+197.717	151	70	-1.645	0	1.645	134+162.717	134+232.717	151.576	151	Sag	42.561
91	134+377.106	151	80	0	3.301	3.301	134+337.106	134+417.106	151	152.321	Sag	24.232
92	134+720.389	162.333	400	3.301	-3.303	-6.604	134+520.389	134+920.389	155.73	155.727	Hog	60.565
93	135+109.299	149.487	170	-3.303	0.715	4.018	135+024.299	135+194.299	152.295	150.095	Sag	42.304
94	135+291.566	150.791	110	0.715	3.304	2.589	135+236.566	135+346.566	150.398	152.608	Sag	42.487
95	135+589.699	160.643	220	3.304	-2.447	-5.751	135+479.699	135+699.699	157.008	157.951	Hog	38.254
96	135+808.182	155.297	100	-2.447	-1.063	1.384	135+758.182	135+858.182	156.52	154.766	Sag	72.254
97	135+949.456	153.796	90	-1.063	-2.268	-1.205	135+904.456	135+994.456	154.274	152.775	Hog	74.674
98	136+095.391	150.486	120	-2.268	2.913	5.181	136+035.391	136+155.391	151.847	152.234	Sag	23.161
99	136+263.952	155.397	125	2.913	-3.297	-6.210	136+201.452	136+326.452	153.576	153.336	Hog	20.127
100	136+399.030	150.943	120	-3.297	3.299	6.596	136+339.030	136+459.030	152.921	152.922	Sag	18.191
101	136+542.279	155.669	110	3.299	-1.739	-5.038	136+487.279	136+597.279	153.854	154.713	Hog	21.833
102	136+650.000	153.796	60	-1.739	0.378	2.117	136+620.000	136+680.000	154.317	153.909	Sag	28.347
103	136+829.311	154.473	100	0.378	3.3	2.922	136+779.311	136+879.311	154.284	156.123	Sag	34.218
104	137+244.312	168.168	220	3.3	-3.298	-6.598	137+134.312	137+354.312	164.538	164.54	Hog	33.344
105	137+634.390	155.304	70	-3.298	0.547	3.845	137+599.390	137+669.390	156.458	155.496	Sag	18.205
106	137+734.327	155.851	60	0.547	-0.047	-0.594	137+704.327	137+764.327	155.687	155.837	Hog	100.95
107	137+857.709	155.793	60	-0.047	1.224	1.271	137+827.709	137+887.709	155.807	156.16	Sag	47.207
108	137+953.543	156.966	110	1.224	-0.35	-1.574	137+898.543	138+008.543	156.293	156.773	Hog	69.869
109	138+060.000	156.593	60	-0.35	-0.09	0.260	138+030.000	138+090.000	156.698	156.566	Sag	230.847
110	138+132.957	156.527	60	-0.09	0.62	0.710	138+102.957	138+162.957	156.554	156.713	Sag	84.437
111	138+209.393	157.001	60	0.62	0.261	-0.359	138+179.393	138+239.393	156.815	157.079	Hog	166.993
112	138+336.878	157.334	60	0.261	0.827	0.566	138+306.878	138+366.878	157.255	157.582	Sag	105.964
113	138+417.054	157.997	60	0.827	0.352	-0.475	138+387.054	138+447.054	157.749	158.102	Hog	126.374
114	138+534.113	158.409	65	0.352	1.904	1.552	138+501.613	138+566.613	158.295	159.028	Sag	41.887
115	138+625.443	160.148	80	1.904	-1.947	-3.851	138+585.443	138+665.443	159.386	159.369	Hog	20.775
116	138+720.105	158.305	60	-1.947	-0.313	1.634	138+690.105	138+750.105	158.889	158.211	Sag	36.729
117	138+932.580	157.64	100	-0.313	-0.725	-0.412	138+882.580	138+982.580	157.797	157.277	Hog	242.63
118	139+156.907	156.013	95	-0.725	0.734	1.459	139+109.407	139+204.407	156.357	156.362	Sag	65.084
119	139+269.574	156.84	75	0.734	2.192	1.458	139+232.074	139+307.074	156.565	157.662	Sag	51.467
120	139+423.913	160.223	170	2.192	-3.104	-5.296	139+338.913	139+508.913	158.36	157.585	Hog	32.104
121	139+610.707	154.426	85	-3.104	1.364	4.468	139+568.207	139+653.207	155.745	155.005	Sag	19.027
122	139+689.560	155.501	40	1.364	3.182	1.818	139+669.560	139+709.560	155.228	156.137	Sag	22.003

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		
123	139+750.000	157.424	60	3.182	1.157	-2.025	139+720.000	139+780.000	156.469	157.771	Hog	29.638
124	140+020.000	160.549	80	1.157	3.297	2.140	139+980.000	140+060.000	160.086	161.867	Sag	37.388
125	140+234.756	167.629	150	3.297	0.882	-2.415	140+159.756	140+309.756	165.156	168.29	Hog	62.107
126	140+396.249	169.053	80	0.882	-3.295	-4.177	140+356.249	140+436.249	168.7	167.735	Hog	19.154
127	140+498.252	165.692	80	-3.295	-0.498	2.797	140+458.252	140+538.252	167.01	165.493	Sag	28.601
128	140+942.531	163.48	150	-0.498	-3.26	-2.762	140+867.531	141+017.531	163.853	161.035	Hog	54.302
129	141+179.864	155.742	170	-3.26	3.027	6.287	141+094.864	141+264.864	158.514	158.315	Sag	27.04
130	141+495.403	165.293	210	3.027	-3.183	-6.210	141+390.403	141+600.403	162.115	161.951	Hog	33.815
131	141+726.983	157.921	165	-3.183	2.26	5.443	141+644.483	141+809.483	160.547	159.786	Sag	30.311
132	141+916.610	162.207	180	2.26	-2.433	-4.693	141+826.610	142+006.610	160.173	160.017	Hog	38.353
133	142+054.779	158.845	60	-2.433	0.806	3.239	142+024.779	142+084.779	159.575	159.087	Sag	18.525
134	142+136.077	159.5	60	0.806	0	-0.806	142+106.077	142+166.077	159.258	159.5	Hog	74.472
135	142+270.000	159.5	100	0	3.225	3.225	142+220.000	142+320.000	159.5	161.113	Sag	31.003
136	142+435.526	164.839	60	3.225	2.303	-0.922	142+405.526	142+465.526	163.871	165.53	Hog	65.046
137	142+758.967	172.288	85	2.303	-2.124	-4.427	142+716.467	142+801.467	171.309	171.385	Hog	19.201
138	142+847.014	170.418	70	-2.124	-0.542	1.582	142+812.014	142+882.014	171.161	170.228	Sag	44.252
139	142+940.000	169.914	75	-0.542	-1.53	-0.988	142+902.500	142+977.500	170.117	169.34	Hog	75.912
140	143+010.000	168.843	60	-1.53	-2.441	-0.911	142+980.000	143+040.000	169.302	168.111	Hog	65.877
141	143+103.822	166.553	65	-2.441	-3.302	-0.861	143+071.322	143+136.322	167.346	165.48	Hog	75.432
142	143+214.435	162.9	60	-3.302	0	3.302	143+184.435	143+244.435	163.891	162.9	Sag	18.168
143	143+298.672	162.9	60	0	-0.015	-0.015	143+268.672	143+328.672	162.9	162.895	Hog	3934.297
144	143+423.258	162.881	40	-0.015	1.115	1.130	143+403.258	143+443.258	162.884	163.104	Sag	35.399
145	143+490.000	163.625	40	1.115	-0.71	-1.825	143+470.000	143+510.000	163.402	163.483	Hog	21.923
146	143+578.051	163	60	-0.71	0	0.710	143+548.051	143+608.051	163.213	163	Sag	84.529
147	143+760.000	163	100	0	2.552	2.552	143+710.000	143+810.000	163	164.276	Sag	39.186
148	143+916.528	166.995	170	2.552	-3.04	-5.592	143+831.528	144+001.528	164.825	164.41	Hog	30.4
149	144+044.631	163.1	60	-3.04	0	3.040	144+014.631	144+074.631	164.012	163.1	Sag	19.736
150	144+135.231	163.1	60	0	3.277	3.277	144+105.231	144+165.231	163.1	164.083	Sag	18.307
151	144+396.113	171.65	120	3.277	-3.221	-6.498	144+336.113	144+456.113	169.684	169.718	Hog	18.467
152	144+660.865	163.123	120	-3.221	3.613	6.834	144+600.865	144+720.865	165.055	165.291	Sag	17.561
153	144+842.147	169.672	85	3.613	1.189	-2.424	144+799.647	144+884.647	168.137	170.178	Hog	35.078
154	144+959.595	171.069	40	1.189	3.13	1.941	144+939.595	144+979.595	170.831	171.695	Sag	20.616
155	145+200.000	178.593	150	3.13	0.808	-2.322	145+125.000	145+275.000	176.246	179.199	Hog	64.618
156	145+544.269	181.376	120	0.808	2.429	1.621	145+484.269	145+604.269	180.891	182.833	Sag	74.063
157	145+663.596	184.274	80	2.429	0.288	-2.141	145+623.596	145+703.596	183.303	184.389	Hog	37.38

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

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		
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
69	131+182.493	146.745	60	0.623	3.994	3.371	131+152.493	131+212.493	146.558	147.943	Sag	17.797
70	131+406.592	155.696	150	3.994	-3.972	-7.966	131+331.592	131+481.592	152.701	152.717	Hog	18.829
71	131+550.000	150	70	-3.972	0	3.972	131+515.000	131+585.000	151.39	150	Sag	17.623
72	131+673.858	150	90	0	3.272	3.272	131+628.858	131+718.858	150	151.473	Sag	27.503
73	131+879.002	156.713	100	3.272	0.4	-2.872	131+829.002	131+929.002	155.077	156.913	Hog	34.82
74	131+992.387	157.167	60	0.4	2.473	2.073	131+962.387	132+022.387	157.047	157.909	Sag	28.949
75	132+090.000	159.581	60	2.473	0.812	-1.661	132+060.000	132+120.000	158.839	159.825	Hog	36.125
76	132+165.604	160.195	60	0.812	3.2	2.388	132+135.604	132+195.604	159.951	161.155	Sag	25.126
77	132+312.224	164.887	215	3.2	-3.237	-6.437	132+204.724	132+419.724	161.447	161.407	Hog	33.4
78	132+462.361	160.027	65	-3.237	-0.711	2.526	132+429.861	132+494.861	161.079	159.796	Sag	25.732
79	132+610.608	158.973	85	-0.711	-3.297	-2.586	132+568.108	132+653.108	159.275	157.572	Hog	32.873
80	132+837.232	151.502	80	-3.297	1.121	4.418	132+797.232	132+877.232	152.821	151.95	Sag	18.11
81	132+972.587	153.019	150	1.121	-3.235	-4.356	132+897.587	133+047.587	152.178	150.592	Hog	34.434

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		
82	133+143.167	147.5	85	-3.235	0	3.235	133+100.667	133+185.667	148.875	147.5	Sag	26.272
83	133+270.045	147.5	60	0	-0.978	-0.978	133+240.045	133+300.045	147.5	147.207	Hog	61.351
84	133+333.646	146.878	60	-0.978	-0.119	0.859	133+303.646	133+363.646	147.171	146.842	Sag	69.87
85	133+442.677	146.748	40	-0.119	1.544	1.663	133+422.677	133+462.677	146.772	147.057	Sag	24.045
86	133+498.365	147.608	40	1.544	0.031	-1.513	133+478.365	133+518.365	147.299	147.614	Hog	26.44
87	133+641.467	147.653	130	0.031	1.437	1.406	133+576.467	133+706.467	147.633	148.587	Sag	92.464
88	133+802.453	149.967	95	1.437	0.195	-1.242	133+754.953	133+849.953	149.284	150.06	Hog	76.45
89	133+950.339	150.255	80	0.195	1.973	1.778	133+910.339	133+990.339	150.177	151.044	Sag	44.998
90	134+083.411	152.88	120	1.973	-1.645	-3.618	134+023.411	134+143.411	151.696	151.893	Hog	33.174
91	134+197.717	151	70	-1.645	0	1.645	134+162.717	134+232.717	151.576	151	Sag	42.561
92	134+377.106	151	80	0	3.301	3.301	134+337.106	134+417.106	151	152.321	Sag	24.232
93	134+720.389	162.333	400	3.301	-3.303	-6.604	134+520.389	134+920.389	155.73	155.727	Hog	60.565
94	135+109.299	149.487	170	-3.303	0.715	4.018	135+024.299	135+194.299	152.295	150.095	Sag	42.304
95	135+291.566	150.791	110	0.715	3.304	2.589	135+236.566	135+346.566	150.398	152.608	Sag	42.487
96	135+595.641	160.839	220	3.304	-3.09	-6.394	135+485.641	135+705.641	157.204	157.44	Hog	34.405
97	135+785.704	154.966	100	-3.09	-0.714	2.376	135+735.704	135+835.704	156.511	154.609	Sag	42.096
98	135+949.456	153.796	90	-0.714	-3.282	-2.568	135+904.456	135+994.456	154.118	152.319	Hog	35.054
99	136+082.395	149.433	120	-3.282	3.285	6.567	136+022.395	136+142.395	151.402	151.404	Sag	18.274
100	136+263.952	155.397	125	3.285	-3.297	-6.582	136+201.452	136+326.452	153.344	153.336	Hog	18.99
101	136+399.030	150.943	120	-3.297	3.299	6.596	136+339.030	136+459.030	152.921	152.922	Sag	18.191
102	136+542.279	155.669	110	3.299	-2.222	-5.521	136+487.279	136+597.279	153.854	154.447	Hog	19.925
103	136+630.000	153.72	60	-2.222	0.064	2.286	136+600.000	136+660.000	154.387	153.739	Sag	26.25
104	136+810.000	153.836	100	0.064	3.3	3.236	136+760.000	136+860.000	153.804	155.486	Sag	30.903
105	137+244.312	168.168	220	3.3	-3.298	-6.598	137+134.312	137+354.312	164.538	164.54	Hog	33.344
106	137+634.390	155.304	70	-3.298	0.547	3.845	137+599.390	137+669.390	156.458	155.496	Sag	18.205
107	137+734.327	155.851	60	0.547	-0.047	-0.594	137+704.327	137+764.327	155.687	155.837	Hog	100.95
108	137+857.709	155.793	60	-0.047	1.224	1.271	137+827.709	137+887.709	155.807	156.16	Sag	47.207
109	137+953.543	156.966	110	1.224	-0.35	-1.574	137+898.543	138+008.543	156.293	156.773	Hog	69.869
110	138+075.352	156.539	60	-0.35	0.424	0.774	138+045.352	138+105.352	156.644	156.666	Sag	77.528
111	138+539.061	158.503	65	0.424	1.904	1.480	138+506.561	138+571.561	158.366	159.122	Sag	43.901
112	138+610.000	159.854	60	1.904	-0.962	-2.866	138+580.000	138+640.000	159.283	159.565	Hog	20.931
113	138+671.102	159.266	60	-0.962	-1.958	-0.996	138+641.102	138+701.102	159.555	158.679	Hog	60.293
114	138+734.070	158.033	60	-1.958	-0.411	1.547	138+704.070	138+764.070	158.621	157.91	Sag	38.8
115	138+820.000	157.68	60	-0.411	-0.039	0.372	138+808.096	138+831.904	157.729	157.675	Sag	63.991
116	138+932.969	157.636	100	-0.039	-1.017	-0.978	138+882.969	138+982.969	157.656	157.128	Hog	102.281
117	139+034.863	156.6	60	-1.017	-0.481	0.536	139+004.863	139+064.863	156.905	156.456	Sag	111.989
118	139+156.907	156.013	95	-0.481	0.527	1.008	139+109.407	139+204.407	156.241	156.263	Sag	94.271
119	139+267.395	156.595	75	0.527	2.318	1.791	139+229.895	139+304.895	156.397	157.464	Sag	41.872
120	139+423.913	160.223	170	2.318	-3.301	-5.619	139+338.913	139+508.913	158.253	157.417	Hog	30.256
121	139+602.811	154.318	85	-3.301	1.364	4.665	139+560.311	139+645.311	155.721	154.898	Sag	18.223
122	139+689.560	155.501	40	1.364	3.182	1.818	139+669.560	139+709.560	155.228	156.137	Sag	22.003

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		
123	139+750.000	157.424	60	3.182	1.157	-2.025	139+720.000	139+780.000	156.469	157.771	Hog	29.638
124	140+020.000	160.549	80	1.157	3.297	2.140	139+980.000	140+060.000	160.086	161.867	Sag	37.388
125	140+234.756	167.629	150	3.297	0.882	-2.415	140+159.756	140+309.756	165.156	168.29	Hog	62.107
126	140+396.249	169.053	80	0.882	-3.295	-4.177	140+356.249	140+436.249	168.7	167.735	Hog	19.154
127	140+498.252	165.692	80	-3.295	-0.498	2.797	140+458.252	140+538.252	167.01	165.493	Sag	28.601
128	140+942.531	163.48	150	-0.498	-3.26	-2.762	140+867.531	141+017.531	163.853	161.035	Hog	54.302
129	141+179.864	155.742	170	-3.26	3.027	6.287	141+094.864	141+264.864	158.514	158.315	Sag	27.04
130	141+495.403	165.293	210	3.027	-3.183	-6.210	141+390.403	141+600.403	162.115	161.951	Hog	33.815
131	141+726.983	157.921	165	-3.183	2.26	5.443	141+644.483	141+809.483	160.547	159.786	Sag	30.311
132	141+916.610	162.207	180	2.26	-2.433	-4.693	141+826.610	142+006.610	160.173	160.017	Hog	38.353
133	142+054.779	158.845	60	-2.433	0.806	3.239	142+024.779	142+084.779	159.575	159.087	Sag	18.525
134	142+136.077	159.5	60	0.806	0	-0.806	142+106.077	142+166.077	159.258	159.5	Hog	74.472
135	142+270.000	159.5	100	0	3.225	3.225	142+220.000	142+320.000	159.5	161.113	Sag	31.003
136	142+435.526	164.839	60	3.225	2.303	-0.922	142+405.526	142+465.526	163.871	165.53	Hog	65.046
137	142+754.613	172.188	85	2.303	-1.272	-3.575	142+712.113	142+797.113	171.209	171.647	Hog	23.774
138	143+041.309	168.54	90	-1.272	-3.258	-1.986	142+996.309	143+086.309	169.113	167.074	Hog	45.331
139	143+214.435	162.9	60	-3.258	0	3.258	143+184.435	143+244.435	163.877	162.9	Sag	18.418
140	143+298.672	162.9	60	0	-0.015	-0.015	143+268.672	143+328.672	162.9	162.895	Hog	3934.297
141	143+423.258	162.881	40	-0.015	1.115	1.130	143+403.258	143+443.258	162.884	163.104	Sag	35.399
142	143+490.000	163.625	40	1.115	-0.71	-1.825	143+470.000	143+510.000	163.402	163.483	Hog	21.923
143	143+578.051	163	60	-0.71	0	0.710	143+548.051	143+608.051	163.213	163	Sag	84.529
144	143+760.000	163	100	0	2.552	2.552	143+710.000	143+810.000	163	164.276	Sag	39.186
145	143+916.528	166.995	170	2.552	-3.04	-5.592	143+831.528	144+001.528	164.825	164.41	Hog	30.4
146	144+044.631	163.1	60	-3.04	0	3.040	144+014.631	144+074.631	164.012	163.1	Sag	19.736
147	144+135.231	163.1	60	0	3.277	3.277	144+105.231	144+165.231	163.1	164.083	Sag	18.307
148	144+396.113	171.65	120	3.277	-3.221	-6.498	144+336.113	144+456.113	169.684	169.718	Hog	18.467
149	144+660.865	163.123	120	-3.221	3.613	6.834	144+600.865	144+720.865	165.055	165.291	Sag	17.561
150	144+842.147	169.672	85	3.613	1.189	-2.424	144+799.647	144+884.647	168.137	170.178	Hog	35.078
151	144+959.595	171.069	40	1.189	3.13	1.941	144+939.595	144+979.595	170.831	171.695	Sag	20.616
152	145+200.000	178.593	150	3.13	0.808	-2.322	145+125.000	145+275.000	176.246	179.199	Hog	64.618
153	145+544.269	181.376	120	0.808	2.429	1.621	145+484.269	145+604.269	180.891	182.833	Sag	74.063
154	145+663.596	184.274	80	2.429	0.288	-2.141	145+623.596	145+703.596	183.303	184.389	Hog	37.38

Annexure 2.4 : Widening Scheme (Section 6+7 Combined)

Design Chainage		Length (m)	TCS Type	Description
From	To			
113830	114020	190	2	Eccentric Widening - Right side
114020	114200	180	2	Eccentric Widening - Left side
114200	114420	220	9	New Construction - Both side Cutting
114420	114470	50	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	135	2	Eccentric Widening - Left side
116634.7	116645.3	11	STR	MNB
116645.3	117050	405	2	Eccentric Widening - Left side
117050	117120	70	2	Eccentric Widening - Right side
117120	117163	43	1A	New Construction - Realignment
117163	117178	15	1A	New Construction - Realignment
117178	117260	82	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	118000	190	9	New Construction - Realignment (Both Side Cutting)
118000	118150	150	8A	New Construction - Realignment (Left Side Hill)
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	118665.5	55	2	Eccentric Widening - Right side
118665.5	118680.6	15	STR	MNB
118680.6	118710	29	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

Annexure 2.4 : Widening Scheme (Section 6+7 Combined)

Design Chainage		Length (m)	TCS Type	Description
From	To			
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	2	Eccentric Widening - Right side
121191.1	121208.9	18	STR	MNB
121208.9	121330	121	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	125560	220	1A	New Construction - Bypass
125560	125880	320	9	New Construction - Bypass (Both Side Cutting)
125880	126000	120	1A	New Construction - Bypass
126000	126300	300	9	New Construction - Bypass (Both Side Cutting)
126300	126866.3	566	1A	New Construction - Bypass
126866.3	126873.8	8	STR	MNB
126873.8	128292.5	1419	1A	New Construction - Bypass
128292.5	128307.5	15	STR	MNB
128307.5	128704	397	1A	New Construction - Bypass
128704	128716	12	STR	MNB
128716	128800	84	1A	New Construction - Bypass
128800	128932	132	1A	New Construction - Bypass
128932	129022	90	STR	MJB
129022	129100	78	1A	New Construction - Bypass
129100	129130	30	1A	New Construction - Bypass
129130	129370	240	9	New Construction - Bypass (Both Side Cutting)
129370	129480	110	1A	New Construction - Bypass
129480	129740	260	9	New Construction - Bypass (Both Side Cutting)
129740	129870	130	1A	New Construction - Bypass
129870	130000	130	8A	New Construction - Bypass (Left Side Hill)
130000	130300	300	9	New Construction - Bypass (Both Side Cutting)
130300	130400	100	1A	New Construction - Bypass
130400	130600	200	9	New Construction - Bypass (Both Side Cutting)
130600	130630	30	1A	New Construction - Bypass
130630	130840	210	2	Eccentric Widening - Left side

Annexure 2.4 : Widening Scheme (Section 6+7 Combined)

Design Chainage		Length (m)	TCS Type	Description
From	To			
130840	130990	150	1A	New Construction - Realignment
130990	131152	162	2	Eccentric Widening - Left side
131+152	131+350	198	2	Eccentric Widening - Left side
131+350	131+470	120	8	Eccentric Widening - Left Hill
131+470	131+592	122	8	Eccentric Widening - Right Hill
131+592	131+607	15		MNB
131+607	131+720	113	2	Eccentric Widening - Right side
131+720	131+800	80	8	Eccentric Widening - Left Hill
131+800	131+920	120	8	Eccentric Widening - Right Hill
131+920	132+030	110	2	Eccentric Widening - Left side
132+030	132+110	80	8	Eccentric Widening - Right Hill
132+110	132+250	140	2	Eccentric Widening - Left side
132+250	132+420	170	8	Eccentric Widening - Right Hill
132+420	132+500	80	2	Eccentric Widening - Left side
132+500	132+600	100	8	Eccentric Widening - Left Hill
132+600	132+900	300	2	Eccentric Widening - Left side
132+900	133+200	300	1A	New Construction - Realignment
133+200	133+208	8		MNB
133+208	133+720	512	2	Eccentric Widening - Left side
133+720	134+200	480	8	Eccentric Widening - Right Hill
134+200	134+268	68	2	Eccentric Widening - Right side
134+268	134+292	24		MNB
134+292	134+430	138	1A	New Construction - Realignment
134+430	134+550	120	8A	New Construction - Realignment (Left Side Hill)
134+550	134+700	150	8	Eccentric Widening - Left Hill
134+700	134+900	200	8A	New Construction - Realignment (Left Side Hill)
134+900	134+950	50	8	Eccentric Widening - Right Hill
134+950	135+200	250	2	Eccentric Widening - Right side
135+200	135+300	100	8	Eccentric Widening - Right Hill
135+300	135+400	100	2	Eccentric Widening - Right side
135+400	135+500	100	8	Eccentric Widening - Right Hill
135+500	135+600	100	2	Eccentric Widening - Left side
135+600	135+800	200	8	Eccentric Widening - Right Hill
135+800	136+000	200	1A	New Construction - Realignment
136+000	136+100	100	2	Eccentric Widening - Left side
136+100	136+200	100	1A	New Construction - Realignment
136+200	136+300	100	8	Eccentric Widening - Left Hill
136+300	136+400	100	8	Eccentric Widening - Left Hill
136+400	136+600	200	8	Eccentric Widening - Right Hill
136+600	136+900	300	2	Eccentric Widening - Left side
136+900	136+950	50	1A	New Construction - Realignment
136+950	137+220	270	9	New Construction - Both side Cutting
137+220	137+500	280	1A	New Construction - Realignment
137+500	137+900	400	2	Eccentric Widening - Left side
137+900	138+500	600	2	Eccentric Widening - Right side
138+500	138+600	100	2	Eccentric Widening - Left side
138+600	138+800	200	8	Eccentric Widening - Left Hill
138+800	139+000	200	2	Eccentric Widening - Left side
139+000	139+150	150	1A	New Construction - Realignment
139+150	139+200	50	8A	New Construction - Realignment (Right Side Hill)

Annexure 2.4 : Widening Scheme (Section 6+7 Combined)

Design Chainage		Length (m)	TCS Type	Description
From	To			
139+200	139+430	230	2	Eccentric Widening - Left side
139+430	139+550	120	1A	New Construction - Realignment
139+550	139+800	250	2	Eccentric Widening - Left side
139+800	139+904	104	1A	New Construction - Realignment
139+904	139+927	23		MNB
139+927	140+100	173	1A	New Construction - Realignment
140+100	140+500	400	2	Eccentric Widening - Left side
140+500	140+600	100	1A	New Construction - Realignment
140+600	140+750	150	8A	New Construction - Realignment (Right Side Hill)
140+750	140+950	200	8	Eccentric Widening - Right Hill
140+950	141+030	80	8	Eccentric Widening - Left Hill
141+030	141+200	170	1A	New Construction - Realignment
141+200	141+700	500	2	Eccentric Widening - Left side
141+700	141+800	100	2	Eccentric Widening - Right side
141+800	141+950	150	8	Eccentric Widening - Left Hill
141+950	142+289	339	2	Eccentric Widening - Left side
142+289	142+301	12		MNB
142+301	142+600	299	2	Eccentric Widening - Right side
142+600	142+750	150	2	Eccentric Widening - Left side
142+750	143+249	499	2	Eccentric Widening - Right side
143+249	143+261	12		MNB
143+261	143+500	239	2	Eccentric Widening - Right side
143+500	143+616	116	1A	New Construction - Realignment
143+616	143+624	8		MNB
143+624	143+800	176	1A	New Construction - Realignment
143+800	144+093	293	2	Eccentric Widening - Right side
144+093	144+107	14		MNB
144+107	144+150	43	2	Eccentric Widening - Right side
144+150	144+330	180	8	Eccentric Widening - Left Hill
144+330	144+400	70	8	Eccentric Widening - Right Hill
144+400	144+530	130	8A	New Construction - Realignment (Right Side Hill)
144+530	144+700	170	1A	New Construction - Realignment
144+700	145+150	450	2	Eccentric Widening - Left side
145+150	145+200	50	9	New Construction - Both side Cutting
145+200	145+350	150	9	New Construction - Both side Cutting
145+350	145+550	200	1A	New Construction - Realignment
145+550	145+712	162	2	Eccentric Widening - Left side

PART-II

DESIGN OF STRUCTURES

DETAIL DESIGN CALCULATION OF PIER WITH PILE FOUNDATION.

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	

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

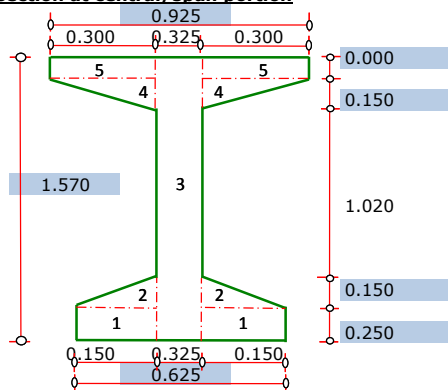
Diameter of cast-in-situ piles	=	1.2 m
Pile edge distance	=	0.15 m
Pile no's	long. direction (col.)	2
	trans. direction (row)	4
Pile spacing	long. direction (col.)	3.6 m
	trans. direction (row)	3.6 m
Pile cap depth	=	1.8 m
Pier dimension in long. direction	=	1.0 m
Pier dimension in trans. direction	=	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

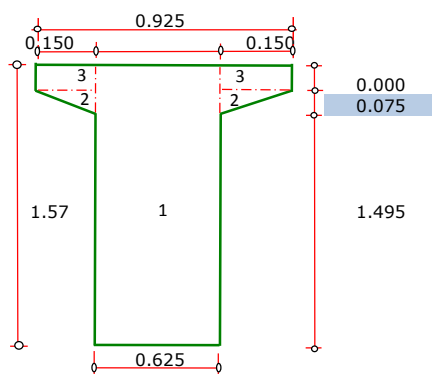
Span c/L to c/L of Expansion joint	=	22.000 m	at deck slab level beyond c/LI of bearing
Span c/L to c/L of Bearing	=	20.600 m	
Expansion joint	=	0.040 m	
Overall Span of girder	=	21.200 m	
Distance betw Exp. joint and c/L of Brg.	=	0.700 m	
Length of end thickning zone of girder	=	0.700 m	
Length of 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 diaphragm/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			29.5

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
						Total Dead Load=	
						401.0 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)

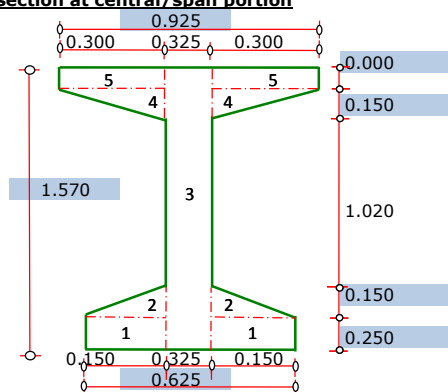
Description	Nos	Width(m)	Depth(m)	Area(m ²)	Length(m)	Unit Wt (t/m ³)	Total Wt (t)
300 mm thk. Footpath	2	1.50	0.300	0.90	22.00	2.5	49.5
						Total FPDL=	
						49.5 t	
						Total DL+Surfacing+ SIDL+FPDL=	
						535.0 t	

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

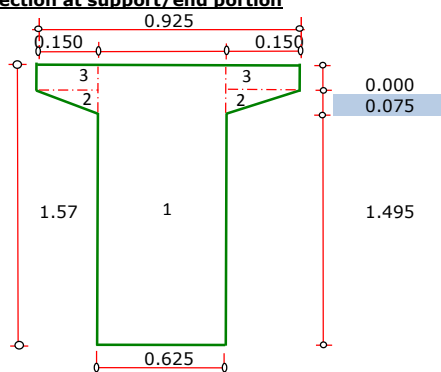
Span c/L to c/L of Expansion joint	=	22.000 m	
Span c/L to c/L of Bearing	=	20.600 m	
Expansion joint	=	0.040 m	
Overall Span of girder	=	21.200 m	
Dist bet. exp joint and c/l of bearing	=	0.700 m	at deck slab level
Length of end thickning zone of girder	=	0.700 m	beyond c/LI of bearing
Length of tapering 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

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		Check by	S.G	

Dead load of diaphragm/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. Footpath	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

Basic Footpath live load	=	LHS Span	400 Kg/m ²	RHS Span	400 Kg/m ²
Actual Footpath live load	=		342 Kg/m ²		342 Kg/m ²
	=		0.34 t/m ²		0.34 t/m ²
Footpath area	=		66.00 m ²		66.00 m ²
Total Footpath live load	=		22.6 t		22.6 t

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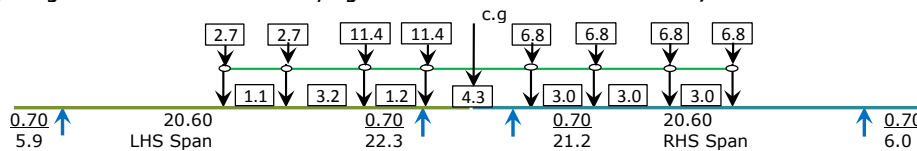
Case 1 - Max. Live Load Reaction & Max. Transverse Moment

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 coincide with c/L of Pier.

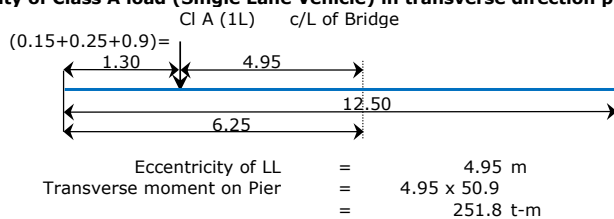


Reaction due to LL (Class A)		
Single Lane Vehicle	=	22.3 t on LHS Span
	=	21.2 t on RHS Span
Two Lane Vehicle	=	44.6 t on LHS Span
	=	42.5 t on RHS Span
Three Lane Vehicle	=	60.2 t on LHS Span
	=	57.4 t on RHS Span
Four Lane Vehicle	=	71.3 t on LHS Span
	=	68.0 t on RHS Span

Reaction due to LL (Class A) with Impact factor

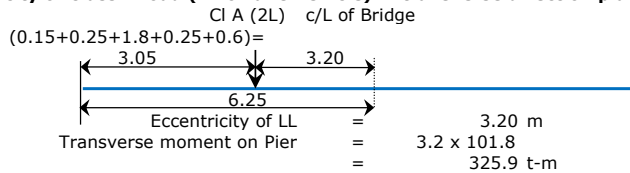
Single Lane Vehicle	=	50.9 t
Two Lane Vehicle	=	101.8 t
Three Lane Vehicle	=	137.4 t
Four Lane Vehicle	=	162.9 t

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

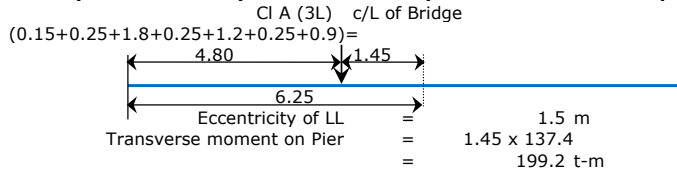


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		Check by	S.G	

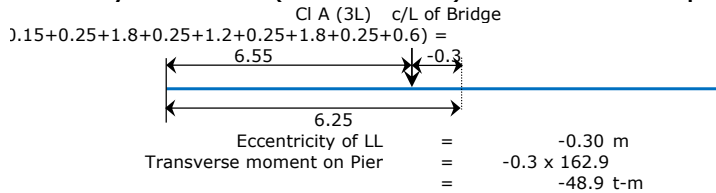
Eccentricity of Class A load (Two Lane Vehicle) in transverse direction placed in median side.



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



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



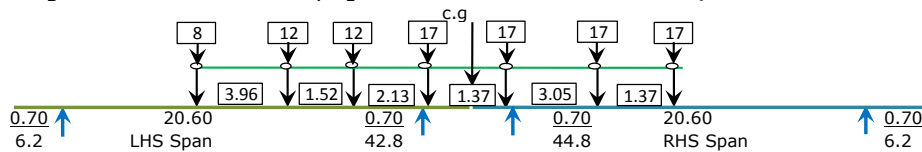
Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
CI A(1lane)	26.0	24.8	50.9	4.950	0.700	0.700	251.9	0.8
CI A(2lane)	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7
CI A(3lane)	70.3	67.1	137.4	1.450	0.700	0.700	199.2	2.3
CI A(4lane)	83.4	79.5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

2) Class 70 R Wheeled vehicle

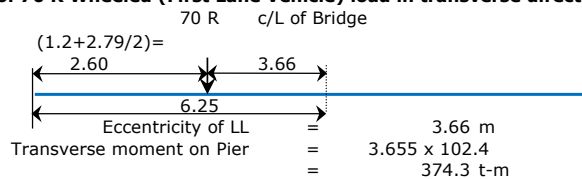
Impact factor for 70 R Wheel = 16.9%

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

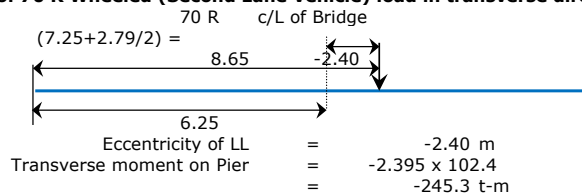
CI.208.3 of IRC:6-2017



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



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



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

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		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
Total	71.0	72.0	143.1	-	-	-	363.0	2.4

Multiplied by 0.9 for 3 Lane effect

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)

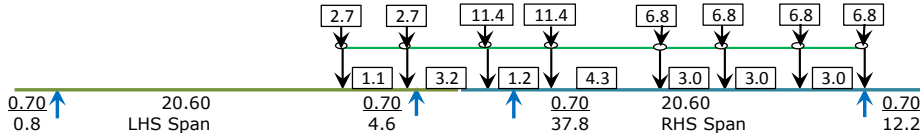
Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
Cl A 2Lane	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7
70R Wheel	40.0	41.9	81.9	-2.295	0.700	0.700	-188.0	1.4
Total	92.1	91.6	183.7	-	-	-	137.7	3.0

Multiplied by 0.8 for 4 Lane effect

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

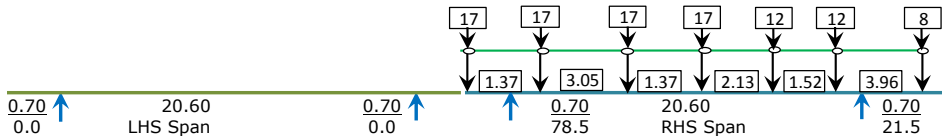
Reaction due to LL (Class A) with Impact factor

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
Total	5.4	126.9	132.2	-	-	-	310.6	76.5

Multiplied by 0.9 for 3 Lane effect

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		Check by	S.G	

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
Total	10.8	161.9	172.7	-	-	-	134.0	95.2

Multiplied by 0.8 for 4 Lane effect

Case 3 - Max. Longitudinal Force

Length of superstructure for breaking = 22.00 m

Cl.211.2 of
IRC:6-2017

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

		<u>Sevice condition-</u>		<u>One Span Dislodqe condition-</u>	
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	

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Design horizontal frictional force		<u>LL1</u> 48.5 t	<u>LL2</u> 47.1 t	<u>LL3</u> 48.5 t	
POT-PTFE Bearing:		Fixed			Fixed
Coefficient of Friction μ	=	0.05 0.03			0.05 0.03
DL & SIDL reaction at free end, Rg	=	535.0 t			267.5 t
Live load reaction at at free end, Rq	=	<u>LL1</u> 143.1 t	<u>LL2</u> 102.4 t	<u>LL3</u> 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
Zone factor Z	=	0.24
Importance factor I	=	1.2

Fig:4.1 of IRC:SP:114 - 2018

Seismic Coefficient in Longitudinal Direction

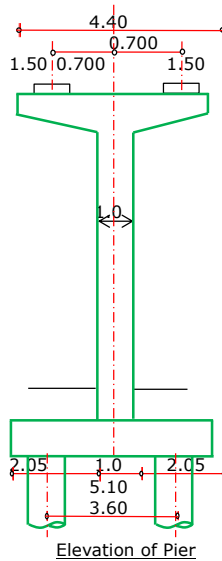
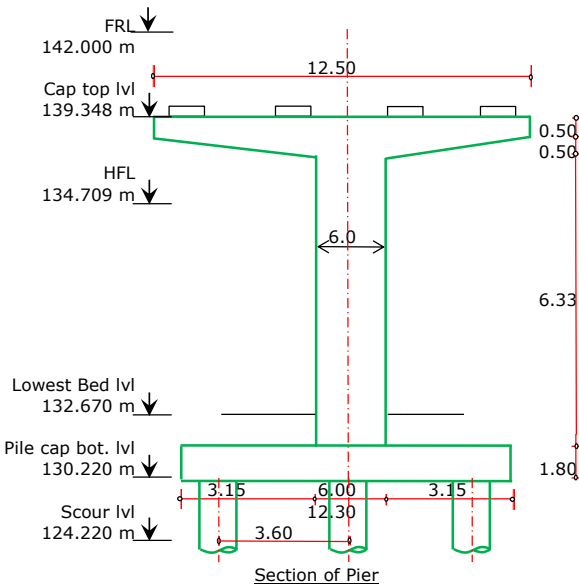
Horizontal seismic coefficient A_{HL}	=	0.180
Vertical seismic coefficient V_h	=	0.120

Seismic Coefficient in Transverse Direction

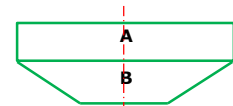
Horizontal seismic coefficient A_{HT}	=	0.180
Vertical seismic coefficient V_h	=	0.120

7.0 Load calculation of Substructure & Foundation

Dimensional details of Pier cap, Shaft & Footing



Height of pedestals,	
PD1 =	0.200 m
PD2 =	0.269 m
PD3 =	0.338 m
PD4 =	0.406 m
PD5 =	0.475 m



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

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		Check by	S.G	

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 & FPD	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

Total load of DL, SIDL & Surfacing			=	LHS Span	RHS Span	267.5 t
Seismic Longitudinal force:-						
Longitudinal seismic coefficient	A _{HL}	=	0.180			
Seismic component of DL, SIDL & Surfacing		=	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 & Surfacing		=	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 & Surfacing		=	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		
Total Live load		=	LHS Span 71.0	RHS Span 72.0 t	LHS Span 50.0	RHS Span 52.4 t	LHS Span 5.4	RHS Span 126.9 t
<u>Seismic Longitudinal force:-</u> No Live load seismic component is considered in longitudinal direction								
<u>Seismic Transverse force:-</u>								
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
<u>Seismic Vertical force:-</u>								
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

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

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 coefficient 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 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 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 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 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

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 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	-

(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	-

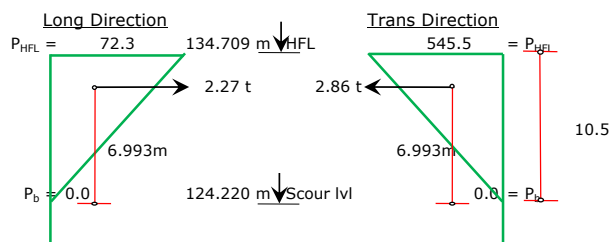
V	H _L	H _T	M _{L/Shaft}	M _{T/Shaft}
t	t	t	t-m	t-m
-	35.7	-	354.3	-
-	-	35.7	-	354.3
23.8	-	-	0.0	-

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}	=	√ 2 x V
Max. current effect in long dire		
V _L	=	4.24 m/sec
	=	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

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Pressure due to water current, $P = 52 \times K \times V_{\max}^2$ Kg/m²



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,	F_v	=	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
Transverse wind load,	F_T	=	16.4 t

$b / t = 7.50$ where, Width=b & Depth=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

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		Check by	S.G	

11.0 Summary of Forces at Founding Level

Summary of Forces Founding level (LWL)						Span Dislodge Condition				
						Forces about c/L of foundation				
						V	H _L	H _T	M _L	M _T
Description		t	t	t	t-m	t	t	t	t-m	t-m
1) DL of Superstructure		450.5			0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier		38.5			0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat		46.0			0.0	23.0			16.1	0.0
4) Carriageway Live load,	LL1	143.1		0.0	2.4					
	LL2	102.4		0.0	1.7					
	LL3	132.2		0.0	76.5					
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)						Span Dislodge Condition				
						Forces about c/L of foundation				
						V	H _L	H _T	M _L	M _T
Description		t	t	t	t-m	t	t	t	t-m	t-m
1) DL of Superstructure		450.5			0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier		38.5			0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat		46.0			0.0	23.0			16.1	0.0
4) Carriageway Live load,	LL1	143.1		0.0	2.4					
	LL2	102.4		0.0	1.7					
	LL3	132.2		0.0	76.5					
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

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							Check by	S.G	
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

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	

11.3 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)						Span Dislodge Condition				
						Forces about c/L of Shaft				
Description						V	H _L	H _T	M _L	M _T
						t	t	t	t-m	t-m
1) DL of Superstructure		450.5				225.2			157.7	0.0
2) SIDL - Crash Barrier		38.5				19.3			13.5	0.0
3) SIDL - Wearing Coat		46.0				23.0			16.1	0.0
4) Carriageway Live load,	LL1	143.1		0.0	2.4					
	LL2	102.4		0.0	1.7					
	LL3	132.2		0.0	76.5					
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				48.1		457.8
13) Cariageway Live load,	LL1			25.7						
	LL2			18.4						
	LL3			23.8						
14) Substructure component				38.9				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	

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	

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

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	

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

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

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.

- Span is less than 15.0 m.
- 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)

Horizontal seismic force,	A_v	=	0.667	$\times A_h$
where,	F_{eq}	=	$A_h \times (DL + b \times LL)$	(Refer Cl. 5.2.1 of IRC:SP:114-2018)
	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 Strata Soil Type	Notation
Rock/Hard Soil (I)	HS
Medium/ Stiff Soil (II)	MS
Soft Soil (III)	SS

$$\text{Horizontal force, } F = 6 EI / \{x^2 (3L - x)\}$$

where,

F = Force applied at cg of Superstructure or top of Bearing to produce 1mm deflection at Pier/Abutment 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

$$\text{Horizontal Seismic coefficient, } A_h = (Z/2) \times (S_a/g) / (R/I)$$

where,

$$\text{Zone Factor, } Z = 0.36$$

$$\text{Importance Factor, } I = 1.2$$

Resonance Reduction Factor, R

$$\text{Longitudinal direction, } R_L = 3.0$$

$$\text{Transverse direction, } R_T = 3.0$$

Elastic Seismic Acceleration Coefficient (ESAM), S_a/g

$$\text{Foundation soil strata} = SS$$

Elastic Seismic Acceleration Coefficient (ESAM) for 5% damping,

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

$$\text{Type of Pier/ Abutment} = RC$$

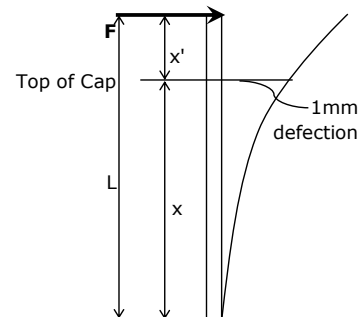
$$\text{Grade of Concrete} = M 35$$

$$\text{Youngs Modulus, } E = 3.2E+07 \text{ KN/m}^2$$

$$x = 7.328 \text{ m}$$

$$x'_L = 0.303 \text{ m}$$

$$x'_T = 1.207 \text{ m}$$



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.

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

Seismic Coefficient in Longitudinal Direction in Service Condition:

Size of Rectangular Pier/ Abutment		=	1.00	x	6.00	m
Moment of Inertia	I	=	$6 \times 1^3 / 12$			
		=	0.5	m ⁴		
Dia of Solid Pier	D	=	1.00	m		
Moment of Inertia	I	=	$\pi() \times 1^4 / 64$			
		=	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	=	$\pi() \times (1^4 - 0^4) / 64$			
		=	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	=	$2 \times (D / (1000 \times F))^{0.5}$			
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			
Horizontal Seismic Coefficient, A _{hL}		=	$(0.36/ 2) \times 2.5 / (3 / 1.2)$			
		=	0.180			
Vertical Seismic Coefficient, A _v		=	A _h *2/3 or 0			
		=	0.120			

HS	2.02
MS	2.50
SS	2.50

Seismic Coefficient in Transverse Direction in Service Condition:

Size of Rectangular Pier/ Abutment		=	1.00	x	6.00	m
Moment of Inertia	I	=	$1 \times 6^3 / 12$			
		=	18.0	m ⁴		
Dia of Solid Pier	D	=	1.00	m		
Moment of Inertia	I	=	$\pi() \times 1^4 / 64$			
		=	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	=	$\pi() \times (1^4 - 0^4) / 64$			
		=	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	=	$2 \times (D / (1000 \times F))^{0.5}$			
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			
Horizontal Seismic Coefficient, A _{hT}		=	$(0.36 / 2) \times 2.5 / (3 / 1.2)$			
		=	0.18			
Vertical Seismic Coefficient, A _v		=	A _h *2/3 or 0			
		=	0.12			

HS	2.50
MS	2.50
SS	2.50

Grade of concrete of cast-insitu Pile,	=	M 35
<u>Salient dimensions:</u> (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
<u>Salient dimensions:</u> (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
<u>Other Salient Design Data</u>		
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

Area of pile,	$A = \pi \cdot d^2 / 4$	=	1.131 m ²	
Moment of inertia pile,	$I = \pi \cdot d^4 / 64$	=	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,	$E_c = 22 \cdot ((f_{ck} + 10) / 12.5)^{0.3}$	=	3.23E+07 KN/m ²	3.29E+06

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

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

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

$$R = \sqrt[4]{\frac{EL}{\eta_b}}$$

Thus,
$$R = \frac{\sqrt{KB}}{4\sqrt{EI}}$$

k_1 = Modulus of Subgrade Reaction for Cohesive Soil, in KN/m^3 (see Table 4)
Type of top layer soil providing lateral resistance = Granular soil

From Table we get,	η_b	=	Silty sand	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

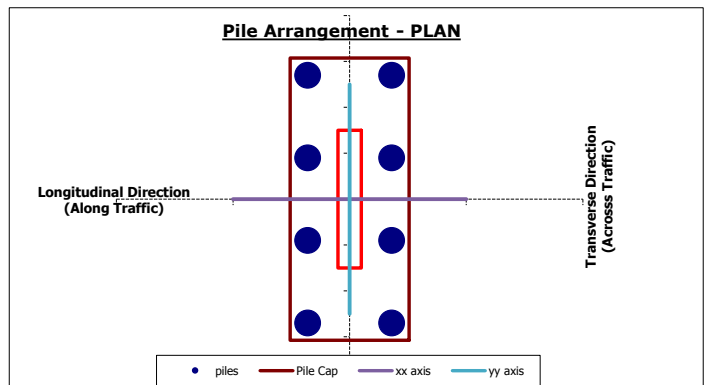
Considering equation of parabola,	y	=	αL^2
	α	=	0.000185
	y_2	=	0.037 m

Design lateral load for fixed head pile, $H = \frac{12 E I_y}{(e + z_f)^3}$

		=	519 kN
Design moment for fixed head pile,	M_F	=	$m \cdot H \cdot (e + z_f)/2$
Moment reduction factor,	m	=	0.82

The diagram illustrates a pile of total length L fixed at the bottom. A lateral load P is applied at a height L_f from the fixed end. The pile is divided into two segments: L_1 (above the load) and L_f (below the load). A dashed curve represents the deflection of the pile. The horizontal deflection at the pile top is labeled y_2 , and the horizontal deflection at the scour level is labeled y_1 . The scour level is indicated by a horizontal line. A label '1% of pile dia' points to the scour level, indicating the scour depth. The top of the pile is labeled 'Pile top level'.

Cl. 79.3.5.1
of IRC:78

[illegible]

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:

Characteristic 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
Characteristic 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,	$\epsilon_s = f_{yd} / E_s$	=	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	=	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)	1 st layer	=	20 mm
	2 nd layer	=	20 mm
Number of long bars	1 st layer	=	16
	2 nd layer	=	16
Area of steel provided,	A_{sc}	=	10053 mm ²
Percentage of steel provided		=	0.89 %
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

OK

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

Diameter of long bars (L1)	1 st layer	=	20 mm
Number of long bars	1 st 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}	=	$20 \cdot A \cdot B \cdot C / \sqrt{\dots}$
where,	A	=	$1 / (1 + 0.2 \cdot \phi_{ef})$
Creep Coefficient,	$\phi(\infty, 0)$	=	1.00
First order BM Quasi-Permanent load	M_{0Ed}	=	93 kN-m
First order BM in design load in ULS	M_{0Ed}	=	235 kN-m
Effective creep ratio,	$\phi_{ef} = \phi(\infty, 0) \cdot M_{0Ed} / M_{0Fd}$	=	0.40
	A	=	0.93
where,	B	=	$\sqrt{(1 + 2\omega)}$
Mechanical r/f ratio,	$\omega = A_s \cdot f_{yd} / A_c \cdot f_{cd}$	=	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	=	$N_{Ed} / A_c \cdot f_{cd}$

Eccentricity due to slenderness:

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

Eq.11.1
of IRC:112

2.51

Pile Moment Capacity Check at ULS :-* check for α value

Load case	N_{Ed} kN	M_{0Ed} kN-m	$n = N_{Ed} / (A_c \cdot f_{cd})$	λ_{lim}	K_r	e_2 (m)	M_{Ed} kN-m	$P_U / f_{ck} \cdot D^2$	$M_U / f_{ck} \cdot D^3$	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.
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 at ULS :-

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

Design of section without shear reinforcement:-

Factor for concrete cracked in shear, $v = 0.6(1 - f_{ck}/310) = 0.53$
 $K = \text{Min}.[1 + (200/d)^{0.5}, 2] = 1.43$
 $\rho_1 = \text{Min}.[A_{st} / b_w \cdot d, 0.02] = 0.008$
 $v_{min} = 0.031 \cdot K^{3/2} \cdot f_{ck}^{1/2} = 0.312 \text{ MPa}$
Mean Comp. stress in concrete, $\sigma_{cd} = \text{Min}[N_{Ed}/A_c, 0.2f_{cd}] = 0 \text{ MPa}$
Min. Design Shear resistance, $V_{Rd,c1} = (v_{min} + 0.15 \cdot \sigma_{cd}) b_w \cdot d = 412 \text{ kN}$
 $V_{Rd,c2} = [0.12 \cdot K \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} + 0.15 \cdot \sigma_{cd}] b_w \cdot d = 620 \text{ kN}$
Design Shear resistance, $V_{Rdc} = \text{Max}.[V_{Rd,c1} \text{ \& } V_{Rd,c2}] = 620 \text{ kN}$

Check for Shear R/f (SR) requirement, **SR NRQ**

Design of section with shear reinforcement:-

Dia. of Shear r/f, stirrups/ links $\phi = 16 \text{ mm}$
Number of stirrups/ links $n = 1$
Spacing of stirrups/ links $s = 95 \text{ mm}$
Area of Shear r/f provided, $A_{sw} = 201 \text{ mm}^2$
 $A_{sw} / s = 2.1 \text{ mm}^2/\text{mm}$
Minimum area of Shear r/f required, $\text{Min. } A_{sw} / s = 1.0 \text{ mm}^2/\text{mm}$
Status, Area of Shear r/f, **OK**
Lever arm factor, $\alpha_{cw} = 0.9$
Coeff. depends on stress in compression cord, $\alpha_{cw} = 1.0$
Factor for concrete cracked in shear, $v_1 = v = 0.53$
Shear capacity of concrete strut corresponding, $\theta = 45 \text{ degree}$
 $V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) = 4938 \text{ kN}$
Status, Check for Redesign of section, **OK**
Angle of concrete strut due shear, $\theta = 2.3 \text{ degree}$
Shear capacity of strut, $V_{Rd} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) = 392 \text{ kN}$
0
Angle of concrete strut considered, $\theta = 21.8 \text{ degree}$
Shear resistance provided by r/f, $V_{Rd,s} = A_{sw} / s \cdot z \cdot f_{ywd} \cdot \cot \theta = 2094 \text{ kN}$
Status, Check for shear capacity of section, **OK**
5.3

*RQ - Required

*NRQ - Not Required

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, $L_0 = 8.054 \text{ m}$
Effective length factor, $= 1.00$
Effective length of column $L_e = 8.054 \text{ m}$
Radius of Gyration, $r = 0.300 \text{ m}$
 $L_e / r = 26.8$

Short column

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	

Confinement 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, $A_{sh} = 0.09SD_k f_{ck} / f_y [A_g / A_k - 1.0]$	=	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_g	=	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, $\omega_{wd.c}$	≥	$\max(1.4 \omega_{wreq}; 0.18)$

Normalised axial force, $n_k = N_{Ed} / (A_c * f_{ck}) > 0.08$ = 0.08 0.08

R/f ratio of longitudinal r/f, $\rho_L = A_s / A_c$ = 0.009

$\omega_{wreq} = 0.37 * A_c / A_{cc} * n_k + 0.13 * f_{yd} / f_{cd} * (\rho_L - 0.01)$ = 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

Eq. 17.7
of IRC:112

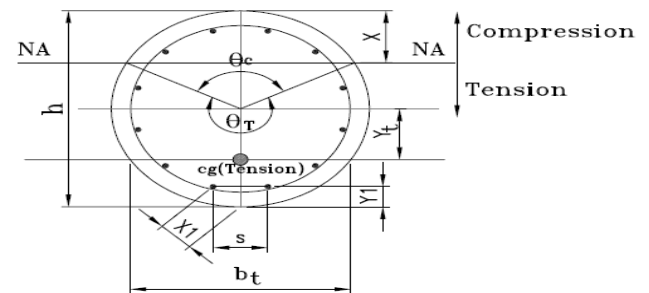
Flexure design of Pile at SLS :-**Material Data**

Concrete strength of cast-in-situ Pile, f_{ck}	=	35 MPa
Modulus of Elasticity of Concrete, E_{cm}	=	32308 MPa
Characteristic 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

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, 1 st layer	=	20 mm
2 nd layer	=	20 mm
Number of long bars, 1 st layer	=	16 Nos.
2 nd 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

at SLS Rare load combination

**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-max	1540	93	1.7	0.8	Uncracked	2155	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-max	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-max	1248	93	1.4	0.6	Uncracked	1861	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-max	836	55	0.9	0.4	Uncracked	2029	6.3	0.0	1130973	1.018E+11	1245385

* check formula

* check formula

Load	Eccentricity	Net eccentricity	Eff. MoI of cracked section	Calculated N.A	Yassume	Ycal - Yassume	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-max	0.0	60.7	1.176E+11	1555	1555	0	0.0	1.7	10.4
SQ-P:MT-max	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

Max. Concrete Compressive Stress	=	2.7 MPa	OK
Max. Tensile Stress in Steel	=	-7.8 MPa	OK

Max. Concrete Compressive Stress	=	1.7 MPa	OK
Max. Tensile Stress in Steel	=	0 MPa	OK

Cl 12.3 of
IRC:112

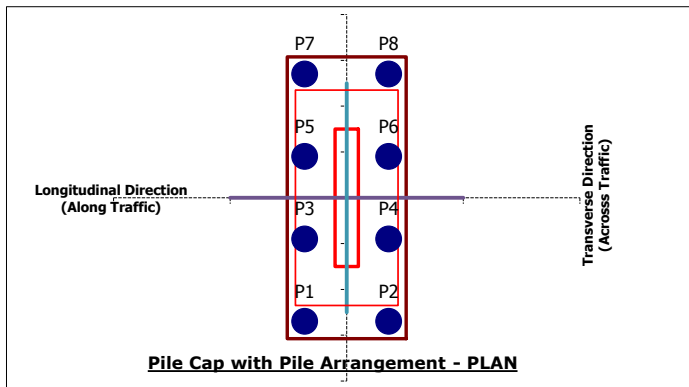
Coefficient depends on duration of load	K_t	=	0.50
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Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

15.0 Design of Pile Cap by Bending theory :-

Grade of concrete of Pile cap,	=	M 35
<u>Salient dimensions:</u> (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
<u>Salient dimensions:</u> (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
<u>Material property</u>		
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
<u>Reinforcement details</u>		
Clear cover to reinforcement	=	75 mm
Dia. of longitudinal r/f at bottom	=	25 mm
Dia. of transverse 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

**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.
		0.000 m, Pile no.
along Transverse direction	=	2.400 m, Pile no.
	=	-1.200 m, Pile no.

P1	P3	P5	P7		
P1	P2				
P3	P4				

For opposite face check:-

Lever arm of piles from face,		
along Longitudinal direction	=	1.300 m, Pile no.
	=	0.000 m, Pile no.
along Transverse direction	=	2.400 m, Pile no.
	=	-1.200 m, Pile no.

P2	P4	P6	P8		
P7	P8				
P5	P6				

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		Design by	D.S.R	
		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

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,	$\epsilon_s = f_{yd} / E_s$	=	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	=	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,	$p_{t,req} = f_{ck} / (2f_{yk}) \cdot (1 - (1 - (4.598 \cdot R / f_{ck})^2))$	=	0.0018	0.0003
Min. % of steel required,	$p_t = \max.[0.26 \cdot f_{ctm} / f_{yk}, 0.0013]$	=	0.0014	0.0014
Area of steel required for bending,	$A_{st, req}$	=	15732 mm ²	29698 mm ²
R/f provided,	Dia. 1 st layer	=	25 mm	25 mm
	Spacing	=	125 mm c/c	125 mm c/c
	Dia. 2 nd 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,	$x_{u,max} = 0.456 \cdot d$	=	776 mm	764 mm
Actual depth of N.A,	$x = (0.87 \cdot f_{yk} \cdot A_{st}) / (0.362 \cdot f_{ck} \cdot b)$	=	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,	$T = A_{st} \cdot f_s$	=	8368 kN	20661 kN
Total compressive force in concrete,	$C = 0.362 \cdot f_{ck} \cdot x \cdot b$	=	8368 kN	20661 kN
Difference of force,	$C - T$	=	0	0
Lever arm of compressive force in concrete,	$z = (d - 0.416 \cdot x)$	=	1647 mm	1620 mm
Moment of resistance,	$M_{Rd} = C \cdot z$	=	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 :-

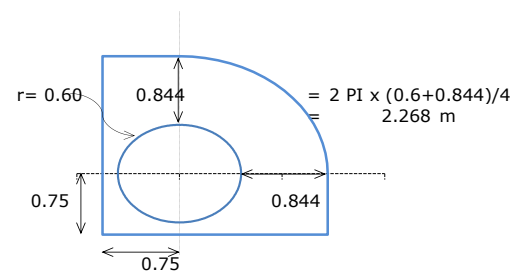
		Long. Direction	Trans. Direction	
Design Shear Force of the concrete section,	$V_{NS} = V_{Ed}$	=	3575 kN	3042 kN
Design of section without shear reinforcement:-				
Factor for concrete cracked in shear,	$v=0.6(1-f_{ck}/310)$	=	0.53	0.53
Max. $V_{Ed} = 0.5 b_w \cdot d \cdot v \cdot f_{cd}$		=	36082 kN	85742 kN
Status,			OK	OK
$K = \text{Min.}[1 + (200/d)^{0.5}, 2]$		=	1.34	1.35
$\rho_1 = \text{Min.}[A_{sl} / b_w \cdot d, 0.02]$		=	0.002	0.002
$v_{min} = 0.031 \cdot K^{3/2} \cdot f_{rk}^{1/2}$		=	0.285 MPa	0.286 MPa
Mean Comp. stress in concrete,	$\sigma_{cd} = \text{Min}[N_{Ed}/A_{cr}, 0.2f_{cd}]$ Mpa	=	0 MPa	0 MPa
Min. Design Shear resistance,	$V_{Rd,c1} = (v_{min} + 0.15 \cdot \sigma_{cd}) b_w \cdot d$	=	2475 kN	5899 kN
$V_{Rd,c2} = [0.12 \cdot K \cdot (80 \cdot \rho_1 \cdot f_{rk})^{0.33} + 0.15 \cdot \sigma_{cd}] b_w \cdot d$		=	2554 kN	6158 kN
Design Shear resistance,	$V_{Rdc} = \text{Max.}[V_{Rd,c1} \& V_{Rd,c2}]$	=	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
Minimum area of Shear r/f required,	A_{sw} / s	=	9.0 mm ² /mm	14.5 mm ² /mm
Status, Area of Shear r/f,	Min. A_{sw} / s	=	5.4 mm ² /mm	13.1 mm ² /mm
Lever arm factor,		=	OK	OK
Coeff. depends on stress in compression cord,	α_{cw}	=	0.9	0.9
Factor for concrete cracked in shear,	$v_1 = v$	=	1.0	1.0
Shear capacity of concrete strut corresponding,	θ	=	0.53	0.53
$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$		=	45 degree	45 degree
Status, Check for Redesign of section,		=	32474 kN	77168 kN
Angle of concrete strut due shear,	θ	=	OK	OK
Shear capacity of strut,	$V_{Rd} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$	=	3.2 degree	1.1 degree
Angle of concrete strut considered,	θ	=	3575 kN	3042 kN
Shear resistance provided by r/f,	$V_{Rd,s} = A_{sw} / s \cdot z \cdot f_{vw} \cdot \cot\theta$	=	0	0
Status, Check for shear capacity of section,		=	21.8 degree	21.8 degree
		=	13848 kN	21831 kN
		=	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 pile 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,	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$	=	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,	$v_{Ed} = \beta V_{Ed,red} / (u_o \cdot d)$	=	1.57 MPa
Max. Punching shear resistance,	$V_{Rd,max} = 1/2 \cdot v \cdot f_{cd}$	=	4.16 MPa
		=	OK
Check at a distance 'd/2' from Pile perimeter,			
Factored Shear due to s/w of Cap (incl. 15% subm	ΔV_{Ed}	=	844 mm
Net applied Shear force,	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$	=	249 kN
Control perimeter,	u_i	=	3478 kN
Punching Shear stress,	$v_{Ed} = V_{Ed,red} / (u_i \cdot d)$	=	2268 mm
		=	0.91 MPa
$K = \text{Min.}[1 + (200/d)^{0.5}, 2]$		=	1.34
$v_{min} = 0.031 \cdot K^{3/2} \cdot f_{rk}^{1/2}$		=	0.286 MPa
Mean value of percentage of steel provided,	ρ_l	=	0.003
Punching shear resistance at control perimeter,		=	
$V_{Rd} = \text{Min.}[0.12K(80\rho_l f_{rk})^{0.33} \cdot 2d/a, v_{min} \cdot 2d/a]$		=	1.14 MPa
		=	OK

where, $v=0.6(1-f_{ck}/310)$

**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
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BM* _L (kN-m)								
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BM _T (kN-m)	1693	649	1272	1047	1972	1470	1972	1155
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BM* _T (kN-m)								
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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,	$T = A_{st} \cdot \sigma_s$	= 3670	3215 kN	= 1088	1266 kN	from Stress diagram
Total comp. force in concrete,	$C = 0.5 \cdot \sigma_c \cdot x \cdot b$	= 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,	$M_s = T \cdot (d - x_1)$	= 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,	$\alpha_e = E_s / E_{cm}'$	=	0.30	mm	for	Moderate	Exposure
		=	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
Calculated spacing b/w bars,	$5 \cdot (c + \phi_{eq}/2)$	=	437.5	mm	438
Status		OK		OK	
Overall depth of Pile cap,	D	=	1800	mm	1800
Depth of Neutral axis,	x	=	355	mm	355
Tensile stress in R/f,	σ_{sc}	=	167.1	MPa	26.6
Area of steel,	A_s	=	19246	mm ²	47521
Eff. depth in tension,	$h_{c,eff} = \text{Min.}[2.5 \cdot (h-d), (h-x)/3, h/2]$	=	249	mm	311
Eff. area of concrete within tensile zone,	$A_{c,eff} = h_{c,eff} \cdot b$	=	1268625	mm ²	3828375
	$\rho_{peff} = A_s / A_{c,eff}$	=	0.015		0.012
	$\epsilon_{sm} - \epsilon_{cm} = \text{Max}\{[\sigma_{sc} - k_t \cdot f_{ct,eff}(1 + \alpha_e \cdot \rho_{peff}) / E_s] \& (0.6 \cdot \sigma_{sc} / E_s)\}$	=	0.0005		0.0001
	$S_{r,max} = 3.4c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{p,eff}$	=	535		597
	$W_k = S_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm})$	=	0.268	mm	0.048
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
<u>Material data:</u>		
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,	$\epsilon_s = f_{yd} / E_s$	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	0.00417
Creep Coefficient,	ϕ	1.50

Geometric data: (Pier / Abutment)

		<u>Long. Direction</u>	<u>Trans. Direction</u>	
Dimension of Pier/ Abutment,	=	1000 mm	6000 mm	Wall type Pier
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	
<u>Vertical longitudinal r/f detail of Column on each face:</u>				
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}	$20 \cdot A \cdot B \cdot C / \sqrt{n}$	
where,	A	$1 / (1 + 0.2 \cdot \phi_{ef})$	
Creep Coefficient,	$\phi(\infty, 0)$	1.50	
First order BM Quasi-Permanent load	M_{oEdo}	1693 kN-m	
First order BM in design load in ULS	M_{oEd}	7831 kN-m	
Effective creep ratio,	$\phi_{ef} = \phi(\infty, 0) \cdot M_{oFnn} / M_{oFrd}$	0.32	

where,	B	$\sqrt{(1 + 2\omega)}$	
Mechanical r/f ratio,	$\omega = A_s \cdot f_{yd} / A_c \cdot f_{cd}$	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	$N_{Ed} / A_c \cdot f_{cd}$	

Eccentricity due to slenderness:

Mominal second order moment,	M_2	$N_{Ed} \cdot e_2$	
Deflection,	e_2	$(1/r) \cdot l_e^2 / c$	
Factor for constant cross section,	c	10	
Curvature,	$1/r$	$K_r \cdot K_\phi \cdot 1/r_0$	
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	$1 + \beta \cdot \phi_{ef}$	
	$\beta = 0.35 + (f_{rk}/200) - (\lambda/150)$	0.14	0.46
	K_ϕ	1.04	1.15
	$1/r_0 = f_{yd} / (E_s \cdot 0.45d)$	0.0030 1/m	

Eq.11.1
of IRC:112

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

Pier Moment Capacity Check at ULS :-

Load case	N _{Ed} kN	M _{0EdIL} kN-m	M _{0EdIT} kN-m	H _L kN	H _T kN	n = N _{Ed} / (A _c * f _{cd})	λ _{lim}	K _r	β _L	β _T	K _{φL}	K _{φT}
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 _{EdIL} kN-m	M _{EdIT} kN-m	M _{UL} kN-m	M _{UT} kN-m	N _{Ed} / N _{Rd}	α	(M _{Ed} / M _u) ^α	
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
<u>Design of section without shear reinforcement:-</u>			
Factor for concrete cracked in shear,	v = 0.6(1 - f _{ck} /310)	0.53	0.53
	K = Min.[1 + (200/d) ^{0.5} , 2]	1.47	1.18
	ρ ₁ = Min.[A _{sl} / b _w d, 0.02]	0.003	0.003
	v _{min} = 0.031 * K ^{3/2} * f _{rk} ^{1/2}	0.325 MPa	0.236 MPa
Mean Comp. stress in concrete,	σ _{cd} = Min[N _{Ed} /A _c , 0.2f _{cd}] Mpa	1.8 MPa	1.8 MPa
Min. Design Shear resistance,	V _{Rd,c1} = (v _{min} + 0.15*σ _{cd})b _w d	3321 kN	3029 kN
	V _{Rd,c2} = [0.12*K*(80+ρ ₁ *f _{rk}) ^{0.33} + 0.15*σ _{cd}]b _w d	3449 kN	3261 kN
Design Shear resistance,	V _{Rdc} = Max.[V _{Rd,c1} & V _{Rd,c2}]	3449 kN	3261 kN
Check for Shear R/f (SR) requirement,		SR NRQ	SR NRQ
<u>Design of section with shear reinforcement:-</u>			
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,	v ₁ = v	0.53	0.53
Shear capacity of concrete strut corresponding,	θ	45 degree	45 degree
	V _{Rd,max} = α _{cw} *b _w *Z*v ₁ *f _{cd} /(cotθ+tanθ)	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} = α _{cw} *b _w *Z*v ₁ *f _{cd} /(cotθ+tanθ)	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 _{wd} *cotθ	-44470 kN	-1673253 kN
		-19.8	-795.0

*RQ - Required
*NRQ - Not Required

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}	=	ρ _w f _{yd} / f _{cd}
Where,	ρ _w	=	A _{sw} / (b * S ₁)
Area of spiral/ hoop,	A _{sw}	=	1608 mm ²
Dia. of spiral or hoop	b	=	5900 mm
For rectangular section,	ω _{wd,c}	≥	max(ω _{wd,req} ; 0.12)
Confining r/f ratio,	ω _{wd,req}	=	0.37*A _g /A _{cc} *ρ _k +0.13*f _{yd} /f _{cd} *(ρ _L -0.01)
Where,	n _k = N _{Ed} /(A _c *f _{ck})	=	0.08
	ρ _L = A _g /(b * D)	=	0.01
	ω _{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

Eq. 17.5
of IRC:112

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

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 Footpath/ Kerb	=	0.000	m	
4)	Width of Footpath/ 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/ Diaphragm	=	0.300	m	
15)	Width of End Cross girder/ Diaphragm	=	0.400	m	
16)	Girder length beyond c/l of Brg. on each side	=	0.450	m	

1.2 Material Properties:

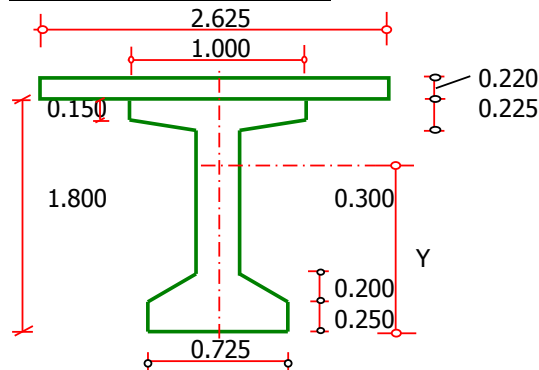
1) Concrete Grade for Deck slab	=	M 45
2) Concrete Grade for Cross girder	=	M 45
3) Concrete Grade of PSC Longitudinal girder	=	M 45
4) Reinforcement	=	Fe 500
5) Clear Cover	=	40 mm
6) Condition of Exposure	=	Moderate
7) Cable/ Strands	=	19 T 13 conforming to IS 14268 (Stress-relieved multiply strands of low relaxa 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 flange	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 = $\Sigma beff,i + bw$	=	7.465 m
c/c Spacing of Longitudinal girder, b	=	2.625 m

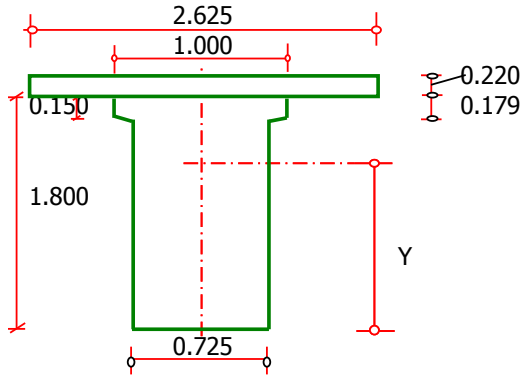
SL.No.	A	y	Ay	Ay ²	I _o	I _o + Ay ²	u
1. Deck Slab	0.57750	1.91000	1.10303	2.10678	0.002329	2.10911	4.690
2. Rectangular top Flange	0.15000	1.72500	0.25875	0.44634	0.000281	0.44663	0.300
3. Triangular top Flange	0.02625	1.62500	0.04266	0.06932	0.000008	0.06932	0.026
4. Web + Top & Bottom Rect. Haunch	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)	$Y = \Sigma(A.y) / \Sigma A$	=	1.3163 m
Moment of inertia of composite girder	$I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.6594 m ⁴

Section Properties of Girder

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

Girder Section at the Support**Calculation of Torsional Constant**

Component	Width	Depth	Area	I _z
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 flange	0.725	0.450	0.326	0.0143

$$I_z = 0.1166$$

$$\begin{aligned} \text{Modulus of Elasticity of Deck Slab} &= 34313 \text{ MPa} \\ \text{Modulus of Elasticity of Girder} &= 34313 \text{ MPa} \end{aligned}$$

$$\begin{aligned} b_{eff} &= \Sigma b_{eff,i} + b_w = 7.805 \text{ m} \\ \text{c/c Spacing of Longitudinal girder, } b &= 2.625 \text{ m} \end{aligned}$$

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. Rectangular 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

$$\begin{aligned} \text{Area} & \Sigma A = 1.9278 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} & Y = \Sigma(A.y) / \Sigma A = 1.2218 \text{ m} \\ \text{Moment of inertia of end intermediate girder} & I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2 = 0.7746 \text{ m}^4 \end{aligned}$$

Section Properties of Girder

$$\begin{aligned} \text{Area} & \Sigma A = 1.3503 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} & Y = \Sigma(A.y) / \Sigma A = 0.9274 \text{ m} \\ \text{Moment of inertia of end intermediate girder} & I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2 = 0.3817 \text{ m}^4 \end{aligned}$$

1.1 Summary of Section Properties of Longitudinal Girder (Outer)**A. Section Properties of Composite Section at the Span**

$$\begin{aligned} \text{Area} &= 1.3975 \text{ m}^2 & Y_b &= 1.3163 \text{ m} & Y_{ts} &= 0.7037 \text{ m} \\ I_z &= 0.6594 \text{ m}^4 & Z_b &= 0.5010 \text{ m}^3 & Z_{ts} &= 0.9371 \text{ m}^3 \\ & & & & Y_{tg} &= 0.4837 \text{ m} \\ & & & & Z_{tg} &= 1.3633 \text{ m}^3 \end{aligned}$$

B. Section Properties of Girder at the Span

$$\begin{aligned} \text{Area} &= 0.8200 \text{ m}^2 & Y_b &= 0.8982 \text{ m} & Y_t &= 0.9018 \text{ m} \\ I_z &= 0.3102 \text{ m}^4 & Z_b &= 0.3453 \text{ m}^3 & Z_t &= 0.3440 \text{ m}^3 \end{aligned}$$

C. Section Properties of Composite Section at the Support

$$\begin{aligned} \text{Area} &= 1.9278 \text{ m}^2 & Y_b &= 1.2218 \text{ m} & Y_{ts} &= 0.7982 \text{ m} \\ I_z &= 0.7746 \text{ m}^4 & Z_b &= 0.6340 \text{ m}^3 & Z_{ts} &= 0.9704 \text{ m}^3 \\ & & & & Y_{tg} &= 0.5782 \text{ m} \\ & & & & Z_{tg} &= 1.3395 \text{ m}^3 \end{aligned}$$

D. Section Properties of Girder at the Support

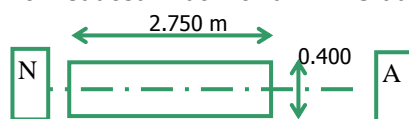
$$\begin{aligned} \text{Area} &= 1.3503 \text{ m}^2 & Y_b &= 0.9274 \text{ m} & Y_t &= 0.8726 \text{ m} \\ I_z &= 0.3817 \text{ m}^4 & Z_b &= 0.4116 \text{ m}^3 & Z_t &= 0.4374 \text{ m}^3 \end{aligned}$$

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphragm 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.)



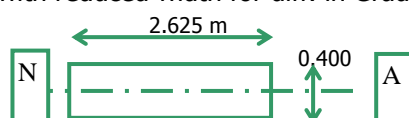
Modulus of Elasticity ratio = 1.000

Area = 1.1000 m²

Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0147 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**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 m²

Distance of cg from bottom fibre (y) = 0.2000 m

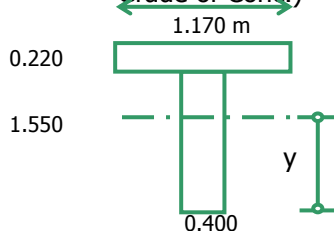
Moment of inertia (I_z) = 0.0140 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**End Cross Girder****_ENDCG**

(with reduced width for diff. in

beff = Min{ $\sum beff_{i,i} + bw, b$ } = 1.170 m

Grade of Conc.)

Modulus of Elasticity ratio = 1.000

Area = 0.8774 m²

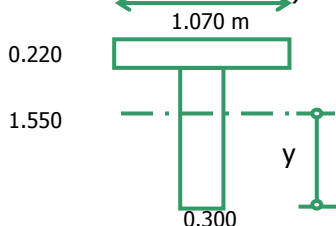
Distance of cg from bottom fibre (y) = 1.0346 m

Moment of inertia of end x-girder (I_z) = 0.2676 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴

Intermediate/ Inner Cross Girder

(with reduced width for diff. in

Grade of Conc.)



$$b_{eff} = \text{Min}\{ \sum b_{eff,i} + b_w, b \}$$

Modulus of Elasticity ratio

_INTCG

= 1.070 m

= 1.000

Area = 0.7004 m²

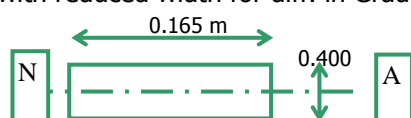
Distance of cg from bottom fibre (y) = 1.0724 m

Moment of inertia of end x-girder (I_z) = 0.2165 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

Cantilever Slab (At Expansion End)

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



Modulus of Elasticity ratio = 1.000

Area = 0.0660 m²

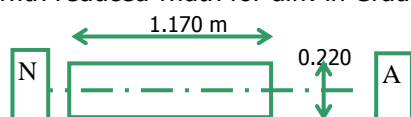
Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2574 m²

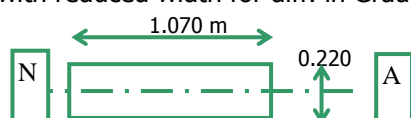
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0010 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2354 m²

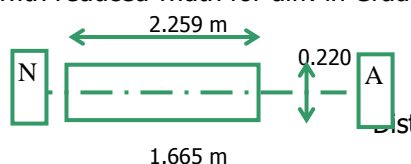
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSCANT2**Intermediate Slab (Next to End Cross Girder)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.4969 m²

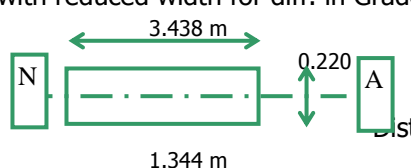
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0020 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB1**Intermediate Slab (At Splayed/ Tapered Portion)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.7563 m²

Distance of cg from bottom fibre (y) = 0.1100 m

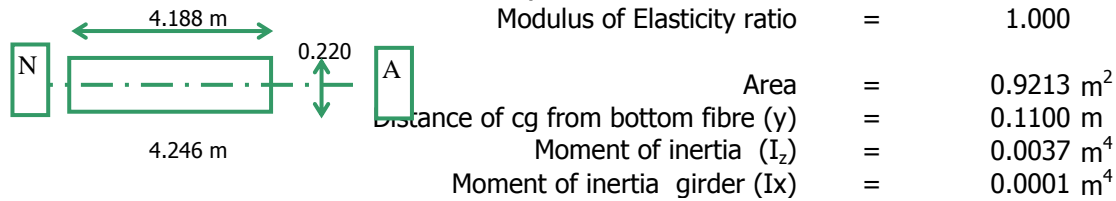
Moment of inertia (I_z) = 0.0031 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB2

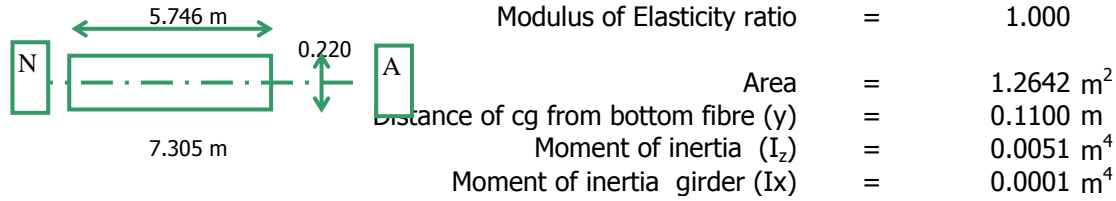
Intermediate Slab (Adjacent to First Inner Cross Girder) _TRANSLAB3

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



Intermediate Slab (Between Inner Cross Girders) _TRANSLAB4

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



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 6-2014
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 ratio 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 Obstruction		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
Traverse wind Force		F_T	=	$P_z \times A_1 \times G \times C_D$
			=	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	=	$P_z \times A_3 \times G \times C_L$
			=	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	<u>Load Case 1</u>	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	<u>Load Case 2</u>	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	<u>Load Case 3</u>	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	<u>Load Case 4</u>	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	<u>Load Case 5</u>	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	<u>Load Case 6</u>	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	<u>Load Case 7</u>	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	<u>Load Case 8</u>	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	<u>Load Case 9</u>	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 <u>Load Case 10</u> 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 <u>Load Case 11</u> 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 <u>Load Case 12</u> 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 <u>Load Case 13</u> 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 <u>Load Case 14</u> Class A 4 Lane Placed most Eccentric (Left side)			
1st Lane Z1 = $0.5 + 0 + 0.15 + 0.50/2$	=	0.90 m	From CB
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 <u>Load Case 15</u> 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 <u>Load Case 16</u> Class A 4 Lane Placed most Eccentric (Right side)			
1st Lane Z1 = $13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8$	=	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	<u>Load Case 17</u>	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	<u>Load Case 18</u>	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	<u>Load Case 19</u>	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	<u>Load Case 20</u>	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	<u>Load Case 21</u>	70R W for combination of Class A 2 Lane Eccentric on Left	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
	2nd Lane	$Z3 = 2.7 + 0.5 + 1.2$	=	4.40 m
	Class A	$Z4 = 4.4 + 1.8$	=	6.20 m
	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
				1.800
				5.300
				9.045
22	<u>Load Case 22</u>	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
Density of Concrete Girder	=	25.0 KN/m ³
Density 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	= 0.82×25	=	20.5 KN/m
Weight of Girder at End section	= 1.35×25	=	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
Weight of Intermediate X-girder	=	=	3.6 KN

Weight of the Shuttering Slab during Stage-2 Loading

Weight of Shuttering	=	5.0 KN/m
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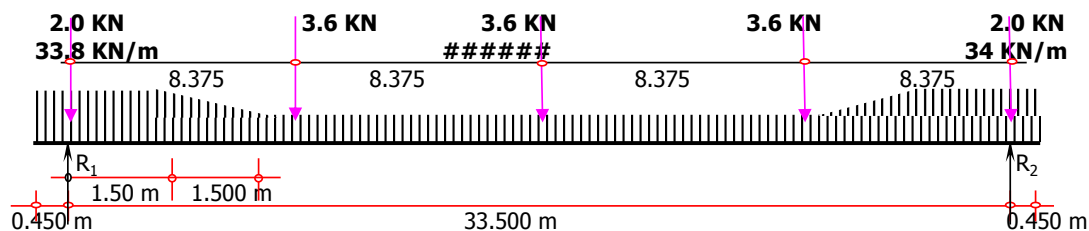
Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	= $2.8 \times 0.22 \times 26$	=	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

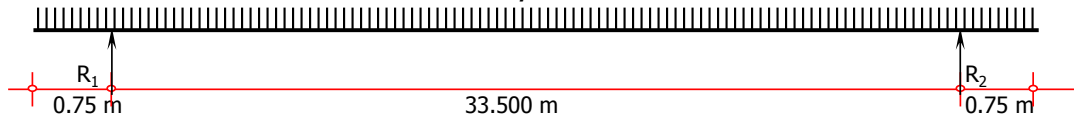


Support Reaction

$R_1 = 395.7 \text{ KN}$

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

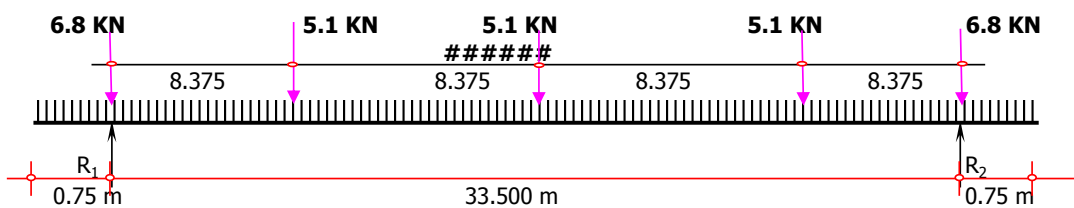
Stage-2 loading

Shuttering Load for Deck Slab**5.0 KN/m****Support Reaction**

$R_1 = 87.5 \text{ KN}$

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**

$R_1 = 289.7 \text{ KN}$

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 Momen	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
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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

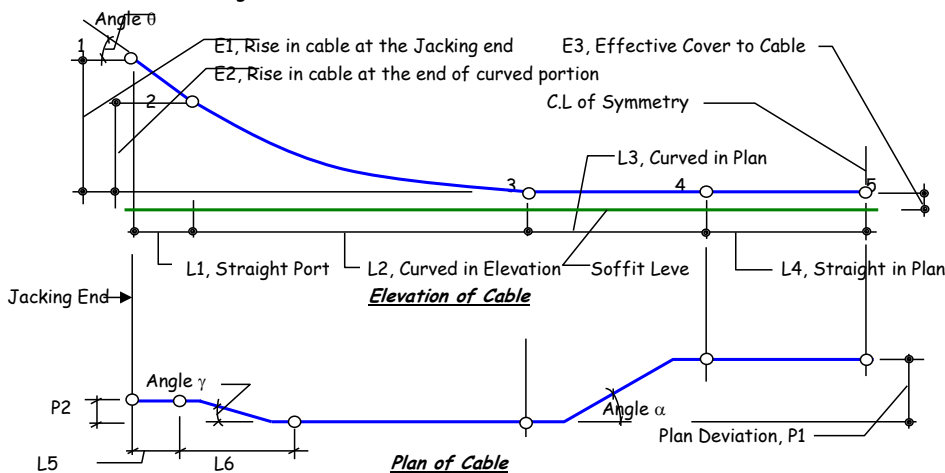
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	378.5	262.9	173.5	87.6	0.0	1.0
2nd Stage DL	271.1	205.2	134.3	68.4	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat)	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	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 Corrugated HDPE sheathing (Refer Table 6.2 , IRC:112) (Refer Table 6.2 , IRC:112)
13) Wobble Friction Coefficient	k	0.002	per metre	
14) Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		
15) Relaxation loss at 1000 hrs at 50% UTS (as % of initial stress)	Re2	0		
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 Stage stressing in 14 days	Fc _j	40.6	MPa	
21) Modulus of Elasticity of Concrete at 1st Stage stressing	Ec _j	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



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

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
	P_f	2478.218	2438.394	2408.571	2388.088	2367.605
	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
	P_f	2478.218	2438.394	2408.571	2388.088	2367.605
	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
	P_f	2340.543	2322.244	2272.127	2252.852	2233.577
	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)	$P_s \cdot \cos\theta$ (KN)	$P_s \cdot \cos\theta \cdot Y_{ord}$ (KN.m)	$P_s \cdot \sin\theta$ (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		

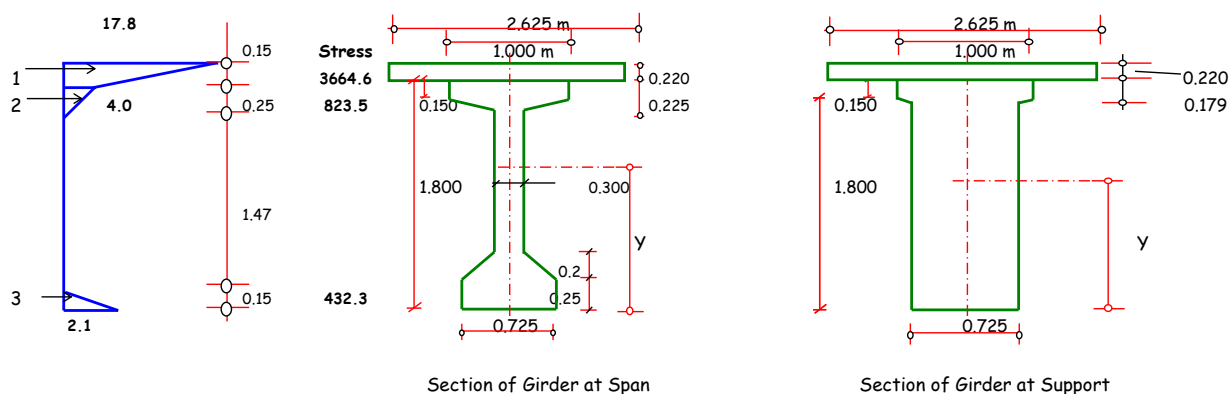
(*) Notations Used :

 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	E_c	=	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



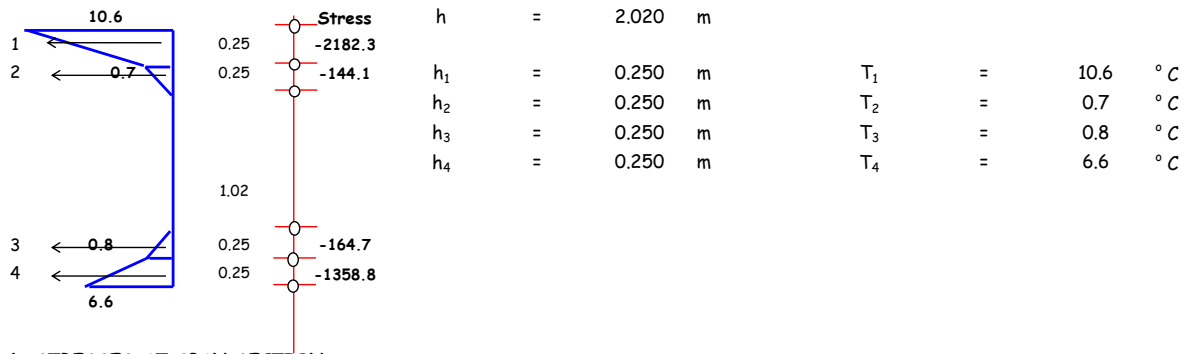
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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	0.150	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	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	0.150	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



4.2.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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 _†	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 _†	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 _†)	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 _†	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						
Stress after P-E loss	KN	377.5	377.5	377.5	377.5	377.5
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 _†	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.	%	1.846	1.959	2.142	2.241	2.315
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs.	%	83.99				
(Ref. Table 6.2, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 7 to 28 days	KN	= 83.99/100 * 2.315/100 * (0.75*183.7*83) * 1				
	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	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at 28 days	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	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at 28 days	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				
	=	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.	%	1.846	1.959	2.142	2.241	2.315
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs.	%	12.81				
(Ref. Table 6.3, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 28 to 49 days	KN	= 12.81/100 * 2.315/100 * (0.75*183.7*83) * 1				
	KN	27.2	28.8	31.5	33.0	34.1
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at 28 days	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at 49 days	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	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at 49 days	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 to Infinity	KN	= 203.2/100 * 2.315/100 * (0.75*183.7*83) * 1				
	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. Stress 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 with 75% 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

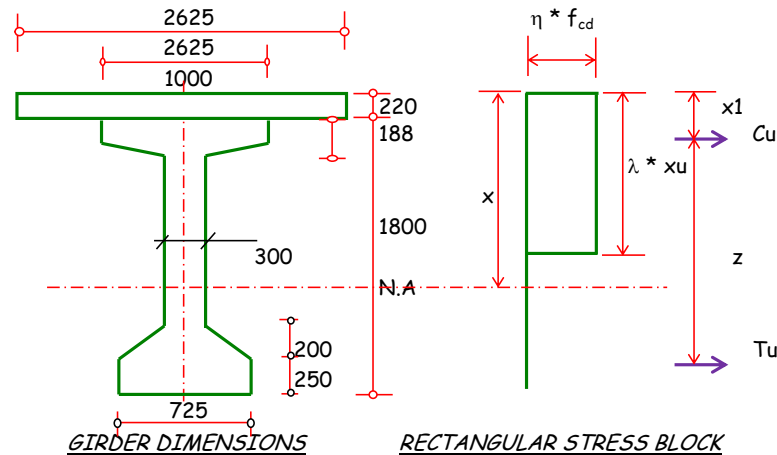
Design Parameters

Grade of Concrete of Longitudinal PSC Girder = M 45 MPa
 Modulus of Elasticity of Concrete of PSC Girder $E_{cm/g}$ = 34 GPa
 Grade 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
 Nominal Diameter of Prestressing Strands = 12.7 mm
 Nominal Area of each Strands = 98.7 mm²
 Nos of Strands = 89
 Modulus of Elasticity of Prestressing Steel E_p = 195 GPa
 Partial factor for Prestressing Steel γ_p = 1.15
 Characteristic tensile strength of Prestressing steel f_{pk} = 1860 MPa

Dia. of Untensioned Reinforcement ϕ = 12 mm
 c.g of Untensioned Reinforcement 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**Bending Moment at Different Section (KN-m) for Inner Girder**

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor ULS
	0m	4.1875m	8.375m	12.5625m	16.75m	
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	
Characteristic 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, E_s (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	
Characteristics compressive strength, f_{ck} (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, E_p (GPa)	195	195	195	195	195	
Characteristic 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
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	
Nominal Diameter of Prestressing Strands	=	12.7	mm	
Nominal Area of each Strands	=	98.7	mm ²	
Nos of Strands	=	89		8784.31

(Table 6.5 of IRC: 112 - 2011)

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	4.188	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, $f_{ctk,0.05}$ (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 $\gamma_m = 1.5$
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}/A_c or $0.2 \times f_{cd}$
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) $v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	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/($b_{wc} \times d$)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk0.5}/\gamma_m$	-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 $V_{Rdc} \min = (v_{min} + 0.15 * \sigma_{cp}) b_w * d$	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	$V_{Ns} = V_u - V_{pd}$
b) Design yield 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	1.17	1.48	1.64	1.67	1.67	= Difference between cg of tension force and compression force
e) Steel Shear capacity $V_{Rd,s} = (A_{sw} / s) z f_{ywd} \cot \theta$, KN	2726.0	3449.1	3815.2	2186.9	2187.1	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity $V_{Rd,max} = \alpha_{cw} * b * z * v1 * f_{cd} / (\cot \theta + \tan \theta)$, KN	3968.3	1882.8	2082.6	2122.3	2122.5	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of $V_{Rd,s}$ & $V_{Rd,max}$	2726.0	1882.8	2082.6	2122.3	2122.5	
h) Design value for concrete compressive strength ($f_{cd} = \alpha f_{ck} / \gamma_m$), Mpa	20.1	20.1	20.1	20.1	20.1	
i) $v1 =$	0.60	0.60	0.60	0.60	0.60	as f_{ck} is less than 80 Mpa
j) $\alpha_{cw} =$	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, $A_{sw} =$	402	402	402	226	226	in mm^2
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 Footpath/ Kerb	=	0.000 m	
4) Width of Footpath/ 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/ Diaphragm	=	0.300 m	
15) Width of End Cross girder/ Diaphragm	=	0.400 m	
16) Girder length beyond c/l of Brg. on each side	=	0.450 m	

1.2 Material Properties:

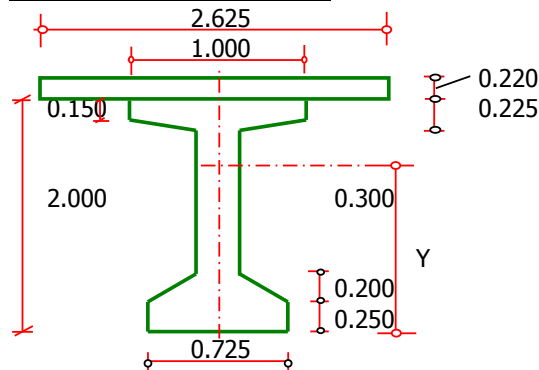
1) Concrete Grade for Deck slab	=	M 45
2) Concrete Grade for Cross girder	=	M 45
3) Concrete Grade of PSC Longitudinal girder	=	M 45
4) Reinforcement	=	Fe 500
5) Clear Cover	=	40 mm
6) Condition of Exposure	=	Moderate
7) Cable/ Strands	=	19 T 13 conforming to IS 14268
		(Stress-relieved multiply strands 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 flange	0.631	0.450	0.284	0.0114

$$I_z = 0.0291$$

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

$b_{eff} = \Sigma b_{eff,i} + b_w$	=	7.865 m
c/c Spacing of Longitudinal girder, b	=	2.625 m

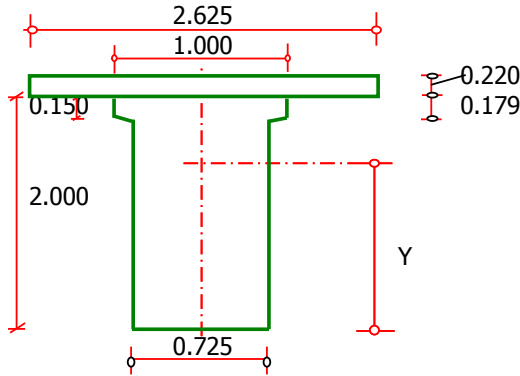
SL.No.	A	y	Ay	Ay ²	I _o	I _o + Ay ²	u
1. Deck Slab	0.57750	2.11000	1.21853	2.57109	0.002329	2.57342	4.690
2. Rectangular top Flange	0.15000	1.92500	0.28875	0.55584	0.000281	0.55613	0.300
3. Triangular top Flange	0.02625	1.82500	0.04791	0.08743	0.000008	0.08744	0.026
4. Web + Top & Bottom Rect. Haunch	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)	$Y = \Sigma(A.y) / \Sigma A$	=	1.4376 m
Moment of inertia of composite girder	$I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.8445 m ⁴

Section Properties of Girder

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

Girder Section at the Support**Calculation of Torsional Constant**

Component	Width	Depth	Area	I _z
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 flange	0.725	0.450	0.326	0.0143

$$I_z = 0.1423$$

$$\begin{aligned} \text{Modulus of Elasticity of Deck Slab} &= 34313 \text{ MPa} \\ \text{Modulus of Elasticity of Girder} &= 34313 \text{ MPa} \end{aligned}$$

$$\begin{aligned} b_{eff} &= \sum b_{eff,i} + b_w = 8.205 \text{ m} \\ \text{c/c Spacing of Longitudinal girder, } b &= 2.625 \text{ m} \end{aligned}$$

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. Rectangular 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

$$\begin{aligned} \text{Area} & \Sigma A = 2.0728 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} & Y = \Sigma(A \cdot y) / \Sigma A = 1.3293 \text{ m} \\ \text{Moment of inertia of end intermediate girder} & I_z = \Sigma(I_o + A \cdot y^2) - \Sigma A \cdot Y^2 = 1.0107 \text{ m}^4 \end{aligned}$$

Section Properties of Girder

$$\begin{aligned} \text{Area} & \Sigma A = 1.4953 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} & Y = \Sigma(A \cdot y) / \Sigma A = 1.0278 \text{ m} \\ \text{Moment of inertia of end intermediate girder} & I_z = \Sigma(I_o + A \cdot y^2) - \Sigma A \cdot Y^2 = 0.5204 \text{ m}^4 \end{aligned}$$

1.1 Summary of Section Properties of Longitudinal Girder (Outer)**A. Section Properties of Composite Section at the Span**

$$\begin{aligned} \text{Area} &= 1.4575 \text{ m}^2 & Y_b &= 1.4376 \text{ m} & Y_{ts} &= 0.7824 \text{ m} \\ I_z &= 0.8445 \text{ m}^4 & Z_b &= 0.5875 \text{ m}^3 & Z_{ts} &= 1.0794 \text{ m}^3 \\ & & & & Y_{tg} &= 0.5624 \text{ m} \\ & & & & Z_{tg} &= 1.5016 \text{ m}^3 \end{aligned}$$

B. Section Properties of Girder at the Span

$$\begin{aligned} \text{Area} &= 0.8800 \text{ m}^2 & Y_b &= 0.9963 \text{ m} & Y_t &= 1.0037 \text{ m} \\ I_z &= 0.4097 \text{ m}^4 & Z_b &= 0.4112 \text{ m}^3 & Z_t &= 0.4082 \text{ m}^3 \end{aligned}$$

C. Section Properties of Composite Section at the Support

$$\begin{aligned} \text{Area} &= 2.0728 \text{ m}^2 & Y_b &= 1.3293 \text{ m} & Y_{ts} &= 0.8907 \text{ m} \\ I_z &= 1.0107 \text{ m}^4 & Z_b &= 0.7603 \text{ m}^3 & Z_{ts} &= 1.1347 \text{ m}^3 \\ & & & & Y_{tg} &= 0.6707 \text{ m} \\ & & & & Z_{tg} &= 1.5069 \text{ m}^3 \end{aligned}$$

D. Section Properties of Girder at the Support

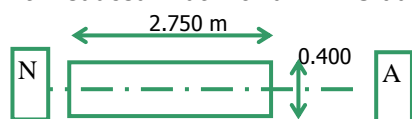
$$\begin{aligned} \text{Area} &= 1.4953 \text{ m}^2 & Y_b &= 1.0278 \text{ m} & Y_t &= 0.9722 \text{ m} \\ I_z &= 0.5204 \text{ m}^4 & Z_b &= 0.5063 \text{ m}^3 & Z_t &= 0.5353 \text{ m}^3 \end{aligned}$$

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphragm 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.)



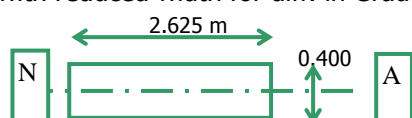
Modulus of Elasticity ratio = 1.000

Area = 1.1000 m²

Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0147 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**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 m²

Distance of cg from bottom fibre (y) = 0.2000 m

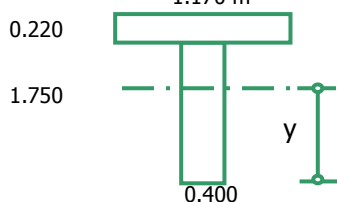
Moment of inertia (I_z) = 0.0140 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**End Cross Girder****_ENDCG**

(with reduced width for diff. in

beff = Min{ Σ beff,i + bw, b} = 1.170 m

Modulus of Elasticity ratio = 1.000

Grade of Conc.)

Area = 0.9574 m²

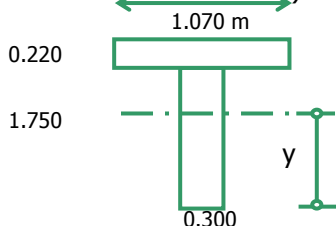
Distance of cg from bottom fibre (y) = 1.1398 m

Moment of inertia of end x-girder (I_z) = 0.3623 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴

Intermediate/ Inner Cross Girder

(with reduced width for diff. in

Grade of Conc.)



$$b_{eff} = \text{Min}\{\sum b_{eff,i} + b_w, b\}$$

Modulus of Elasticity ratio

_INTCG

= 1.070 m

= 1.000

Area = 0.7604 m²

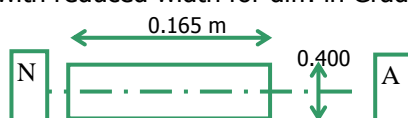
Distance of cg from bottom fibre (y) = 1.1799 m

Moment of inertia of end x-girder (I_z) = 0.2926 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

Cantilever Slab (At Expansion End)

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



Modulus of Elasticity ratio = 1.000

Area = 0.0660 m²

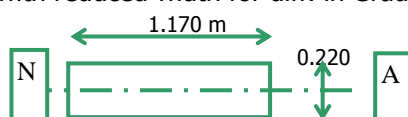
Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2574 m²

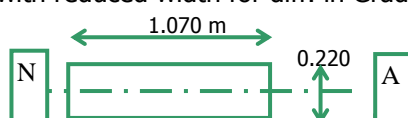
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0010 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2354 m²

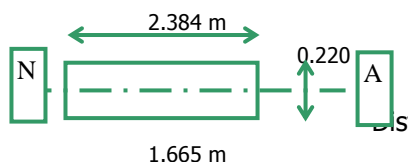
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSCANT2**Intermediate Slab (Next to End Cross Girder)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.5244 m²

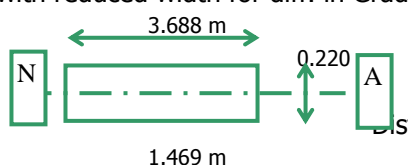
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0021 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB1**Intermediate Slab (At Splayed/ Tapered Portion)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.8113 m²

Distance of cg from bottom fibre (y) = 0.1100 m

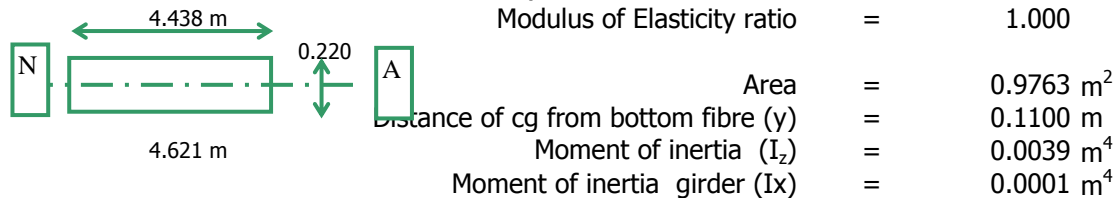
Moment of inertia (I_z) = 0.0033 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB2

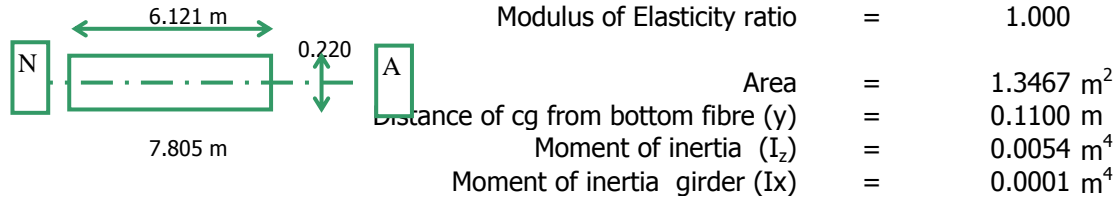
Intermediate Slab (Adjacent to First Inner Cross Girder) _TRANSLAB3

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



Intermediate Slab (Between Inner Cross Girders) _TRANSLAB4

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



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 6-2014
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 ratio 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**Wind speed & design wind pressure for Basic wind speed of 39m/s**

H (m)	Plain Terrain		Terrain with Obstruction		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
Traverse wind Force		F_T	=	$P_z \times A_1 \times G \times C_D$
			=	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	=	$P_z \times A_3 \times G \times C_L$
			=	536.3 KN
UDL on girder			=	1.1 KN/m ²
Outer girder			=	2.8 KN/m
Inner girder			=	3.0 KN/m
				2.82 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	<u>Load Case 1</u>			
	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	<u>Load Case 2</u>			
	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	<u>Load Case 3</u>			
	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	<u>Load Case 4</u>			
	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	<u>Load Case 5</u>			
	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	<u>Load Case 6</u>			
	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	<u>Load Case 7</u>			
	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	<u>Load Case 8</u>			
	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	<u>Load Case 9</u>			
	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 <u>Load Case 10</u> 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 <u>Load Case 11</u> 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 <u>Load Case 12</u> 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 <u>Load Case 13</u> 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 <u>Load Case 14</u> Class A 4 Lane Placed most Eccentric (Left side)			
1st Lane Z1 = $0.5 + 0 + 0.15 + 0.50/2$	=	0.90 m	From CB
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 <u>Load Case 15</u> 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 <u>Load Case 16</u> Class A 4 Lane Placed most Eccentric (Right side)			
1st Lane Z1 = $13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8$	=	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	<u>Load Case 17</u>	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	<u>Load Case 18</u>	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	<u>Load Case 19</u>	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	<u>Load Case 20</u>	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	<u>Load Case 21</u>	70R W for combination of Class A 2 Lane Eccentric on Left	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
	2nd Lane	$Z3 = 2.7 + 0.5 + 1.2$	=	4.40 m
	Class A	$Z4 = 4.4 + 1.8$	=	6.20 m
	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
				1.800
				5.300
				9.045
22	<u>Load Case 22</u>	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
Density of Concrete Girder	=	25.0 KN/m ³
Density 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	= 0.88×25	=	22.0 KN/m
Weight of Girder at End section	= 1.495×25	=	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
Weight of Intermediate X-girder	=	=	4.1 KN

Weight of the Shuttering Slab during Stage-2 Loading

Weight of Shuttering	=	5.0 KN/m
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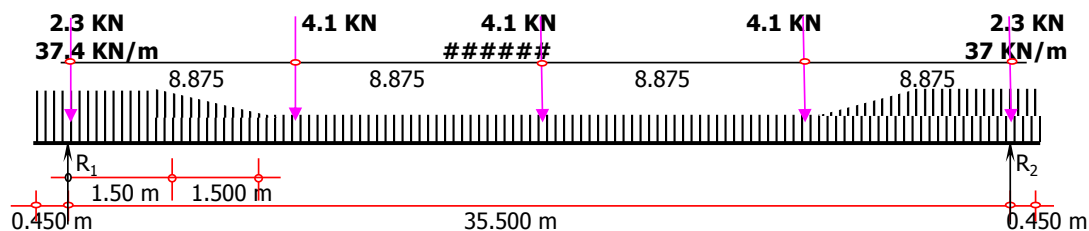
Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	= $2.8 \times 0.22 \times 26$	=	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

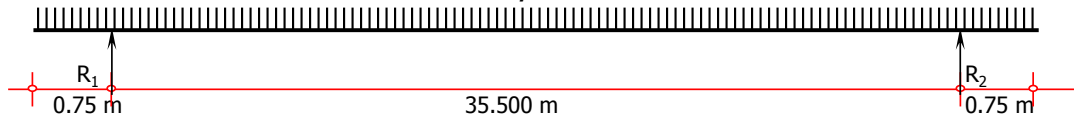


Support Reaction

$R_1 = 450.3 \text{ KN}$

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

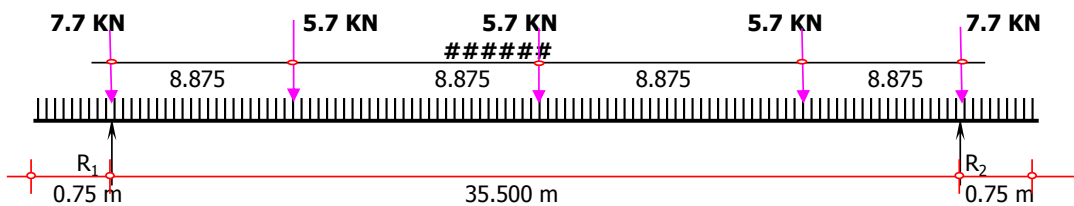
Stage-2 loading

Shuttering Load for Deck Slab**5.0 KN/m****Support Reaction**

$R_1 = 92.5 \text{ KN}$

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 Momen	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**

$R_1 = 307.3 \text{ KN}$

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
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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

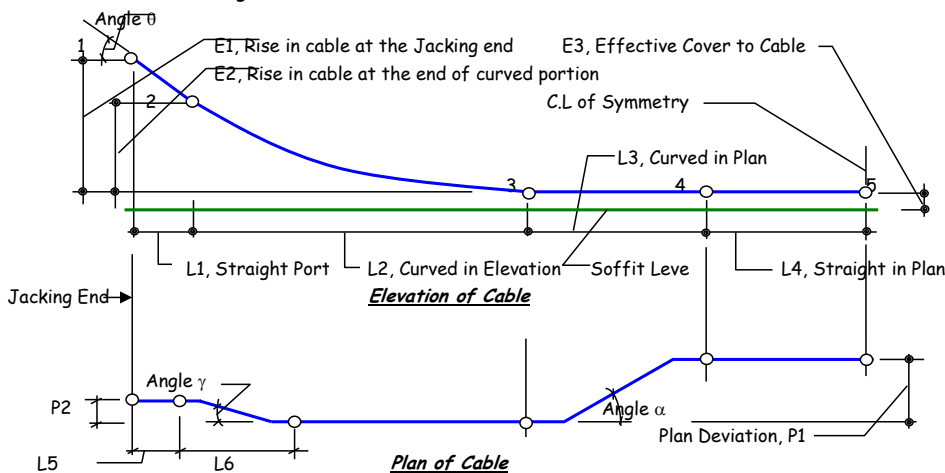
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	431.3	299.0	197.3	99.7	0.0	1.0
2nd Stage DL	287.8	218.0	142.5	72.7	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat)	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	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 Corrugated HDPE sheathing (Refer Table 6.2 , IRC:112)
13) Wobble Friction Coefficient	k	0.002	per metre	
14) Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		
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 Stage stressing in 14 days	Fc _j	40.6	MPa	
21) Modulus of Elasticity of Concrete at 1st Stage stressing	Ec _j	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



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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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

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
	P_f	2478.218	2436.692	2406.951	2385.289	2363.627
	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
	P_f	2478.218	2436.692	2406.951	2385.289	2363.627
	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
	P_f	2478.222	2457.612	2403.897	2382.316	2360.734
	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)	$P_s \cdot \cos\theta$ (KN)	$P_s \cdot \cos\theta \cdot Y_{ord}$ (KN.m)	$P_s \cdot \sin\theta$ (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
	<i>Eff. Eccentricity of Cable</i>					0.2007		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.4814		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6236		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6555		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6559		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					0.0000		

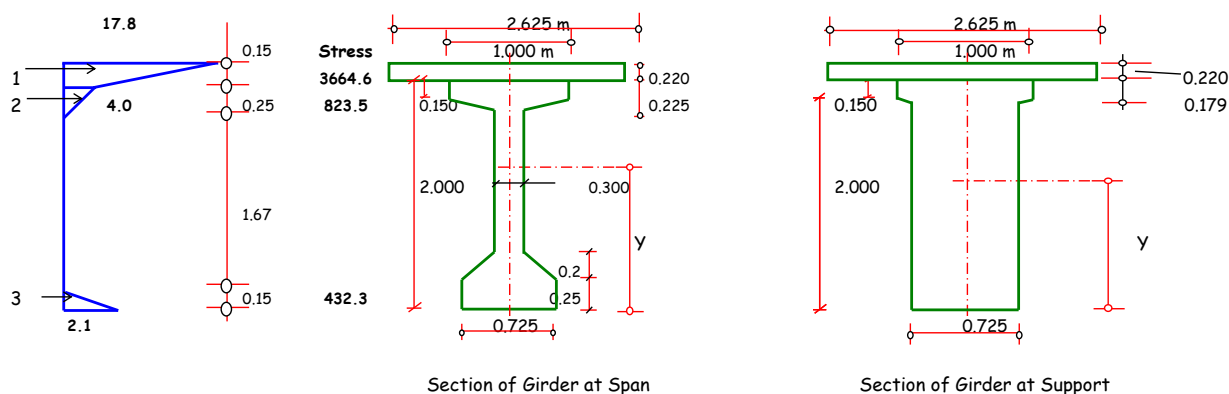
(*) Notations Used :

 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	E_c	=	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



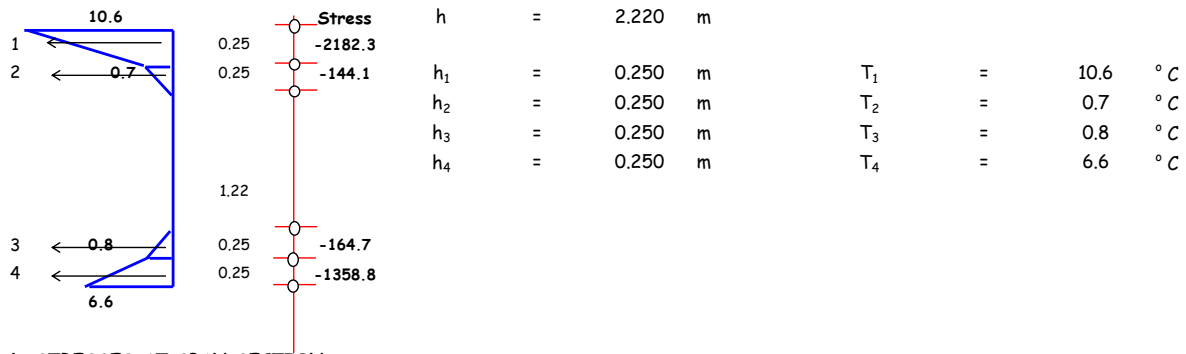
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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	0.150	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	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	0.150	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



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	0.698	-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			

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 _†	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 _†	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 _†)	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 _†	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						
Stress after P-E loss	KN	357.0	357.0	357.0	357.0	357.0
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 _†	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.	%	1.869	1.988	2.168	2.265	2.337
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs.	%	83.99				
(Ref. Table 6.2, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 7 to 28 days	KN	= 83.99/100 * 2.337/100 * (0.75*183.7*82) * 1				
	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	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at 28 days	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	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at 28 days	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				
	=	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 Stage 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.	%	1.869	1.988	2.168	2.265	2.337
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs.	%	12.81				
(Ref. Table 6.3, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 28 to 49 days	KN	= 12.81/100 * 2.337/100 * (0.75*183.7*82) * 1				
	KN	26.9	28.6	31.2	32.6	33.7
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at 28 days	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at 49 days	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	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at 49 days	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				
	=	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 to Infinity	KN	= 203.2/100 * 2.337/100 * (0.75*183.7*82) * 1				
	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. Stress 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 with 75% 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 :-

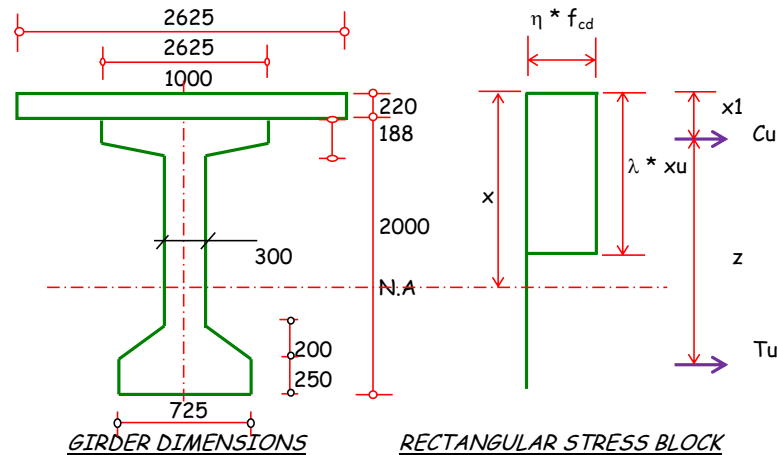
Span Length (c/c of Brg.) = 35.500 m

Design Parameters

Grade of Concrete of Longitudinal PSC Girder = M 45 MPa
 Modulus of Elasticity of Concrete of PSC Girder $E_{cm/g}$ = 34 GPa
 Grade 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
 Nominal Diameter of Prestressing Strands = 12.7 mm
 Nominal Area of each Strands = 98.7 mm²
 Nos of Strands = 88
 Modulus of Elasticity of Prestressing Steel E_p = 195 GPa
 Partial factor for Prestressing Steel γ_p = 1.15
 Characteristic tensile strength of Prestressing steel f_{pk} = 1860 MPa
 Dia. of Untensioned Reinforcement ϕ = 12 mm
 c.g of Untensioned Reinforcement 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**Bending Moment at Different Section (KN-m) for Inner Girder**

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor ULS
	0m	4.4375m	8.875m	13.3125m	17.75m	
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	
Characteristic 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, E_s (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	
Characteristics compressive strength, f_{ck} (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, E_p (GPa)	195	195	195	195	195	
Characteristic 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	
Nominal 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, $f_{ctk,0.05}$ (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 $\gamma_m = 1.5$
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}/A_c or $0.2 \times f_{cd}$
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) $v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	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/($b_{wc} \times d$)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk0.5}/\gamma_m$	-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 $V_{Rdc} \min = (v_{min} + 0.15 * \sigma_{cp}) b_w * d$	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	$V_{Ns} = V_u - V_{pd}$
b) Design yield 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 $V_{Rd,s} = (A_{sw} / s) z f_{ywd} \cot \theta$, KN	3247.0	3974.7	4306.2	2464.1	2464.6	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity $V_{Rd,max} = \alpha_{cw} * b * z * v1 * [f_{cd} / (\cot \theta + \tan \theta)]$, KN	4723.7	2169.8	2350.7	2391.3	2391.8	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of $V_{Rd,s}$ & $V_{Rd,max}$	3247.0	2169.8	2350.7	2391.3	2391.8	
h) Design value for concrete compressive strength ($f_{cd} = \alpha f_{ck} / \gamma_m$), Mpa	20.1	20.1	20.1	20.1	20.1	
i) $v1 =$	0.60	0.60	0.60	0.60	0.60	as f_{ck} is less than 80 Mpa
j) $\alpha_{cw} =$	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, $A_{sw} =$	402	402	402	226	226	in mm^2
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)

$$f_b = 0.48 * f_{cj} * (A2/A1)^{0.5} \text{ or } 0.8 * f_{cj} \text{ whichever is lesser}$$

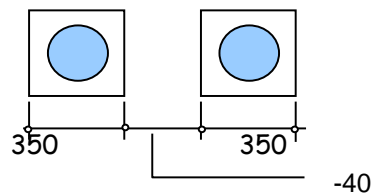
Where :

$$f_b = \text{Allowable stress behind anchorages}$$

f_{cj} = Concrete Strength at the time of stressing (in Mpa)

A_1 = Bearing area of anchorage (Eq. Square of dimension "a")

A2 = As defined in Clause 7.3 of IRC : 18 (Eq. Square of dimension "b")



$$f_{cj} = 40.59 \text{ Mpa}$$

$a = 350 \text{ mm}$

b = 300 mm

$$f_b = 16.7 \text{ Mpa}$$

B. Force acting at the anchorage

a) Force per cable at Jacking end after friction & Slip	=	2235.6 KN
b) Elastic Shortening Loss per cable	=	77.1 KN
c) Relaxation Loss per cable	=	136.1 KN
d) Net force after instantaneous 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 $0.2 \cdot Y_o$ to $2.0 \cdot Y_o$.

$$0.2 \cdot y_o = 30 \text{ mm} \qquad 2.0 \cdot y_o = 300 \text{ mm}$$

$$\text{Area of Stirrups required} = F_{bst}/0.87 \cdot f_v = \mathbf{912 \text{ mm}^2}$$

Provided, 6-legged T-16 in 2 layers, in the form of spiral, $A_s = 2412.0 \text{ mm}^2$

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_0	=	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 calculation:-***(a) Top Slab***

Thickness of wearing coat	=	0.1 m
Load (UDL) due to WC	0.1×22	= 2.2 kN/m width
Thickness of earth/profile corrective course fill	=	0.0 m
Load (UDL) due to earth fill	0×20	= 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)			Submerged Pressure	Water Pressure
		Dry Earth Pressure				
0.388	0.000	$0.5 \times 20 \times 0.3875$	=	3.9	0.0	0.0
0.288	0.675	$0.5 \times 20 \times 0.675$	=	6.8	2.9	0.0
0.300	0.975	$0.5 \times 20 \times 0.975$	=	9.8	4.4	0.0
2.900	3.875	$0.5 \times 20 \times 3.875$	=	38.8	18.9	26.0
0.300	4.175	$0.5 \times 20 \times 4.175$	=	41.8	20.4	29.0
0.300	4.475	$0.5 \times 20 \times 4.475$	=	44.8	21.9	32.0

Live Load Surcharge calculation:-

Equivalent height	=	1.2 m
Uniform Intensity of loading	$1.2 \times 0.5 \times 20$	= 12.0 KN/m ²

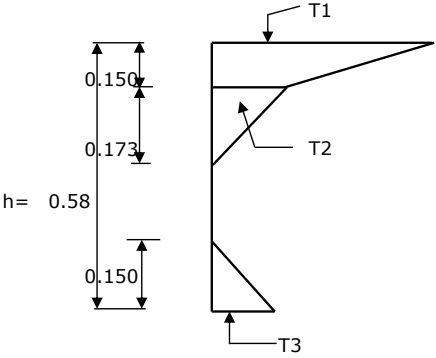
Braking Load calculation:-

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:-

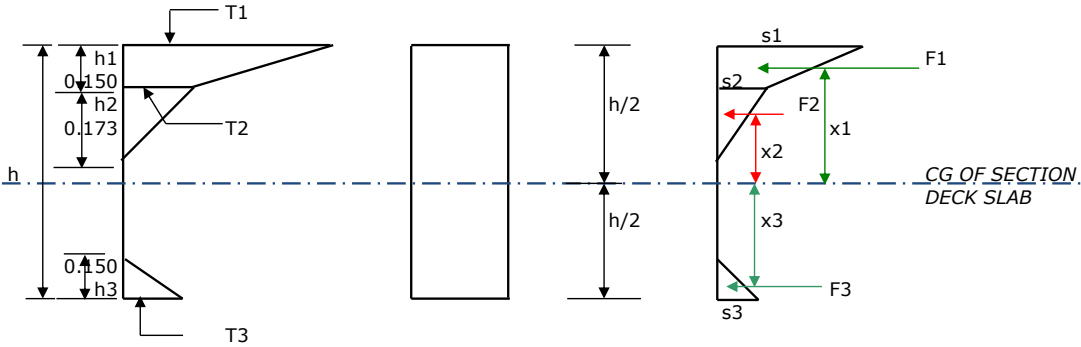
The top slab is designed for the effects of the distribution of the temperature across the deck depth as given in the sketch below.



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	$\alpha =$	0.000012	/ °C
Stress	$\sigma =$	$E \alpha t$	
Force	$F =$	$F1 + F2 + F3$	
Moment	$M =$	$F1x1 + F2x2 - F3x3$	

Generalised Temperature & corresponding Force Diagram



Force & Moment calculation:-

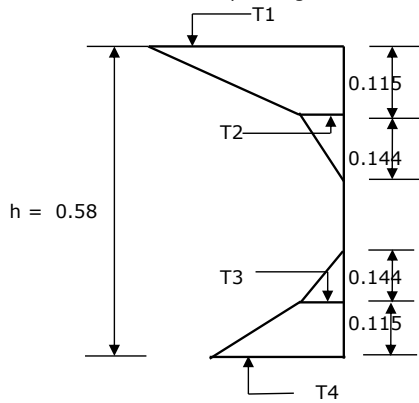
Member	h	h1	h2	h3	T1	T2	T3
	(m)	(m)	(m)	(m)	°C	°C	°C
Top slab	0.575	0.150	0.173	0.150	17.80	4.00	2.10

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

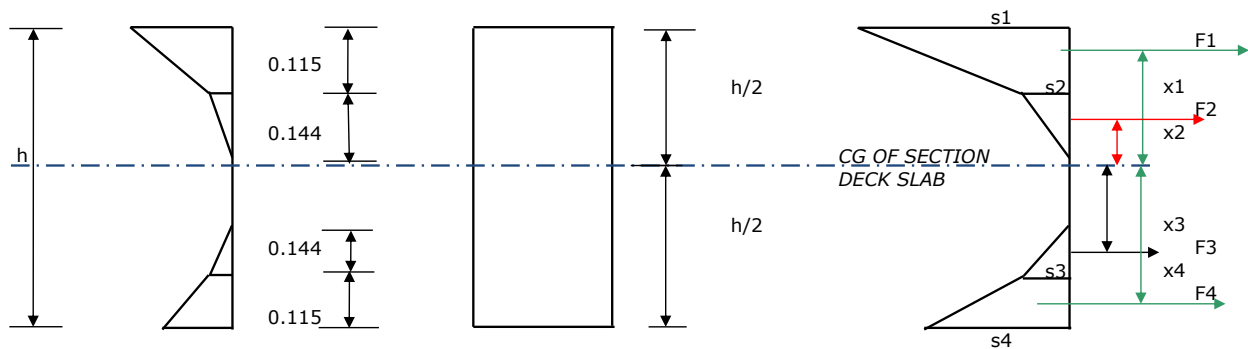
Member	cg from Top	cg from Bottom	cg of Top Block from Top	cg of Mid Block from Top	cg of Bottom Block from Bottom	x1	x2	x3	Moment
						m	m	m	Tm
Top slab	0.2875	0.2875	0.0592	0.2075	0.0500	0.2283	0.0800	0.2375	7.0

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	$\alpha =$	$1.2E-05$	$/ ^\circ C$
Stress	$\sigma =$	$E \alpha t$	
Force	$F =$	$F1 + F2 + F3 + F4$	
Moment	$M =$	$F1x1 + F2x2 - F3x3 - F4x4$	

Generalised Temperature & corresponding Force Diagram**Force & Moment calculation:-**

Member	h (m)	$h1$ (m)	$h2$ (m)	$h3$ (m)	$h4$ (m)	$T1$ $^\circ C$	$T2$ $^\circ C$	$T3$ $^\circ C$	$T4$ $^\circ C$
Top slab	0.575	0.115	0.144	0.144	0.115	10.60	0.70	0.80	6.60

Member	$s1$ $T / m2$	$s2$ $T / m2$	$s3$ $T / m2$	$s4$ $T / m3$	$F1$ T	$F2$ T	$F3$ T	$F4$ T	Force 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 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

Total load on 70 R-Wheel
Wearing coat thickness

=
=

1220
0.100

kN
m

with I.F =

0.22
1.22

Load	Dimension of tyre		Load kN	Dist. Betwn. each load	Final Load kN
	Length	Width			
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
= Effective Span
= C/C span
= Width Of Slab
= 12.85
= 6.00
= 6.43
= 13.50
= 2.25

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

[illegible]

Effective Live Load for 70R-Bogie Load

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

[illegible]

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

Total load on 70 R-Track = 770 kN with I.F = 0.10
Wearing coat thickness = 0.100 m 1.10

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
a constant,
α = depends on value = 2.60
of b / L o
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

[illegible]

Effective Live Load for Class-A Wheel Load

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

[illegible][illegible]

	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				

[illegible]

Summary of Shear force (SF) & Bending Moment (BM):-

	1	2	3	
4				7
5				8
6				9
	10	11	12	

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 q 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
Grade of Steel Reinforcement	=	Fe 500	MPa	Page:235 of IRC:112-2011 & Table 6.5 of IRC:112-2011
Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa
Design yield of strength of shear Reinforcement	f_{ywd}	=	400	MPa
Modulus of Elasticity of Steel	E_s	=	200000	MPa
Modulus of Elasticity of Concrete, for Short Term loading	E_{cm}	=	32308	MPa
for Long Term loading	$E_{cm}^{'}$	=	13842	MPa
			13726	MPa
			13576	MPa
			16154	MPa
				for Top/Deck slab
				for Outer wall
				for Inner wall
				for Bottom/Base slab
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.8	MPa
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)	$x_{u,max}/d$	=	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
			1.35	for Outer wall
			1.38	for Inner wall
			1.00	for Bottom/Base slab

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Refer:- Creep coefficient calculation

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:-

$A_{s,min} = \max [0.26*f_{ctm}/f_{yk}*b_t*d , 0.0013*b_t*d]$

where,
 b_t = mean width of tension zone

Refer IRC:112, cl 16.5.1.1)

Maximum Reinforcement:-

Tension Reinf.	\leq	$0.025*A_c$	At section other than lap
Tension + Compression Reinf.	\leq	$0.04*A_c$	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					x _{u,max}	x _u =0.87f _{yk} *A _{st} /0.362f _{ck} *b	Check	z=(d-0.416x _u)	A _{st,cal} =M _{Ed} /0.87f _{yk} *z	A _{st,max}	Check	A _{st,cal} <A _{st,pro} <A _{st,max}	Check	ΔF _d = 0.5V _{Ed}	M _{Ed} /Z+	M _{Ed} /Z+ΔF _d	M _{Ed} =0.87 * f _{yk} * A _{st} * z	M _{Rd} /Z	Check	
							Dia.	Spacing	Dia.	Spacing	A _{st, pro}							A _{st,min}		A _{st,max}						M _{Rd} /Z	
							Tm	mm	mm	mm	mm							mm		mm						mm	mm ²
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			Check
				Dia.	Spacing	$A_{st, pro}$	
			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}	v_l	$V_{Ed,max} = \alpha_{cw} * b_w * z * v_l * f_{td}/2$	Check	$v = 0.6[1 - f_{td}/310]$	$0.5 * b_w * d * v * f_{td}$	$V_{Ed} \leq 0.5 * b_w * d * v * f_{td}$	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	β=a _s /z _d	βV _{Ed}	K=Min[1+(200/d) ^{0.5} , 2]	ρ1=Min[Asl/b _w d, 0.02]	V _{min} =0.031 K ^{3/2} f _{ck} ^{1/2}	σ _{cp}	V _{Red,1} =(V _{min} +0.15σ _{cp})b _w d	V _{Red,2} = [0.12K(80 ρ1 f _{ck}) ^{0.33} +0.15σ _c] _ρ b _w d	V _{Red,c} =Max[V _{Red,1} & V _{Red,2}]	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}		Overall depth, D		A _{st, pro}	I	Y/2	Stress 'σ _c '	f _{ctm,fl}	Cracked/ Uncracked		Modular ratio 'm'	N.A depth 'x'	I _{NA}	Comp. stress	Per. stress	Check	Tensile stress	Per. stress	Check
			Tm	mm	mm	mm	mm ²	mm ⁴	mm	Mpa	Mpa				mm	mm ⁴	Mpa	Mpa		Mpa	Mpa	
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}	b	Overall depth, D	d	A _{st, pro}	I	Y/2	Stress 'σ _c '	f _{ctm,fl}	Cracked/UnCracked		Modular ratio 'm'	N.A depth 'x'	I _{NA}	Comp. stress	Per. stress	Check	Tensile stress	Per. stress	Check
			Tm	mm	mm	mm	mm ²	mm ⁴	mm	Mpa	Mpa				mm	mm ⁴	Mpa	Mpa		Mpa	Mpa	
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	$A_{s,min} = k_c * k * f_{ct} / \sigma_c$	$A_{st,pro}$ or $A_{s,pro}$	Check	Bar dia	Clear cover 'c'	$h_{c,eff} = \min[2.5(h-d), (h-x/3), h/2]$	$A_{c,eff} = h_{c,eff} * b$	$\rho_{peff} = A_s / A_{c,eff}$	$S_{max} = \frac{3.4c + 0.17\phi}{\rho_{peff}}$	σ_{sc}	N.A depth 'x'	k_t	$\alpha_e = E_s / E_{cm}$	$\epsilon_{sm} - \epsilon_{cm}$	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	;

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

```

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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

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

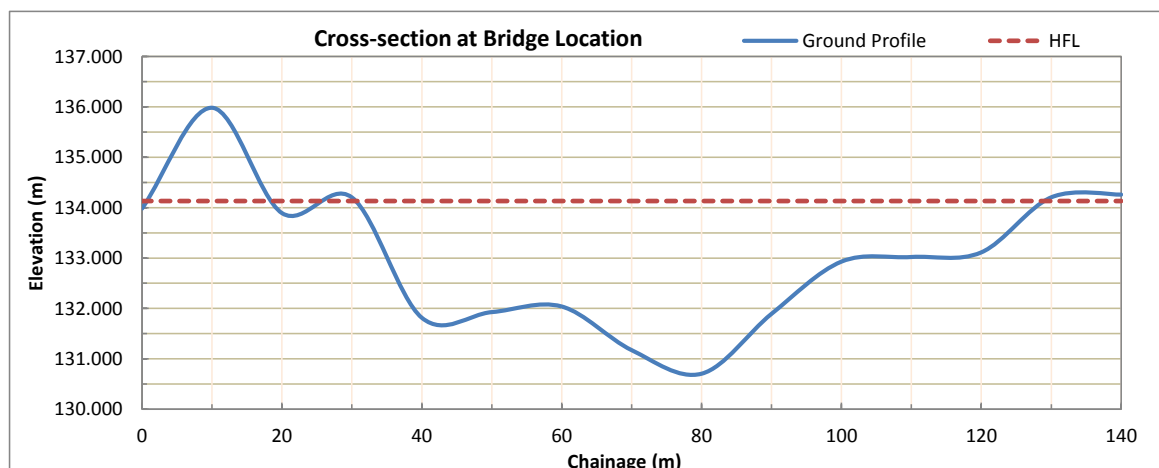
Bridge Data:-

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)	$A \times V$	=	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% safety 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 , $4.8 \sqrt{Q}$ = **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, $d_{sm} = 1.34 \times (D_b^2 / K_{sf})^{1/3}$
 D_b = Design discharge per metre width = 8.977 Cumecs / m
 K_{sf} = Silt factor = 1.500
 d_{sm} = **5.056 m**
Maximum Scour Depth:
For Piers, **$2.0 \times d_{sm}$** = 10.112 m
For Abutments, **$1.27 \times d_{sm}$** = 6.421 m

Maximum Scour Level :

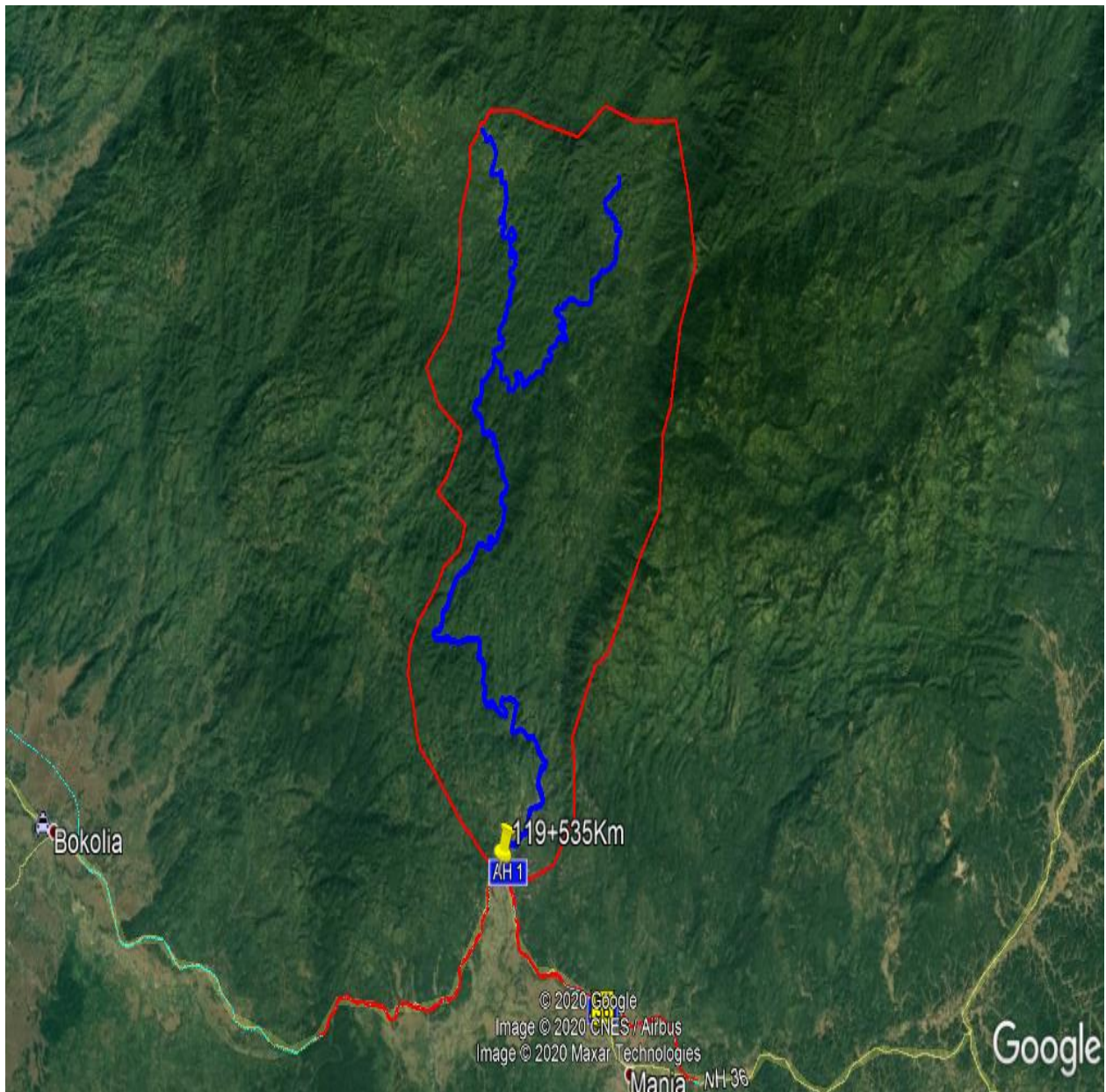
Maximum Scour Level = HFL - Maximum Scour Depth
For Piers, = 124.439 m
For Abutments, = 128.130 m

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 (3x30) at design Ch.128+977KM**

Hydrological Calculation for Major Bridge : Yammuna River

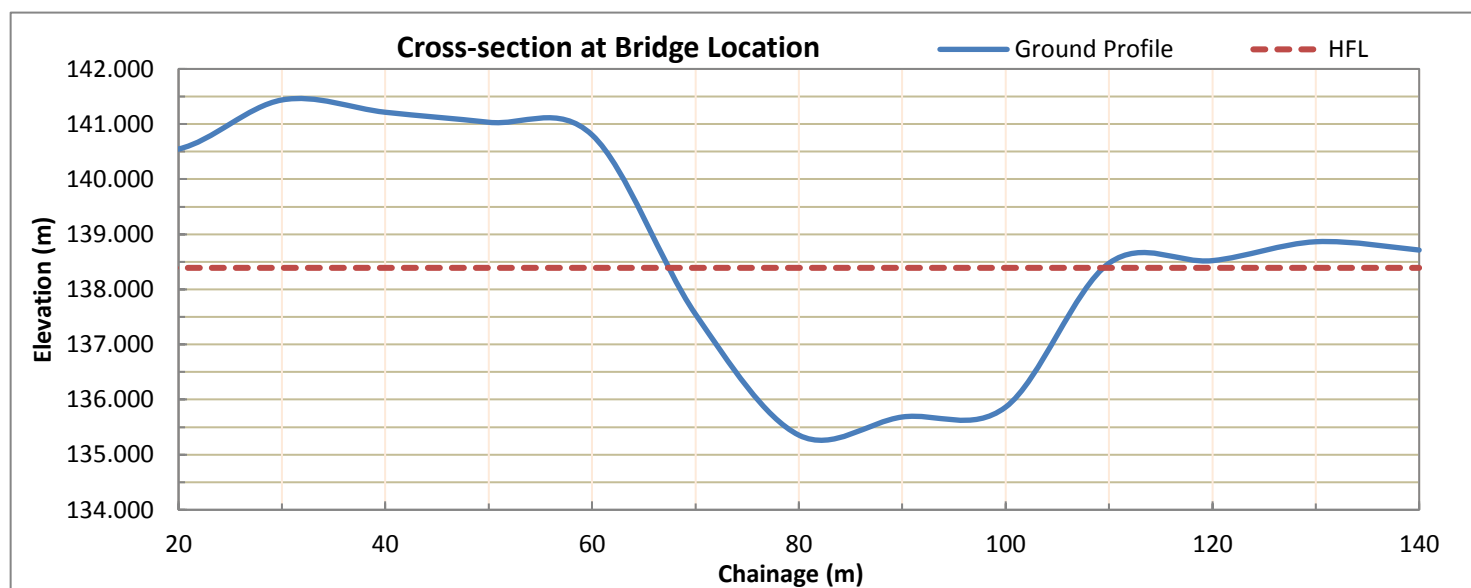
Bridge Data:-

Road Name	=	Daboka-Manja
Bridge Name/ID No.	=	-
Name of Nallah / Stream / River	=	Yammuna River
Existing Ch.	=	-
Design Ch.	=	128+977 KM
Type of Proposed bridge	=	PSC I Girder
Span Arrangement of proposed bridge	=	3x30
Existing FRL	=	-
Design FRL	=	152.000 m
HFL at bridge location	=	138.390 m
Rugosity Co-efficient,n (Reff: IRC: SP: 13 - 2004 , Table 5.1)	=	0.04
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	140.594	140.594	138.3903					
2	10	140.585	140.585	138.3903					
3	20	140.547	140.547	138.3903					
5	30	141.437	141.437	138.3903					
7	40	141.214	141.214	138.3903					
9	50	141.032	141.032	138.3903					
11	60	140.802	140.802	138.3903					
13	70	137.550	137.550	138.3903	0.840306	0.420	2.101	5.070	5.000
15	80	135.355	135.355	138.3903	3.035306	1.938	19.378	10.238	10.000
17	90	135.684	135.684	138.3903	2.706306	2.871	28.708	10.005	10.000
19	100	135.863	135.863	138.3903	2.527306	2.617	26.168	10.002	10.000
21	110	138.474	138.474	138.3903		1.264	6.318	5.602	5.000
23	120	138.521	138.521	138.3903					
25	130	138.864	138.864	138.3903					
27	140	138.713	138.713	138.3903					
TOTAL							82.673	40.918	40.000



Hydraulic Mean Radius (R)	A / P	=	2.020 m
Velocity of Water (V)	$(1/n)(R)^{2/3}(S)^{1/2}$	=	4.148 m/sec
Discharge (Q)	$A \times V$	=	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	:	128+977 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, $Q = C M^{3/4}$
 $C = 14 - 19$ 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 $Q = C M^{2/3}$
 $C = 6.8$ 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	$I_o = (F / T) \times (T + 1) / (1 + 1) =$	=	88.54 mm / Hr
Time of Concentration	$t_c = (0.87 \times L^3 / H)^{0.385}$	=	4.35 Hrs
Critical Rainfall Intensity	$I_c = I_o \times (2 / (1 + t_c))$	=	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	$Q = 0.028 \times P \times f \times A \times I_c$	=	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% safety 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	=	82.67 m ²
Velocity of Water,	V	=	4.563 m/sec
Observed HFL	HFL	=	138.390 m
BED Level	OGL	=	135.355 m
Depth of Water	d	=	3.035 m
Available Water way	a	=	181.261 m ²
Using Molesworth's Formula, $\{(V^2/17.9)+0.015\} \times \{(A/a)^2-1\}$		=	-0.932906 ~ 0.000m
The negative afflux obtained by above formula reflects that there is no heading up of water due to obstruction, thus its considered as zero.			

7.0 Vertical Clearance : (Refer IRC : 5 - 1998, Clause : 106.2.1)

Design Discharge,	Q	=	377.200 Cumecs
Vertical Clearance (adopted),		=	1.20 m

Minimum Soffit Level of Deck Slab = HFL + Afflux + Vertical Clearance = 139.590 m

Hydrological Calculation for Major Bridge : Yammuna River

Name of Nallah / Stream / River : Yammuna River

8.0 Linear Waterway :

As per Lacy's formula required linear water way , $4.8 \sqrt[3]{Q}$ = **93.224 m**
HFL Spread including afflux at Proposed Bridge Location = **40.000 m**
Adopted obstructed linear waterway = 87.700 m
Restriction to the linear waterway = 5.9 % < **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, $d_{sm} = 1.34 \times (D_b^2 / K_{sf})^{1/3}$
 D_b = Design discharge per metre width = 12.259 Cumecs / m
 K_{sf} = Silt factor = 1.500
 $d_{sm} =$ = **6.224 m**
Maximum Scour Depth:
For Piers, **$2.0 \times d_{sm}$** = 12.447 m
For Abutments, **$1.27 \times d_{sm}$** = 7.904 m

Maximum Scour Level :

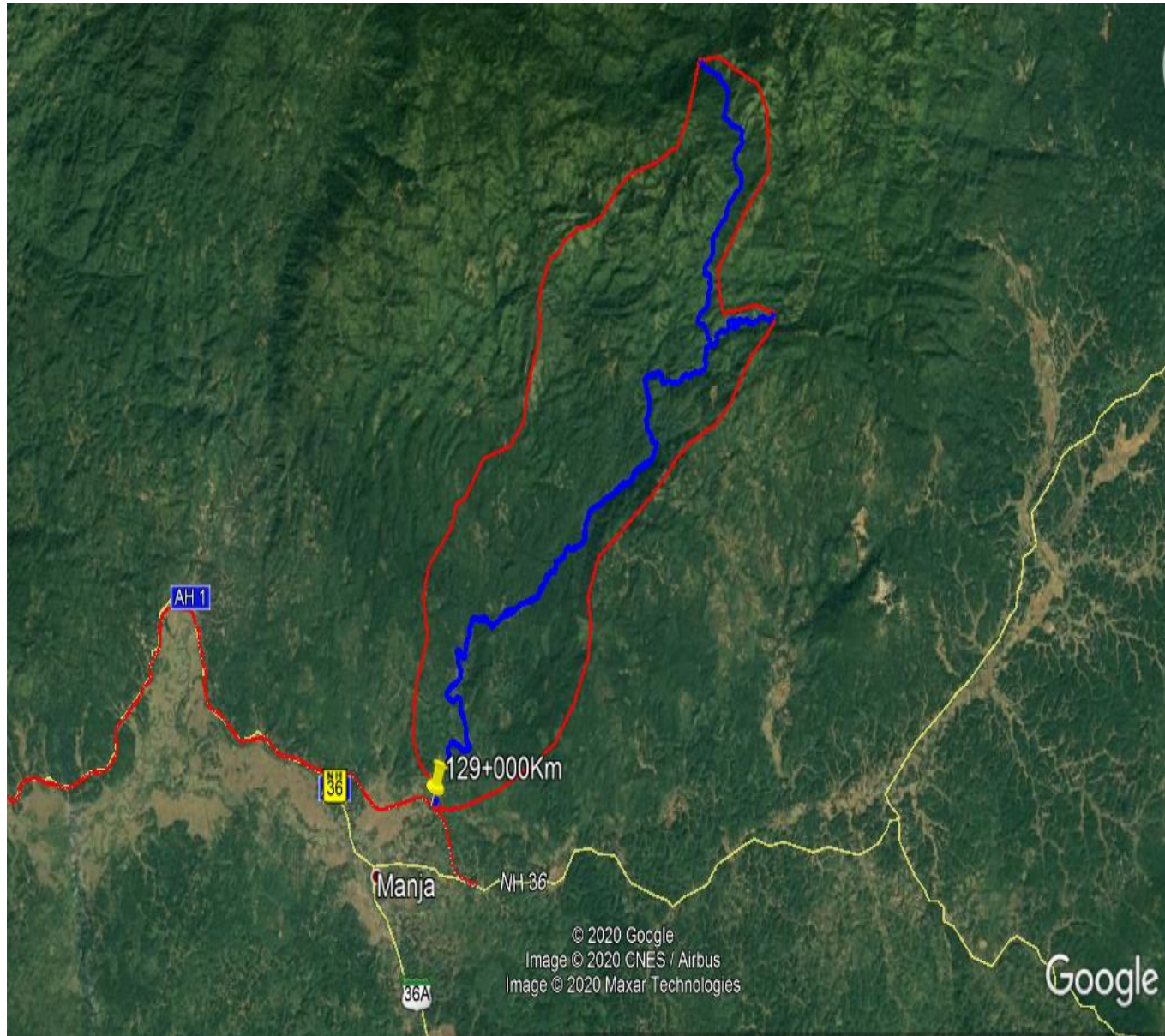
Maximum Scour Level = HFL - Maximum Scour Depth
For Piers, = 125.943 m
For Abutments, = 130.486 m

Proposed Bridge Span Arrangement :

Total Length of Proposed Bridge = 3x30 m
Span Length for each = 30 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	138.390	138.390	
2	Lowest Bed Level at Proposed Location	135.355	135.355	
3	Minimum Soffit Level at proposed location	139.590	149.810	Safe
4	Proposed Formation Level	141.780	152.000	Safe



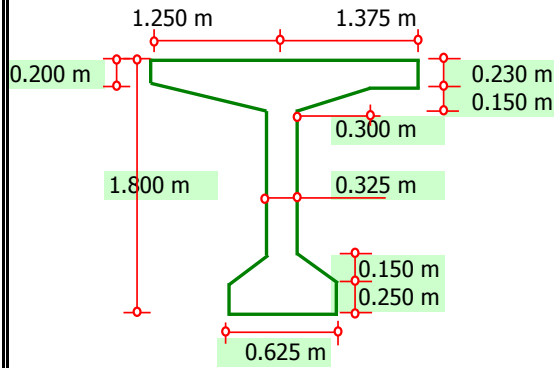
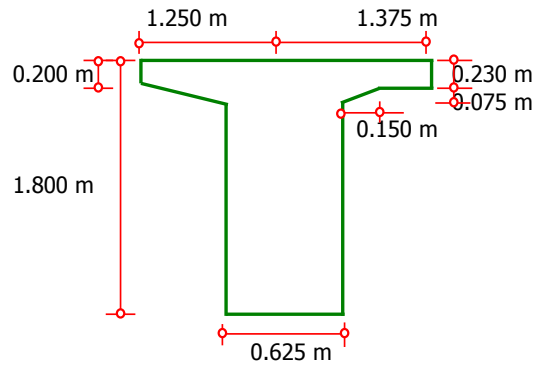
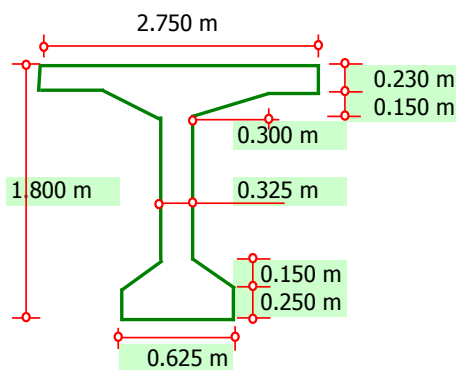
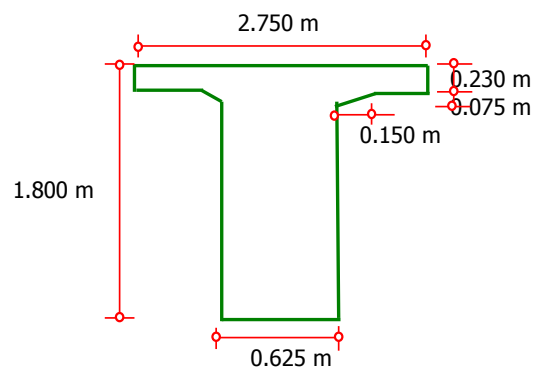
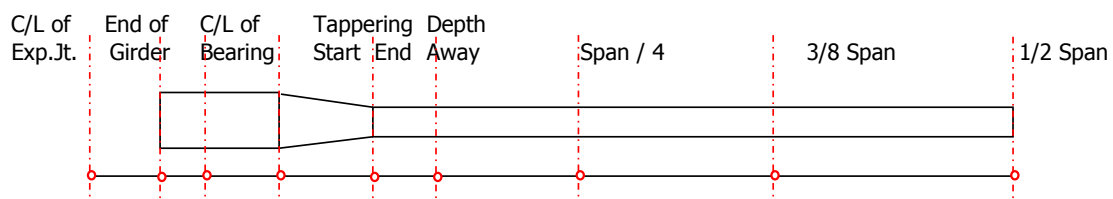
CATCHMENT AREA= 63.4 Sq.Km

DETAIL DESIGN CALCULATION OF RCC T GIRDER 22M .

1.0 INPUT DATA

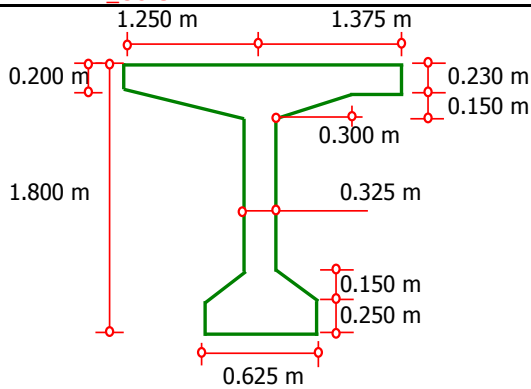
Span length c/c of Expansion Joint
 Span length c/c of Bearing
 Skew Angle
 Distance between C/L of Brg. and C/L of Exp. Joint
 No. of Longitudinal Girders
 No. of Cross Girders
 C/C spacing of Cross Girder
 C/C spacing of Longitudinal Girder
 Total Width of Superstructure
 Length of Girder Beyond Bearing
 Thickness of Deck slab
 Thickness of Deck slab at Exp. Joint
 Length of Solid Portion of Web
 Length of Tapering Portion of Web
 Depth of T Girder
 Depth of Cross Girder
 Cantilever length
 C/C spacing of Longitudinal Girder (Skew)
 Distance between C/L of Brg. and C/L of Exp. Joint (Normal)
 Bearing size in longitudinal direction (POT-PTFE/Elastomeric)

= 22.000 m
 = 21.000 m
 = 0.0 Degree Clockwise (-)
 = 0.500 m
 = 5 Nos
 = 3 Nos
 = 10.500 m
 = 2.750 m
 = 13.500 m
 = 0.300 m
 = 0.230 m
 = 0.400 m
 = 0.700 m
 = 0.900 m
 = 1.500 m
 = 1.250 m
 = 1.250 m
 = 2.750 m
 = 0.500 m
 = 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 SpanMember no: GO-SPAN *

Effective Span,

$$l_0 = \text{Min.} \begin{cases} \text{Distance from Brg. to Brg.} \\ \text{Clear distance b/w supports + Eff. Depth*} \end{cases}$$

$$l_0 = \text{Min.} \begin{cases} 21.000 \text{ m} + \\ 20.400 \text{ m} + 1.530 \text{ m} \end{cases}$$

* Effective depth assumed 0.85 times of Overall depth

$$l_0 = 21.000 \text{ m}$$

$$b_1 = 1.088 \text{ m}$$

$$b_2 = 1.213 \text{ m}$$

$$beff_1 = \text{Min.} \begin{cases} 0.2 b_1 + 0.1 l_0 \\ 0.2 l_0 \end{cases} = \begin{cases} 2.318 \text{ m} \\ 4.200 \text{ m} \end{cases}$$

$$beff_2 = \text{Min.} \begin{cases} 0.2 b_2 + 0.1 l_0 \\ 0.2 l_0 \end{cases} = \begin{cases} 2.343 \text{ m} \\ 4.200 \text{ m} \end{cases}$$

$$beff = \text{Min.} \begin{cases} \sum beff_i + bw \\ b \end{cases} = \begin{cases} 4.985 \text{ m} \\ 2.625 \text{ m} \end{cases}$$

$$beff = 2.625 \text{ m}$$

SL.No.		A	y	Ay	Ay ²	I _o	I _o + Ay ²
1.	Rectangular top Flange Cantilever	0.25000	1.70000	0.42500	0.72250	0.000833	0.72333
2.	Rectangular top Flange Inner	0.31625	1.68500	0.53288	0.89790	0.001394	0.89930
3.	Triangular top Flange Cantilever	0.09788	1.54000	0.15073	0.23212	0.000203	0.23232
4.	Triangular top Flange Haunch	0.02250	1.52000	0.03420	0.05198	0.000028	0.05201
5.	Web	0.42900	0.91000	0.39039	0.35525	0.062291	0.41755
5.	Bottom Bulb, Triangle	0.02250	0.30000	0.00675	0.00203	0.000028	0.00205
6.	Bottom Bulb, Rectangle	0.15625	0.12500	0.01953	0.00244	0.000814	0.00326
Total Section Σ		1.29438		1.55948	2.2642	0.066	2.32982
		ΣA		ΣAy	ΣAy ²	Σ I _o	Σ (I _o + Ay ²)

0.7191 1.8000

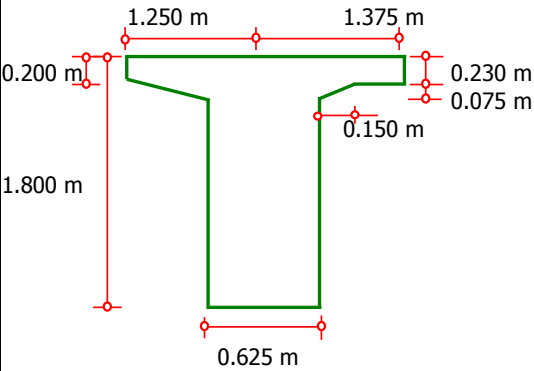
Section Properties of Total Section

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 (I _z) = Σ(I _o + A.y ²) - ΣA.Y ²		=	0.4509	m ⁴
Perimeter in contact with atmosphere u		=	8.9945	m

u
1.45000
2.51750
1.10230
0.33541
2.04000
0.42426
1.12500
8.99447
Σu

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						
	ΣA		ΣAy	ΣAy ²	Σ I _o	Σ (I _o + Ay ²)	Σu

Section Properties of Total Section

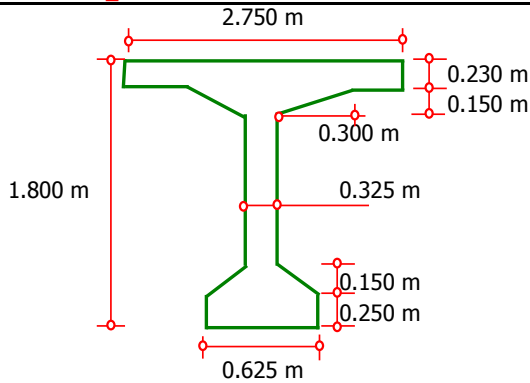
Area ΣA = 1.6023 m²

Distance of cg from bottom fibre (Y) $Y = \Sigma(A \cdot y) / \Sigma A$ = 1.1320 m

Moment of inertia of Girder (I_z) = $\Sigma(I_o + A \cdot y^2) - \Sigma A \cdot Y^2$ = 0.5095 m⁴

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 SpanMember no: GI-SPAN *

Effective Span,

$$l_0 = \text{Min.} \begin{cases} \text{Distance from Brg. to Brg.} \\ \text{Clear distance b/w supports + Eff. Depth*} \end{cases}$$

$$l_0 = \text{Min.} \begin{cases} 21.000 \text{ m} + \\ 20.400 \text{ m} + 1.530 \text{ m} \end{cases}$$

* Effective depth assumed 0.85 times of Overall depth

$$l_0 = 21.000 \text{ m}$$

$$b_{1,2} = 1.2125 \text{ m}$$

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

$$beff = 2.343 \text{ m}$$

$$beff = \text{Min.} \begin{cases} \Sigma beff_{i,j} + bw \\ b \end{cases} = \begin{cases} 5.010 \text{ m} \\ 2.750 \text{ m} \end{cases}$$

$$beff = 2.750 \text{ m}$$

SL.No.		A	y	Ay	Ay ²	I _o	I _o + Ay ²
1. Rectangular top Flange		0.63250	1.68500	1.06576	1.79581	0.002788	1.79860
2. Triangular top Flange		0.04500	1.52000	0.06840	0.10397	0.000056	0.10402
3. Web		0.42900	0.91000	0.39039	0.35525	0.062291	0.41755
4. Bottom Bulb, Triangle		0.02250	0.30000	0.00675	0.00203	0.000028	0.00205
5. Bottom Bulb, Rectangle		0.15625	0.12500	0.01953	0.00244	0.000814	0.00326
Total Section	Σ	1.28525		1.55083	2.25950	0.06598	2.32548
		ΣA		ΣAy	ΣAy ²	Σ I _o	Σ (I _o + Ay ²)

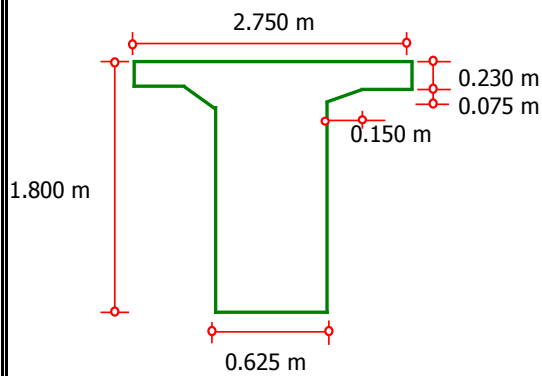
0.7140 1.8000

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 (I _z) = Σ(I _o + A.y ²) - ΣA.Y ²		=	0.454179	m ⁴
Perimeter in contact with atmosphere u		=	9.2951	m

u
5.03500
0.67082
2.04000
0.42426
1.12500
9.295084
Σu

0.13827

3.2 Inner T - Girder at the SupportMember no: GI-SUP *

SL.No.	A	y	Ay	Ay ²	I _o	I _o + Ay ²
1. Rectangular top Flange	0.63250	1.68500	1.06576	1.79581	0.002788	1.79860
2. Triangular top Flange	0.01125	1.54500	0.01738	0.02685	0.000004	0.02686
3. Web	0.98125	0.78500	0.77028	0.60467	0.201557	0.80623
Total Section Σ	1.62500		1.85343	2.4273	0.204	2.63168
	ΣA		ΣAy	ΣAy ²	Σ I _o	Σ (I _o + Ay ²)
	0.9028	1.8000				

u
5.03500
0.33541
3.61500

8.985410
Σu

0.18085

Section Properties of Total Section

Area ΣA = $\boxed{1.625} \text{ m}^2$

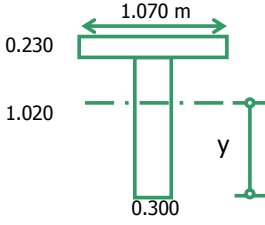
Distance of cg from bottom fibre (Y) $Y = \Sigma(A \cdot y) / \Sigma A$ = $\boxed{1.141} \text{ m}$

Moment of inertia of Girder (I_z) = $\Sigma(I_o + A \cdot y^2) - \Sigma A \cdot Y^2$ = $\boxed{0.518} \text{ m}^4$

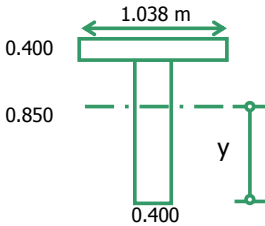
4.0 Section Properties of Transverse Members

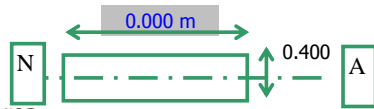
Span length c/c of Bearing	=	21.000	m	
C/C spacing of Longitudinal Girder	=	2.750	m	
C/C spacing of Cross Girder	=	10.500	m	
Distance between C/L of Brg. and C/L of Exp. Joint	=	0.500	m	0.500 (Normal)
Cantilever length	=	1.250	m	
Depth of Cross-Girder	=	1.250	m	
Thickness of Deck slab	=	0.230	m	
Thickness of Deck slab at Exp. Joint	=	0.400	m	
Length of Solid Portion of Web	=	0.700	m	
Length of Tapering 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 GirderMember no: INT-CG

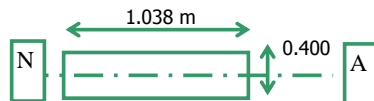
	Effective span	l_0	=	1.925	m	
		$b_{1,2}$	=	5.100	m	
		$beff_{1,2}$	=	Min. $\begin{cases} 0.2 b_{1,2} + 0.1 l_0 \\ 0.2 l_0 \end{cases}$	= 1.213 m = 0.385 m	
		$beff_{1,2}$	=	0.385	m	
		$beff$	=	Min. $\begin{cases} \sum beff_{i,j} + bw \\ b \end{cases}$	= 1.070 m = 10.500 m	
		$beff$	=	1.070	m	
Area			=	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 (I_z)			=	0.08090	m ⁴	
Moment of inertia girder (I_x)			=	0.00947	m ⁴	

4.2 End Cross GirderMember no: END-CG

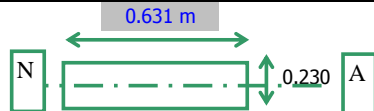
	Effective span	l_0	=	1.925	m	
		b_1	=	0.300	m	
		b_2	=	5.050	m	
		$beff_1$	=	Min. $\begin{cases} 0.2 b_1 + 0.1 l_0 \\ 0.2 l_0 \end{cases}$	= 0.253 m = 0.385 m	
		$beff_1$	=	0.253	m	
		$beff_2$	=	Min. $\begin{cases} 0.2 b_2 + 0.1 l_0 \\ 0.2 l_0 \end{cases}$	= 1.203 m = 0.385 m	
		$beff_2$	=	0.385	m	
		$beff$	=	Min. $\begin{cases} \sum beff_{i,j} + bw \\ b \end{cases}$	= 1.038 m = 5.500 m	
		$beff$	=	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 (I_z)			=	0.099	m ⁴	
Moment of inertia girder (I_x)			=	0.022	m ⁴	

4.3 Edge Cantilever Slab beyond supportMember no: **_DUMMY-T**

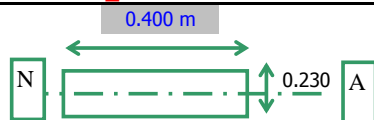
Area	=	0.0000 m ²
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0000 m ⁴
Moment of inertia girder (I_x)	=	0.0000 m ⁴

4.4 Cantilever Slab (End of End Cross Girder)Member no: **_T-SLAB1**

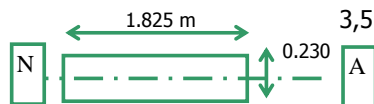
Area	=	0.4150 m ²
Distance of cg from bottom fibre (y)	=	0.2000 m
Moment of inertia (I_z)	=	0.0055 m ⁴
Moment of inertia girder (I_x)	=	0.0087 m ⁴

4.5 Intermediate Slab Next to End Cross GirderMember no: **_T-SLAB2**

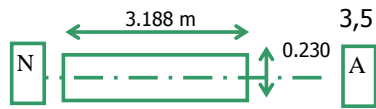
Area	=	0.1452 m ²
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0006 m ⁴
Moment of inertia girder (I_x)	=	0.0010 m ⁴

4.6 Slab at Tapered portionMember no: **_T-SLAB3**

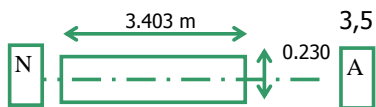
Area	=	0.0920 m ²
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0004 m ⁴
Moment of inertia girder (I_x)	=	0.0005 m ⁴

4.7 Slab at Depth away from supportMember no: **_T-SLAB4**

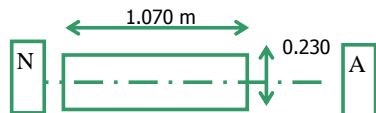
Area	=	0.4198 m ²
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0019 m ⁴
Moment of inertia girder (I_x)	=	0.0033 m ⁴

4.8 Slab Adjacent to Intermediate Cross GirdersMember 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 GirdersMember 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**

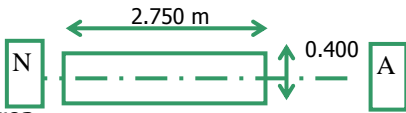
Area	=	0.2461 m ²
Distance of cg from bottom fibre (y)	=	0.1150 m
Moment of inertia (I_z)	=	0.0011 m ⁴
Moment of inertia girder (I_x)	=	0.0019 m ⁴

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

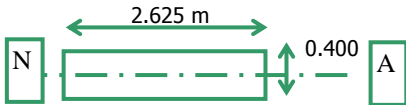
5.0 Section Properties of Longitudinal Members (Slab Portion)

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 GirderMember no: **L-SLAB1**


The diagram shows a rectangular slab portion between two girders, labeled 'N' and 'A'. The horizontal dimension (width) is 2.750 m, indicated by a double-headed arrow above the slab. The vertical dimension (thickness) is 0.400 m, indicated by a double-headed arrow to the right of the slab. A dashed horizontal line represents the centerline of the slab.

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 GirderMember no: **L-SLAB2**


The diagram shows a rectangular slab portion between two girders, labeled 'N' and 'A'. The horizontal dimension (width) is 2.625 m, indicated by a double-headed arrow above the slab. The vertical dimension (thickness) is 0.400 m, indicated by a double-headed arrow to the right of the slab. A dashed horizontal line represents the centerline of the slab.

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

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	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	32.9	647.6	21.0	266.1
15	34.5	715.8	23.2	321.9
20	35.8	769.0	24.8	370.5
30	37.1	824.3	26.9	436.0
50	39.1	920.7	29.4	517.3
60	39.7	944.6	30.3	548.8
70	40.2	968.7	31.0	576.6
80	40.7	993.3	31.8	605.2
90	41.2	1018.2	32.5	634.4
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/m²

Upper Bound Height = 10.0 m

P_z = 647.6 N/m²P_z = Design wind pressure at bridge for h = 10.0 m= 647.6 N/m²

Transverse wind Force

$$F_T = P_z \times A_1 \times G \times C_D$$

$$= 150.0 \text{ KN}$$

1.7

Longitudinal wind Force

$$F_L = 25\% \text{ of } F_T$$

$$= 37.5 \text{ KN}$$

0.1

Vertical wind Force

$$F_V = P_z \times A_3 \times G \times C_L$$

$$= 288.5 \text{ KN}$$

UDL on girder

$$= 1.0 \text{ KN/m}^2$$

Outer girder

$$= 2.6 \text{ KN/m}$$

Inner girder

$$= 2.7 \text{ KN/m}$$

Calculation of Effective Bridge Temperature loading:-**Calculation of Uniform Temperature Rise & Fall effect:-**

Maximum Air Shade temperature

$$= 47.5 \text{ }^{\circ}\text{C}$$

Minimum Air Shade temperature

$$= 2.5 \text{ }^{\circ}\text{C}$$

Difference between Max. & Min. of Air Shade temperature

$$= 45.0 \text{ }^{\circ}\text{C}$$

Maximum Effective Bridge temperature

$$= 32.50 \text{ }^{\circ}\text{C}$$

Minimum Effective Bridge temperature

$$= 12.50 \text{ }^{\circ}\text{C}$$

For Expansion, temperature Rise

$$= 35.00 \text{ }^{\circ}\text{C}$$

For Contraction, temperature Fall

$$= 35.00 \text{ }^{\circ}\text{C}$$

Coefficient of Thermal Expansion

$$\alpha = 0.000012$$

For Expansion, Strain on Top Slab

$$= 0.000420$$

For Contraction, Strain on Top Slab

$$= 0.000420$$

Max. Longitudinal force:-

Class A

$$= 110.8 \text{ KN}$$

70 R Wheel

$$= 200 \text{ KN}$$

$$227.7 \text{ KN}$$

$$57 \text{ KN}$$

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:-

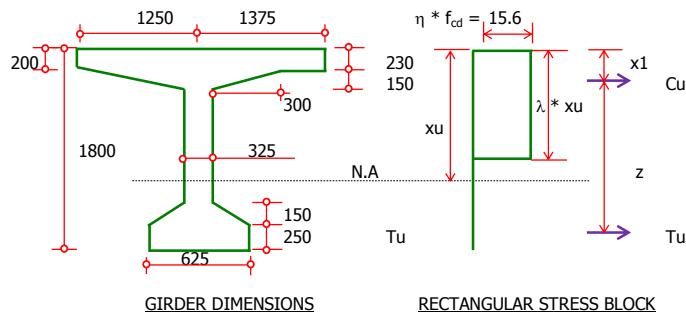
Relative Humidity	RH	=	80	%
Exposure condition considered		=	Moderate	
Coefficient of thermal expansion	α	=	1.2E-05	/°C

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_{vwd}	=	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	$x_{u,max} / d$	=	0.464	

Modular ration E_{cmP} / E_{cmd} = 1 (Precast Beam / Cast in situ Deck)**Min. & Max. Longitudinal Reinforcement Percentage:-**

Min. Reinforcement percentage for beam section	$A_{s,min}$	=	$\max [0.26 * f_{ctm} / f_{yk} * b_t * d , 0.0013 * b_t * d]$
Max. Reinforcement percentage for beam section	$A_{s,max}$	=	$0.025 * A_c$ Clause 16.5.1.1

Section At	Unit	c/L Brg.	Tapered 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 A_s , 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
$A_{s,min}$	mm ²	1511	786	786	781	781	781
Check $A_{s,min} < A_s$, provided		OK	OK	OK	OK	OK	OK
A_c	mm ²	1602344	1294375	1294375	1294375	1294375	1294375
$A_{s,max}$	mm ²	40059	32359	32359	32359	32359	32359
Check $A_{s,max} > A_s$, 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

$\lambda * x_u$	mm	91	91	91	105	105	105
C_{Area}	mm ²	238525	238525	238525	275221	275221	275221
x_1	mm	45	45	45	52	52	52
C_u	KN	3729	3729	3729	4303	4303	4303
T_u	KN	4546	4546	4546	5245	5245	5245
Check ($C_u - T_u = 0$)		-817	-817	-817	-942	-942	-942
x_u	mm	114	114	114	131	131	131
x_u / d_{eff}		0.068	0.068	0.068	0.079	0.079	0.079
Check $x_u / d_{eff} < x_{u,max} / d_{eff}$		UR,OK	UR,OK	UR,OK	UR,OK	UR,OK	UR,OK
$z = d_{eff} - x_1$	mm	1633	1633	1633	1616	1616	1616
$M_{RD} = T_u * z$	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
$A_{s,cal}$	mm ²	676	2546	2546	3438	4411	4630
Check $A_{s,cal} < A_s$, 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.	Tapered 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
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. $t_{ef,j} = A/u$	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 A_k , u_k (m)		4.643	4.463	4.463	4.463	4.463	4.463
Torsional Stress, $\tau_{t,j}$	KN/m ²	0	0	0	0	0	0
$z_l = h$	m	1.800	1.800	1.800	1.800	1.800	1.800
Torsional Shear, $V_{Ed,l}$	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
ρ_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
$V_{Rd,c \text{ min}}$	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

Clause 10.3.3.3

Check if shear force V_{ED}' calculated with out reduction factor β

Clause 10.3.2 (5)

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

Check for Section Maximum Shear Capacity:-

Clause 10.3

Shear V_{cdd} Conc. Cord	KN	0	0	0	0	0	0
Shear V_{td} Reinf. Cord	KN	0	0	0	0	0	0
$V_{NS} = V_{ED}' - V_{cdd} - V_{td}$	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
v_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
$\rho_{min} = (0.072 * f_{ck}^{0.5}) / f_{yk}$		0.00085	0.00085	0.00085	0.00085	0.00085	0.00085
$A_{sw,min} = \rho_{min} * s * b_w$	mm ² /m	106	55	55	55	55	55
Provide A_{sw}	Legs	No's	2	2	2	2	2
	Dia.	mm	12	12	12	10	10
	Spacing	mm	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
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
$T_{Ed} / T_{Rd,max} + V_{NS} / V_{Rd,max} \leq 1.0$		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

Clause 10.3.3.2

Clause 10.3.3.5

Clause 10.3.3.2

Anchorage of Span Reinforcement at End:-

Clause 16.5.1.4

Tensile force to be resisted where,	F_s	=	$V_{ED} * (a_1/d_{eff}) + N_{ED}$
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 support	d_{eff}	=	1678 mm
	z	=	1510 mm
	a_1	=	$z * (\cot \theta + \cot \alpha) / 2$
for $\theta = 21.8 \text{ deg.}$ &	$\alpha = 90.0 \text{ deg.}$		
Tensile force	a_1	=	1888 mm
Area of tensile reinforcement at support section	F_s	=	902 KN
Tensile capacity of anchorage reinforcement	A_s	=	10455 mm ²
	F_t	=	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 mmFor 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. dimention.**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. $(0.3 * l_b, 10\phi, 100\text{mm})$
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.	Tapered 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

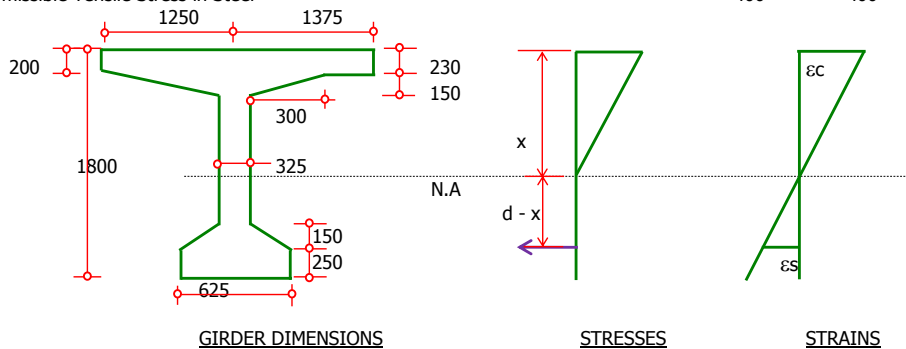
10.0 SLS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT (OUTER GIRDER)

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

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

**10.1 SLS STRESSES CHECK**

Section	Unit	c/L Brg.	Tapered	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	425	790	790	1850	2280	2309
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	0.57	1.08	1.08	2.46	3.03	3.07
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 ²	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

3.41E+11

0.34

Computation of Stresses in Girder section at SLS Quasi-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
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

OK

10.1 SLS CRACK WIDTH CHECK

Minimum area of reinforcement required to Crack control	$A_{s,min}$	=	$(k_c * k * f_{ct,eff} * A_{ct}) / \sigma_c$, where
Coefficient	k_c	=	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. $\left\{ \begin{array}{l} 2.5 * (h - d) \\ (h - x/3) \\ h/2 \end{array} \right.$
	$\epsilon_{sm} - \epsilon_{cm}$	=	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 Brq.	Tapered	Deff	L/4	3L/8	L/2
Dist. from c/L brq.	m	0.000	1.600	1.500	5.250	7.875	10.500
$A_{s,pro}$	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
$A_{ct} = (h - x) * bw$	mm ²	959967	499183	499183	493698	493698	493698
$A_{s,min}$	mm ²	1383	719	719	711	711	711
Check $A_{s,min} < A_{s,pro}$		OK	OK	OK	OK	OK	OK

Calculation of Crack Width:-

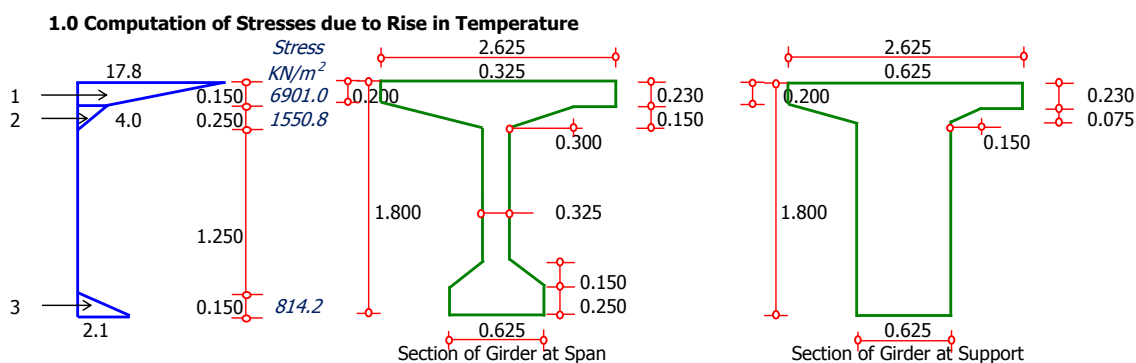
d_{eff}	mm	1678	1678	1678	1668	1668	1668
$h_{c,eff}$	mm	305	305	305	330	330	330
$A_{c,eff}$	mm ²	190625	99125	99125	107250	107250	107250
$\rho_{p,eff}$		0.055	0.105	0.105	0.112	0.112	0.112
$S_{r,max}$	mm	235.2	187.6	187.6	184.4	184.4	184.4
$S_{pro.}$	mm	100.0	100.0	100.0	100.0	100.0	100.0
Check		OK	OK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

1) Crack Width calculation at SLS Quasi-Peak

Section At	Unit	c/L Brq.	Tapered	Deff	L/4	3L/8	L/2
Dist. from c/L brq.	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
$\epsilon_{sm} - \epsilon_{cm}$		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

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		°C
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	°C	h_1	= 0.150 m
T_2	=	4.0	°C	h_2	= 0.250 m
T_3	=	2.1	°C	h_3	= 0.150 m

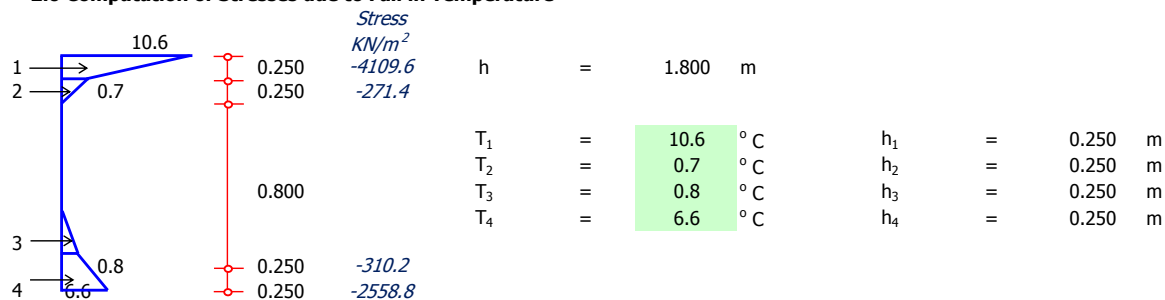
**1.1 Stresses at Span section**

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	0.625	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



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							

2.2 Stresses at Support section

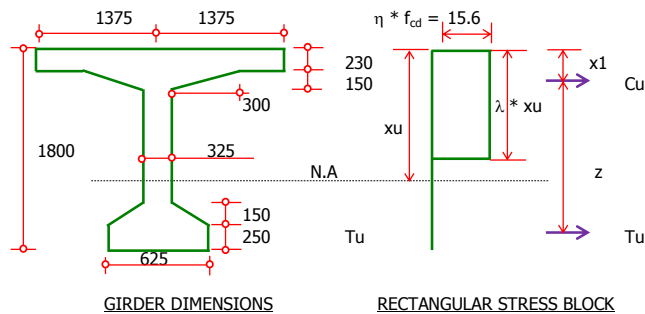
Segment	Height	Stress	Width b	Force F _{te}	y from top	e	Moment M _{te} =F _{te} *e	Stresses Assuming End Restrained	Stress due to release of Axial Force	Stress due to release of Moment	Final Stress
Number	m	KN/m ²	m	KN	m	m	KNm	KN/m ²	KN/m ²	KN/m ²	KN/m ²
1	0.230	-4109.6	2.625	-1415.2	0.086	0.582	-823.4	-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							

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_{vwd}	=	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	$x_{u,max} / d$	=	0.464	

Modular ration E_{cmP} / E_{cmd} = 1 (Precast Beam / Cast in situ Deck)**Min. & Max. Longitudinal Reinforcement Percentage:-**

Min. Reinforcement percentage for beam section	$A_{s,min}$	=	$\max [0.26 * f_{ctm} / f_{yk} * b_t * d , 0.0013 * b_t * d]$
Max. Reinforcement percentage for beam section	$A_{s,max}$	=	$0.025 * A_c$ Clause 16.5.1.1

Section At	Unit	c/L Brg.	Tapered 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 A_s , 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
$A_{s,min}$	mm ²	1511	786	786	781	781	781
Check $A_{s,min} < A_s$, provided		OK	OK	OK	OK	OK	OK
A_c	mm ²	1625000	1285250	1285250	1285250	1285250	1285250
$A_{s,max}$	mm ²	40625	32131	32131	32131	32131	32131
Check $A_{s,max} > A_s$, 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

$\lambda * x_u$	mm	91	91	91	105	105	105
C_{Area}	mm ²	249883	249883	249883	288326	288326	288326
x_1	mm	45	45	45	52	52	52
C_u	KN	3907	3907	3907	4508	4508	4508
T_u	KN	4546	4546	4546	5245	5245	5245
Check ($C_u - T_u = 0$)		-639	-639	-639	-738	-738	-738
x_u	mm	114	114	114	131	131	131
x_u / d_{eff}		0.068	0.068	0.068	0.079	0.079	0.079
Check $x_u / d_{eff} < x_{u,max} / d_{eff}$		UR,OK	UR,OK	UR,OK	UR,OK	UR,OK	UR,OK
$z = d_{eff} - x_1$	mm	1633	1633	1633	1616	1616	1616
$M_{RD} = T_u * z$	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
$A_{s,cal}$	mm ²	676	2408	2408	3913	4283	4441
Check $A_{s,cal} < A_s$, 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.	Tapered 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
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 A_k , 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
ρ_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_{cd}	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.

Clause10.3.3.3

Check if shear force V_{ED}' calculated with out reduction factor β

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

Clause10.3.2 (5)

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
v_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	476	463	463	382	246	200
$\rho_{min} = (0.072 * f_{ck}^{0.5}) / f_{yk}$		0.00085	0.00085	0.00085	0.00085	0.00085	0.00085
$A_{sw,min} = \rho_{min} * s * b_w$	mm ² /m	106	55	55	55	55	55
Provide							
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	899	803	803	659	463	375
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
$T_{Ed} / T_{Rd,max} + V_{NS} / V_{Rd,max} \leq 1.0$		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

Clause10.3.3.2

Clause10.3.3.5

Clause10.3.3.2

Anchorage of Span Reinforcement at End:-

Clause 16.5.1.4

Tensile force to be resisted	F_s	=	$V_{ED} * (a_1/d_{eff}) + N_{ED}$
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 support	d_{eff}	=	1678 mm
	z	=	1510.2 mm
	a_1	=	$z * (\cot \theta + \cot \alpha) / 2$
for $\theta = 21.8 \text{ deg.}$ &	$\alpha = 90.0 \text{ deg.}$		
	a_1	=	1887.88468 mm
Tensile force	F_s	=	809 KN
Area of tensile reinforcement at support section	A_s	=	10455 mm ²
Tensile capacity of anchorage reinforcement	F_t	=	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 mmFor 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. dimention.**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. $(0.3 * l_b, 10\phi, 100\text{mm})$
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.	Tapered 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

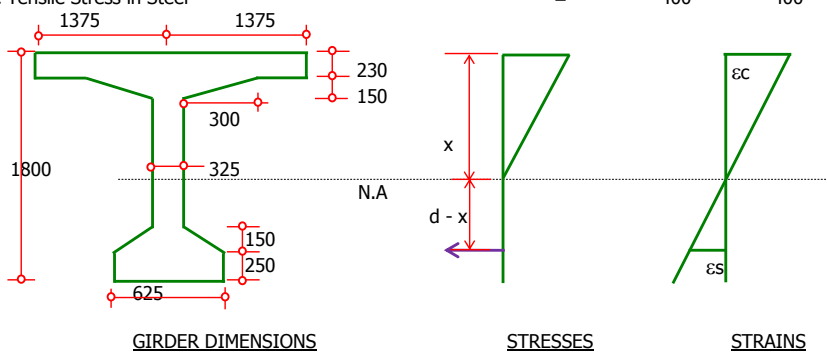
12.0 SLS CHECK OF RCC T-GIRDER SECTIONS FOR BENDING MOMENT (*INNER GIRDER*)

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	

Max. Permissible Concrete Compressive Stress	=	16.8	12.6	MPa
Max. Permissible Tensile Stress in Steel	=	-400	-400	MPa

**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 ²	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	$A_{s,min}$	=	$(k_c * k * f_{ct,eff} * A_{ct}) / \sigma_c$, where
Coefficient	k_c	=	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. $\begin{cases} 2.5 * (h - d) \\ (h - x/3) \\ h/2 \end{cases}$
	$\epsilon_{sm} - \epsilon_{cm}$	=	Max. $\begin{cases} [\sigma_{sc} - k_t * f_{ct,eff} (1 + \alpha_e * \rho_{p,eff}) / \rho_{eff}] / E \\ 0.6 * \sigma_{sc} / E_s \end{cases}$
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.	Tapered	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
$A_{s,pro}$	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
$A_{ct} = (h - x) * bw$	mm ²	963444	500991	500991	495608	495608	495608
$A_{s,min}$	mm ²	1388	722	722	714	714	714
Check $A_{s,min} < A_{s,pro}$		OK	OK	OK	OK	OK	OK

Calculation of Crack Width:-

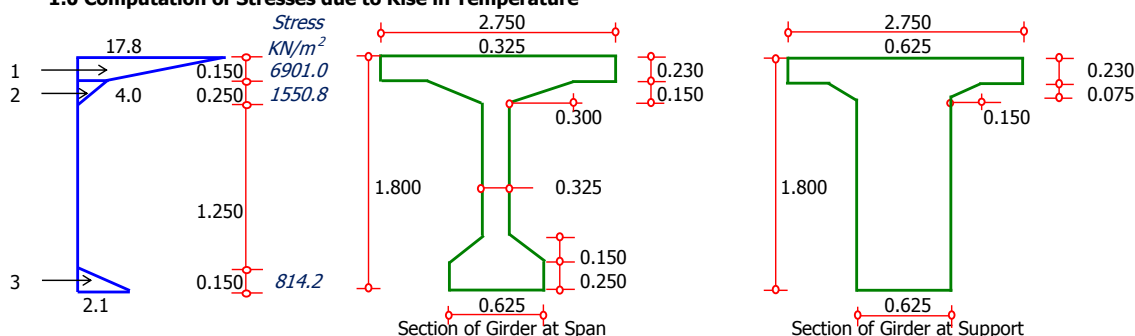
d_{eff}	mm	1678	1678	1678	1668	1668	1668
$h_{c,eff}$	mm	305	305	305	330	330	330
$A_{c,eff}$	mm ²	190625	99125	99125	107250	107250	107250
$\rho_{p,eff}$		0.055	0.105	0.105	0.112	0.112	0.112
$S_{r,max}$	mm	235.2	187.6	187.6	184.4	184.4	184.4
$S_{pro.}$	mm	100.0	100.0	100.0	100.0	100.0	100.0
Status		OK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

Crack Width calculation for Quasi-Permanent Load Combination:

Section At	Unit	c/L Brg.	Tapered	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
$\epsilon_{sm} - \epsilon_{cm}$		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

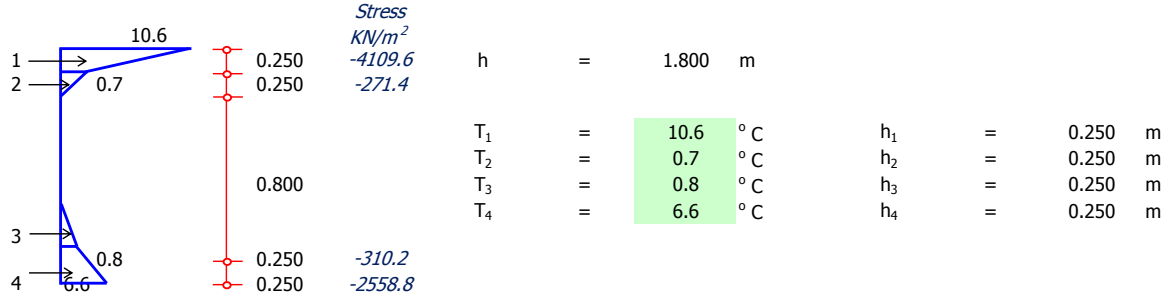
Total height of T-girder	h	=	1.800	1.800	m
Center of gravity (c.g) of T-girder from bottom	y	=	1.207	1.141	m
Moment of Inertia of girder section	I	=	0.454	0.518	m ⁴
Area of girder section	A	=	1.285	1.625	m ²
Modulus of Elasticity of Concrete	E_{cm}	=	3.23E+07		KN/m ²
Coefficient of thermal expansion of concrete	α	=	0.000012		°C
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	°C	h_1	= 0.150 m
T_2	=	4.0	°C	h_2	= 0.250 m
T_3	=	2.1	°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	2.750	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**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	2.750	-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							

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	2.750	-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							

DETAIL DESIGN CALCULATION OF PIER WITH PILE FOUNDATION.

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	

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

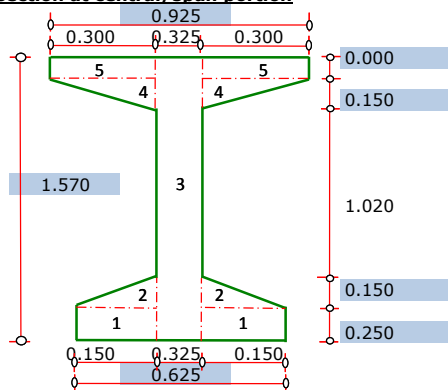
Diameter of cast-in-situ piles	=	1.2 m
Pile edge distance	=	0.15 m
Pile no's	long. direction (col.)	2
		4
Pile spacing	long. direction (col.)	3.6 m
	trans. direction (row)	3.6 m
Pile cap depth	=	1.8 m
Pier dimension in long. direction	=	1.0 m
Pier dimension in trans. direction	=	6.0 m
Pier thickness 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

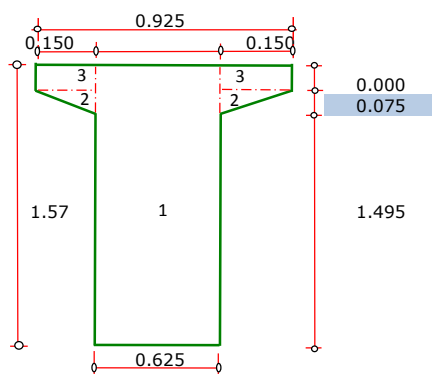
Span c/L to c/L of Expansion joint	=	22.000 m	at deck slab level beyond c/LI of bearing
Span c/L to c/L of Bearing	=	20.600 m	
Expansion joint	=	0.040 m	
Overall Span of girder	=	21.200 m	
Distance betw Exp. joint and c/L of Brg.	=	0.700 m	
Length of end thickning zone of girder	=	0.700 m	
Length of 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 diaphragm/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			29.5

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)

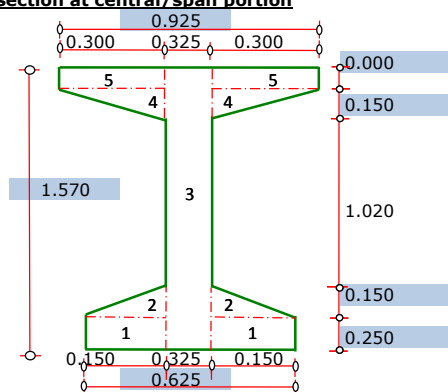
Description	Nos	Width(m)	Depth(m)	Area(m ²)	Length(m)	Unit Wt (t/m ³)	Total Wt (t)
300 mm thk. Footpath	2	1.50	0.300	0.90	22.00	2.5	49.5
						Total FPDL=	
						49.5 t	
						Total DL+Surfacing+ SIDL+FPDL=	
						535.0 t	

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

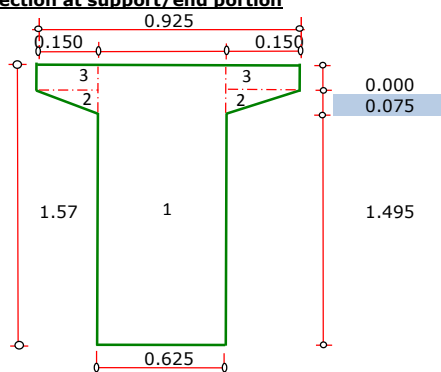
Span c/L to c/L of Expansion joint	=	22.000 m	
Span c/L to c/L of Bearing	=	20.600 m	
Expansion joint	=	0.040 m	
Overall Span of girder	=	21.200 m	
Dist bet. exp joint and c/l of bearing	=	0.700 m	at deck slab level
Length of end thickning zone of girder	=	0.700 m	beyond c/LI of bearing
Length of tapering 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

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	

Dead load of diaphragm/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. Footpath	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

Basic Footpath live load	=	LHS Span	400 Kg/m ²	RHS Span	400 Kg/m ²
Actual Footpath live load	=		342 Kg/m ²		342 Kg/m ²
	=		0.34 t/m ²		0.34 t/m ²
Footpath area	=		66.00 m ²		66.00 m ²
Total Footpath live load	=		22.6 t		22.6 t

Cl.206.3 of
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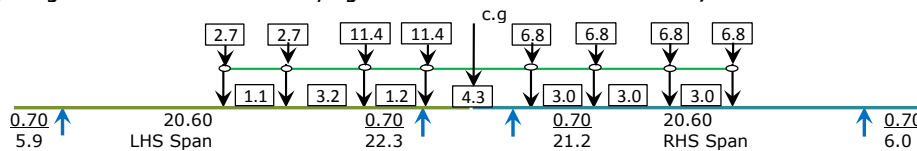
Case 1 - Max. Live Load Reaction & Max. Transverse Moment

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 coincide with c/L of Pier.

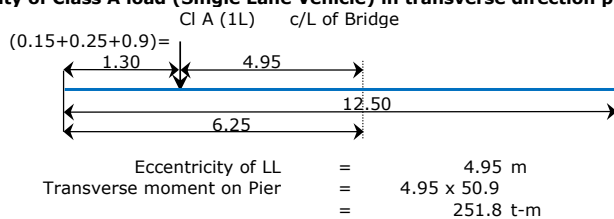


Reaction due to LL (Class A)		
Single Lane Vehicle	=	22.3 t on LHS Span
	=	21.2 t on RHS Span
Two Lane Vehicle	=	44.6 t on LHS Span
	=	42.5 t on RHS Span
Three Lane Vehicle	=	60.2 t on LHS Span
	=	57.4 t on RHS Span
Four Lane Vehicle	=	71.3 t on LHS Span
	=	68.0 t on RHS Span

Reaction due to LL (Class A) with Impact factor

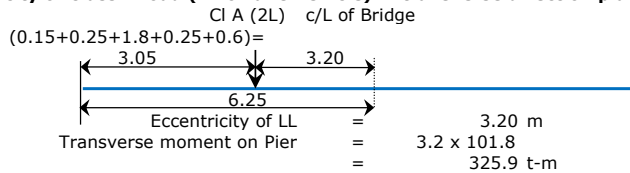
Single Lane Vehicle	=	50.9 t
Two Lane Vehicle	=	101.8 t
Three Lane Vehicle	=	137.4 t
Four Lane Vehicle	=	162.9 t

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

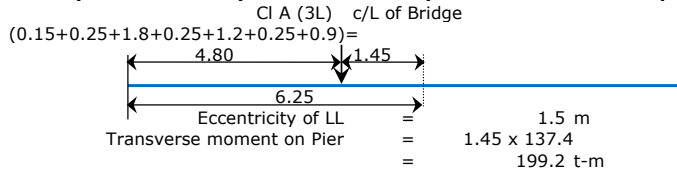


Component: Design of Substructure & Pile Foundation	Detail Project Report-	Rev.	R0	Ref.
Details: Design of Substructure & Pile Foundation for Pier no P1 of Bridge at Ch: 129+905		Date	15 Apr,20	
		Design by	D.S.R	
		Check by	S.G	

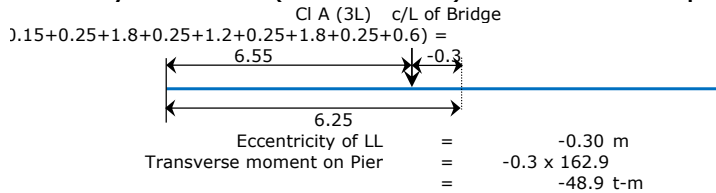
Eccentricity of Class A load (Two Lane Vehicle) in transverse direction placed in median side.



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



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



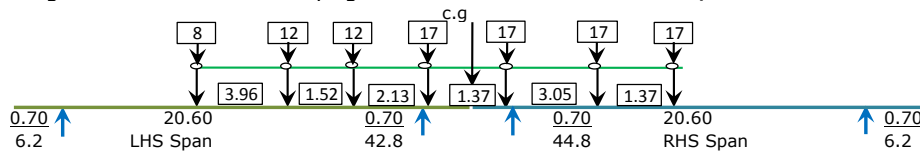
Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
CI A(1lane)	26.0	24.8	50.9	4.950	0.700	0.700	251.9	0.8
CI A(2lane)	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7
CI A(3lane)	70.3	67.1	137.4	1.450	0.700	0.700	199.2	2.3
CI A(4lane)	83.4	79.5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

2) Class 70 R Wheeled vehicle

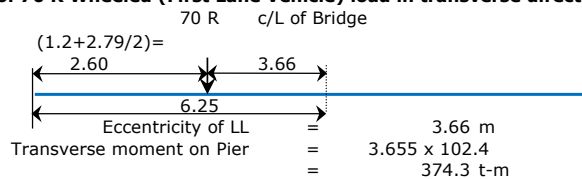
Impact factor for 70 R Wheel = 16.9%

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

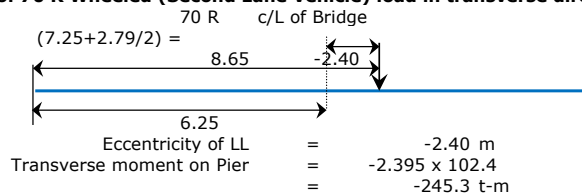
CI.208.3 of IRC:6-2017



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



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



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.
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		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
Total	71.0	72.0	143.1	-	-	-	363.0	2.4

Multiplied by 0.9 for 3 Lane effect

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)

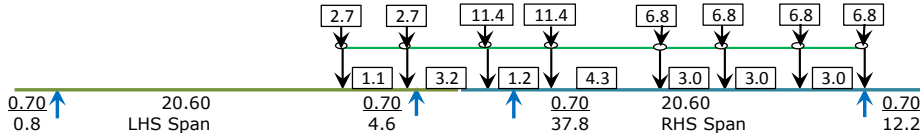
Type	Load (t)			Eccentricity (m)			Moment (t-m)	
	LHS Span	RHS Span	Total	Trans.	Long. LHS	Long. RHS	Trans.	Long.
Cl A 2Lane	52.1	49.7	101.8	3.200	0.700	0.700	325.7	1.7
70R Wheel	40.0	41.9	81.9	-2.295	0.700	0.700	-188.0	1.4
Total	92.1	91.6	183.7	-	-	-	137.7	3.0

Multiplied by 0.8 for 4 Lane effect

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

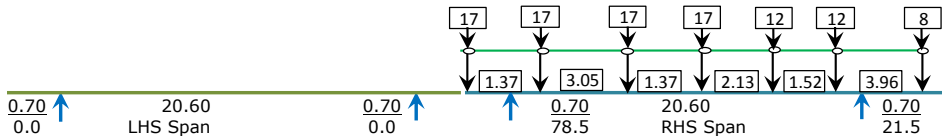
Reaction due to LL (Class A) with Impact factor

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
Total	5.4	126.9	132.2	-	-	-	310.6	76.5

Multiplied by 0.9 for 3 Lane effect

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	

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
Total	10.8	161.9	172.7	-	-	-	134.0	95.2

Multiplied by 0.8 for 4 Lane effect

Case 3 - Max. Longitudinal Force

Length of superstructure for breaking = 22.00 m

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IRC:6-2017

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- POT-PTFE		One Span Dislodqe condition-	
Type of bearing used	=				
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		Rev.	R0	Ref.
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		Design by	D.S.R	
		Check by	S.G	

Design horizontal frictional force		<u>LL1</u> 48.5 t	<u>LL2</u> 47.1 t	<u>LL3</u> 48.5 t	
POT-PTFE Bearing:		Fixed			Fixed
Coefficient of Friction μ	=	0.05 0.03			0.05 0.03
DL & SIDL reaction at free end, Rg	=	535.0 t			267.5 t
Live load reaction at at free end, Rq	=	<u>LL1</u> 143.1 t	<u>LL2</u> 102.4 t	<u>LL3</u> 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
Zone factor Z	=	0.24
Importance factor I	=	1.2

Fig:4.1 of IRC:SP:114 - 2018

Seismic Coefficient in Longitudinal Direction

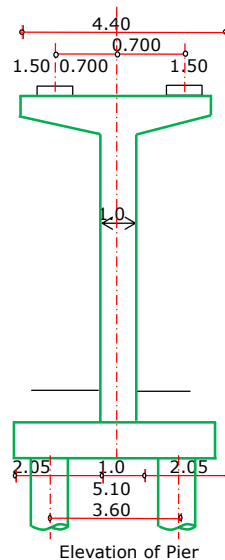
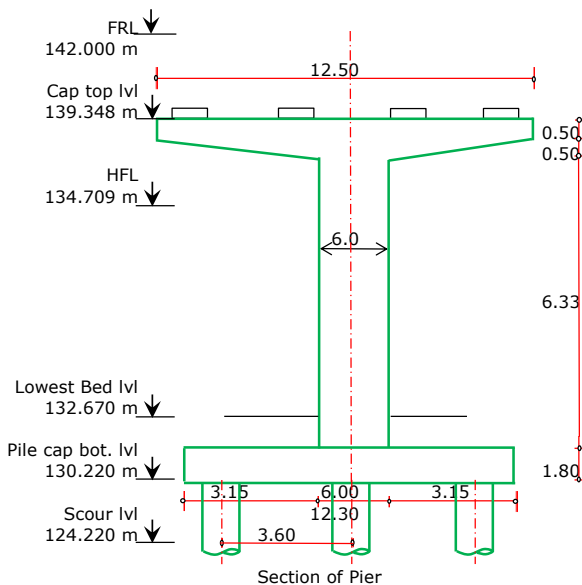
Horizontal seismic coefficient A_{HL}	=	0.180
Vertical seismic coefficient V_h	=	0.120

Seismic Coefficient in Transverse Direction

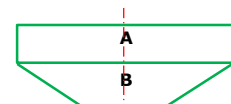
Horizontal seismic coefficient A_{HT}	=	0.180
Vertical seismic coefficient V_h	=	0.120

7.0 Load calculation of Substructure & Foundation

Dimensional details of Pier cap, Shaft & Footing



Height of pedestals,	
PD1 =	0.200 m
PD2 =	0.269 m
PD3 =	0.338 m
PD4 =	0.406 m
PD5 =	0.475 m



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

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		Check by	S.G	

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 & FPD	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 coefficient A _{HL}	=	0.180			
Seismic component of DL, SIDL & Surfacing	=	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 & Surfacing	=	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 & Surfacing	=	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			LHS Span			LHS Span		
Total Live load	=	71.0	RHS Span	=	50.0	RHS Span	=	5.4
Seismic Longitudinal force:- No Live load seismic component is considered in longitudinal direction								
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
Lever arm above base slab e _{V/Base}	=	12.980	12.980 m		12.980	12.980 m		12.980
Transverse moment M _T	=	166.0	168.2 t-m		116.8	122.4 t-m		12.6
Lever arm above Shaft e _{V/Shaft}	=	11.180	11.180 m		11.180	11.180 m		11.180
Longitudinal moment M _{T/Shaft}	=	143.0	144.9 t-m		100.6	105.5 t-m		10.8
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
Lever arm about c/L of base sl e _L	=	0.700	0.700 m		0.700	0.700 m		0.700
Longitudinal moment M _L	=	6.0	6.0 t-m		4.2	4.4 t-m		0.5

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

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

Total load of Substructure & Foundation	=	498.3 t
Total load of Substructure	=	216.0 t
Seismic Longitudinal force:-		
Longitudinal seismic coefficient A _{HL}	=	0.180
Seismic component of Substructure & Foun	=	89.7 t
Lever arm above base slab e _{V/Base}	=	3.533 m
Longitudinal moment M _L	=	316.9 t-m

Seismic component of Substructure	=	38.9 t
Lever arm above Shaft e _{V/Shaft}	=	5.175 m
Longitudinal moment M _{L/Shaft}	=	464.1 t-m

Seismic Transverse force:-

Transverse seismic coefficient A _{hT}	=	0.180
Seismic component of Substructure & Foun	=	89.7 t
Lever arm above base slab e _{V/Base}	=	3.533 m
Transverse moment M _T	=	316.9 t-m

Seismic component of Substructure	=	38.9 t
Lever arm above Shaft e _{V/Shaft}	=	5.175 m
Longitudinal moment M _{L/Shaft}	=	464.1 t-m

Seismic Vertical force:-

Vertical seismic coefficient A _v	=	0.120
Seismic component of Substructure & Foun	=	59.8 t
Lever arm about c/L of base sl e _L	=	0.000 m
Longitudinal moment M _L	=	0.0 t-m

Seismic component of Substructure	=	25.9 t
Lever arm about c/L of base sl e _L	=	0.000 m
Longitudinal moment M _L	=	0.0 t-m

HFL condition

Total load of Substructure & Foundation	=	367.7 t
Total load of Substructure	=	198.3 t
Seismic component of Substructure & Foun	=	66.2 t
Lever arm above base slab e _{V/Base}	=	4.273 m
Longitudinal moment M _L	=	282.8 t-m

Seismic component of Substructure	=	35.7 t
Lever arm above Shaft e _{V/Shaft}	=	5.354 m
Longitudinal moment M _{L/Shaft}	=	354.3 t-m

Seismic component of Substructure & Foun	=	66.2 t
Lever arm above base slab e _{V/Base}	=	4.273 m
Transverse moment M _T	=	282.8 t-m

Seismic component of Substructure	=	35.7 t
Lever arm above Shaft e _{V/Shaft}	=	5.354 m
Longitudinal moment M _{L/Shaft}	=	354.3 t-m

Seismic component of Substructure & Foun	=	44.1 t
Lever arm about c/L of base sl e _L	=	0.000 m
Longitudinal moment M _L	=	0.0 t-m

Seismic component of Substructure	=	23.8 t
Lever arm about c/L of base sl e _L	=	0.000 m
Longitudinal moment M _L	=	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 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	-

(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	-

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	-

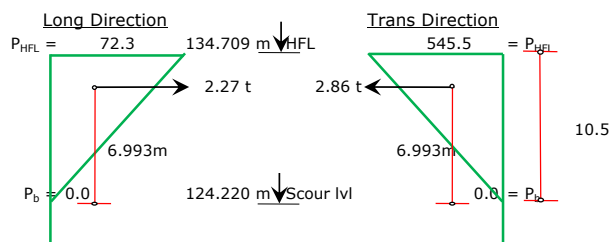
9.0 Water Current force calculation at HFL condition

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}	=	√ 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

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		Check by	S.G	

Pressure due to water current, $P = 52 \times K \times V_{\max}^2$ Kg/m²



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,	F_v	=	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
Transverse wind load,	F_T	=	16.4 t

$b / t = 7.50$ where, Width=b & Depth=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

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		Check by	S.G	

11.0 Summary of Forces at Founding Level

Summary of Forces Founding level (LWL)						Span Dislodge Condition				
						Forces about c/L of foundation				
						V	H _L	H _T	M _L	M _T
Description		t	t	t	t-m	t	t	t	t-m	t-m
1) DL of Superstructure		450.5			0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier		38.5			0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat		46.0			0.0	23.0			16.1	0.0
4) Carriageway Live load,	LL1	143.1		0.0	2.4					
	LL2	102.4		0.0	1.7					
	LL3	132.2		0.0	76.5					
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)						Span Dislodge Condition				
						Forces about c/L of foundation				
						V	H _L	H _T	M _L	M _T
Description		t	t	t	t-m	t	t	t	t-m	t-m
1) DL of Superstructure		450.5			0.0	225.2			157.7	0.0
2) SIDL - Crash Barrier		38.5			0.0	19.3			13.5	0.0
3) SIDL - Wearing Coat		46.0			0.0	23.0			16.1	0.0
4) Carriageway Live load,	LL1	143.1		0.0	2.4					
	LL2	102.4		0.0	1.7					
	LL3	132.2		0.0	76.5					
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

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	
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

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	

11.3 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					
		V	H _L	H _T	M _L	M _T	V	H _L	H _T	M _L	M _T
		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	

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	

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

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	

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

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

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.

- Span is less than 15.0 m.
- 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)

Horizontal seismic force,	A_v	=	0.667	$\times A_h$
where,	F_{eq}	=	$A_h \times (DL + b \times LL)$	(Refer Cl. 5.2.1 of IRC:SP:114-2018)
	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 Strata Soil Type	Notation
Rock/Hard Soil (I)	HS
Medium/ Stiff Soil (II)	MS
Soft Soil (III)	SS

$$\text{Horizontal force, } F = 6 EI / \{x^2 (3L - x)\}$$

where,

F = Force applied at cg of Superstructure or top of Bearing to produce 1mm deflection at Pier/Abutment 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

$$\text{Horizontal Seismic coefficient, } A_h = (Z/2) \times (S_a/g) / (R/I)$$

where,

$$\text{Zone Factor, } Z = 0.36$$

$$\text{Importance Factor, } I = 1.2$$

Resonance Reduction Factor, R

$$\text{Longitudinal direction, } R_L = 3.0$$

$$\text{Transverse direction, } R_T = 3.0$$

Elastic Seismic Acceleration Coefficient (ESAM), S_a/g

$$\text{Foundation soil strata} = SS$$

Elastic Seismic Acceleration Coefficient (ESAM) for 5% damping,

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

$$\text{Type of Pier/ Abutment} = RC$$

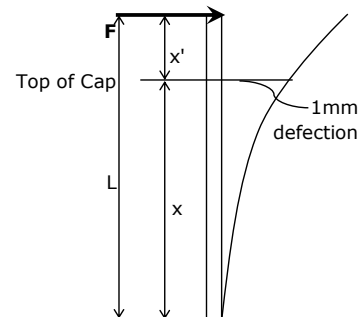
$$\text{Grade of Concrete} = M 35$$

$$\text{Youngs Modulus, } E = 3.2E+07 \text{ KN/m}^2$$

$$x = 7.328 \text{ m}$$

$$x'_L = 0.303 \text{ m}$$

$$x'_T = 1.207 \text{ m}$$



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.

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

Seismic Coefficient in Longitudinal Direction in Service Condition:

Size of Rectangular Pier/ Abutment		=	1.00	x	6.00	m
Moment of Inertia	I	=	$6 \times 1^3 / 12$			
		=	0.5	m ⁴		
Dia of Solid Pier	D	=	1.00	m		
Moment of Inertia	I	=	$\pi() \times 1^4 / 64$			
		=	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	=	$\pi() \times (1^4 - 0^4) / 64$			
		=	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	=	$2 \times (D / (1000 \times F)) ^{0.5}$			
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			
Horizontal Seismic Coefficient, A _{hL}		=	$(0.36/ 2) \times 2.5 / (3 / 1.2)$			
		=	0.180			
Vertical Seismic Coefficient, A _v		=	A _h *2/3 or 0			
		=	0.120			

HS	2.02
MS	2.50
SS	2.50

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		
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			

HS	2.50
MS	2.50
SS	2.50

Grade of concrete of cast-insitu Pile,	=	M 35
<u>Salient dimensions:</u> (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
<u>Salient dimensions:</u> (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
<u>Other Salient Design Data</u>		
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

Area of pile,	$A = \pi \cdot d^2 / 4$	=	1.131 m ²	
Moment of inertia pile,	$I = \pi \cdot d^4 / 64$	=	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,	$E_c = 22 \cdot ((f_{ck} + 10) / 12.5)^{0.3}$	=	3.23E+07 KN/m ²	3.29E+06
The depth of fixity (in m) is determined from the codal provisions and graphs,				

$$R = \sqrt{\frac{EI}{L}}$$

Thus, $R = \sqrt[4]{\frac{EI}{\pi}}$

Type of top layer soil providing lateral resistance = Granular soil

From table we get,

t_B	=	3.661 m
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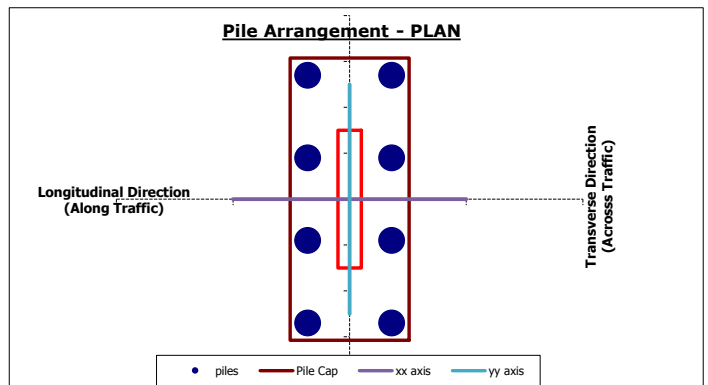
$$L_1 / T = 1.639$$
$$L_f / z_f = 8.054 \text{ m}$$
$$\text{Allowable deflection at scour level, } y_1 = 0.012 \text{ m}$$
$$\alpha = 0.000185$$
$$\text{Design lateral load for fixed head pile, } H = \frac{12 E I_y}{L^3}$$
$$= 519 \text{ kN}$$

Moment reduction factor, $m = 0.82$

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The diagram illustrates a pile of total length L subjected to a lateral load P at the top. The pile is divided into two segments: a fixed segment of length L_1 and a free segment of length L_f . The deflection curve is shown as a dashed line, with the maximum deflection at the top labeled y_2 . The scour level is indicated by a horizontal line at a depth L_f from the top, with the deflection at this level labeled y_1 . The scour level is also labeled as "1% of pile dia".

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,	$\epsilon_s = f_{yd} / E_s$	=	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	=	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)	1 st layer	=	20 mm
	2 nd layer	=	20 mm
Number of long bars	1 st layer	=	16
	2 nd layer	=	16
Area of steel provided,	A_{sc}	=	10053 mm ²
Percentage of steel provided		=	0.89 %
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

OK

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

Diameter of long bars (L1)	1 st layer	=	20 mm
Number of long bars	1 st 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}	=	$20 \cdot A \cdot B \cdot C / \sqrt{\dots}$
where,	A	=	$1 / (1 + 0.2 \cdot \phi_{ef})$
Creep Coefficient,	$\phi(\infty, 0)$	=	1.00
First order BM Quasi-Permanent load	M_{0Ed}	=	93 kN-m
First order BM in design load in ULS	M_{0Ed}	=	235 kN-m
Effective creep ratio,	$\phi_{ef} = \phi(\infty, 0) \cdot M_{0Ed} / M_{0Fd}$	=	0.40
	A	=	0.93
where,	B	=	$\sqrt{(1 + 2\omega)}$
Mechanical r/f ratio,	$\omega = A_s \cdot f_{yd} / A_c \cdot f_{cd}$	=	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	=	$N_{Ed} / A_c \cdot f_{cd}$

Eccentricity due to slenderness:

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

Eq.11.1
of IRC:112

2.51

Pile Moment Capacity Check at ULS :-* check for α value

Load case	N_{Ed} kN	M_{0Ed} kN-m	$n = N_{Ed} / (A_c \cdot f_{cd})$	λ_{lim}	K_r	e_2 (m)	M_{Ed} kN-m	$P_U / f_{ck} \cdot D^2$	$M_U / f_{ck} \cdot D^3$	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.
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 at ULS :-

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

Design of section without shear reinforcement:-

Factor for concrete cracked in shear, $v = 0.6(1 - f_{ck}/310) = 0.53$
 $K = \text{Min}.[1 + (200/d)^{0.5}, 2] = 1.43$
 $\rho_1 = \text{Min}.[A_{st} / b_w \cdot d, 0.02] = 0.008$
 $v_{min} = 0.031 \cdot K^{3/2} \cdot f_{ck}^{1/2} = 0.312 \text{ MPa}$
Mean Comp. stress in concrete, $\sigma_{cd} = \text{Min}[N_{Ed}/A_c, 0.2f_{cd}] = 0 \text{ MPa}$
Min. Design Shear resistance, $V_{Rd,c1} = (v_{min} + 0.15 \cdot \sigma_{cd}) b_w \cdot d = 412 \text{ kN}$
 $V_{Rd,c2} = [0.12 \cdot K \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} + 0.15 \cdot \sigma_{cd}] b_w \cdot d = 620 \text{ kN}$
Design Shear resistance, $V_{Rdc} = \text{Max}.[V_{Rd,c1} \text{ \& } V_{Rd,c2}] = 620 \text{ kN}$

Check for Shear R/f (SR) requirement, **SR NRQ**

Design of section with shear reinforcement:-

Dia. of Shear r/f, stirrups/ links $\phi = 16 \text{ mm}$
Number of stirrups/ links $n = 1$
Spacing of stirrups/ links $s = 95 \text{ mm}$
Area of Shear r/f provided, $A_{sw} = 201 \text{ mm}^2$
 $A_{sw} / s = 2.1 \text{ mm}^2/\text{mm}$
Minimum area of Shear r/f required, $\text{Min. } A_{sw} / s = 1.0 \text{ mm}^2/\text{mm}$
Status, Area of Shear r/f, **OK**
Lever arm factor, $\alpha_{cw} = 0.9$
Coeff. depends on stress in compression cord, $\alpha_{cw} = 1.0$
Factor for concrete cracked in shear, $v_1 = v = 0.53$
Shear capacity of concrete strut corresponding, $\theta = 45 \text{ degree}$
 $V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) = 4938 \text{ kN}$
Status, Check for Redesign of section, **OK**
Angle of concrete strut due shear, $\theta = 2.3 \text{ degree}$
Shear capacity of strut, $V_{Rd} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot \theta + \tan \theta) = 392 \text{ kN}$
 0
Angle of concrete strut considered, $\theta = 21.8 \text{ degree}$
Shear resistance provided by r/f, $V_{Rd,s} = A_{sw} / s \cdot z \cdot f_{ywd} \cdot \cot \theta = 2094 \text{ kN}$
Status, Check for shear capacity of section, **OK**
5.3

*RQ - Required

*NRQ - Not Required

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, $L_0 = 8.054 \text{ m}$
Effective length factor, $= 1.00$
Effective length of column $L_e = 8.054 \text{ m}$
Radius of Gyration, $r = 0.300 \text{ m}$
 $L_e / r = 26.8$
Short column

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

Confinement 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, $A_{sh} = 0.09SD_k f_{ck}/f_y [A_g/A_k - 1.0]$	=	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_g	=	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, $\omega_{wd.c}$	≥	$\max(1.4 \omega_{wreq}; 0.18)$

Normalised axial force, $n_k = N_{Ed} / (A_c * f_{ck}) > 0.08$ = 0.08 0.08

R/f ratio of longitudinal r/f, $\rho_L = A_s / A_c$ = 0.009

$\omega_{wreq} = 0.37 * A_c / A_{cc} * n_k + 0.13 * f_{yd} / f_{cd} * (\rho_L - 0.01)$ = 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

Eq. 17.7
of IRC:112

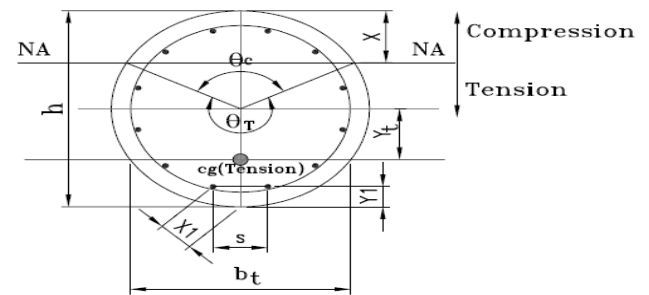
Flexure design of Pile at SLS :-**Material Data**

Concrete strength of cast-in-situ Pile, f_{ck}	=	35 MPa
Modulus of Elasticity of Concrete, E_{cm}	=	32308 MPa
Characteristic 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

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, 1 st layer	=	20 mm
2 nd layer	=	20 mm
Number of long bars, 1 st layer	=	16 Nos.
2 nd 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

at SLS Rare load combination

**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-max	1540	93	1.7	0.8	Uncracked	2155	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-max	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-max	1248	93	1.4	0.6	Uncracked	1861	6.3	0.0	1130973	1.018E+11	1245385
SQ-P:MT-max	836	55	0.9	0.4	Uncracked	2029	6.3	0.0	1130973	1.018E+11	1245385

* check formula

* check formula

Load	Eccentricity	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-max	0.0	60.7	1.176E+11	1555	1555	0	0.0	1.7	10.4
SQ-P:MT-max	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

Max. Concrete Compressive Stress	=	2.7 MPa	OK
Max. Tensile Stress in Steel	=	-7.8 MPa	OK

Max. Concrete Compressive Stress	=	1.7 MPa	OK
Max. Tensile Stress in Steel	=	0 MPa	OK

Cl 12.3 of
IRC:112

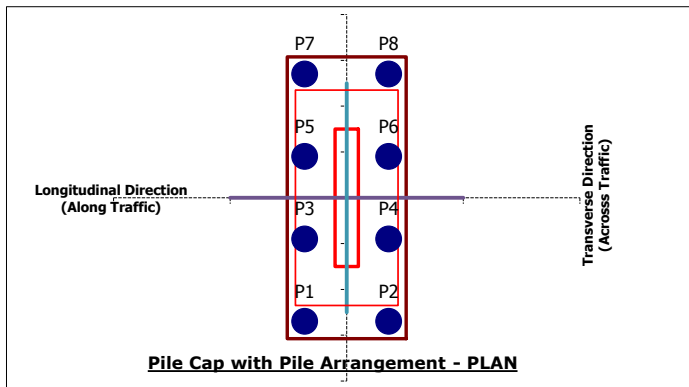
Coefficient depends on duration of load	K_t	=	0.50
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[illegible]

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

15.0 Design of Pile Cap by Bending theory :-

Grade of concrete of Pile cap,	=	M 35
<u>Salient dimensions:</u> (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
<u>Salient dimensions:</u> (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
<u>Material property</u>		
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
<u>Reinforcement details</u>		
Clear cover to reinforcement	=	75 mm
Dia. of longitudinal r/f at bottom	=	25 mm
Dia. of transverse 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

**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.
		0.000 m, Pile no.
along Transverse direction	=	2.400 m, Pile no.
	=	-1.200 m, Pile no.

P1	P3	P5	P7		
P1	P2				
P3	P4				

For opposite face check:-

Lever arm of piles from face,		
along Longitudinal direction	=	1.300 m, Pile no.
	=	0.000 m, Pile no.
along Transverse direction	=	2.400 m, Pile no.
	=	-1.200 m, Pile no.

P2	P4	P6	P8		
P7	P8				
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

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,	$\epsilon_s = f_{yd} / E_s$	=	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	=	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,	$p_{t,req} = f_{ck} / (2f_{yk}) \cdot (1 - (1 - (4.598 \cdot R / f_{ck})^2))$	=	0.0018	0.0003
Min. % of steel required,	$p_t = \max.[0.26 \cdot f_{ctm} / f_{yk}, 0.0013]$	=	0.0014	0.0014
Area of steel required for bending,	$A_{st, req}$	=	15732 mm ²	29698 mm ²
R/f provided,	Dia. 1 st layer	=	25 mm	25 mm
	Spacing	=	125 mm c/c	125 mm c/c
	Dia. 2 nd 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,	$x_{u,max} = 0.456 \cdot d$	=	776 mm	764 mm
Actual depth of N.A,	$x = (0.87 \cdot f_{yk} \cdot A_{st}) / (0.362 \cdot f_{ck} \cdot b)$	=	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,	$T = A_{st} \cdot f_s$	=	8368 kN	20661 kN
Total compressive force in concrete,	$C = 0.362 \cdot f_{ck} \cdot x \cdot b$	=	8368 kN	20661 kN
Difference of force,	$C - T$	=	0	0
Lever arm of compressive force in concrete,	$z = (d - 0.416 \cdot x)$	=	1647 mm	1620 mm
Moment of resistance,	$M_{Rd} = C \cdot z$	=	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

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

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

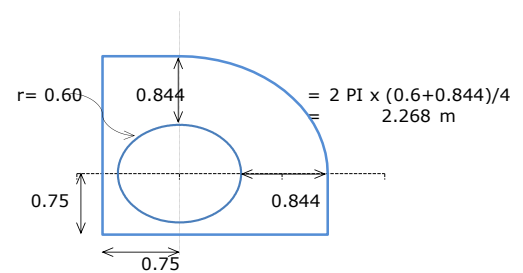
		Long. Direction	Trans. Direction	
Design Shear Force of the concrete section,	$V_{NS} = V_{Ed}$	=	3575 kN	3042 kN
Design of section without shear reinforcement:-				
Factor for concrete cracked in shear,	$v=0.6(1-f_{ck}/310)$	=	0.53	0.53
Max. $V_{Ed} = 0.5 b_w \cdot d \cdot v \cdot f_{cd}$		=	36082 kN	85742 kN
Status,			OK	OK
$K = \text{Min.}[1 + (200/d)^{0.5}, 2]$		=	1.34	1.35
$\rho_1 = \text{Min.}[A_{sl} / b_w \cdot d, 0.02]$		=	0.002	0.002
$v_{min} = 0.031 \cdot K^{3/2} \cdot f_{rk}^{1/2}$		=	0.285 MPa	0.286 MPa
Mean Comp. stress in concrete,	$\sigma_{cd} = \text{Min}[N_{Ed}/A_{cr}, 0.2f_{cd}]$ Mpa	=	0 MPa	0 MPa
Min. Design Shear resistance,	$V_{Rd,c1} = (v_{min} + 0.15 \cdot \sigma_{cd}) b_w \cdot d$	=	2475 kN	5899 kN
$V_{Rd,c2} = [0.12 \cdot K \cdot (80 \cdot \rho_1 \cdot f_{rk})^{0.33} + 0.15 \cdot \sigma_{cd}] b_w \cdot d$		=	2554 kN	6158 kN
Design Shear resistance,	$V_{Rdc} = \text{Max.}[V_{Rd,c1} \& V_{Rd,c2}]$	=	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
Minimum area of Shear r/f required,	A_{sw} / s	=	9.0 mm ² /mm	14.5 mm ² /mm
Status, Area of Shear r/f,	Min. A_{sw} / s	=	5.4 mm ² /mm	13.1 mm ² /mm
Lever arm factor,		=	OK	OK
Coeff. depends on stress in compression cord,	α_{cw}	=	0.9	0.9
Factor for concrete cracked in shear,	$v_1 = v$	=	1.0	1.0
Shear capacity of concrete strut corresponding,	θ	=	0.53	0.53
$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$		=	45 degree	45 degree
Status, Check for Redesign of section,		=	32474 kN	77168 kN
Angle of concrete strut due shear,	θ	=	OK	OK
Shear capacity of strut,	$V_{Rd} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$	=	3.2 degree	1.1 degree
Angle of concrete strut considered,	θ	=	3575 kN	3042 kN
Shear resistance provided by r/f,	$V_{Rd,s} = A_{sw} / s \cdot z \cdot f_{vw} \cdot \cot\theta$	=	0	0
Status, Check for shear capacity of section,		=	21.8 degree	21.8 degree
		=	13848 kN	21831 kN
		=	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 pile 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,	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$	=	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,	$v_{Ed} = \beta V_{Ed,red} / (u_o \cdot d)$	=	1.57 MPa
Max. Punching shear resistance,	$V_{Rd,max} = 1/2 \cdot v \cdot f_{cd}$	=	4.16 MPa
		=	OK
Check at a distance 'd/2' from Pile perimeter,			
Factored Shear due to s/w of Cap (incl. 15% subm	ΔV_{Ed}	=	844 mm
Net applied Shear force,	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$	=	249 kN
Control perimeter,	u_i	=	3478 kN
Punching Shear stress,	$v_{Ed} = V_{Ed,red} / (u_i \cdot d)$	=	2268 mm
		=	0.91 MPa
$K = \text{Min.}[1 + (200/d)^{0.5}, 2]$		=	1.34
$v_{min} = 0.031 \cdot K^{3/2} \cdot f_{rk}^{1/2}$		=	0.286 MPa
Mean value of percentage of steel provided,	ρ_l	=	0.003
Punching shear resistance at control perimeter,		=	
$V_{Rd} = \text{Min.}[0.12K(80\rho_l f_{rk})^{0.33} \cdot 2d/a, v_{min} \cdot 2d/a]$		=	1.14 MPa
		=	OK

where, $v=0.6(1-f_{ck}/310)$

**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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		Check by	S.G	

BM _L (kN-m)	5807	1848	3640	4520	5088	4001	5088	2942
------------------------	------	------	------	------	------	------	------	------

BM* _L (kN-m)								
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BM _T (kN-m)	1693	649	1272	1047	1972	1470	1972	1155
------------------------	------	-----	------	------	------	------	------	------

BM* _T (kN-m)								
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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,	$T = A_{st} \cdot \sigma_s$	= 3670	3215 kN	= 1088	1266 kN	from Stress diagram
Total comp. force in concrete,	$C = 0.5 \cdot \sigma_c \cdot x \cdot b$	= 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,	$M_s = T \cdot (d - x_1)$	= 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	
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	
Transformed Moment of Inertia,	I_{NA}	= 5.074E+11	5.074E+11	= 1.209E+12	1.209E+12	
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,	$\alpha_e = E_s / E_{cm}'$	=	0.30	mm	for	Moderate	Exposure
		=	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
Calculated spacing b/w bars,	$5 \cdot (c + \phi_{eq}/2)$	=	437.5	mm	438
Status		OK		OK	
Overall depth of Pile cap,	D	=	1800	mm	1800
Depth of Neutral axis,	x	=	355	mm	355
Tensile stress in R/f,	σ_{sc}	=	167.1	MPa	26.6
Area of steel,	A_s	=	19246	mm ²	47521
Eff. depth in tension,	$h_{c,eff} = \text{Min}.[2.5 \cdot (h-d), (h-x)/3, h/2]$	=	249	mm	311
Eff. area of concrete within tensile zone,	$A_{c,eff} = h_{c,eff} \cdot b$	=	1268625	mm ²	3828375
	$\rho_{peff} = A_s / A_{c,eff}$	=	0.015		0.012
	$\epsilon_{sm} - \epsilon_{cm} = \text{Max}\{[\sigma_{sc} - k_t \cdot f_{ct,eff}(1 + \alpha_e \cdot \rho_{peff}) / E_s] \& (0.6 \cdot \sigma_{sc} / E_s)\}$	=	0.0005		0.0001
	$S_{r,max} = 3.4c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{p,eff}$	=	535		597
	$W_k = S_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm})$	=	0.268	mm	0.048
Status		OK		OK	

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		Check by	S.G	

16.0 Design of Pier Shaft:-

Grade of concrete of cast-insitu Pile,	=	M 35
<u>Material data:</u>		
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,	$\epsilon_s = f_{yd} / E_s$	0.00217
Max. strain of steel,	$\epsilon_{s,max} = 0.002 + f_{yd} / E_s$	0.00417
Creep Coefficient,	ϕ	1.50

Geometric data: (Pier / Abutment)

		<u>Long. Direction</u>	<u>Trans. Direction</u>	
Dimension of Pier/ Abutment,	=	1000 mm	6000 mm	Wall type Pier
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	
<u>Vertical longitudinal r/f detail of Column on each face:</u>				
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}	=	$20 \cdot A \cdot B \cdot C / \sqrt{n}$	Eq.11.1
where,	A	=	$1 / (1 + 0.2 \cdot \phi_{ef})$	of IRC:112
Creep Coefficient,	$\phi(\infty, 0)$	=	1.50	
First order BM Quasi-Permanent load	M_{oEao}	=	1693 kN-m	
First order BM in design load in ULS	M_{oEd}	=	7831 kN-m	
Effective creep ratio,	$\phi_{ef} = \phi(\infty, 0) \cdot M_{oFnn} / M_{oFrd}$	=	0.32	

where,	B	=	$\sqrt{(1 + 2\omega)}$
Mechanical r/f ratio,	$\omega = A_s \cdot f_{yd} / A_c \cdot f_{cd}$	=	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	=	$N_{Ed} / A_c \cdot f_{cd}$

Eccentricity due to slenderness:

Mominal second order moment,	M_2	=	$N_{Ed} \cdot e_2$
Deflection,	e_2	=	$(1/r) \cdot l_e^2 / c$
Factor for constant cross section,	c	=	10
Curvature,	$1/r$	=	$K_r \cdot K_\phi \cdot 1/r_0$
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	=	$1 + \beta \cdot \phi_{ef}$
	$\beta = 0.35 + (f_{rk}/200) - (\lambda/150)$	=	0.14
	K_ϕ	=	1.04
	$1/r_0 = f_{yd} / (E_s \cdot 0.45d)$	=	0.0030 1/m

0.46
1.15

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		Check by	S.G	

Pier Moment Capacity Check at ULS :-

Load case	N _{Ed} kN	M _{0EdIL} kN-m	M _{0EdIT} kN-m	H _L kN	H _T kN	n = N _{Ed} / (A _c * f _{cd})	λ _{lim}	K _r	β _L	β _T	K _{φL}	K _{φT}
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 _{EdIL} kN-m	M _{EdIT} kN-m	M _{UL} kN-m	M _{UT} kN-m	N _{Ed} / N _{Rd}	α	(M _{Ed} / M _u) ^α	
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
<u>Design of section without shear reinforcement:-</u>			
Factor for concrete cracked in shear,	v = 0.6(1 - f _{ck} /310)	0.53	0.53
	K = Min.[1 + (200/d) ^{0.5} , 2]	1.47	1.18
	ρ ₁ = Min.[A _{sl} / b _w d, 0.02]	0.003	0.003
	v _{min} = 0.031 * K ^{3/2} * f _{rk} ^{1/2}	0.325 MPa	0.236 MPa
Mean Comp. stress in concrete,	σ _{cd} = Min[N _{Ed} /A _c , 0.2f _{cd}] Mpa	1.8 MPa	1.8 MPa
Min. Design Shear resistance,	V _{Rd,c1} = (v _{min} + 0.15*σ _{cd})b _w d	3321 kN	3029 kN
	V _{Rd,c2} = [0.12*K*(80+ρ ₁ *f _{rk}) ^{0.33} + 0.15*σ _{cd}]b _w d	3449 kN	3261 kN
Design Shear resistance,	V _{Rdc} = Max.[V _{Rd,c1} & V _{Rd,c2}]	3449 kN	3261 kN
Check for Shear R/f (SR) requirement,		SR NRQ	SR NRQ
<u>Design of section with shear reinforcement:-</u>			
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,	v ₁ = v	0.53	0.53
Shear capacity of concrete strut corresponding,	θ	45 degree	45 degree
	V _{Rd,max} = α _{cw} *b _w *Z*v ₁ *f _{cd} /(cotθ+tanθ)	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} = α _{cw} *b _w *Z*v ₁ *f _{cd} /(cotθ+tanθ)	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 _{wd} *cotθ	-44470 kN	-1673253 kN
		-19.8	-795.0

*RQ - Required
*NRQ - Not Required

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}	=	ρ _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,	ω _{wd,c}	≥	max(ω _{wd,req} ; 0.12)
Confining r/f ratio,	ω _{wd,req}	=	0.37*A _g /A _{cc} *ρ _k +0.13*f _{yd} /f _{cd} *(ρ _L -0.01)
Where,	n _k = N _{Ed} /(A _c *f _{ck})	=	0.08
	ρ _L = A _g /(b * D)	=	0.01
	ω _{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

Eq. 17.5
of IRC:112

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		Check by	S.G	

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 Footpath/ Kerb	=	0.000 m	
4) Width of Footpath/ 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/ Diaphragm	=	0.300 m	
15) Width of End Cross girder/ Diaphragm	=	0.400 m	
16) Girder length beyond c/l of Brg. on each side	=	0.450 m	

1.2 Material Properties:

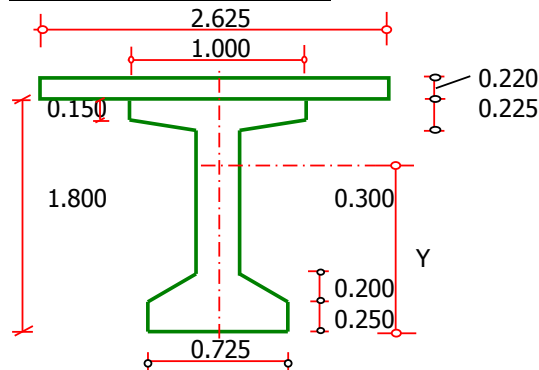
1) Concrete Grade for Deck slab	=	M 45
2) Concrete Grade for Cross girder	=	M 45
3) Concrete Grade of PSC Longitudinal girder	=	M 45
4) Reinforcement	=	Fe 500
5) Clear Cover	=	40 mm
6) Condition of Exposure	=	Moderate
7) Cable/ Strands	=	19 T 13 conforming to IS 14268
		(Stress-relieved multiply strands 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	I _z
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 flange	0.631	0.450	0.284	0.0114
I _z =				0.0274

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

beff = $\Sigma beff,i + bw$	=	7.465 m
c/c Spacing of Longitudinal girder, b	=	2.625 m

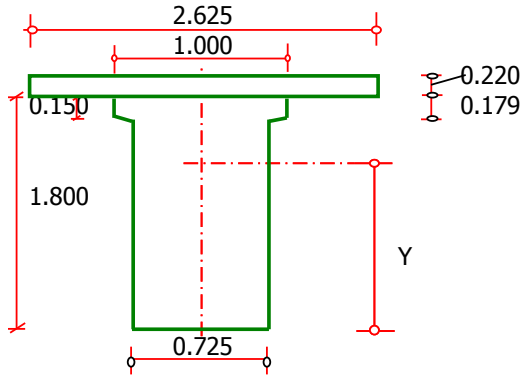
SL.No.	A	y	Ay	Ay ²	I _o	I _o + Ay ²	u
1. Deck Slab	0.57750	1.91000	1.10303	2.10678	0.002329	2.10911	4.690
2. Rectangular top Flange	0.15000	1.72500	0.25875	0.44634	0.000281	0.44663	0.300
3. Triangular top Flange	0.02625	1.62500	0.04266	0.06932	0.000008	0.06932	0.026
4. Web + Top & Bottom Rect. Haunch	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)	$Y = \Sigma(A.y) / \Sigma A$	=	1.3163 m
Moment of inertia of composite girder	$I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.6594 m ⁴

Section Properties of Girder

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

Girder Section at the Support**Calculation of Torsional Constant**

Component	Width	Depth	Area	I _z
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 flange	0.725	0.450	0.326	0.0143

$$I_z = 0.1166$$

$$\begin{aligned} \text{Modulus of Elasticity of Deck Slab} &= 34313 \text{ MPa} \\ \text{Modulus of Elasticity of Girder} &= 34313 \text{ MPa} \end{aligned}$$

$$\begin{aligned} b_{eff} &= \Sigma b_{eff,i} + b_w = 7.805 \text{ m} \\ \text{c/c Spacing of Longitudinal girder, } b &= 2.625 \text{ m} \end{aligned}$$

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. Rectangular 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

$$\begin{aligned} \text{Area} &= \Sigma A = 1.9278 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} &= Y = \Sigma(A.y) / \Sigma A = 1.2218 \text{ m} \\ \text{Moment of inertia of end intermediate girder} &= I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2 = 0.7746 \text{ m}^4 \end{aligned}$$

Section Properties of Girder

$$\begin{aligned} \text{Area} &= \Sigma A = 1.3503 \text{ m}^2 \\ \text{Distance of cg from bottom fibre (Y)} &= Y = \Sigma(A.y) / \Sigma A = 0.9274 \text{ m} \\ \text{Moment of inertia of end intermediate girder} &= I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2 = 0.3817 \text{ m}^4 \end{aligned}$$

1.1 Summary of Section Properties of Longitudinal Girder (Outer)**A. Section Properties of Composite Section at the Span**

$$\begin{aligned} \text{Area} &= 1.3975 \text{ m}^2 & Y_b &= 1.3163 \text{ m} & Y_{ts} &= 0.7037 \text{ m} \\ I_z &= 0.6594 \text{ m}^4 & Z_b &= 0.5010 \text{ m}^3 & Z_{ts} &= 0.9371 \text{ m}^3 \\ & & & & Y_{tg} &= 0.4837 \text{ m} \\ & & & & Z_{tg} &= 1.3633 \text{ m}^3 \end{aligned}$$

B. Section Properties of Girder at the Span

$$\begin{aligned} \text{Area} &= 0.8200 \text{ m}^2 & Y_b &= 0.8982 \text{ m} & Y_t &= 0.9018 \text{ m} \\ I_z &= 0.3102 \text{ m}^4 & Z_b &= 0.3453 \text{ m}^3 & Z_t &= 0.3440 \text{ m}^3 \end{aligned}$$

C. Section Properties of Composite Section at the Support

$$\begin{aligned} \text{Area} &= 1.9278 \text{ m}^2 & Y_b &= 1.2218 \text{ m} & Y_{ts} &= 0.7982 \text{ m} \\ I_z &= 0.7746 \text{ m}^4 & Z_b &= 0.6340 \text{ m}^3 & Z_{ts} &= 0.9704 \text{ m}^3 \\ & & & & Y_{tg} &= 0.5782 \text{ m} \\ & & & & Z_{tg} &= 1.3395 \text{ m}^3 \end{aligned}$$

D. Section Properties of Girder at the Support

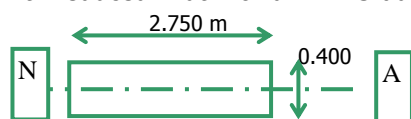
$$\begin{aligned} \text{Area} &= 1.3503 \text{ m}^2 & Y_b &= 0.9274 \text{ m} & Y_t &= 0.8726 \text{ m} \\ I_z &= 0.3817 \text{ m}^4 & Z_b &= 0.4116 \text{ m}^3 & Z_t &= 0.4374 \text{ m}^3 \end{aligned}$$

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphragm 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.)



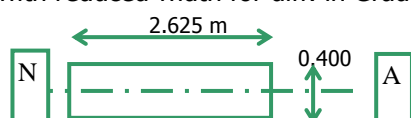
Modulus of Elasticity ratio = 1.000

Area = 1.1000 m²

Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0147 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**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 m²

Distance of cg from bottom fibre (y) = 0.2000 m

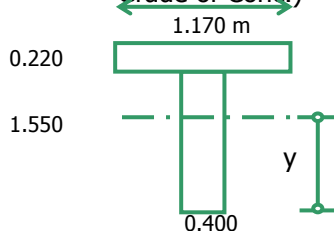
Moment of inertia (I_z) = 0.0140 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**End Cross Girder****_ENDCG**

(with reduced width for diff. in

beff = Min{ $\sum beff_{i,i} + bw, b$ } = 1.170 m

Grade of Conc.)

Modulus of Elasticity ratio = 1.000

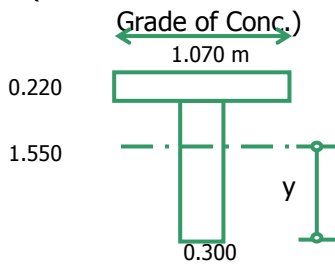
Area = 0.8774 m²

Distance of cg from bottom fibre (y) = 1.0346 m

Moment of inertia of end x-girder (I_z) = 0.2676 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴

Intermediate/ Inner Cross Girder

(with reduced width for diff. in



$$b_{eff} = \text{Min}\{\sum b_{eff,i} + b_w, b\}$$

Modulus of Elasticity ratio

_INTCG

= 1.070 m

= 1.000

Area = 0.7004 m²

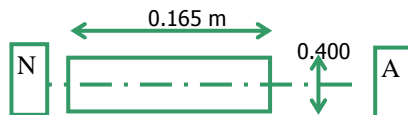
Distance of cg from bottom fibre (y) = 1.0724 m

Moment of inertia of end x-girder (I_z) = 0.2165 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

Cantilever Slab (At Expansion End)

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



Modulus of Elasticity ratio = 1.000

Area = 0.0660 m²

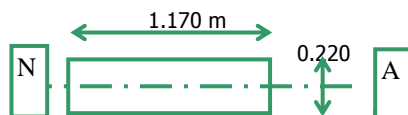
Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2574 m²

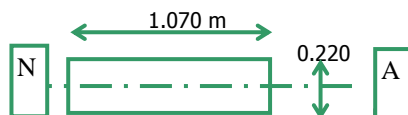
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0010 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2354 m²

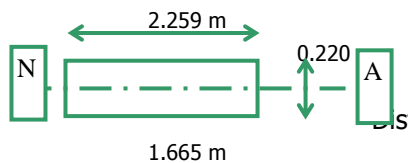
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSCANT2**Intermediate Slab (Next to End Cross Girder)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.4969 m²

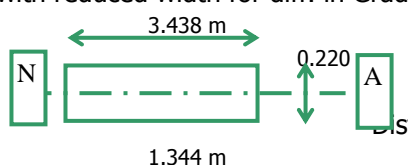
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0020 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB1**Intermediate Slab (At Splayed/ Tapered Portion)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.7563 m²

Distance of cg from bottom fibre (y) = 0.1100 m

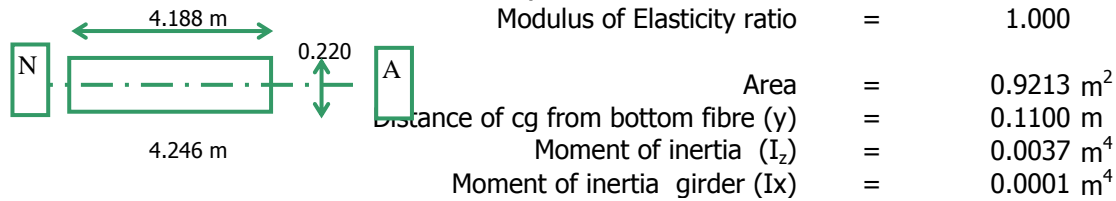
Moment of inertia (I_z) = 0.0031 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB2

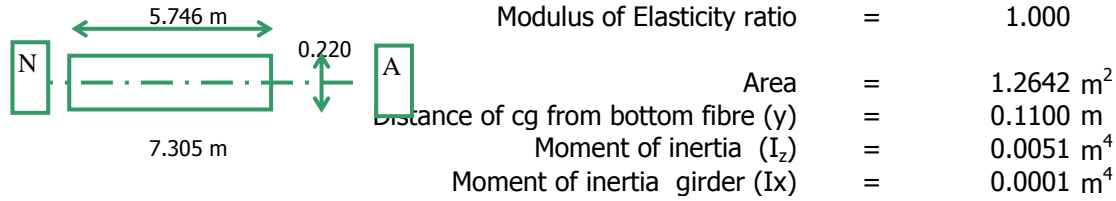
Intermediate Slab (Adjacent to First Inner Cross Girder) _TRANSLAB3

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



Intermediate Slab (Between Inner Cross Girders) _TRANSLAB4

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



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 6-2014
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 ratio 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 Obstruction		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
Traverse wind Force		F_T	=	$P_z \times A_1 \times G \times C_D$
			=	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	=	$P_z \times A_3 \times G \times C_L$
			=	507.3 KN
UDL on girder			=	1.1 KN/m ²
Outer girder			=	2.8 KN/m
Inner girder			=	3.0 KN/m
				2.82 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	<u>Load Case 1</u>	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	<u>Load Case 2</u>	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	<u>Load Case 3</u>	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	<u>Load Case 4</u>	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	<u>Load Case 5</u>	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	<u>Load Case 6</u>	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	<u>Load Case 7</u>	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	<u>Load Case 8</u>	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	<u>Load Case 9</u>	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 <u>Load Case 10</u> 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 <u>Load Case 11</u> 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 <u>Load Case 12</u> 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 <u>Load Case 13</u> 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 <u>Load Case 14</u> Class A 4 Lane Placed most Eccentric (Left side)			
1st Lane Z1 = $0.5 + 0 + 0.15 + 0.50/2$	=	0.90 m	From CB
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 <u>Load Case 15</u> 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 <u>Load Case 16</u> Class A 4 Lane Placed most Eccentric (Right side)			
1st Lane Z1 = $13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8$	=	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	<u>Load Case 17</u>	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	<u>Load Case 18</u>	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	<u>Load Case 19</u>	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	<u>Load Case 20</u>	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	<u>Load Case 21</u>	70R W for combination of Class A 2 Lane Eccentric on Left	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
	2nd Lane	$Z3 = 2.7 + 0.5 + 1.2$	=	4.40 m
	Class A	$Z4 = 4.4 + 1.8$	=	6.20 m
	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
				1.800
				5.300
				9.045
22	<u>Load Case 22</u>	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
Density of Concrete Girder	=	25.0 KN/m ³
Density 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	= 0.82×25	=	20.5 KN/m
Weight of Girder at End section	= 1.35×25	=	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
Weight of Intermediate X-girder	=	=	3.6 KN

Weight of the Shuttering Slab during Stage-2 Loading

Weight of Shuttering	=	5.0 KN/m
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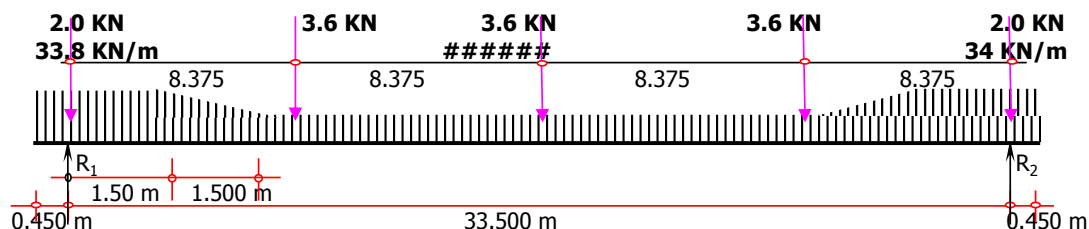
Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	= $2.8 \times 0.22 \times 26$	=	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

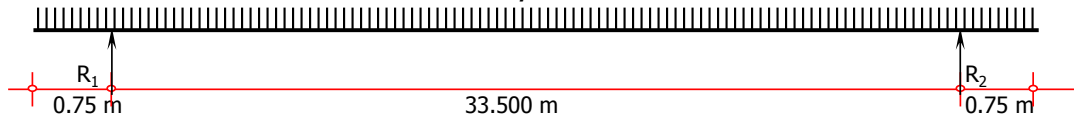


Support Reaction

$R_1 = 395.7 \text{ KN}$

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

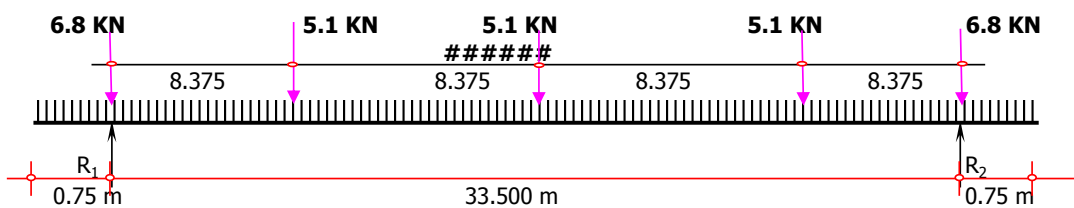
Stage-2 loading

Shuttering Load for Deck Slab**5.0 KN/m****Support Reaction**

$R_1 = 87.5 \text{ KN}$

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**

$R_1 = 289.7 \text{ KN}$

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 Momen	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
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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

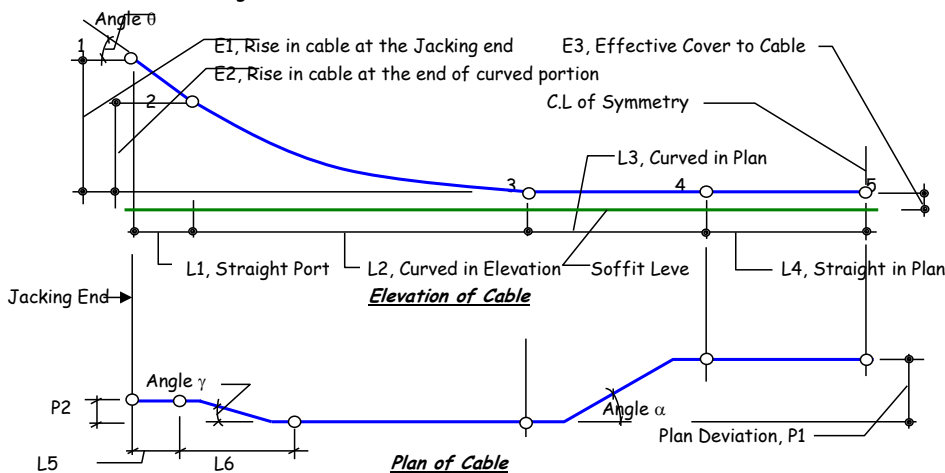
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.1875m	8.375m	12.5625m	16.75m	
1st Stage DL	378.5	262.9	173.5	87.6	0.0	1.0
2nd Stage DL	271.1	205.2	134.3	68.4	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat)	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	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 Corrugated HDPE sheathing (Refer Table 6.2 , IRC:112) (Refer Table 6.2 , IRC:112)
13) Wobble Friction Coefficient	k	0.002	per metre	
14) Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		
15) Relaxation loss at 1000 hrs at 50% UTS (as % of initial stress)	Re2	0		
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 Stage stressing in 14 days	Fc _j	40.6	MPa	
21) Modulus of Elasticity of Concrete at 1st Stage stressing	Ec _j	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



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

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
	P_f	2478.218	2438.394	2408.571	2388.088	2367.605
	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
	P_f	2478.218	2438.394	2408.571	2388.088	2367.605
	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
	P_f	2340.543	2322.244	2272.127	2252.852	2233.577
	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)	$P_s \cdot \cos\theta$ (KN)	$P_s \cdot \cos\theta \cdot Y_{ord}$ (KN.m)	$P_s \cdot \sin\theta$ (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		

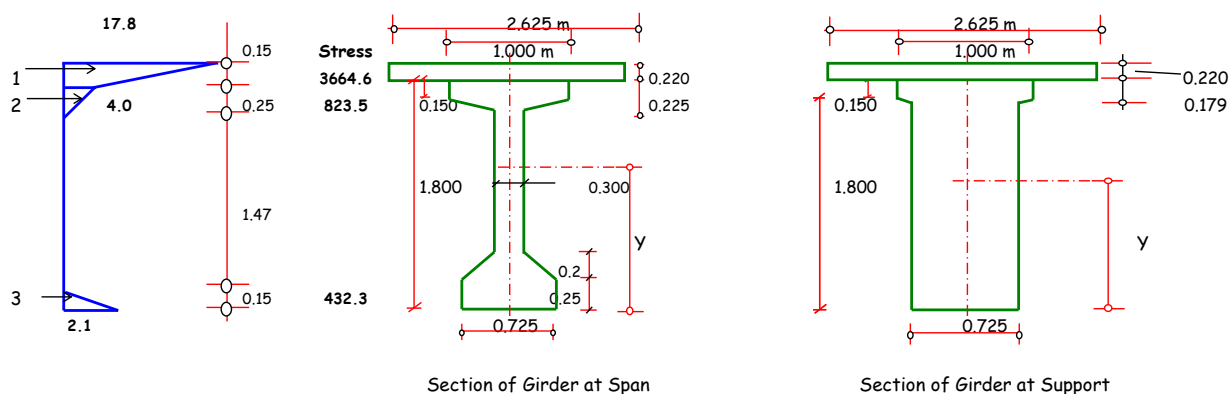
(*) Notations Used :

 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	E_c	=	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



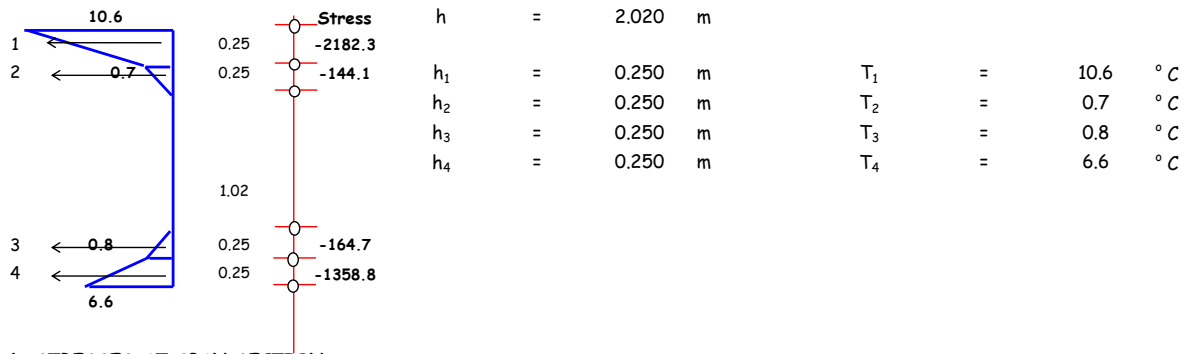
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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	0.150	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	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	0.150	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



4.2.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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 _†	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 _†	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 _†)	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 _†	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						
Stress after P-E loss	KN	377.5	377.5	377.5	377.5	377.5
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 _†	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.	%	1.846	1.959	2.142	2.241	2.315
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs.	%	83.99				
(Ref. Table 6.2, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 7 to 28 days	KN	= 83.99/100 * 2.315/100 * (0.75*183.7*83) * 1				
	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	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at 28 days	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	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at 28 days	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				
	=	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.	%	1.846	1.959	2.142	2.241	2.315
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs.	%	12.81				
(Ref. Table 6.3, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 28 to 49 days	KN	= 12.81/100 * 2.315/100 * (0.75*183.7*83) * 1				
	KN	27.2	28.8	31.5	33.0	34.1
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at 28 days	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at 49 days	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	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at 49 days	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 to Infinity	KN	= 203.2/100 * 2.315/100 * (0.75*183.7*83) * 1				
	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. Stress 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 with 75% 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

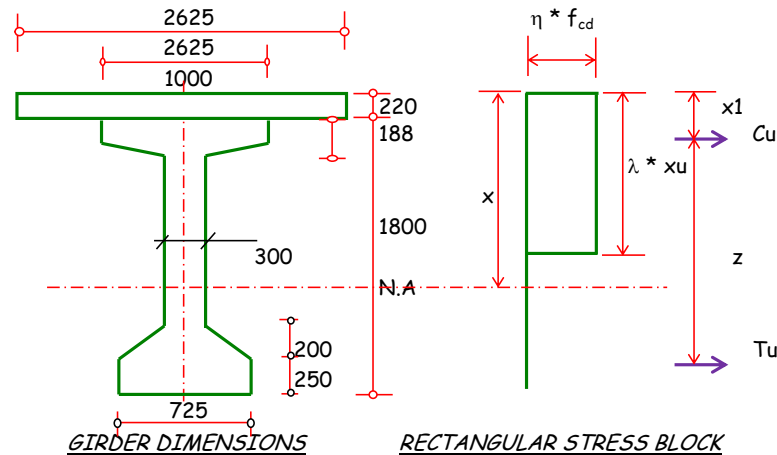
Design Parameters

Grade of Concrete of Longitudinal PSC Girder = M 45 MPa
 Modulus of Elasticity of Concrete of PSC Girder $E_{cm/g}$ = 34 GPa
 Grade 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
 Nominal Diameter of Prestressing Strands = 12.7 mm
 Nominal Area of each Strands = 98.7 mm²
 Nos of Strands = 89
 Modulus of Elasticity of Prestressing Steel E_p = 195 GPa
 Partial factor for Prestressing Steel γ_p = 1.15
 Characteristic tensile strength of Prestressing steel f_{pk} = 1860 MPa

Dia. of Untensioned Reinforcement ϕ = 12 mm
 c.g of Untensioned Reinforcement 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**Bending Moment at Different Section (KN-m) for Inner Girder**

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor ULS
	0m	4.1875m	8.375m	12.5625m	16.75m	
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	
Characteristic 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, E_s (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	
Characteristics compressive strength, f_{ck} (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, E_p (GPa)	195	195	195	195	195	
Characteristic 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	
Nominal 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 6-201
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, $f_{ctk,0.05}$ (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 $\gamma_m = 1.5$
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}/A_c or $0.2 \times f_{cd}$
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) $v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	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/($b_{wc} \times d$)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk0.5}/\gamma_m$	-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 $V_{Rdc} \min = (v_{min} + 0.15 * \sigma_{cp}) b_w * d$	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	$V_{Ns} = V_u - V_{pd}$
b) Design yield 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	1.17	1.48	1.64	1.67	1.67	= Difference between cg of tension force and compression force
e) Steel Shear capacity $V_{Rd,s} = (A_{sw} / s) z f_{ywd} \cot \theta$, KN	2726.0	3449.1	3815.2	2186.9	2187.1	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity $V_{Rd,max} = \alpha_{cw} * b * z * v1 * [f_{cd} / (\cot \theta + \tan \theta)]$, KN	3968.3	1882.8	2082.6	2122.3	2122.5	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of $V_{Rd,s}$ & $V_{Rd,max}$	2726.0	1882.8	2082.6	2122.3	2122.5	
h) Design value for concrete compressive strength ($f_{cd} = \alpha f_{ck} / \gamma_m$), Mpa	20.1	20.1	20.1	20.1	20.1	
i) $v1 =$	0.60	0.60	0.60	0.60	0.60	as f_{ck} is less than 80 Mpa
j) $\alpha_{cw} =$	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, $A_{sw} =$	402	402	402	226	226	in mm^2
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 Footpath/ Kerb	=	0.000 m	
4) Width of Footpath/ 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/ Diaphragm	=	0.300 m	
15) Width of End Cross girder/ Diaphragm	=	0.400 m	
16) Girder length beyond c/l of Brg. on each side	=	0.450 m	

1.2 Material Properties:

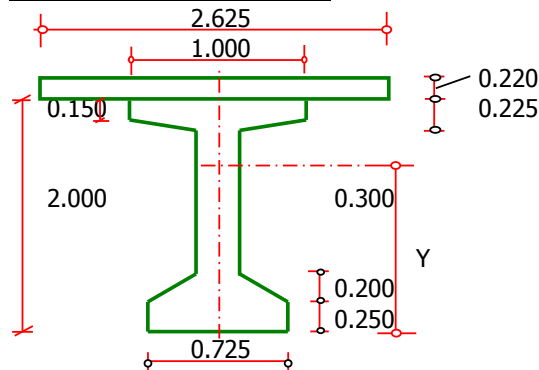
1) Concrete Grade for Deck slab	=	M 45
2) Concrete Grade for Cross girder	=	M 45
3) Concrete Grade of PSC Longitudinal girder	=	M 45
4) Reinforcement	=	Fe 500
5) Clear Cover	=	40 mm
6) Condition of Exposure	=	Moderate
7) Cable/ Strands	=	19 T 13 conforming to IS 14268
		(Stress-relieved multiply strands 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 flange	0.631	0.450	0.284	0.0114

$$I_z = 0.0291$$

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

$b_{eff} = \Sigma b_{eff,i} + b_w$	=	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. Rectangular 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)	$Y = \Sigma(A.y) / \Sigma A$	=	1.4376 m
Moment of inertia of composite girder	$I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.8445 m ⁴

Section Properties of Girder

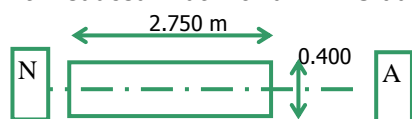
Area	ΣA	=	0.8800 m ²
Distance of cg from bottom fibre (Y)	$Y = \Sigma(A.y) / \Sigma A$	=	0.9963 m
Moment of inertia of girder	$I_z = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.4097 m ⁴

1.1 Section Properties of Top/ Deck Slab & Cross girder/ Diaphragm 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.)



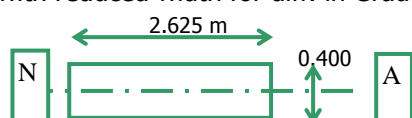
Modulus of Elasticity ratio = 1.000

Area = 1.1000 m²

Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0147 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**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 m²

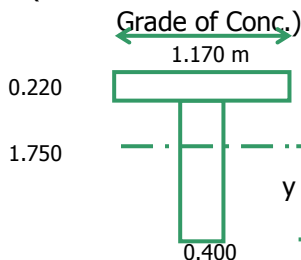
Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0140 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴**End Cross Girder****_ENDCG**

(with reduced width for diff. in

beff = Min{ $\sum beff_{i,i} + bw, b$ } = 1.170 m

Modulus of Elasticity ratio = 1.000

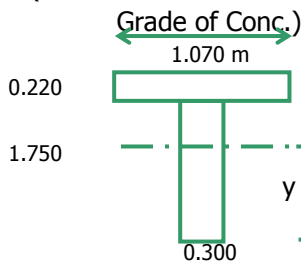
Area = 0.9574 m²

Distance of cg from bottom fibre (y) = 1.1398 m

Moment of inertia of end x-girder (I_z) = 0.3623 m⁴Moment of inertia girder (I_x) = 0.0001 m⁴

Intermediate/ Inner Cross Girder

(with reduced width for diff. in



$$b_{eff} = \text{Min}\{\sum b_{eff,i} + b_w, b\}$$

Modulus of Elasticity ratio

_INTCG

= 1.070 m

= 1.000

Area = 0.7604 m²

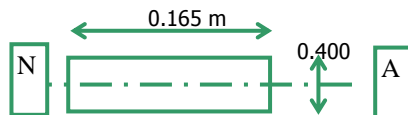
Distance of cg from bottom fibre (y) = 1.1799 m

Moment of inertia of end x-girder (I_z) = 0.2926 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

Cantilever Slab (At Expansion End)

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



Modulus of Elasticity ratio = 1.000

Area = 0.0660 m²

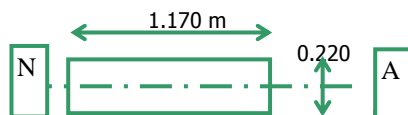
Distance of cg from bottom fibre (y) = 0.2000 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2574 m²

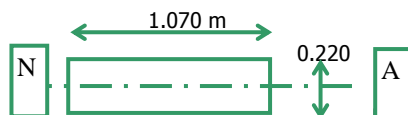
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0010 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

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

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



Modulus of Elasticity ratio = 1.000

Area = 0.2354 m²

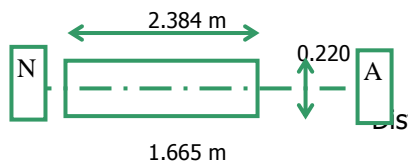
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0009 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSCANT2**Intermediate Slab (Next to End Cross Girder)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.5244 m²

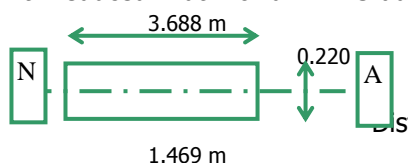
Distance of cg from bottom fibre (y) = 0.1100 m

Moment of inertia (I_z) = 0.0021 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB1**Intermediate Slab (At Splayed/ Tapered Portion)**

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



Modulus of Elasticity ratio = 1.000

Area = 0.8113 m²

Distance of cg from bottom fibre (y) = 0.1100 m

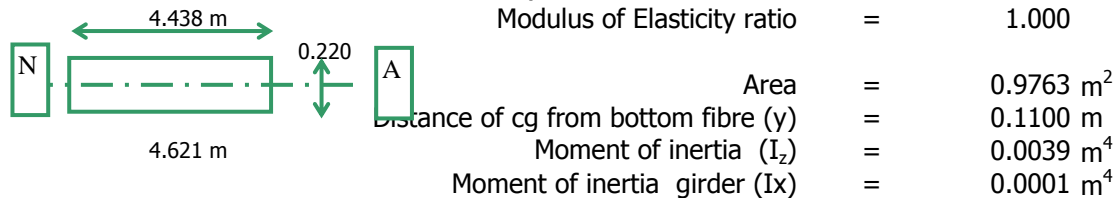
Moment of inertia (I_z) = 0.0033 m⁴

Moment of inertia girder (I_x) = 0.0001 m⁴

_TRANSLAB2

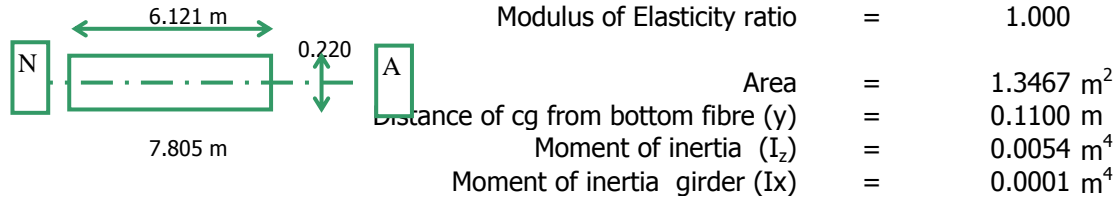
Intermediate Slab (Adjacent to First Inner Cross Girder) _TRANSLAB3

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



Intermediate Slab (Between Inner Cross Girders) _TRANSLAB4

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



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 6-2014
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 ratio 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**Wind speed & design wind pressure for Basic wind speed of 39m/s**

H (m)	Plain Terrain		Terrain with Obstruction		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
Traverse wind Force		F_T	=	$P_z \times A_1 \times G \times C_D$
			=	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	=	$P_z \times A_3 \times G \times C_L$
			=	536.3 KN
UDL on girder			=	1.1 KN/m ²
Outer girder			=	2.8 KN/m
Inner girder			=	3.0 KN/m
				2.82 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	<u>Load Case 1</u> 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	<u>Load Case 2</u> 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	<u>Load Case 3</u> 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	<u>Load Case 4</u> 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	<u>Load Case 5</u> 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	<u>Load Case 6</u> 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	<u>Load Case 7</u> 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	<u>Load Case 8</u> 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	<u>Load Case 9</u> 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 <u>Load Case 10</u> 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 <u>Load Case 11</u> 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 <u>Load Case 12</u> 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 <u>Load Case 13</u> 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 <u>Load Case 14</u> Class A 4 Lane Placed most Eccentric (Left side)			
1st Lane Z1 = $0.5 + 0 + 0.15 + 0.50/2$	=	0.90 m	From CB
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 <u>Load Case 15</u> 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 <u>Load Case 16</u> Class A 4 Lane Placed most Eccentric (Right side)			
1st Lane Z1 = $13.5 - 0.5 - 0.15 - 0.25 - 1.8 - 1.7 - 1.8 - 1.7 - 1.8$	=	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	<u>Load Case 17</u>	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	<u>Load Case 18</u>	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	<u>Load Case 19</u>	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	<u>Load Case 20</u>	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	<u>Load Case 21</u>	70R W for combination of Class A 2 Lane Eccentric on Left	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
	2nd Lane	$Z3 = 2.7 + 0.5 + 1.2$	=	4.40 m
	Class A	$Z4 = 4.4 + 1.8$	=	6.20 m
	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
				1.800
				5.300
				9.045
22	<u>Load Case 22</u>	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
Density of Concrete Girder	=	25.0 KN/m ³
Density 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	= 0.88×25	=	22.0 KN/m
Weight of Girder at End section	= 1.495×25	=	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
Weight of Intermediate X-girder	=	=	4.1 KN

Weight of the Shuttering Slab during Stage-2 Loading

Weight of Shuttering	=	5.0 KN/m
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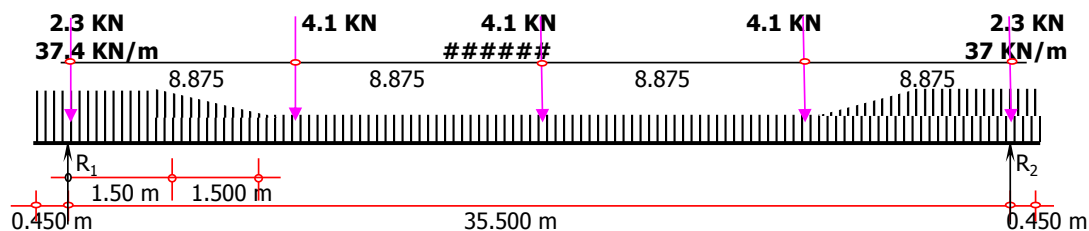
Weight of the Deck Slab & Cross Girder during Stage-2 Loading

Weight of Deck Slab	= $2.8 \times 0.22 \times 26$	=	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

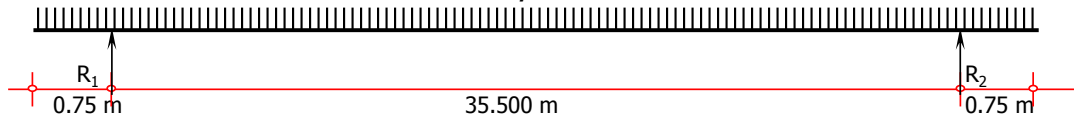


Support Reaction

$R_1 = 450.3 \text{ KN}$

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

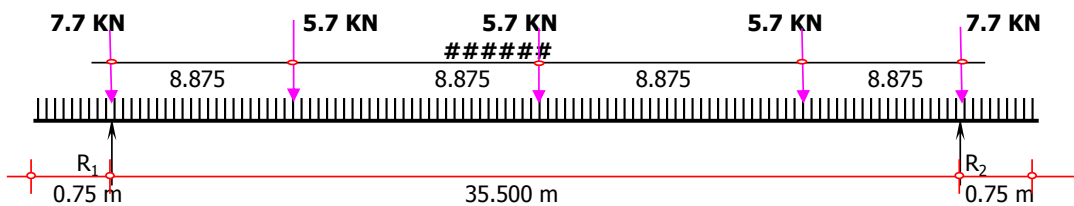
Stage-2 loading

Shuttering Load for Deck Slab**5.0 KN/m****Support Reaction**

$R_1 = 92.5 \text{ KN}$

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 Momen	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**

$R_1 = 307.3 \text{ KN}$

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
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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

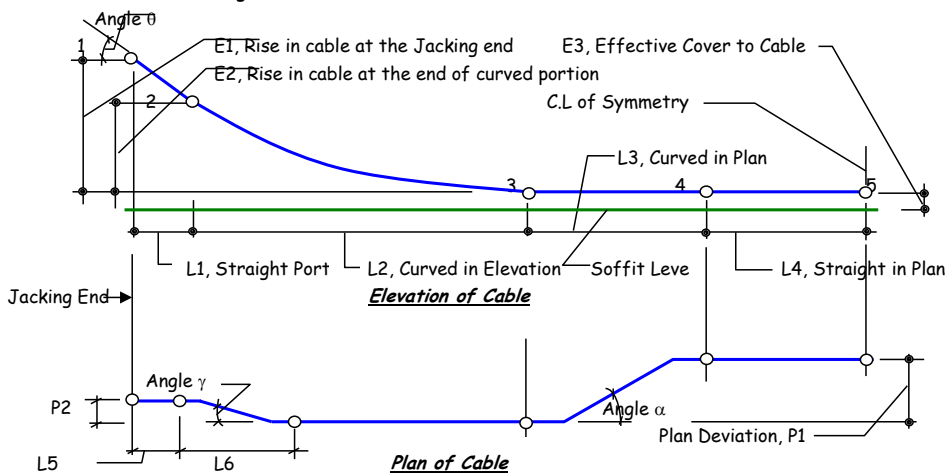
LOADINGS	d' away from	L/8	L/4	3L/8	L/2	Factor SLS Rare
	2m	4.4375m	8.875m	13.3125m	17.75m	
1st Stage DL	431.3	299.0	197.3	99.7	0.0	1.0
2nd Stage DL	287.8	218.0	142.5	72.7	0.0	1.0
SIDL (Crash Barrier)	16.8	17.0	24.3	30.6	35.7	1.0
SIDL (Surfcaing/ Wearing coat)	83.3	72.9	48.1	23.3	4.3	1.2
Pedestrian LL	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 Corrugated HDPE sheathing (Refer Table 6.2 , IRC:112) (Refer Table 6.2 , IRC:112)
13) Wobble Friction Coefficient	k	0.002	per metre	
14) Relaxation loss at 1000 hrs at 70% UTS (as % of initial stress)	Re1	2.5		
15) Relaxation loss at 1000 hrs at 50% UTS (as % of initial stress)	Re2	0		
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 Stage stressing in 14 days	Fc _j	40.6	MPa	
21) Modulus of Elasticity of Concrete at 1st Stage stressing	Ec _j	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



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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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
$\Sigma \theta$ (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

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
	P_f	2478.218	2436.692	2406.951	2385.289	2363.627
	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
	P_f	2478.218	2436.692	2406.951	2385.289	2363.627
	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
	P_f	2478.222	2457.612	2403.897	2382.316	2360.734
	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)	$P_s \cdot \cos\theta$ (KN)	$P_s \cdot \cos\theta \cdot Y_{ord}$ (KN.m)	$P_s \cdot \sin\theta$ (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
	<i>Eff. Eccentricity of Cable</i>					0.2007		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.4814		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6236		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6555		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					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
	<i>Eff. Eccentricity of Cable</i>					0.6559		
	Total Stage 2					0.0	0.0	0.0
	<i>Eff. Eccentricity of Cable</i>					0.0000		

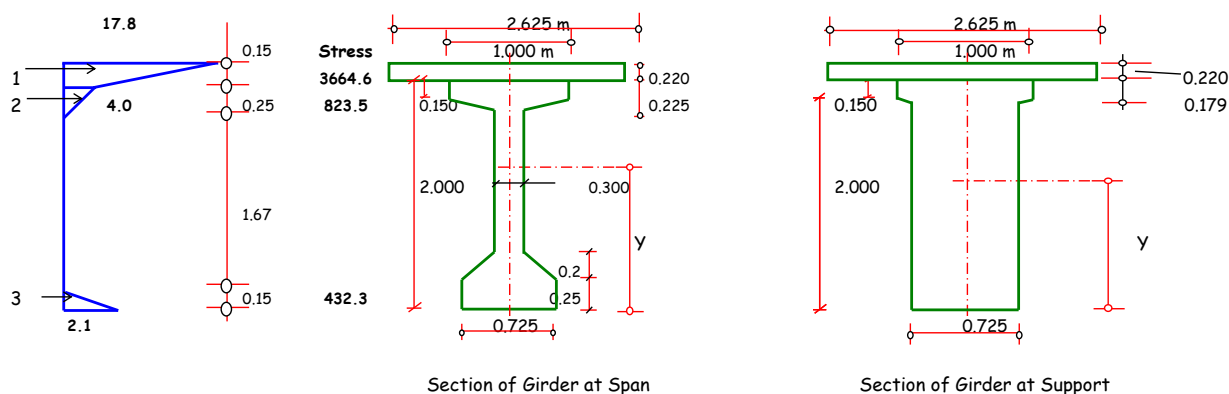
(*) Notations Used :

 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	E_c	=	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



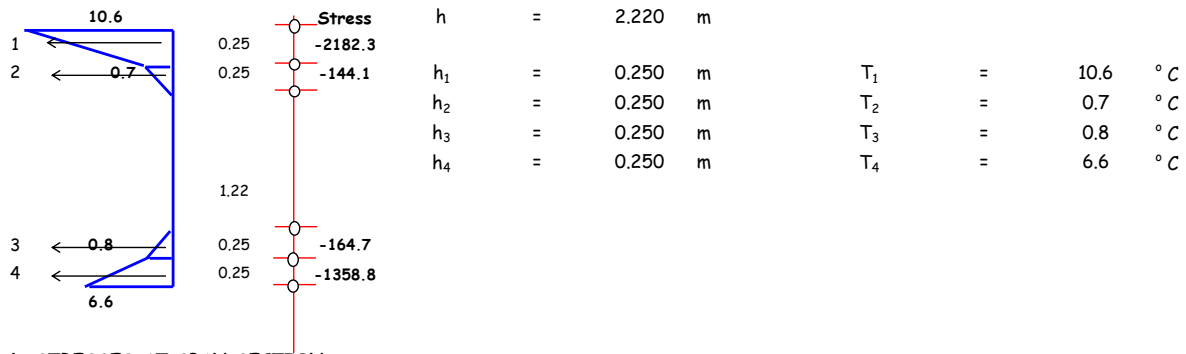
4.1.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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	0.150	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	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	0.150	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



4.2.1 STRESSES AT SPAN SECTION

Segment	Height	Stress	b	Force	y from top	e	Moment	Stresses			
								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	0.698	-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	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	0.806	-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 _†	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 _†	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 _†)	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 _†	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						
Stress after P-E loss	KN	357.0	357.0	357.0	357.0	357.0
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 _†	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.	%	1.869	1.988	2.168	2.265	2.337
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	336				
Relaxation loss as % of loss at 1000 hrs.	%	83.99				
(Ref. Table 6.2, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 7 to 28 days	KN	= 83.99/100 * 2.337/100 * (0.75*183.7*82) * 1				
	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	14 days	0.00003	0.00003	0.00003	0.00003	0.00003
Residual Shrinkage Strain at 28 days	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	14 days	2.426	2.502	2.502	2.502	2.502
Creep Co-efficient at 28 days	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				
	=	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	KNm	0.0	1117.9	1930.5	2407.9	2575.5
1) Sterss due to 2nd Stege DL						
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.	%	1.869	1.988	2.168	2.265	2.337
(Ref. Table 6.2, IRC:112 - 2011)						
Time after prestressing	hr.	840				
Relaxation loss as % of loss at 1000 hrs.	%	12.81				
(Ref. Table 6.3, IRC:112 - 2011)						
Factor For time dependent loss		1.0				
Relaxation loss at mid-span for 28 to 49 days	KN	= 12.81/100 * 2.337/100 * (0.75*183.7*82) * 1				
	KN	26.9	28.6	31.2	32.6	33.7
2) Shrinkage Loss (28-49 days)						
Residual Shrinkage Strain at 28 days	28 days	0.00006	0.00006	0.00006	0.00006	0.00006
Residual Shrinkage Strain at 49 days	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	28 days	2.127	2.194	2.194	2.194	2.194
Creep Co-efficient at 49 days	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				
	=	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 to Infinity	KN	= 203.2/100 * 2.337/100 * (0.75*183.7*82) * 1				
	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. Stress 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 with 75% 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 :-

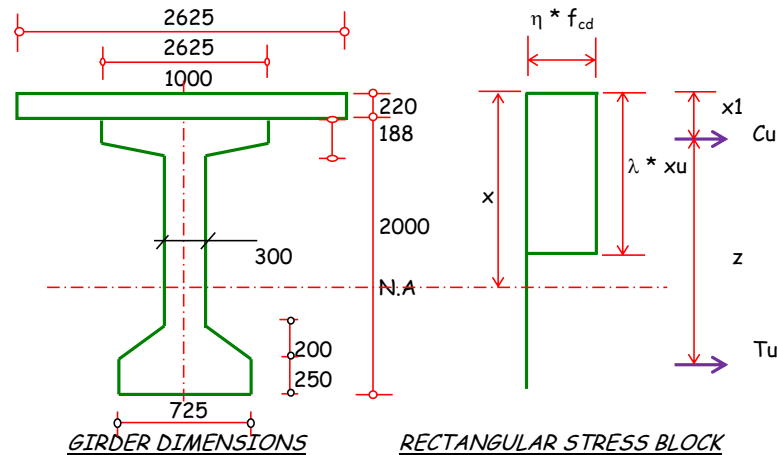
Span Length (c/c of Brg.) = 35.500 m

Design Parameters

Grade of Concrete of Longitudinal PSC Girder = M 45 MPa
 Modulus of Elasticity of Concrete of PSC Girder $E_{cm/g}$ = 34 GPa
 Grade 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
 Nominal Diameter of Prestressing Strands = 12.7 mm
 Nominal Area of each Strands = 98.7 mm²
 Nos of Strands = 88
 Modulus of Elasticity of Prestressing Steel E_p = 195 GPa
 Partial factor for Prestressing Steel γ_p = 1.15
 Characteristic tensile strength of Prestressing steel f_{pk} = 1860 MPa
 Dia. of Untensioned Reinforcement ϕ = 12 mm
 c.g of Untensioned Reinforcement 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**Bending Moment at Different Section (KN-m) for Inner Girder**

Loading	c/L Brg.	L/8	L/4	3L/8	L/2	Factor ULS
	0m	4.4375m	8.875m	13.3125m	17.75m	
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	
Characteristic 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, E_s (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	
Characteristics compressive strength, f_{ck} (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, E_p (GPa)	195	195	195	195	195	
Characteristic 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	
Nominal 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, $f_{ctk,0.05}$ (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 $\gamma_m = 1.5$
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}/A_c or $0.2 \times f_{cd}$
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) $v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	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/($b_{wc} \times d$)

s) Characteristic axial tensile strength of concrete at a strain, 5% fractile of tensile strength, $f_{ctk0.5}$ (KN)	2.50	2.50	2.50	2.50	2.50	Ref. Table 6.5 of, IRC:112
t) $f_{ctk0.5}/\gamma_m$	-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, $V_{Rdc} = (I \cdot b_{wc}/s) \cdot ((f_{ctd})^2 + k1 \cdot \sigma_{cp} \cdot f_{ctd})^{0.5}$	3298.6	1333.7	1333.7	1333.7	1333.7	Ref. Eq. 10.4 of, IRC:113
y) Shear Resistance, $V_{Rdc} = [0.12 \cdot K \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} + 0.15 \cdot \sigma_{cp}] \cdot b_w \cdot d$	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 $V_{Rdc} \min = (v_{min} + 0.15 \cdot \sigma_{cp}) \cdot b_w \cdot d$	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	$V_{Ns} = V_u - V_{pd}$
b) Design yield 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 $V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot \theta$, KN	3247.0	3974.7	4306.2	2464.1	2464.6	Ref. Eq. 10.7 of IRC:112
f) Max Shear capacity $V_{Rd,max} = \alpha_{cw} \cdot b \cdot z \cdot v1 \cdot [f_{cd} / (\cot \theta + \tan \theta)]$, KN	4723.7	2169.8	2350.7	2391.3	2391.8	Ref. Eq. 10.8 of IRC:112
g) V_{RD} Smaller of $V_{Rd,s}$ & $V_{Rd,max}$	3247.0	2169.8	2350.7	2391.3	2391.8	
h) Design value for concrete compressive strength ($f_{cd} = \alpha_{fck}/\gamma_m$), 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) $\alpha_{cw} =$	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, $A_{sw} =$	402	402	402	226	226	in mm ²
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)

$$f_b = 0.48 * f_{cj} * (A2/A1)^{0.5} \text{ or } 0.8 * f_{cj} \text{ whichever is lesser}$$

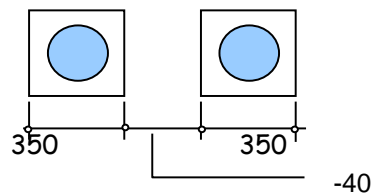
Where :

$$f_b = \text{Allowable stress behind anchorages}$$

f_{cj} = Concrete Strength at the time of stressing (in Mpa)

A_1 = Bearing area of anchorage (Eq. Square of dimension "a")

A2 = As defined in Clause 7.3 of IRC : 18 (Eq. Square of dimension "b")



$$f_{cj} = 40.59 \text{ Mpa}$$

$a = 350 \text{ mm}$

$b = 300 \text{ mm}$

$$f_b = 16.7 \text{ Mpa}$$

B. Force acting at the anchorage

a) Force per cable at Jacking end after friction & Slip	=	2235.6 KN
b) Elastic Shortening Loss per cable	=	77.1 KN
c) Relaxation Loss per cable	=	136.1 KN
d) Net force after instantaneous 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)

$2 \cdot y_o$ = Side of End Block	=	300 mm
$2 \cdot y_{po}$ = 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 $0.2 \cdot Y_o$ to $2.0 \cdot Y_o$.

$$0.2 \cdot y_o = 30 \text{ mm} \qquad 2.0 \cdot y_o = 300 \text{ mm}$$

$$\text{Area of Stirrups required} = F_{bst}/0.87*f_v = 912 \text{ mm}^2$$

Provided, 6-legged T-16 in 2 layers, in the form of spiral, $A_s = 2412.0 \text{ mm}^2$

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_0	=	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 calculation:-***(a) Top Slab***

Thickness of wearing coat	=	0.1 m
Load (UDL) due to WC	0.1×22	= 2.2 kN/m width
Thickness of earth/profile corrective course fill	=	0.0 m
Load (UDL) due to earth fill	0×20	= 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)			Submerged Pressure	Water Pressure
		Dry Earth Pressure				
0.388	0.000	$0.5 \times 20 \times 0.3875$	=	3.9	0.0	0.0
0.288	0.675	$0.5 \times 20 \times 0.675$	=	6.8	2.9	0.0
0.300	0.975	$0.5 \times 20 \times 0.975$	=	9.8	4.4	0.0
2.900	3.875	$0.5 \times 20 \times 3.875$	=	38.8	18.9	26.0
0.300	4.175	$0.5 \times 20 \times 4.175$	=	41.8	20.4	29.0
0.300	4.475	$0.5 \times 20 \times 4.475$	=	44.8	21.9	32.0

Live Load Surcharge calculation:-

Equivalent height	=	1.2 m
Uniform Intensity of loading	$1.2 \times 0.5 \times 20$	= 12.0 KN/m ²

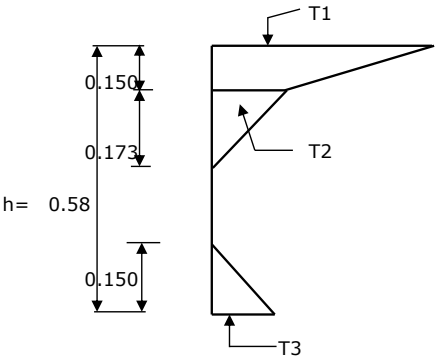
Braking Load calculation:-

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:-

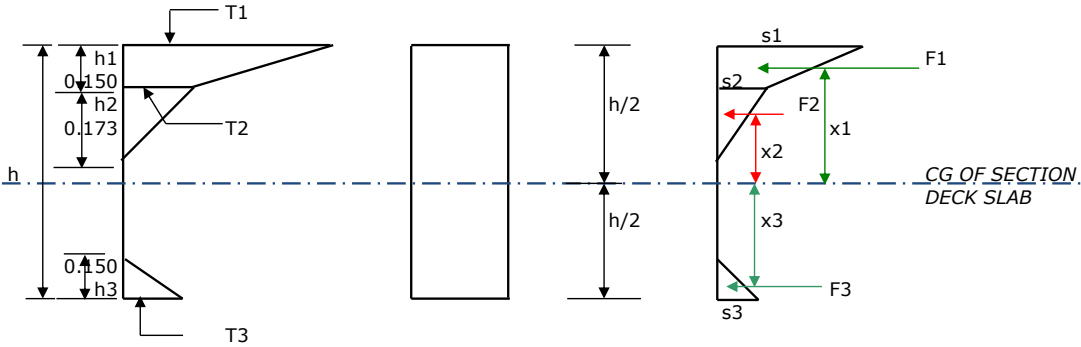
The top slab is designed for the effects of the distribution of the temperature across the deck depth as given in the sketch below.



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:-

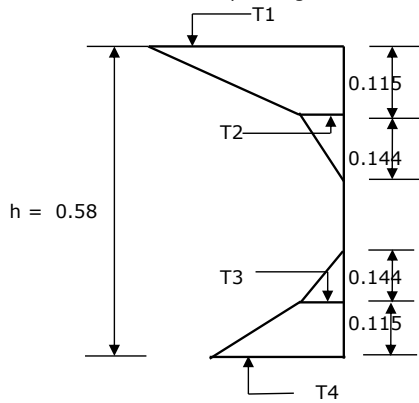
Member	h	h1	h2	h3	T1	T2	T3
	(m)	(m)	(m)	(m)	°C	°C	°C
Top slab	0.575	0.150	0.173	0.150	17.80	4.00	2.10

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

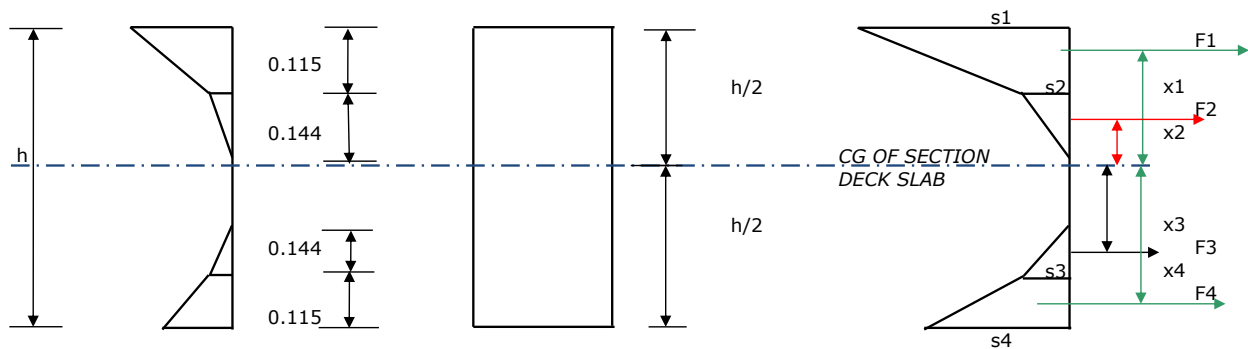
Member	cg from Top	cg from Bottom	cg of Top Block from Top	cg of Mid Block from Top	cg of Bottom Block from Bottom	x1	x2	x3	Moment
						m	m	m	Tm
Top slab	0.2875	0.2875	0.0592	0.2075	0.0500	0.2283	0.0800	0.2375	7.0

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	$\alpha =$	$1.2E-05$	$/ ^\circ C$
Stress	$\sigma =$	$E \alpha t$	
Force	$F =$	$F1 + F2 + F3 + F4$	
Moment	$M =$	$F1x1 + F2x2 - F3x3 - F4x4$	

Generalised Temperature & corresponding Force Diagram**Force & Moment calculation:-**

Member	h (m)	$h1$ (m)	$h2$ (m)	$h3$ (m)	$h4$ (m)	$T1$ $^\circ C$	$T2$ $^\circ C$	$T3$ $^\circ C$	$T4$ $^\circ C$
Top slab	0.575	0.115	0.144	0.144	0.115	10.60	0.70	0.80	6.60

Member	$s1$ $T / m2$	$s2$ $T / m2$	$s3$ $T / m2$	$s4$ $T / m3$	$F1$ T	$F2$ T	$F3$ T	$F4$ T	Force 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 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

Total load on 70 R-Wheel = 1220 kN with I.F = 0.22
Wearing coat thickness = 0.100 m 1.22

Load	Dimension of tyre		Load kN	Dist. Betwn. each load	Final Load kN
	Length	Width			
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
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

[illegible]

Effective Live Load for 70R-Bogie Load

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

[illegible]

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

Total load on 70 R-Track = 770 kN with I.F = 0.10
Wearing coat thickness = 0.100 m 1.10

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
a constant,
α = 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

[illegible]

Effective Live Load for Class-A Wheel Load

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

b1	for 27 kN	= 0.40	m
b1	for 68 kN	= 0.58	m
b1	for 114 kN	= 0.70	m

[illegible]

	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				

[illegible]

Summary of Shear force (SF) & Bending Moment (BM):-

	1	2	3	
4				7
5				8
6				9
	10	11	12	

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 q 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
Grade of Steel Reinforcement	=	Fe 500	MPa	Page:235 of IRC:112-2011 & Table 6.5 of IRC:112-2011
Characteristic yield strength of Reinforcement	f_{yk}	=	500	MPa
Design yield of strength of shear Reinforcement	f_{ywd}	=	400	MPa
Modulus of Elasticity of Steel	E_s	=	200000	MPa
Modulus of Elasticity of Concrete, for Short Term loading	E_{cm}	=	32308	MPa
for Long Term loading	$E_{cm}^{'}$	=	13842	MPa
			13726	MPa
			13576	MPa
			16154	MPa
				for Top/Deck slab
				for Outer wall
				for Inner wall
				for Bottom/Base slab
Mean value of axial tensile strength of Concrete	f_{ctm}	=	2.8	MPa
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)	$x_{u,max}/d$	=	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
			1.35	for Outer wall
			1.38	for Inner wall
			1.00	for Bottom/Base slab

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Refer:- Creep coefficient calculation

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:-

$A_{s,min} = \max [0.26*f_{ctm}/f_{yk}*b_t*d, 0.0013*b_t*d]$ Refer IRC:112, cl 16.5.1.1)

where,
 b_t = mean width of tension zone

Maximum Reinforcement:-

Tension Reinf.	\leq	$0.025*A_c$	At section other than lap
Tension + Compression Reinf.	\leq	$0.04*A_c$	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					x _{u,max}	x _u =0.87f _{yk} *A _{st} /0.362f _{ck} *b	Check	z=(d-0.416x _u)	A _{st,cal} =M _{Ed} /0.87f _{yk} *z	A _{st,max}	Check	A _{st,cal} <A _{st,pro} <A _{st,max}	Check	ΔF _d = 0.5V _{Ed}	M _{Ed} /Z+	M _{Ed} =0.87 * f _{yk} * A _{st} * z	M _{Rd} /Z	Check	
							Dia.	Spacing	Dia.	Spacing	A _{st, pro}							A _{st,min}		A _{st,max}					M _{Rd} /Z > M _{Ed} /Z + ΔF _d	
							Tm	mm	mm	mm	mm							mm		mm					mm	mm ²
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			Check
				Dia.	Spacing	$A_{st, pro}$	
				mm ² /m	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}	v_l	$V_{Ed,max} = \alpha_{cw} * b_w * z * v_l * f_{td}/2$	Check	$v = 0.6[1 - f_{td}/310]$	$0.5 * b_w * d * v * f_{td}$	$V_{Ed} \leq 0.5 * b_w * d * v * f_{td}$	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	$\beta = a_v/zd$	βV_{Ed}	$K = \text{Min}[1 + (200/d)^{0.5}, 2]$	$\rho 1 = \text{Min}[Asl/b_w d, 0.02]$	$V_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	σ_{cp}	$V_{Rd,c1} = (V_{min} + 0.15\sigma_{cp}) b_w d$	$V_{Rd,c2} = [0.12K(8 + \rho 1 f_{ck})^{0.33} + 0.15\sigma_{cp}] b_w d$	$V_{Rd,c} = \text{Max}[V_{Rd,c1}, V_{Rd,c2}]$	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,

$$h = \text{total depth of member in mm}$$

a) For Rare Load Combination:-

Design Component	Section	Face	M _{Rare}		Overall depth, D		A _{st, pro}	I	Y/2	Stress 'σ _c '	f _{ctm,fl}	Cracked/ Uncracked		Modular ratio 'm'	N.A depth 'x'	I _{NA}	Comp. stress	Per. stress	Check	Tensile stress	Per. stress	Check
			Tm	b	mm	d	mm ²	mm ⁴	mm	Mpa	Mpa				mm	mm ⁴	Mpa	Mpa		Mpa	Mpa	
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}	b	Overall depth, D	d	A _{st, pro}	I	Y/2	Stress 'σ _c '	f _{ctm,fl}	Cracked/ UnCracked		Modular ratio 'm'	N.A depth 'x'	I _{NA}	Comp. stress	Per. stress	Check	Tensile stress	Per. stress	Check
			Tm	mm	mm	mm	mm ²	mm ⁴	mm	Mpa	Mpa				mm	mm ⁴	Mpa	Mpa		Mpa	Mpa	
			mm	mm	mm	mm	mm ²	mm ⁴	mm	Mpa	Mpa				mm	mm ⁴	Mpa	Mpa		Mpa	Mpa	
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	$A_{s,min} = k_c * k * f_{ct} / \sigma_c$	$A_{st,pro}$ or $A_{s,pro}$	Check	Bar dia	Clear cover 'c'	$h_{c,eff} = \min[2.5(h-d), (h-x/3), h/2]$	$A_{c,eff} = h_{c,eff} * b$	$\rho_{peff} = A_s / A_{c,eff}$	$S_{max} = \frac{3.4c + 0.17\phi}{\rho_{peff}}$	σ_{sc}	N.A depth 'x'	k_t	$\alpha_e = E_s / E_{cm}$	$\epsilon_{sm} - \epsilon_{cm}$	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	;

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

```

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LOAD 1 SELF WEIGHT
SELFWEIGHT Y -1

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LOAD 2 SIDL
MEMBER LOAD

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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

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 Minor Bridge
Minor Bridge (1x23.6m) at design Ch.134+280KM

Hydrological Calculation for Minor Bridge

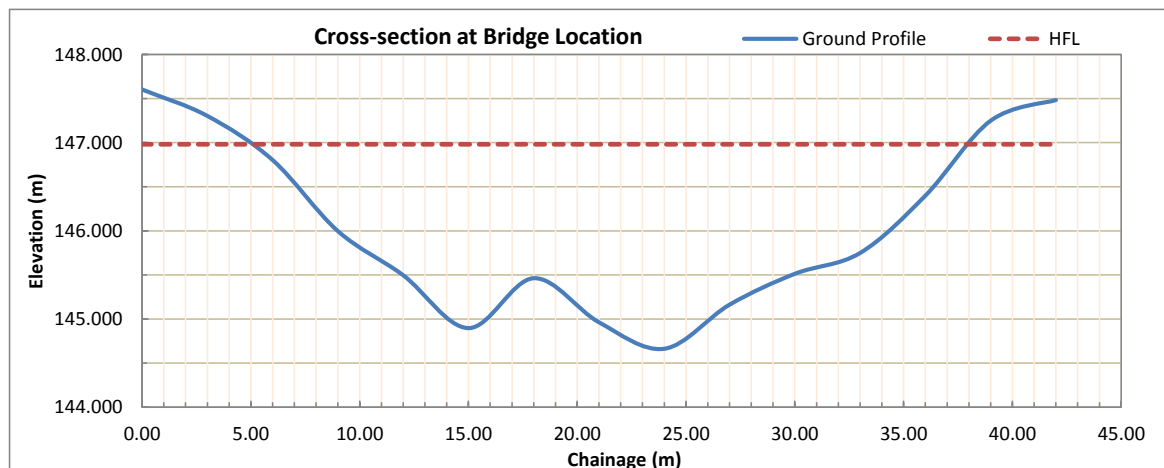
Bridge Data:-

Road Name	=	Daboka-Manja
Bridge Name/ID No.	=	-
Name of Nallah / Stream / River	=	Local stream
Existing Ch.	=	-
Design Ch.	=	134+280 KM
Type of Proposed bridge	=	RCC T-Girder
Span Arrangement of proposed bridge	=	1x23.6m
Existing FRL	=	-
Design FRL	=	151.000 m
HFL at bridge location	=	146.982 m
Rugosity Co-efficient,n	=	0.05
(Reff: IRC: SP: 13 - 2004 , Table 5.1)		
Bed Slope of Stream,S	=	0.001
G.T Sheet No.	=	-
Scale	=	-
Catchment Area	=	2.440 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	0.00	147.603	147.603	146.9817					
2	3.00	147.303	147.303	146.9817					
3	6.00	146.803	146.803	146.9817	0.178667	0.089	0.134	1.511	1.500
5	9.00	145.997	145.997	146.9817	0.984667	0.582	1.745	3.106	3.000
7	12.00	145.497	145.497	146.9817	1.484667	1.235	3.704	3.041	3.000
9	15.00	144.897	144.897	146.9817	2.084667	1.785	5.354	3.059	3.000
11	18.00	145.463	145.463	146.9817	1.52	1.802	5.405	3.053	3.000
13	21.00	144.963	144.963	146.9817	2.02	1.769	5.306	3.041	3.000
15	24.00	144.663	144.663	146.9817	2.32	2.169	6.506	3.015	3.000
17	27.00	145.163	145.163	146.9817	1.82	2.069	6.206	3.041	3.000
19	30.00	145.513	145.513	146.9817	1.47	1.644	4.931	3.020	3.000
21	33.00	145.749	145.749	146.9817	1.23	1.351	4.052	3.009	3.000
23	36.00	146.399	146.399	146.9817	0.58	0.908	2.723	3.070	3.000
25	39.00	147.249	147.249	146.9817		0.291	0.437	1.609	1.500
27	42.00	147.483	147.483	146.9817					
TOTAL							46.503	33.577	33.000



Hydraulic Mean Radius (R)	A / P	=	1.385 m
Velocity of Water (V)	$(1/n)(R)^{2/3}(S)^{1/2}$	=	0.786 m/sec
Discharge (Q)	$A \times V$	=	36.543 Cumecs

Summary of Discharge By Area Velocity Method

1	Discharge at bridge location	=	36.543 Cumecs
	Maximum Discharge By Area velocity method	=	36.543 Cumecs

Hydrological Calculation for Minor Bridge

Name of Nallah / Stream / River	:	Local stream
Location	:	134+280 KM
Catchment Area	:	2.44 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	=	16
Catchment Area,	M	=	2.440 Sq. Km
Discharge,	Q	=	31.236 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	=	2.440 Sq. Km
Discharge,	Q	=	18.124 Cumecs

4.0 Discharge by Rational Formula : (Refer IRC - SP : 13 - 2004, Clause : 4.7)

Catchment Area,	A	=	2.440 Sq. Km
		=	244.00 Hectares
Length of longest path from Toposheet,	L	=	2.832 Km
Difference in levels from Toposheet,	H	=	6 m
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	$t_c = (0.87 \times L^3 / H)^{0.385}$	=	1.58 Hrs
Critical Rainfall Intensity	$I_c = I_o \times (2 / (1 + t_c))$	=	68.58 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)		=	0.975
Critical Intensity of Rainfall, I_c		=	6.858 cm / Hr
Maximum Discharge	$Q = 0.028 \times P \times f \times A \times I_c$	=	36.543 Cumecs

2.0 Design Discharge : (Refer IRC - SP : 13 - 2004, Clause : 6.2.1)

Discharge by Area Velocity Method		=	36.543 Cumecs
Discharge by Dicken's Formula		=	31.236 Cumecs
Discharge by Rational Formula		=	36.543 Cumecs
Design Discharge		=	36.543 Cumecs
Design Discharge with 10% safety factor		=	40.197 Cumecs

3.0 Afflux Calculations : (Refer IRC:5 2015, Clause : 106.6)

Design Discharge	Q	=	40.197 cumecs
Unobstructed area of Cross Section of river	A	=	46.50 m ²
Velocity of Water,	V	=	0.864 m/sec
Observed HFL	HFL	=	146.982 m
BED Level	OGL	=	144.663 m
Depth of Water	d	=	2.319 m
Available Water way	a	=	31.848 m ²
Using Molesworth's Formula,	$\{(V^2/17.9)+0.015\} \times \{(A/a)^2-1\}$	=	0.064239 ~ 0.065m
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.			

4.0 Vertical Clearance : (Refer IRC : 5 - 1998, Clause : 106.2.1)

Design Discharge,	Q	=	40.197 Cumecs
Vertical Clearance (adopted),		=	0.900 m

Minimum Soffit Level of superstructure = HFL + Afflux + Vertical Clearance = 147.947 m

5.0 Linear Waterway :

As per Lacy's formula required linear water way , $4.8 \sqrt[3]{Q}$	=	30.433 m
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Hydrological Calculation for Minor Bridge

Name of Nallah / Stream / River	:	Local stream	
HFL Spread including afflux at Proposed Bridge Location	=		33.000 m
Adopted obstructed linear waterway	=		22.600 m
Restriction to the linear waterway	=		25.7 % < 33%, O.K.

6.0 Scour Depth : (Refer IRC : 78 - 2000, Clause : 703)

% Increase in Design Discharge	=		30 %
So, Increased Design Discharge	=		52.257 Cumecs
Mean Depth of Scour, $d_{sm} = 1.34 \times (D_b^2 / K_{sf})^{1/3}$			
D_b = Design discharge per metre width	=		2.312 Cumecs / m
K_{sf} = Silt factor	=		1.250
d_{sm} =	=		2.175 m

Maximum Scour Depth:

For Abutments,	1.27 x d_{sm}	=	2.762 m
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Maximum Scour Level :

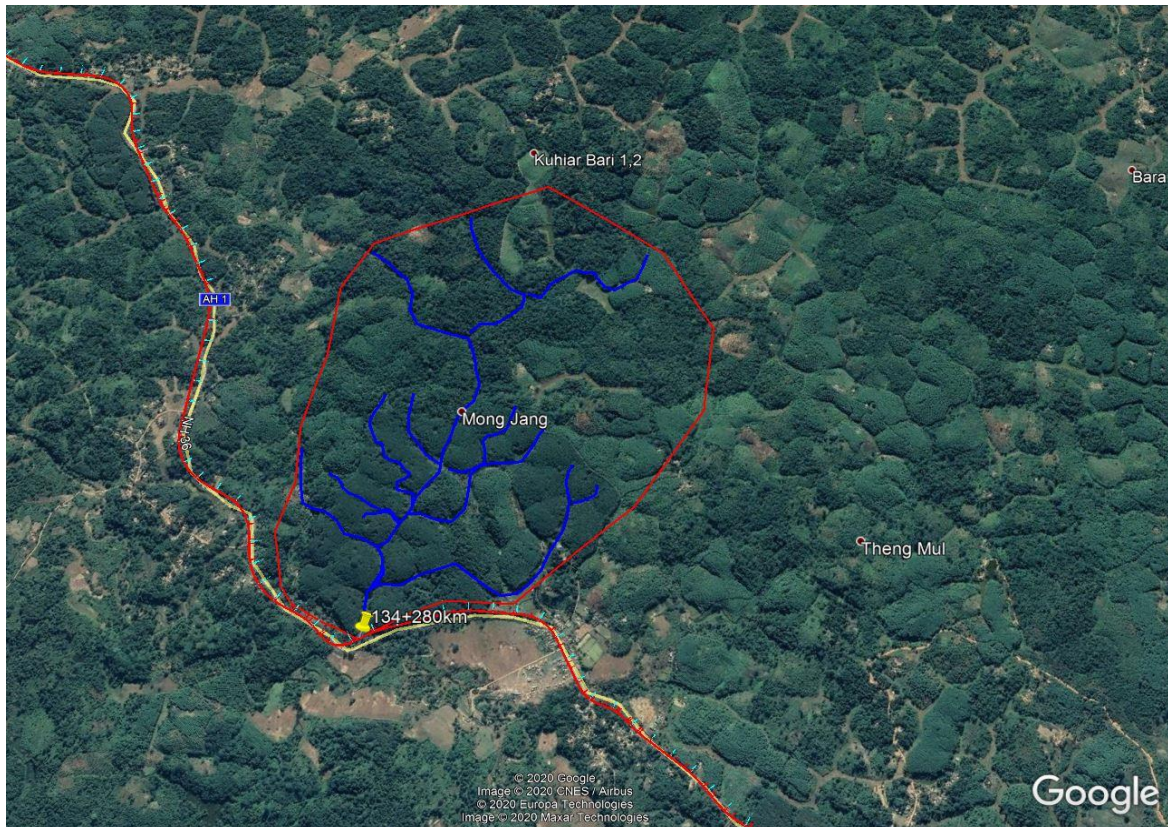
Maximum Scour Level	=	HFL - Maximum Scour Depth	
For Abutments,			144.284 m

Proposed Bridge Span Arrangement :

Total Length of Proposed Bridge	=	1x23.6m m
Span Length for each	=	23.6 m
Number of Spans	=	1
Proposed Depth of Super Structure at Centre	=	2.025 m
Thickness of Wearing Coat	=	0.065 m
Overall depth of superstructure (considering 400mm for camber)	=	2.490 m

7.0 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	147.047	147.047	
2	Lowest Bed Level at Proposed Location	144.663	144.663	
3	Minimum Soffit Level at proposed location	147.947	148.510	Safe
4	Proposed Formation Level	150.437	151.000	Safe



CATCHMENT AREA = 2.44 Sq. Km

Hydrological Calculation for Minor Bridge
Minor Bridge (1x22.5m) at design Ch.139+915KM

Hydrological Calculation for Minor Bridge

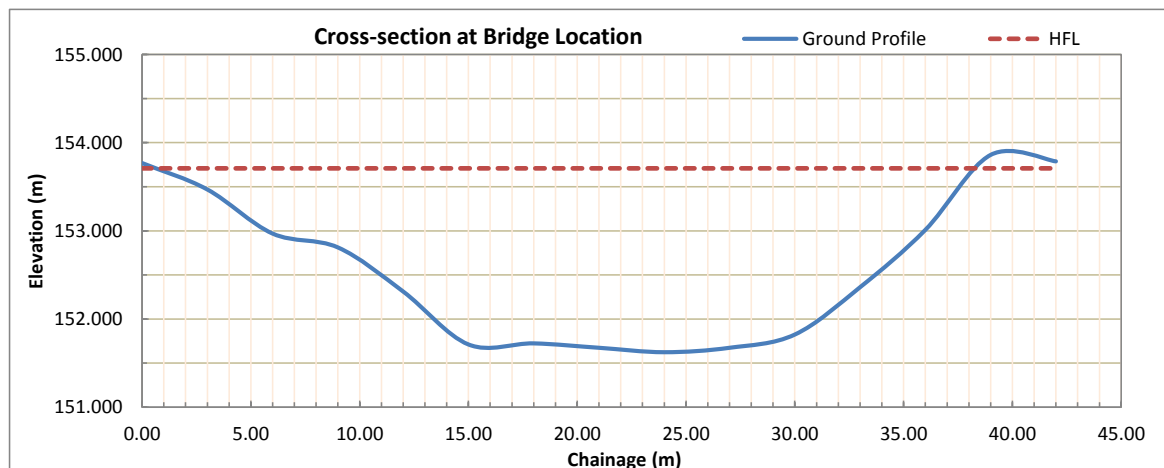
Bridge Data:-

Road Name	=	Daboka-Manja
Bridge Name/ID No.	=	-
Name of Nallah / Stream / River	=	Local stream
Existing Ch.	=	-
Design Ch.	=	139+915 KM
Type of Proposed bridge	=	RCC T-Girder
Span Arrangement of proposed bridge	=	1x22.5m
Existing FRL	=	-
Design FRL	=	159.160 m
HFL at bridge location	=	153.709 m
Rugosity Co-efficient,n	=	0.055
(Ref: IRC: SP: 13 - 2004 , Table 5.1)		
Bed Slope of Stream,S	=	0.001
G.T Sheet No.	=	-
Scale	=	-
Catchment Area	=	2.500 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	0.00	153.769	153.769	153.7086					
2	3.00	153.469	153.469	153.7086	0.239584	0.120	0.180	1.519	1.500
3	6.00	152.969	152.969	153.7086	0.739584	0.490	1.469	3.041	3.000
5	9.00	152.812	152.812	153.7086	0.896584	0.818	2.454	3.004	3.000
7	12.00	152.312	152.312	153.7086	1.396584	1.147	3.440	3.041	3.000
9	15.00	151.712	151.712	153.7086	1.996584	1.697	5.090	3.059	3.000
11	18.00	151.724	151.724	153.7086	1.98	1.991	5.972	3.000	3.000
13	21.00	151.674	151.674	153.7086	2.03	2.010	6.029	3.000	3.000
15	24.00	151.624	151.624	153.7086	2.08	2.060	6.179	3.000	3.000
17	27.00	151.674	151.674	153.7086	2.03	2.060	6.179	3.000	3.000
19	30.00	151.824	151.824	153.7086	1.88	1.960	5.879	3.004	3.000
21	33.00	152.361	152.361	153.7086	1.35	1.616	4.848	3.048	3.000
23	36.00	153.011	153.011	153.7086	0.70	1.023	3.068	3.070	3.000
25	39.00	153.861	153.861	153.7086		0.349	0.523	1.654	1.500
27	42.00	153.788	153.788	153.7086					
TOTAL							51.308	36.442	36.000



Hydraulic Mean Radius (R)	A / P	=	1.408 m
Velocity of Water (V)	$(1/n)(R)^{2/3}(S)^{1/2}$	=	0.722 m/sec
Discharge (Q)	$A \times V$	=	37.058 Cumecs

Summary of Discharge By Area Velocity Method

1	Discharge at bridge location	=	37.058 Cumecs
	Maximum Discharge By Area velocity method	=	37.058 Cumecs

Hydrological Calculation for Minor Bridge

Name of Nallah / Stream / River	:	Local stream
Location	:	139+915 KM
Catchment Area	:	2.5 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	=	16
Catchment Area,	M	=	2.500 Sq. Km
Discharge,	Q	=	31.811 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	=	2.500 Sq. Km
Discharge,	Q	=	18.420 Cumecs

4.0 Discharge by Rational Formula : (Refer IRC - SP : 13 - 2004, Clause : 4.7)

Catchment Area,	A	=	2.500 Sq. Km
		=	250.00 Hectares
Length of longest path from Toposheet,	L	=	2.832 Km
Difference in levels from Toposheet,	H	=	6 m
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	$t_c = (0.87 \times L^3 / H)^{0.385}$	=	1.58 Hrs
Critical Rainfall Intensity	$I_c = I_o \times \{2 / (1 + t_c)\}$	=	68.58 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)		=	0.965
Critical Intensity of Rainfall, I_c		=	6.858 cm / Hr
Maximum Discharge	$Q = 0.028 \times P \times f \times A \times I_c$	=	37.058 Cumecs

2.0 Design Discharge : (Refer IRC - SP : 13 - 2004, Clause : 6.2.1)

Discharge by Area Velocity Method		=	37.058 Cumecs
Discharge by Dicken's Formula		=	31.811 Cumecs
Discharge by Rational Formula		=	37.058 Cumecs
Design Discharge		=	37.058 Cumecs
Design Discharge with 10% safety factor		=	40.764 Cumecs

3.0 Afflux Calculations : (Refer IRC:5 2015, Clause : 106.6)

Design Discharge	Q	=	40.764 cumecs
Unobstructed area of Cross Section of river	A	=	51.31 m ²
Velocity of Water,	V	=	0.794 m/sec
Observed HFL	HFL	=	153.709 m
BED Level	OGL	=	151.624 m
Depth of Water	d	=	2.085 m
Available Water way	a	=	30.642 m ²
Using Molesworth's Formula,	$\{(V^2/17.9)+0.015\} \times \{(A/a)^2-1\}$	=	0.090659 ~ 0.091m
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.			

4.0 Vertical Clearance : (Refer IRC : 5 - 1998, Clause : 106.2.1)

Design Discharge,	Q	=	40.764 Cumecs
Vertical Clearance (adopted),		=	0.900 m

Minimum Soffit Level of superstructure = HFL + Afflux + Vertical Clearance = 154.700 m

5.0 Linear Waterway :

As per Lacy's formula required linear water way , $4.8 \sqrt[3]{Q}$		=	30.646 m
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Hydrological Calculation for Minor Bridge

Name of Nallah / Stream / River	:	Local stream	
HFL Spread including afflux at Proposed Bridge Location	=		36.000 m
Adopted obstructed linear waterway	=		21.500 m
Restriction to the linear waterway	=		29.8 % < 33%, O.K.

6.0 Scour Depth : (Refer IRC : 78 - 2000, Clause : 703)

% Increase in Design Discharge	=		30 %
So, Increased Design Discharge	=		52.993 Cumecs
Mean Depth of Scour, $d_{sm} = 1.34 \times (D_b^2 / K_{sf})^{1/3}$			
D_b = Design discharge per metre width	=		2.465 Cumecs / m
K_{sf} = Silt factor	=		1.250
d_{sm} =	=		2.270 m

As per IRC:78 2014, Clause 703.2.4 for balancer type structure $d_{sm} = (HFL - LBL) / 1.27$

	=		1.713 m
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Maximum Scour Depth:

For Abutments,	1.27 x d_{sm}	=	2.176 m
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Maximum Scour Level :

Maximum Scour Level	=	HFL - Maximum Scour Depth	
For Abutments,	=		151.624 m

Proposed Bridge Span Arrangement :

Total Length of Proposed Bridge	=		1x22.5m m
Span Length for each	=		23.6 m
Number of Spans	=		1
Proposed Depth of Super Structure at Centre	=		1.925 m
Thickness of Wearing Coat	=		0.065 m
Overall depth of superstructure (considering 400mm for camber)	=		2.390 m

7.0 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	153.800	153.800	
2	Lowest Bed Level at Proposed Location	151.624	151.624	
3	Minimum Soffit Level at proposed location	154.700	156.770	Safe
4	Proposed Formation Level	157.090	159.160	Safe

PART-III
GEOTECHNICAL REPORT

Soil Investigation Report of Major Bridge at
Chainage: 119+535

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7. Laboratory Test
8. Soil Profile
9. Foundations
10. Susceptibility of Subsoil to Liquefaction
11. Capacity Calculation
12. Conclusion and General Recommendations

Annexures

- | | |
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| Annexure - 1 | Bore Logs Data Sheets |
| Annexure - 2 | Laboratory Tests Results (Soil Sample) |
| Annexure - 3 | Chemical Test Results |

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 in 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 auger 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		
SI 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		
SI 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 out on 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''): $N'' = 15 + 0.5 \cdot (N' - 15)$

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: $= 5.75 \text{ t/m}^2$.

From Fig 1.of IS: 2131, correction factor = 1.19

So corrected SPT value for overburden, $N' = 1.19 \times 13 = 15.47$.

Corrected SPT for dilatancy $N'' = 15 + \{0.5 \times (15.47 - 15)\} = 15.24$

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.

8.1 Stratum-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

8.2 Stratum-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.

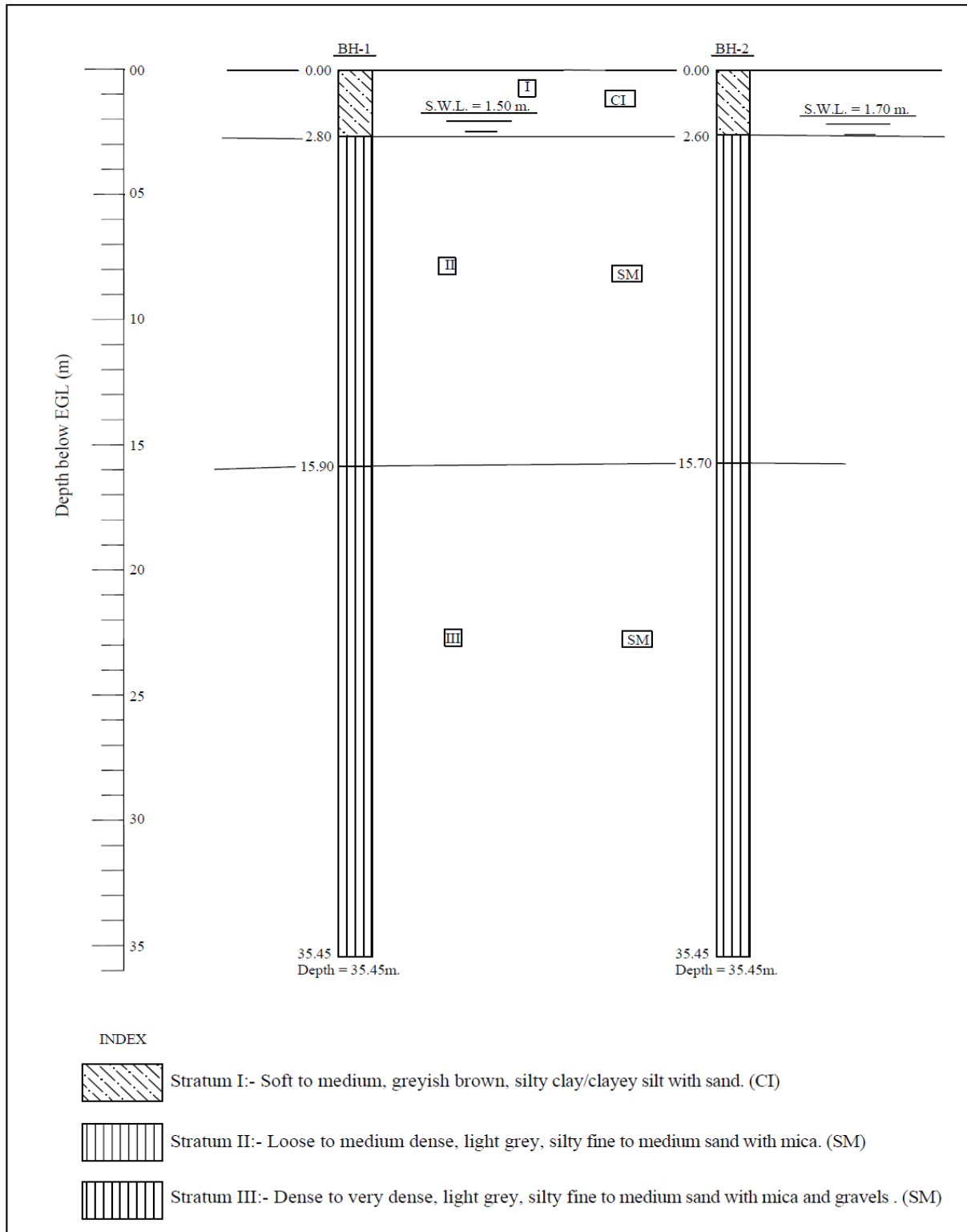


Fig.1 Generalized Sub Soil Profile

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 view of the above sub-soil conditions in mind and the type of structure both the aspects regarding bearing capacity of open foundations and pile capacity for deep foundation in the form of bored cast in-situ pile have been discussed in 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 Standard 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 $CRR/CSR \leq 1.0$ indicates that the soil is prone to liquefaction whereas $CRR/CSR > 1.0$ or corrected $N_1 > 30$ 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) moisture contents 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	1.73	4	3.19	1.47	Silty clay/clayey silt																		Non Liquefiable
	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	Liquefiable			
	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 Liquefiable			
	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	Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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	$(N_1)_{60}>30$					Non Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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						Non Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
BH-2	2.23	5	4.12	1.89	Silty clay/clayey silt																		Non Liquefiable
	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	Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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 Liquefiable			
	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	$(N_1)_{60}>30$					Non Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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						Non Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		
	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 Liquefiable		

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 $\Phi = 31^\circ$ [Ref: Fig. No. 1 of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.35$. $\Phi = 33.25^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 29^\circ$.

$\gamma_{\text{sat}} = 1.90 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.90 \text{ t/m}^3$ [Effective density]

STRATUM-III:

Design corrected "N" = 26. Corresponding $\Phi = 36^\circ$ [Ref: Fig. No. 1 of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.65$. $\Phi = 37.75^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 33^\circ$.

$\gamma_{\text{sat}} = 1.95 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.95 \text{ t/m}^3$ [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 = $\pi \times D \times L_i$

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

$Q_{eg} = A_p(0.5 \times D \times W \times N_r + P_d \times N_q)$

where A_p = 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

N_r = Bearing Capacity factor = 35.19

P_d = Effective overburden pressure at pile tip = 54.26kN/m^2

N_q = Bearing Capacity factor = 35.60

Ultimate End Bearing Capacity $Q_{eg} = 1.130 \cdot (0.5 \cdot 1.200 \cdot 9.693 \cdot 35.19 + 54.265 \cdot 35.60)$
=2415.07kN

For Cohesive Soils

$$Q_{ec} = A_p \cdot N_c \cdot C_p$$

where $A_p = A_s$ defined above = 1.1304 m^2

N_c = Bearing Capacity factor = 9

C_p = Average Cohesion at pile tip = 0.00 kN/m^2

Ultimate End Bearing Capacity $Q_{ec} = 1.130 \cdot 0.000 \cdot 9 = 0.00 \text{ kN/m}^2$

Total Ultimate End Bearing Capacity $Q_u = Q_{eg} + Q_{ec} = 2415.07 \text{ kN}$

Pd Level for this pile= -21.600m

Layer No.1

Effective overburden pressure due to this layer $= 0.500 \times 0.000 + 7.500 \times (0.000 - 9.807346327) = -73.555 \text{ kN/m}^2$

Layer No.2

Effective overburden pressure due to this layer $= 8.000 \times (19.000 - 9.807346327) = 73.541 \text{ kN/m}^2$

Layer No.3

Effective overburden pressure due to this layer $= 5.600 \times (19.500 - 9.807346327) = 54.279 \text{ kN/m}^2$

Layer No.4

Effective overburden pressure due to this layer $= 0.000 \times (19.500 - 9.807346327) = 0.000 \text{ kN/m}^2$

Total effective overburden pressure up to -21.600M level from EGL=54.265 kN/m²

****ULTIMATE SKIN FRICTION CAPACITY****

For Granular Soils

$$Q_{sg} = \sum [K \cdot P_{di} \cdot \tan(d) \cdot A_{si}] \text{ for all layers}$$

where K = Earth Pressure Coeff.

P_{di} = Effective Overburden pressure for i th layer

d = Angle of wall friction for i th layer

A_{si} = Surface area of pile stem for i th layer

Negative Skin Friction $Q_{sg}(\text{ve}) = \sum [0.5 \cdot K \cdot W \cdot L_n \cdot \tan(d) \cdot A_{si}]$ for all layers

W = Bulk Unit Wt. of soil

W = Bulk Unit Wt. of soil

L_n = Thk. of Compressible layer
 K = As defined above
 d = As defined above
 A_{si} = As defined above but with L_n

For Cohesive Soils

$Q_{sc} = \sum [a * C * A_{si}]$ for all layers
where a = Adhesion factor
 C = Average Cohesion
 A_{si} = Surface area of pile stem for i th layer
Negative Skin Friction $Q_{sc}(\text{ve}) = \sum [S * A_{si}]$ for all layers
 S = Shear strength
 A_{si} = As defined above but with L_n

Layer no. 1:

Layer Thickness : 8.000
 P_{di} : -36.78kN/m²
 K : 0.00
 P_{di} : -36.78kN/m²
 $\tan(d)$: 0.00
 A_{si} : 22.62m²
 $Q_{sg} = K * P_{di} * \tan(d) * A_{si} = 0.00 * -36.78 * 0.00 * 22.62 = -0.00\text{kN}$

Adhesion factor(a) : 1.00 for U.Shear(C_u)=0.00
Average Cohesion(C): 0.00kN/m²
 $Q_{sc} = a * C * A_{si} = 1.00 * 0.00 * 22.62 = 0.00\text{kN}$

Total net skin friction of this layer = $[Q_{sg} - Q_{sg}(\text{ve})] + [Q_{sc} - Q_{sc}(\text{ve})]$
= $[-0.00 - 0.00] + [0.00 - 0.00] = 0.00\text{kN/m}^2$

Layer no. 2:

Layer Thickness : 8.000
 P_{di} : -36.78kN/m²
 K : 1.00
 P_{di} : -36.78kN/m²
 $\tan(d)$: 0.55
 A_{si} : 30.16m²
 $Q_{sg} = K * P_{di} * \tan(d) * A_{si} = 1.00 * -36.78 * 0.55 * 30.16 = -614.95\text{kN}$

Adhesion factor(a) : 1.00 for U.Shear(C_u)=0.00
Average Cohesion(C): 0.00kN/m²
 $Q_{sc} = a * C * A_{si} = 1.00 * 0.00 * 30.16 = 0.00\text{kN}$

Total net skin friction of this layer = $[Q_{sg} - Q_{sg}(\text{ve})] + [Q_{sc} - Q_{sc}(\text{ve})]$
= $[-614.95 - 0.00] + [0.00 - 0.00] = -614.95\text{kN/m}^2$

Layer no. 3:

Layer Thickness : 5.600

Pdi : 27.13kN/m²

K : 1.15

Pdi : 27.13kN/m²

tan(d) : 0.65

Asi : 21.11m²

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/m²

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/m²

Layer no. 4:

Layer Thickness : 8.400

Pdi : 54.26kN/m²

K : 1.15

Pdi : 54.26kN/m²

tan(d) : 0.65

Asi : 31.67m²

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/m²

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/m²

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 = $Q_{ut}/FOS = 438.43\text{kN}$

Calculation of lateral pile capacity (Fixed head pile)

Lateral capacity is computed based up on
Allowable top deflection $Y = 12.0\text{mm}$

Free Length $L_1 = 6.000\text{m}$.

For lateral capacity soil is considered as GRANULAR

Fixed Head Pile

Mod. of Subgrade Reaction for Granular Soil(n_b)= $0.800\text{kN/m}^3 \times 1\text{e}3$ (From IS Code)

$E = 29580000.00\text{ kN/m}^2$ $I = 0.101788\text{m}^4$.

$T = (EI/n_b)^{0.2} = (29580000.00 \times 0.101788 / 0.800 \times 1\text{e}3)^{0.2} = 5.189\text{m}$

$L_1/T = 1.156$

Depth of Fixity $L_f = 10.322\text{m}$

Total Ultimate Horizontal Load Capacity H [As per IS Code] = $12EIY/(L_1+L_f)^3$
= 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)**Considering the site condition and to be on a safer side pile foundation is strongly recommended.**

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			
	3.00	3.45	0.45	U	* Not - Received						
	3.50	-		D							
	4.50	4.95	0.45	P	4	5	8	13	Medium dense, light grey, silty fine to medium sand with mica		
	6.00	6.45	0.45	P	4	7	9	16			
	7.50	7.95	0.45	P	5	6	9	15			
	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			
	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			
									2.80m		
									15.90m		

00-01	21.00	21.45	0.45	P	15	19	23	42	Dense to very dense, light grey, silty fine to medium sand with mica .
	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	
	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)		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 to medium, greyish brown, silty clay/clayey silt with sand		
	1.50	1.95	0.45	U							
	2.00	2.45	0.45	P	1	2	3	5	_____ 2.60m _____		
	3.00	3.45	0.45	P	2	3	4	7	Loose to medium dense, light grey, silty fine to medium sand with mica		
	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			
	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			

00-01	17.50	-		D					Dense to very dense, light grey, silty fine to medium sand with mica and gravels.
	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	
	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)

BH 1(A1)																						
BH - 1	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)
	D = Distrubed soil sample, P = SPT Sample, U = Undistrubed Sample UU= Uncompressive 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= Uncompressive undrained test DS= Direct shear test																					

BH 2(P2)

BH - 2	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)	
	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= Uncompressive 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 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 in 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 auger 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		
SI 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		
SI 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: Undisturbed sample 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 out on 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''): $N'' = 15 + 0.5 \times (N' - 15)$

Typical calculation for N value correction:

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: $= 12.35 \text{ t/m}^2$.

From Fig 1.of IS: 2131, correction factor $= 0.93$

So corrected SPT value for overburden, $N' = 0.93 \times 36 = 33.48$.

Corrected SPT for dilatancy $N'' = 15 + \{0.5 \times (33.48 - 15)\} = 24.24$

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
2	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
	2.00	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.

8.1 Stratum-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

8.2 Stratum-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

8.3 Stratum-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

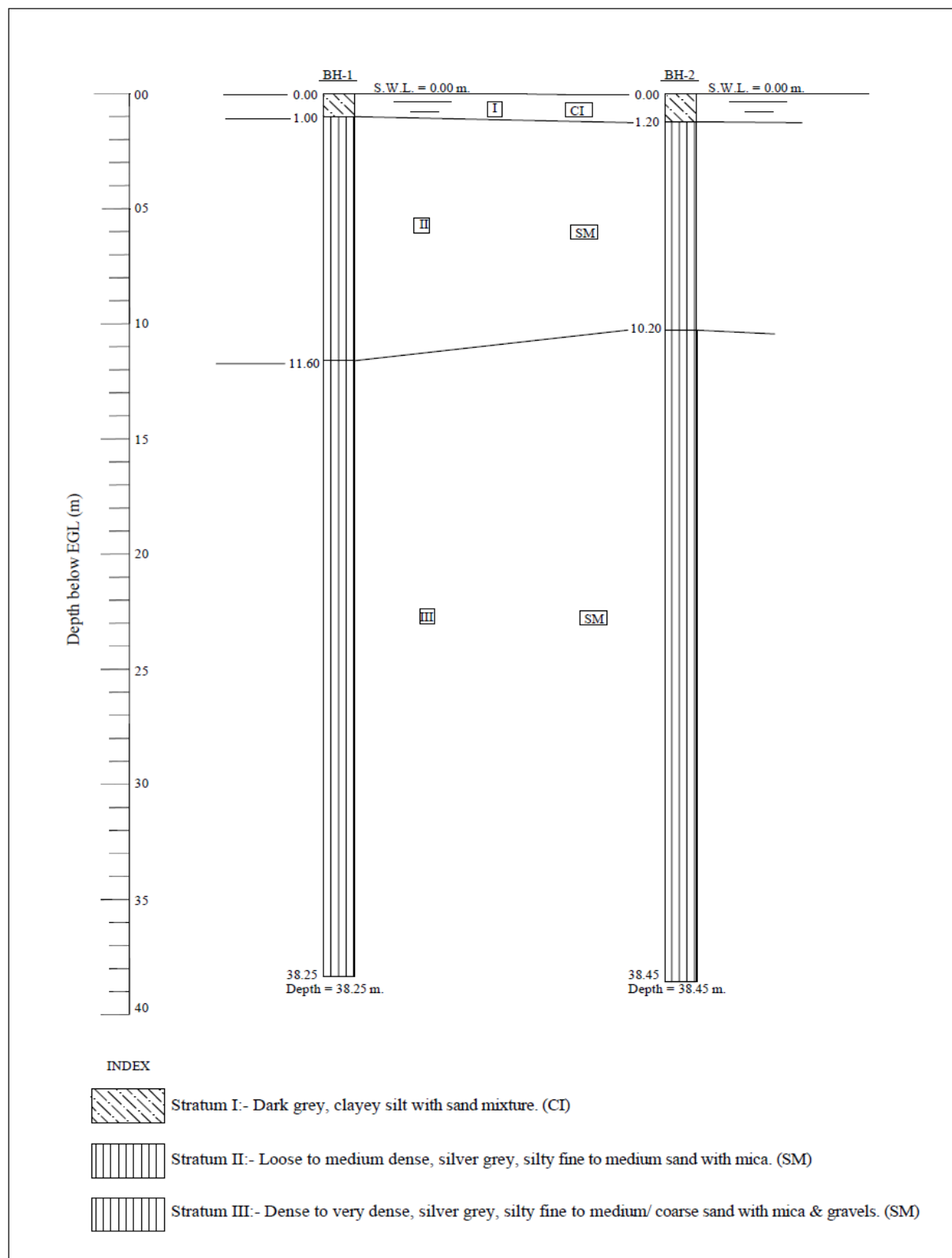


Fig.1 Generalized Sub Soil Profile

9.0 FOUNDATIONS

In view of the above sub-soil conditions in mind and the type of structure both the aspects regarding bearing capacity of open foundations and pile capacity for deep foundation in the form of bored cast in-situ pile have been discussed in 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 Standard 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 $CRR/CSR \leq 1.0$ indicates that the soil is prone to liquefaction whereas $CRR/CSR > 1.0$ or corrected $N_1 > 30$ 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) moisture contents 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	Liquefiable	
	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	Liquefiable	
	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	Liquefiable	
	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 Liquefiable	
	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 Liquefiable	
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
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	Liquefiable	
	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	Liquefiable	
	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 Liquefiable	
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable
	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 Liquefiable

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 6.00 m depth below EGL.

Liquefaction Measures

Liquefaction potential can be improved by following ground improvement techniques.

- 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.
- 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 $\Phi = 31^\circ$ [Ref: Fig. No. 1 of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.35$. $\Phi = 33.25^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 29^\circ$.

$\gamma_{\text{sat}} = 1.90 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.90 \text{ t/m}^3$ [Effective density]

STRATUM-III:

Design corrected “N” = 26. Corresponding $\Phi = 36^\circ$ [Ref: Fig. No. 1 of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.65$. $\Phi = 37.75^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 33^\circ$.

$\gamma_{\text{sat}} = 1.95 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.95 \text{ t/m}^3$ [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 = $\pi \times D \times L_i$

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

$$Q_{eg} = A_p(0.5 \times D \times W \times N_r + P_d \times N_q)$$

where A_p = Cross sectional Area = $1.1304m^2$

D = Pile Stem Dia = 1.20m

W = Effective unit wt. of soil at Pile tip = 9.69kN/m³

Nr = Bearing Capacity factor = 35.19

Pd = Effective overburden pressure at pile tip = 56.46kN/m²

Nq = Bearing Capacity factor = 35.60

Ultimate End Bearing Capacity Qeg

$$=1.130 \times (0.5 \times 1.200 \times 9.693 \times 35.19 + 56.465 \times 35.60) = 2503.60 \text{ kN}$$

For Cohesive Soils

$$Q_{ec} = A_p \times N_c \times C_p$$

where $A_p = A_s$ defined above = 1.1304m²

Nc = Bearing Capacity factor = 9

Cp = Average Cohesion at pile tip = 0.00kN/m²

$$\text{Ultimate End Bearing Capacity } Q_{ec} = 1.130 \times 0.000 \times 9 = 0.00 \text{ kN/m}^2$$

$$\text{Total Ultimate End Bearing Capacity } Q_u = Q_{eg} + Q_{ec} = 2503.60 \text{ kN}$$

Pd Level for this pile = -21.600m

Layer No.1

$$\text{Effective overburden pressure due to this layer} = 0.500 \times 0.000 + 7.500 \times (0.000 - 9.807346327) = -73.555 \text{ kN/m}^2$$

Layer No.2

$$\text{Effective overburden pressure due to this layer} = 3.600 \times (19.000 - 9.807346327) = 33.094 \text{ kN/m}^2$$

Layer No.3

$$\text{Effective overburden pressure due to this layer} = 10.000 \times (19.500 - 9.807346327) = 96.927 \text{ kN/m}^2$$

Layer No.4

Effective overburden pressure due to this layer $= 0.000 \times (19.500 - 9.807346327)$
 $= 0.000 \text{ kN/m}^2$

Total effective overburden pressure up to -21.600M level from EGL $= 56.465$
 kN/m^2

****ULTIMATE SKIN FRICTION CAPACITY****

For Granular Soils

$Q_{sg} = \sum [K * P_{di} * \tan(d) * A_{si}]$ for all layers

where K = Earth Pressure Coeff.

P_{di} = Effective Overburden pressure for i th layer

d = Angle of wall friction for i th layer

A_{si} = Surface area of pile stem for i th layer

Negative Skin Friction $Q_{sg}(\text{ve}) = \sum [0.5 * K * W * L_n * \tan(d) * A_{si}]$ for all layers

W = Bulk Unit Wt. of soil

W = Bulk Unit Wt. of soil

L_n = Thk. of Compressible layer

K = As defined above

d = As defined above

A_{si} = As defined above but with L_n

For Cohesive Soils

$Q_{sc} = \sum [a * C * A_{si}]$ for all layers

where a = Adhesion factor

C = Average Cohesion

A_{si} = Surface area of pile stem for i th layer

Negative Skin Friction $Q_{sc}(\text{ve}) = \sum [S * A_{si}]$ for all layers

S = Shear strength

Asi = As defined above but with Ln

Layer no. 1:

Layer Thickness : 8.000

Pdi : -36.78kN/m²

K : 0.00

Pdi : -36.78kN/m²

tan(d) : 0.00

Asi : 22.62m²

$Q_{sg} = K \cdot P_{di} \cdot \tan(d) \cdot A_{si} = 0.00 \cdot -36.78 \cdot 0.00 \cdot 22.62 = -0.00 \text{ kN}$

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m²

$Q_{sc} = a \cdot C \cdot A_{si} = 1.00 \cdot 0.00 \cdot 22.62 = 0.00 \text{ kN}$

Total net skin friction of this layer = $[Q_{sg} - Q_{sg}(\text{_ve})] + [Q_{sc} - Q_{sc}(\text{_ve})]$
= $[-0.00 - 0.00] + [0.00 - 0.00] = 0.00 \text{ kN/m}^2$

Layer no. 2:

Layer Thickness : 3.600

Pdi : -57.01kN/m²

K : 1.00

Pdi : -57.01kN/m²

tan(d) : 0.55

Asi : 13.57m²

$Q_{sg} = K \cdot P_{di} \cdot \tan(d) \cdot A_{si} = 1.00 \cdot -57.01 \cdot 0.55 \cdot 13.57 = -428.87 \text{ kN}$

Adhesion factor(a) : 1.00 for U.Shear(Cu)=0.00

Average Cohesion(C): 0.00kN/m²

$Q_{sc} = a \cdot C \cdot A_{si} = 1.00 \cdot 0.00 \cdot 13.57 = 0.00 \text{ kN}$

$$\begin{aligned}\text{Total net skin friction of this layer} &= [Q_{sg} - Q_{sg}(\text{_ve})] + [Q_{sc} - Q_{sc}(\text{_ve})] \\ &= [-428.87 - 0.00] + [0.00 - 0.00] = -428.87 \text{ kN/m}^2\end{aligned}$$

Layer no. 3:

Layer Thickness : 10.000

P_{di} : 8.00 kN/m²

K : 1.15

P_{di} : 8.00 kN/m²

tan(*d*) : 0.65

A_{si} : 37.70 m²

$$Q_{sg} = K \cdot P_{di} \cdot \tan(d) \cdot A_{si} = 1.15 \cdot 8.00 \cdot 0.65 \cdot 37.70 = 225.28 \text{ kN}$$

Adhesion factor(*a*) : 1.00 for U.Shear(*C_u*)=0.00

Average Cohesion(*C*): 0.00 kN/m²

$$Q_{sc} = a \cdot C \cdot A_{si} = 1.00 \cdot 0.00 \cdot 37.70 = 0.00 \text{ kN}$$

$$\begin{aligned}\text{Total net skin friction of this layer} &= [Q_{sg} - Q_{sg}(\text{_ve})] + [Q_{sc} - Q_{sc}(\text{_ve})] \\ &= [225.28 - 0.00] + [0.00 - 0.00] = 225.28 \text{ kN/m}^2\end{aligned}$$

Layer no. 4:

Layer Thickness : 8.400

P_{di} : 56.46 kN/m²

K : 1.15

P_{di} : 56.46 kN/m²

tan(*d*) : 0.65

A_{si} : 31.67 m²

$$Q_{sg} = K \cdot P_{di} \cdot \tan(d) \cdot A_{si} = 1.15 \cdot 56.46 \cdot 0.65 \cdot 31.67 = 1335.38 \text{ kN}$$

Adhesion factor(*a*) : 1.00 for U.Shear(*C_u*)=0.00

Average Cohesion(*C*): 0.00 kN/m²

$$Q_{sc} = a \cdot C \cdot A_{si} = 1.00 \cdot 0.00 \cdot 31.67 = 0.00 \text{ kN}$$

$$\text{Total net skin friction of this layer} = [Q_{sg} - Q_{sg}(\text{_ve})] + [Q_{sc} - Q_{sc}(\text{_ve})]$$

$$=[1335.38-0.00]+[0.00-0.00]=1335.38\text{kN/m}^2$$

$$\text{Total Skin Friction Capacity } Q_{us} = Q_{sg} + Q_{sc} = 2127.61\text{kN}$$

$$\text{Total Ultimate Pile Capacity } Q_u = Q_{us} + Q_{ue} = 4631.20\text{kN}$$

$$\text{****Safe Pile Capacity } Q_{us}/\text{FOS} + Q_{ue}/\text{FOS} = 1852.48\text{kN} \text{****}$$

****Computation of Tension capacity****

Tension capacity is computed based upon skin friction of pile only

$$\text{Total ultimate skin friction } Q_{us} = 2127.61\text{ kN}$$

$$\text{Average Void Factor} = 12.00\%$$

$$\text{Total ultimate tension capacity } Q_{ut} = 2127.61\text{kN}$$

$$\text{F.O.S. in tension} = 2.50$$

$$\text{****Safe tension capacity} = Q_{ut}/\text{FOS} = 851.04\text{kN}$$

Calculation of lateral pile capacity (Fixed head pile)

Computation of Lateral Capacity****

Lateral capacity is computed based up on

$$\text{Allowable top deflection } Y = 12.0\text{mm}$$

$$\text{Free Length } L_1 = 6.000\text{m.}$$

For lateral capacity soil is considered as GRANULAR

Fixed Head Pile

Mod. of Subgrade Reaction for Granular Soil(n_b)= $0.800\text{kN/m}^3 \times 1e3$ (From IS Code)

$$E = 29580000.00\text{ kN/m}^2 \quad I = 0.101788\text{m}^4.$$

$$T = (EI/n_b)^{0.2} = (29580000.00 \times 0.101788 / 0.800 \times 1e3)^{0.2} = 5.189\text{m}$$

$$L_1/T = 1.156$$

Depth of Fixity $L_f = 10.322\text{m}$

Total Ultimate Horizontal Load Capacity $H[\text{As per IS Code}] = 12EIY/(L_1 + L_f)^3$
 $= 99.707\text{kN}$

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 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 foundations and 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 liquefiable upto 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) **Considering the site condition and to be on a safer side pile foundation is strongly recommended.**

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)		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 with sand mixture.		
	0.50	-		D					1.00m		
	1.50	1.95	0.45	U	Not received				Loose to medium dense, silver grey, silty fine to medium sand with mica.		
	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							
	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	Dense to very dense, silver grey, silty fine to medium/ coarse sand with mica & gravels.
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	
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	

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= Uncompressive 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= Uncompressive 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.**

Soil Investigation Report of at
Chainage: 134+280

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1.0 INTRODUCTION

A subsoil investigation was necessary for the purpose of the foundation design for the proposed structure. For this purpose borehole exploration work was done. During borehole exploration, disturbed/SPT samples were collected.

2.0 FIELD INVESTIGATION

One no. of borehole was 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).

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 30.05 m.

4.0 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).

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''): $N'' = 15 + 0.5 \times (N' - 15)$

Typical calculation for N value correction:

BH No.-P4. Depth: 13.50 m to 13.95 m. Field $N = 75$. Water table: 2.30 m below EGL.

Effective OVP at the average depth: $= 14.65 \text{ t/m}^2$.

From Fig 1. of IS: 2131, correction factor $= 0.87$.

So corrected SPT value for overburden, $N' = 0.87 \times 75 = 65.55$.

Corrected SPT for dilatancy $N'' = 15 + \{0.5 \times (65.55 - 15)\} = 40.28$

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		
P4	0.50	0.95	0.73	11	18.31	16.66	17	I
	1.50	1.95	1.73	30	41.24	28.12	28	I
	3.00	3.45	3.23	49	59.80	37.40	37	II
	4.50	4.95	4.73	29	33.15	24.08	24	II
	6.00	6.45	6.23	56	60.51	37.75	38	II
	7.50	7.95	7.73	67	68.86	41.93	42	II
	9.00	9.45	9.23	65	63.84	39.42	39	II
	10.50	10.95	10.73	70	65.95	40.48	40	II
	12.00	12.45	12.23	85	77.04	46.02	46	II
	13.50	13.95	13.73	75	65.55	40.28	40	II
	15.00	15.45	15.23	72	60.81	37.90	38	II
	17.00	17.45	17.23	65	52.58	33.79	34	II
	18.50	18.95	18.73	86	67.47	41.23	41	II
	20.00	20.45	20.23	85	64.74	39.87	40	II
	21.50	21.95	21.73	96	71.08	43.04	43	II
	23.00	23.45	23.23	93	66.99	41.00	41	II
	24.50	24.95	24.73	89	62.43	38.71	39	II
	26.00	26.45	26.23	100	68.35	41.68	42	II
	27.50	27.95	27.73	97	64.65	39.83	40	II
	29.00	29.45	29.23	100	65.04	40.02	40	II
	30.50	30.56	30.53	100	63.69	39.35	39	II

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	Loose to medium dense, silty sand	22	28	17
II	Dense to very dense, silty sand.	41	49	27

Note: N Value means Standard Penetration Test (SPT) Values.

5.0 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 2.30 m below EGL.

6.0 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.

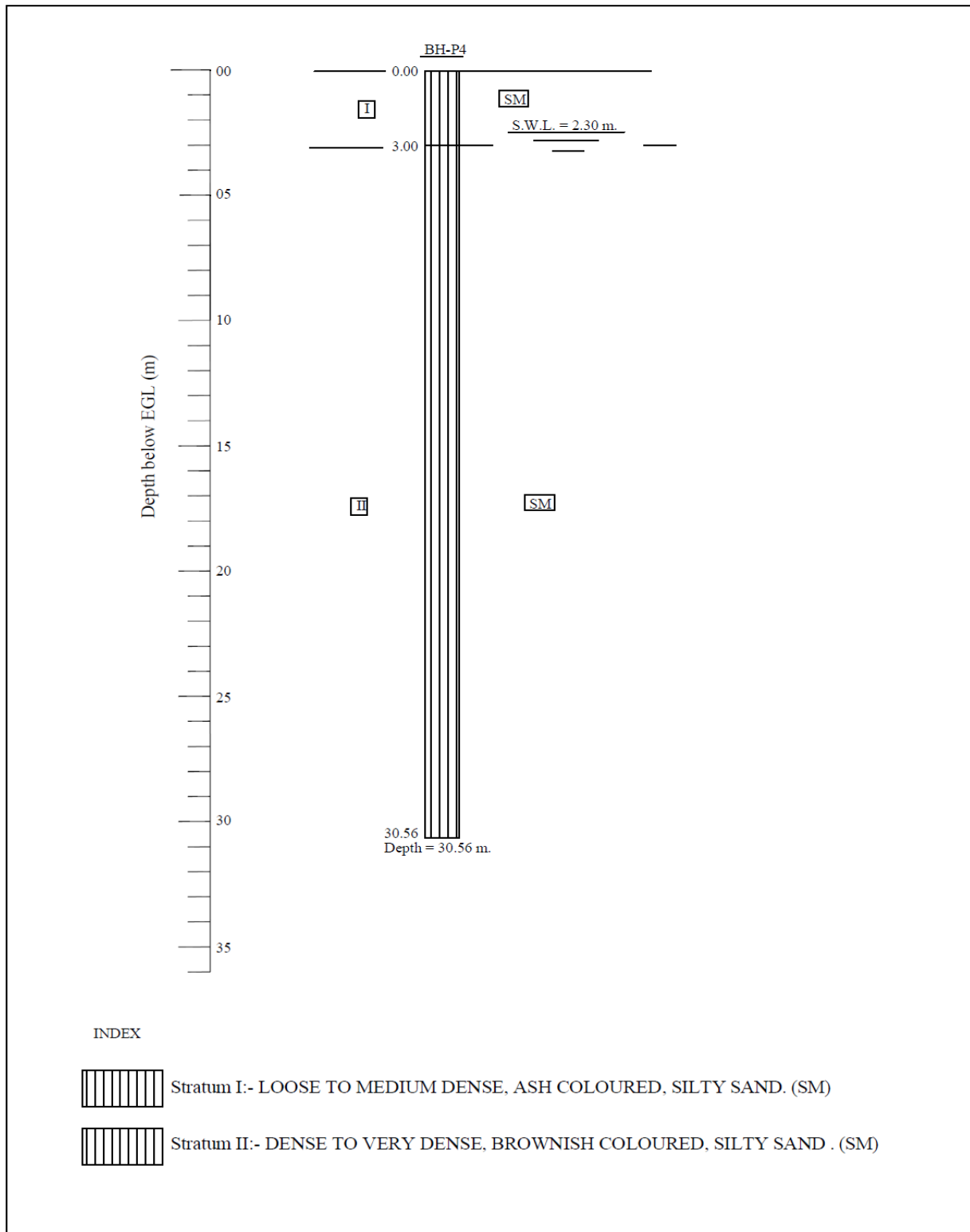


Fig.1 Generalized Sub Soil Profile

6.1 Stratum-I:

The soil in this layer consists of loose to medium dense, ash coloured silty sand. Average corrected “N” value of this layer is 22. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	12
Sand (%)	66
Silt+Clay (%)	22
Specific Gravity	2.65
Free Swell Index (%)	NIL
C (kg/cm ²)	0.12
Phi (Degree)	33.00

Note: Average properties are based on laboratory test results only.

IS Classification: SM

6.2 Stratum-II:

The soil in this layer consists of dense to very dense, brownish coloured silty sand. Average corrected “N” value of this layer is 41. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	05
Sand (%)	72
Silt+Clay (%)	23
Specific Gravity	2.60
Free Swell Index (%)	NIL
C (kg/cm ²)	0.06
Phi (Degree)	39.00

Note: Average properties are based on laboratory test results only.

IS Classification: SM

7.0 FOUNDATIONS

In view of the above sub-soil conditions in mind and the type of structure both the aspects regarding pile capacity for deep foundation in the form of bored cast in-situ pile have been discussed in the following paragraphs. Designer can choose the requirement depending upon the loading & geometry of the structure envisaged.

8.0 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 Standard 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 $CRR/CSR \leq 1.0$ indicates that the soil is prone to liquefaction whereas $CRR/CSR > 1.0$ or corrected $N_1 > 30$ indicates the soil is non liquefiable. Based on the above, the liquefaction resistance of the subsoil is determined and presented below.

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
BH-P4	0.50	11	0.93	0.43	1.70	0.75	0.98	1.05	1.10	0.75	12.0	21	3.78	1.09	16.8	1.00	0.51	0.18	0.617
	1.50	30	2.78	1.28	1.70	0.75	0.98	1.05	1.10	0.75	32.6	21	3.78	1.09	39.2	(N ₁) _{60CS} >30			
	3.00	49	5.55	2.55	1.70	0.75	0.98	1.05	1.20	0.80	62.0	17	3.01	1.06	68.7				
	4.50	29	8.33	3.83	1.62	0.75	0.98	1.05	1.20	0.85	37.1	17	3.01	1.06	42.3				
	6.00	56	11.10	5.10	1.40	0.75	0.98	1.05	1.20	0.95	69.3	17	3.01	1.06	76.5				
	7.50	67	13.88	6.38	1.25	0.75	0.98	1.05	1.20	0.95	74.1	17	3.01	1.06	81.6				
	9.00	65	16.65	7.65	1.14	0.75	0.98	1.05	1.20	0.95	65.7	17	3.01	1.06	72.6				
	10.50	70	19.43	8.93	1.06	0.75	0.98	1.05	1.20	1.00	68.9	17	3.01	1.06	76.1				
	12.00	85	22.20	10.20	0.99	0.75	0.98	1.05	1.20	1.00	79.5	17	3.01	1.06	87.3				
	13.50	75	24.98	11.48	0.93	0.75	0.98	1.05	1.20	1.00	66.2	17	3.01	1.06	73.2				
	15.00	72	27.75	12.75	0.89	0.75	0.98	1.05	1.20	1.00	60.3	17	3.01	1.06	66.9				
	17.00	65	31.45	14.45	0.83	0.75	0.98	1.05	1.20	1.00	51.1	17	3.01	1.06	57.2				
	18.50	86	34.23	15.73	0.80	0.75	0.98	1.05	1.20	1.00	64.8	17	3.01	1.06	71.7				
	20.00	85	37.00	17.00	0.77	0.75	0.98	1.05	1.20	1.00	61.6	17	3.01	1.06	68.3				
	21.50	96	39.78	18.28	0.74	0.75	0.98	1.05	1.20	1.00	67.1	17	3.01	1.06	74.2				
	23.00	93	42.55	19.55	0.72	0.75	0.98	1.05	1.20	1.00	62.9	17	3.01	1.06	69.6				
	24.50	89	45.33	20.83	0.69	0.75	0.98	1.05	1.20	1.00	58.3	17	3.01	1.06	64.8				
	26.00	100	48.10	22.10	0.67	1.00	0.98	1.05	1.20	1.00	84.8	17	3.01	1.06	92.9				
	27.50	97	50.88	23.38	0.65	1.00	0.98	1.05	1.20	1.00	79.9	17	3.01	1.06	87.8				
	29.00	100	53.65	24.65	0.64	1.00	0.98	1.05	1.20	1.00	80.3	17	3.01	1.06	88.1				
	30.50	100	56.43	25.93	0.62	1.00	0.98	1.05	1.20	1.00	78.3	17	3.01	1.06	86.0				

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 2.00 m depth below EGL.

9.0 CAPACITY CALCULATION:

9.1 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,

- Pile capacities shall be ascertained by approved method (IS 2911 (Part 1/Sec 2):2010).
- Assumed Grade of Concrete = M25.
- Diameter of pile used = 1200 mm.
- Depth of liquefaction below EGL = 2.00 m.
- Pile termination depth below existing ground level = 22.00 m.
- Cut-off depth = 2.00 m below existing ground level.

Sample Calculation of Safe Vertical Pile Capacity around BH-P4 [As per IS 2911 (Part 1/Sec 2) : 2010]

Design Strength Parameters:

STRATUM-I:

Design corrected "N" = 15. Corresponding $\Phi = 34^\circ$ [Ref: Fig. No. 1. of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.35$. $\Phi = 33.25^\circ$.

From laboratory test results, $C = 0.12 \text{ kg/cm}^2$ & $\Phi = 33^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 30^\circ$.

$\gamma_{\text{sat}} = 1.90 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.90 \text{ t/m}^3$ [Effective density]

STRATUM-II:

Design corrected "N" = 41. Corresponding $\Phi = 38^\circ$ [Ref: Fig. No. 1. of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 = Very dense. Corresponding $D_r = 0.85$. $\Phi = 40.75^\circ$.

From laboratory test results, $C = 0.06 \text{ kg/cm}^2$ & $\Phi = 39^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 35^\circ$.

$\gamma_{\text{sat}} = 1.95 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.95 \text{ t/m}^3$ [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:

$$Q_u = (A_p \times P_D \times N_q) + (A_p \times N_c \times c_p) + \sum [K_i \times P_{Di} \times \tan \delta_i \times A_{si}] + \sum [\alpha_i \times c_i \times A_{si}]$$

Pile terminating depth below existing ground level = 30.00 m

Cut-off depth = 2.00 m below existing ground level.

Liquefaction depth = 2.00 m below EGL.

Shaft length contributing pile capacity = 28.00 m.

Diameter of pile = 1200 mm.

Bottom depth of Pile =	22.00	m below existing ground level.
Cut-off Length =	2.00	m
Depth of liquefaction =	2.00	m
Diameter of Pile =	1200	mm
FOS of Shaft Resistance =	2.50	
FOS for End Bearing =	2.50	
So, maximum Depth upto which pressure will increase =	18.00	m
	Critical depth	15.00 D
	*Max unit friction on shaft f_s	10 t/m^2
	*Maximum $q(\text{tip})$ =	500 t/m^2
	Maximum OVP =	17.05 t/m^2

Layer No.	Top Depth (m)	Bottom Depth (m)	Eff. Depth (m)	Cohesion (kg/cm ²)	Φ (Degree)	Adhesion Factor (α)	K_0	Eff. Density (t/m ³)	Mean Pressure (t/m ²)
I	2.00	3.00	1.00	0.00	30	1.000	1.00	0.90	0.45
II	3.00	22.00	19.00	0.00	35	1.000	1.25	0.95	9.83

Layer No.	Eff. Depth (m)	Effective σ_z (t/m ²)	$f_{s1} = \alpha C$	$f_{s2} = K_0 \sigma_z \tan \Phi$	$f_s = f_{s1} + f_{s2}$ (t/m ²)	Ultimate Shaft Resistance (t)	Safe Shaft Resistance (t)
I	1.00	0.45	0.00	0.26	0.26	0.98	0.39
II	19.00	9.83	0.00	8.60	8.60	615.96	246.38
	26.90					616.94	246.78

For End Bearing

Cohesion $C = 0$ kg/cm²

Friction Angle = 35 °

Restricting the $q(\text{tip})$ value to 500t/sqm

Uplift Pile Capacity:

Uplift capacity = $1/3 \times [(\text{Shaft Resistance in Vertical Capacity}) + \text{self weight of pile (Buoyant)}]$

= 208.07 t

So, Safe Uplift Pile Capacity = 190 t.

** Note: As per "Pile Design and Construction Practice, Fourth Edition" by M.J. Tomlinson, Page-119 & 122*

$N_q = 51.42$

Eff. Ovp. Pressure= $p(\text{tip}) = 17.05$ t/m²

$q(\text{tip}) = C.N_c + p(\text{tip}).N_q = 876.64$ t/m²

Ultimate End Bearing = 565.49 t

Safe End Bearing = 226.19 t

Safe Shaft Resistance= 246.78 t

So, Total Pile Capacity = 472.97 t

Self weight of pile (Buoyant) = 13.56 t

So, Safe Vertical Pile Capacity = 425 t

Calculation of Lateral Pile Capacity (Fixed head pile)

Design "N" =	25		
Cantilever length(L_1) =	0	cm	
Dia of pile =	1200	mm	
f_{ck} of concrete =	25	MPa	
Deflection =	12	mm	
h =	0.34	Kg/cu cm	Modulus of Subgrade reaction
Moment of inertia (I) =	10178751.60	cm ⁴	
E of concrete =	25000.00	N/sqmm	250000 Kg/sqcm
T =	376.09	cm	
L_1/T =	0.00		
Factor for calculating L_f =	2.18		
L_f =	819.87	cm	
Pile head deflection Y =	0.02	cm	0.1804789 mm
Deflection at cut-off level =	12.00	mm	
Moment =	4.10	t-m per t of Thrust	
Reduction Factor =	0.82		
H =	66.49	t	
Corrected actual moment =	3.36	t-m per t of Thrust	

Safe lateral pile capacity = 45 t.

Based on soil design parameters, safe capacities for pile are presented below

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	
					Load (t)	Moment (t-m/ t of thrust)
30.00	28.00	1200	425	190	45	3.36

Note: The above load should confirmed by Load Test of pile as per IS: 2911(Part 4)

10.0 BORE LOG DATA SHEET:

BORE LOG DATA SHEET																	
Bore Hole No. :- BH-6				Borehole Diameter :- 150 mm Drill Hole:- NX.				Field Tests		Nos	Samples	Nos					
Location :- 134+280				Northing :- -				S		21	U	-					
				Easting :- 549443.035				Vane		-	W	1					
Water Level :- 2.30 m				Termination Depth (m):- 30.56				C		-	D	1					
Depth (m.)	Penetration			N value	Strata Description	Symbol	Thickness (m)	Samples			TCR (%)	RQD (%)	Remarks				
	15 cm	15 cm	15 cm					Type	No.	Depth (m.)							
0.00	4	5	6	<u>11</u>	LOOSE TO MEDIUM DENSE, CLAY SILTY SAND WITH GRAVAL		3.00	S	1	0.50-0.95							
	12	14	16	<u>30</u>				S	2	1.50-1.95							
3.00																	
3.00	16	23	26	<u>49</u>	Dense to very dense, silver grey, silty fine to medium/ coarse sand with mica & gravels.			S	3	3.00-3.15							
	8	12	17	<u>29</u>				S	4	4.50-4.55							
	16	26	30	<u>56</u>				S	5	6.00-6.45							
	20	30	37	<u>67</u>				S	6	7.50-7.95							
	21	33	32	<u>65</u>				S	7	9.00-9.45							
	26	36	34	<u>70</u>				S	8	10.50-10.95							
	30	41	44	<u>85</u>				S	9	12.00-12.45							
	35	36	39	<u>75</u>				S	10	13.50-13.95							
	26	36	36	<u>72</u>				S	11	15.00-15.45							
								D	1	16.50							
	32	30	35	<u>65</u>				S	12	17.00-17.45							
	36	42	44	<u>86</u>				S	13	18.50-18.95							
	32	41	44	<u>85</u>				S	14	20.00-20.45							
	36	45	51	<u>96</u>				S	15	21.50-21.95							
	39	44	49	<u>93</u>				S	16	23.00-23.45							
	41	45	44	<u>89</u>				S	17	24.50-24.95							
	39	49	51	<u>100</u>				S	18	26.00-26.45							
	36	48	49	<u>97</u>				S	19	27.50-27.95							
	7 cm pent. for 79 blows			<u>>100</u>					S	20				29.00-29.07			REFUSAL
30.56	6 cm pent. for 71 blows			<u>>100</u>					S	21				30.50-30.56			REFUSAL
" S " - ' SPT '		" U " - ' UNDISTURBED SAMPLE '		" D " - ' DISTURBED SAMPLE '		" C " - ' CORE '		" W " - ' WATER SAMPLE '		" TCR " - ' TOTAL CORE RECOVERY '		" RQD " - ' ROCK QUALITY DESIGNATION '					

Soil Investigation Report of at
Chainage: 139+915

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1.0 INTRODUCTION

A subsoil investigation was necessary for the purpose of the foundation design for the proposed structure. For this purpose borehole exploration work was done. During borehole exploration, disturbed/SPT samples were collected.

2.0 FIELD INVESTIGATION

One no. of borehole was 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).

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 auger 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 30.05 m.

4.0 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).

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''): $N'' = 15 + 0.5 \times (N' - 15)$

Typical calculation for N value correction:

BH No.-P5. Depth: 13.50 m to 13.95 m. Field $N = 90$. Water table: 1.00 m below EGL.

Effective OVP at the average depth: $= 13.35 \text{ t/m}^2$.

From Fig 1. of IS: 2131, correction factor $= 0.91$.

So corrected SPT value for overburden, $N' = 0.91 \times 90 = 81.46$.

Corrected SPT for dilatancy $N'' = 15 + \{0.5 \times (81.46 - 15)\} = 48.23$

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		
P5	0.50	0.95	0.73	5	8.32	--	8	I
	1.50	1.95	1.73	19	27.71	21.36	21	I
	3.00	3.45	3.23	30	39.49	27.25	27	I
	4.50	4.95	4.73	37	45.03	30.02	30	II
	6.00	6.45	6.23	54	61.59	38.30	38	II
	7.50	7.95	7.73	72	77.65	46.32	46	II
	9.00	9.45	9.23	64	65.66	40.33	40	II
	10.50	10.95	10.73	74	72.57	43.78	44	II
	12.00	12.45	12.23	76	71.50	43.25	43	II
	13.50	13.95	13.73	90	81.46	48.23	48	II
	15.00	15.45	15.23	70	61.10	38.05	38	II
	17.00	17.45	17.23	84	70.08	42.54	43	II
	18.50	18.95	18.73	74	59.79	37.40	37	II
	20.00	20.45	20.23	85	66.61	40.80	41	II
	21.50	21.95	21.73	91	69.24	42.12	42	II
	23.00	23.45	23.23	93	68.78	41.89	42	II
	24.50	24.95	24.73	90	64.77	39.88	40	II
	26.00	26.45	26.23	99	69.37	42.19	42	II
	27.50	27.95	27.73	91	62.14	38.57	39	II
	29.00	29.45	29.23	93	61.93	38.47	38	II
	30.50	30.55	30.53	100	65.19	40.09	40	II

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	Loose to medium dense, silty sand	19	27	8
II	Dense to very dense, silty sand.	41	48	30

Note: N Value means Standard Penetration Test (SPT) Values.

5.0 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 1.00 m below EGL.

6.0 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.

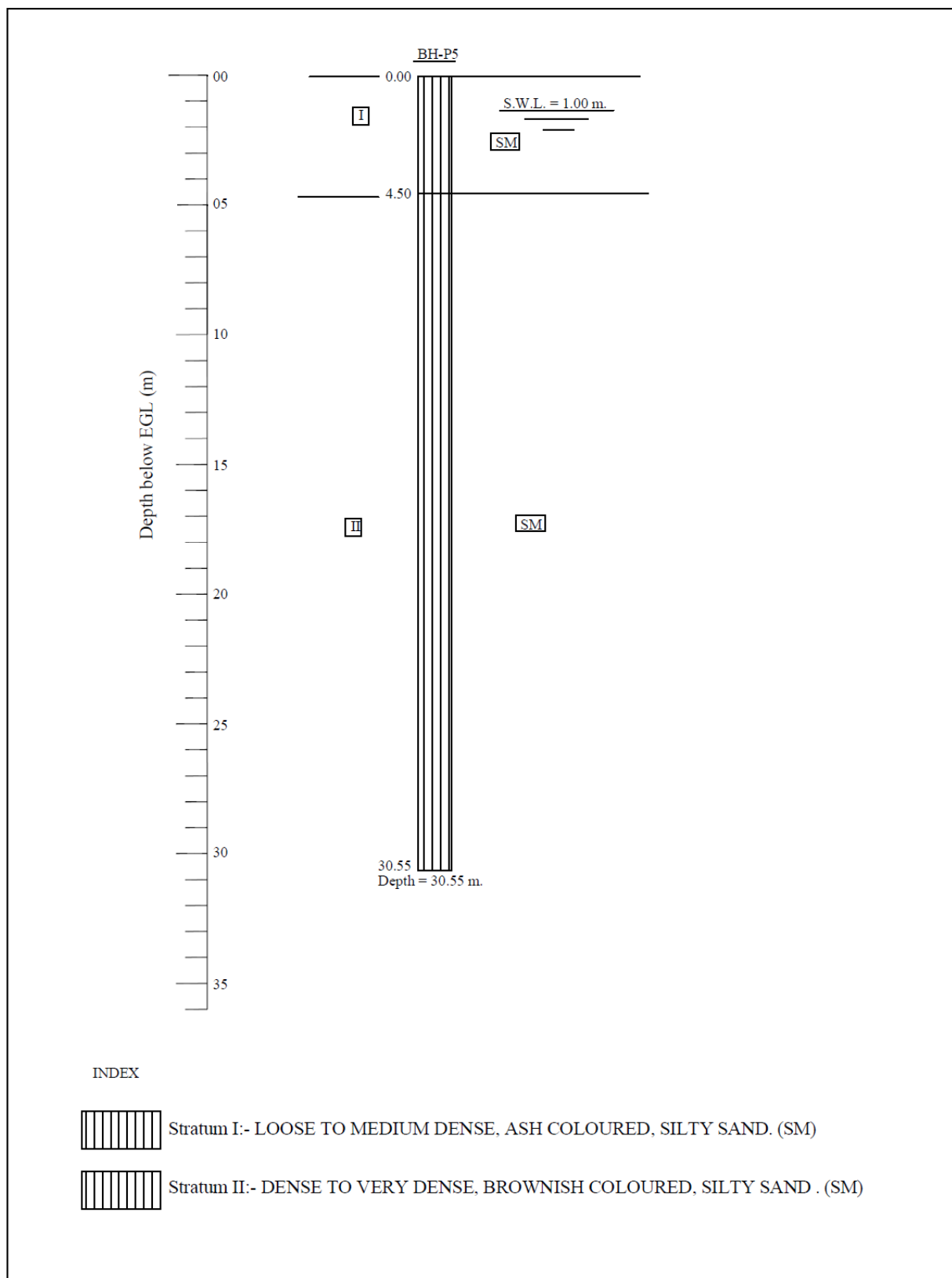


Fig.1 Generalized Sub Soil Profile

6.1 Stratum-I:

The soil in this layer consists of loose to medium dense, ash coloured silty sand. Average corrected “N” value of this layer is 19. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	--
Sand (%)	77
Silt+Clay (%)	23
Specific Gravity	2.66
Free Swell Index (%)	NIL
C (kg/cm ²)	0.12
Phi (Degree)	33.00

Note: Average properties are based on laboratory test results only.

IS Classification: SM

6.2 Stratum-II:

The soil in this layer consists of dense to very dense, brownish coloured, silty sand. Average corrected “N” value of this layer is 41. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	--
Sand (%)	80
Silt+Clay (%)	20
Specific Gravity	2.59
Free Swell Index (%)	NIL
C (kg/cm ²)	0.05
Phi (Degree)	39.00

Note: Average properties are based on laboratory test results only.

IS Classification: SM

7.0 FOUNDATIONS

In view of the above sub-soil conditions in mind and the type of structure both the aspects regarding capacity for deep foundation in the form of bored cast in-situ pile have been discussed in the following paragraphs. Designer can choose the requirement depending upon the loading & geometry of the structure envisaged.

8.0 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 Standard 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 $CRR/CSR \leq 1.0$ indicates that the soil is prone to liquefaction whereas $CRR/CSR > 1.0$ or corrected $N_1 > 30$ indicates the soil is non liquefiable. Based on the above, the liquefaction resistance of the subsoil is determined and presented below.

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-P5	0.50	5	0.93	0.43	1.70	0.75	0.98	1.05	1.10	0.75	5.4	23	4.06	1.10	10.0	1.00	0.51	0.11	0.393	Liquefiable	
	1.50	19	2.78	1.28	1.70	0.75	0.98	1.05	1.10	0.75	20.7	23	4.06	1.10	26.8	0.99	0.50	0.33	1.160	Non Liquefiable	
	3.00	30	5.55	2.55	1.70	0.75	0.98	1.05	1.10	0.80	34.8	23	4.06	1.10	42.3	(N ₁) _{60CS} >30					Non Liquefiable
	4.50	37	8.33	3.83	1.62	0.75	0.98	1.05	1.20	0.85	47.3	20	3.61	1.08	54.7						Non Liquefiable
	6.00	54	11.10	5.10	1.40	0.75	0.98	1.05	1.20	0.95	66.8	20	3.61	1.08	75.7						Non Liquefiable
	7.50	72	13.88	6.38	1.25	0.75	0.98	1.05	1.20	0.95	79.7	20	3.61	1.08	89.6						Non Liquefiable
	9.00	64	16.65	7.65	1.14	0.75	0.98	1.05	1.20	0.95	64.7	20	3.61	1.08	73.4						Non Liquefiable
	10.50	74	19.43	8.93	1.06	0.75	0.98	1.05	1.20	1.00	72.9	20	3.61	1.08	82.3						Non Liquefiable
	12.00	76	22.20	10.20	0.99	0.75	0.98	1.05	1.20	1.00	71.1	20	3.61	1.08	80.4						Non Liquefiable
	13.50	90	24.98	11.48	0.93	0.75	0.98	1.05	1.20	1.00	79.4	20	3.61	1.08	89.3						Non Liquefiable
	15.00	70	27.75	12.75	0.89	0.75	0.98	1.05	1.20	1.00	58.6	20	3.61	1.08	66.9						Non Liquefiable
	17.00	84	31.45	14.45	0.83	0.75	0.98	1.05	1.20	1.00	66.0	20	3.61	1.08	74.9						Non Liquefiable
	18.50	74	34.23	15.73	0.80	0.75	0.98	1.05	1.20	1.00	55.8	20	3.61	1.08	63.8						Non Liquefiable
	20.00	85	37.00	17.00	0.77	0.75	0.98	1.05	1.20	1.00	61.6	20	3.61	1.08	70.1						Non Liquefiable
	21.50	91	39.78	18.28	0.74	0.75	0.98	1.05	1.20	1.00	63.6	20	3.61	1.08	72.3						Non Liquefiable
	23.00	93	42.55	19.55	0.72	0.75	0.98	1.05	1.20	1.00	62.9	20	3.61	1.08	71.5						Non Liquefiable
	24.50	90	45.33	20.83	0.69	0.75	0.98	1.05	1.20	1.00	58.9	20	3.61	1.08	67.2						Non Liquefiable
	26.00	99	48.10	22.10	0.67	1.00	0.98	1.05	1.20	1.00	83.9	20	3.61	1.08	94.2						Non Liquefiable
	27.50	91	50.88	23.38	0.65	1.00	0.98	1.05	1.20	1.00	75.0	20	3.61	1.08	84.6						Non Liquefiable
	29.00	93	53.65	24.65	0.64	1.00	0.98	1.05	1.20	1.00	74.6	20	3.61	1.08	84.2						Non Liquefiable
	30.50	100	56.43	25.93	0.62	1.00	0.98	1.05	1.20	1.00	78.3	20	3.61	1.08	88.1						Non Liquefiable

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 2.00 m depth below EGL.

9.0 CAPACITY CALCULATION:

9.1 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,

- Pile capacities shall be ascertained by approved method (IS 2911 (Part 1/Sec 2):2010).
- Assumed Grade of Concrete = M25.
- Diameter of pile used = 1200 mm.
- Depth of liquefaction below EGL = 2.00 m.
- Pile termination depth below existing ground level = 25.00 m.
- Cut-off depth = 2.00 m below existing ground level.

g)

Sample Calculation of Safe Vertical Pile Capacity around BH-P4 [As per IS 2911 (Part 1/Sec 2) : 2010]

Design Strength Parameters:

STRATUM-I:

Design corrected "N" = 15. Corresponding $\Phi = 34^\circ$ [Ref: Fig. No. 1. of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.35$. $\Phi = 33.25^\circ$.

From laboratory test results, $C = 0.12 \text{ kg/cm}^2$ & $\Phi = 33^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 30^\circ$.

$\gamma_{\text{sat}} = 1.90 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.90 \text{ t/m}^3$ [Effective density]

STRATUM-II:

Design corrected "N" = 41. Corresponding $\Phi = 38^\circ$ [Ref: Fig. No. 1. of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 = Very dense. Corresponding $D_r = 0.85$. $\Phi = 40.75^\circ$.

From laboratory test results, $C = 0.06 \text{ kg/cm}^2$ & $\Phi = 39^\circ$.

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 $C = 0.00 \text{ kg/cm}^2$ and $\Phi = 35^\circ$.

$\gamma_{\text{sat}} = 1.95 \text{ t/m}^3$. $\gamma_{\text{sub}} = 0.95 \text{ t/m}^3$ [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:

$$Q_u = (A_p \times P_D \times N_q) + (A_p \times N_c \times c_p) + \sum [K_i \times P_{Di} \times \tan \delta_i \times A_{si}] + \sum [\alpha_i \times c_i \times A_{si}]$$

Pile terminating depth below existing ground level = 30.00 m

Cut-off depth = 2.00 m below existing ground level.

Liquefaction depth = 2.00 m below EGL.

Shaft length contributing pile capacity = 28.00 m.

Diameter of pile = 1200 mm.

Bottom depth of Pile =	22.00	m below existing ground level.		
Cut-off Length =	2.00	m		
Depth of liquefaction =	2.00	m		
Diameter of Pile =	1200	mm	Critical depth	15.00 D
FOS of Shaft Resistance =	2.50		*Max unit friction on shaft f_s	10 t/m^2
FOS for End Bearing =	2.50		*Maximum $q(\text{tip})$ =	500 t/m^2
So, maximum Depth upto which pressure will increase =	18.00	m	Maximum OVP =	16.98 t/m^2

Layer No.	Top Depth (m)	Bottom Depth (m)	Eff. Depth (m)	Cohesion (kg/cm ²)	Φ (Degree)	Adhesion Factor (α)	K_0	Eff. Density (t/m ³)	Mean Pressure (t/m ²)
I	2.00	4.50	2.50	0.00	30	1.000	1.00	0.90	1.13
II	4.50	22.00	17.50	0.00	35	1.000	1.25	0.95	10.45

Layer No.	Eff. Depth (m)	Effective σ_z (t/m ²)	$f_{s1} = \alpha C$	$f_{s2} = K_0 \sigma_z \tan \Phi$	$f_s = f_{s1} + f_{s2}$ (t/m ²)	Ultimate Shaft Resistance (t)	Safe Shaft Resistance (t)
I	2.50	1.13	0.00	0.65	0.65	6.12	2.45
II	17.50	10.45	0.00	9.15	9.15	603.65	241.46
	26.90					609.77	243.91

For End Bearing

Cohesion $C =$ 0 kg/cm²
Friction Angle = 35 °

Restricting the $q(\text{tip})$ value to 500t/sqm

Uplift Pile Capacity:

Uplift capacity = $1/3 \times [(\text{Shaft Resistance in Vertical Capacity}) + \text{self weight of pile (Buoyant)}]$
= 205.70 t

$N_q =$ 51.42

Eff. Ovp. Pressure= $p(\text{tip})$ 16.98 t/m^2

$q(\text{tip}) = C.N_c + p(\text{tip}).N_q =$ 872.79 t/m^2

Ultimate End Bearing = 565.49 t

Safe End Bearing = 226.19 t

Safe Shaft Resistance= 243.91 t

So, Total Pile Capacity = 470.10 t

Self weight of pile (Buoyant) = 13.56 t

So, Safe Vertical Pile Capacity = 425 t

So, Safe Uplift Pile Capacity = 190 t.

* Note: As per "Pile Design and Construction Practice, Fourth Edition" by M.J. Tomlinson, Page-119 & 122

Calculation of lateral Pile Capacity (Fixed head pile)

Design "N" = 25
Cantilever length(L_1) = 0 cm
Dia of pile = 1200 mm
 f_{ck} of concrete = 25 MPa
Deflection = 12 mm

$h = 0.34$ Kg/cucm Modulus of Subgrade reaction
Moment of inertia (I) = 10178751.60 cm^4
E of concrete = 25000.00 N/sqmm 250000 Kg/sqcm
T = 376.09 cm
 $L_1/T = 0.00$
Factor for calculating L_f = 2.18
 $L_f = 819.87$ cm
Pile head deflection Y = 0.02 cm 0.1804789 mm
Deflection at cut-off level = 12.00 mm
Moment = 4.10 t-m per t of Thrust
Reduction Factor = 0.82

H = 66.49 t
Corrected actual moment = 3.36 t-m per t of Thrust

Safe lateral Pile Capacity = 45 t.

Based on soil design parameters, safe capacities for pile are presented below

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	
					Load (t)	Moment (t-m/ t of thrust)
30.00	28.00	1200	425	190	45	3.36

Note: The above load should confirmed by Load Test of pile as per IS: 2911(Part 4)

10.0 BORE LOG DATA SHEET:

BORE LOG DATA SHEET																			
Bore Hole No. :- BH-7				Borehole Diameter :- 150 mm Drill Hole:- NX.				Field Tests		Nos	Samples		Nos						
Location :- 139+915				Northing :- -				S		21	U		-						
				Easting :- 554173.229				Vane		-	W		1						
Water Level : 1.00 m				Termination Depth (m):- 30.55				C		-	D		1						
Depth (m.)	Penetration			N value	Strata Description	Symbol	Thickness (m)	Samples			TCR (%)	RQD (%)	Remarks						
	15 cm	15 cm	15 cm					Type	No.	Depth (m.)									
0.00	2	2	3	<u>5</u>	LOOSE TO MEDIUM DENSE, ASH COLOURED, SILTY SAND.		4.50	S	1	0.50-0.95									
	6	8	11	<u>19</u>				S	2	1.50-1.95									
	9	13	17	<u>30</u>				S	3	3.00-3.15									
4.50																			
4.50	12	16	21	<u>37</u>	DENSE TO VERY DENSE, BROWNISH COLOURED, SILTY SAND			S	4	4.50-4.55									
	16	23	31	<u>54</u>				S	5	6.00-6.45									
	22	30	42	<u>72</u>				S	6	7.50-7.95									
	23	32	32	<u>64</u>				S	7	9.00-9.45									
	31	33	41	<u>74</u>				S	8	10.50-10.95									
	32	32	44	<u>76</u>				S	9	12.00-12.45									
	30	41	49	<u>90</u>				S	10	13.50-13.95									
	32	35	35	<u>70</u>				S	11	15.00-15.45									
								D	1	16.50									
	40	40	44	<u>84</u>				S	12	17.00-17.45									
	35	39	35	<u>74</u>				S	13	18.50-18.95									
	34	41	44	<u>85</u>				S	14	20.00-20.45									
	35	45	46	<u>91</u>				S	15	21.50-21.95									
	33	42	51	<u>93</u>				S	16	23.00-23.45									
	36	39	51	<u>90</u>				S	17	24.50-24.95									
	42	49	50	<u>99</u>				S	18	26.00-26.45									
	40	42	49	<u>91</u>				S	19	27.50-27.95									
	42	44	49	<u>93</u>				S	20	29.00-29.45									
30.55	5 cm pent. for 62 blows			<u>≥100</u>										S	21	30.50-30.55			REFUSAL
" S " - 'SPT'		" U " - 'UNDISTURBED SAMPLE'		" D " - 'DISTURBED SAMPLE'				" C " - 'CORE'		" W " - 'WATER SAMPLE'				" TCR " - 'TOTAL CORE RECOVERY'		" RQD " - 'ROCK QUALITY DESIGNATION'			

Soil Investigation Report of at
Chainage: 142+195

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1.0 INTRODUCTION

A subsoil investigation was necessary for the purpose of the foundation design for the proposed structure. For this purpose borehole exploration work was done. During borehole exploration, disturbed/SPT samples were collected.

2.0 FIELD INVESTIGATION

One no. of borehole was 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).

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 auger 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 30.05 m.

4.0 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).

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''): $N'' = 15 + 0.5 \times (N' - 15)$

Typical calculation for N value correction:

BH No.-P1. Depth: 4.50 m to 4.95 m. Field $N = 34$. Water table: 2.00 m below EGL.

Effective OVP at the average depth: $= 6.25 \text{ t/m}^2$.

From Fig 1. of IS: 2131, correction factor $= 1.16$

So corrected SPT value for overburden, $N' = 1.16 \times 34 = 39.40$.

Corrected SPT for dilatancy $N'' = 15 + \{0.5 \times (39.40 - 15)\} = 27.20$

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		
P1	0.50	0.95	0.73	8	13.32	--	13	I
	1.50	1.95	1.73	12	16.50	15.75	16	I
	3.00	3.45	3.23	19	23.56	19.28	19	I
	4.50	4.95	4.73	34	39.40	27.20	27	II
	6.00	6.45	6.23	50	54.67	34.84	35	II
	7.50	7.95	7.73	53	55.06	35.03	35	II
	9.00	9.45	9.23	61	60.50	37.75	38	II
	10.50	10.95	10.73	73	69.40	42.20	42	II
	12.00	12.45	12.23	76	69.46	42.23	42	II
	13.50	13.95	13.73	81	71.36	43.18	43	II
	15.00	15.45	15.23	91	77.43	46.22	46	II
	16.50	16.95	16.73	88	72.45	43.73	44	II
	18.00	18.45	18.23	94	75.00	45.00	45	II
	19.50	19.95	19.73	92	71.22	43.11	43	II
	21.00	21.45	21.23	95	71.45	43.22	43	II
	22.50	22.95	22.73	93	68.01	41.51	42	II
	24.00	24.45	24.23	96	68.33	41.67	42	II
	25.50	25.95	25.73	95	65.87	40.43	40	II
	27.00	27.45	27.23	97	65.56	40.28	40	II
	28.50	28.95	28.73	100	65.92	40.46	40	II
	30.00	30.05	30.03	100	64.55	39.77	40	II

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	Loose to medium dense, silty sand	16	19	13
II	Dense to very dense, silty sand.	40	46	27

Note: N Value means Standard Penetration Test (SPT) Values.

5.0 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 2.00 m below EGL.

6.0 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.

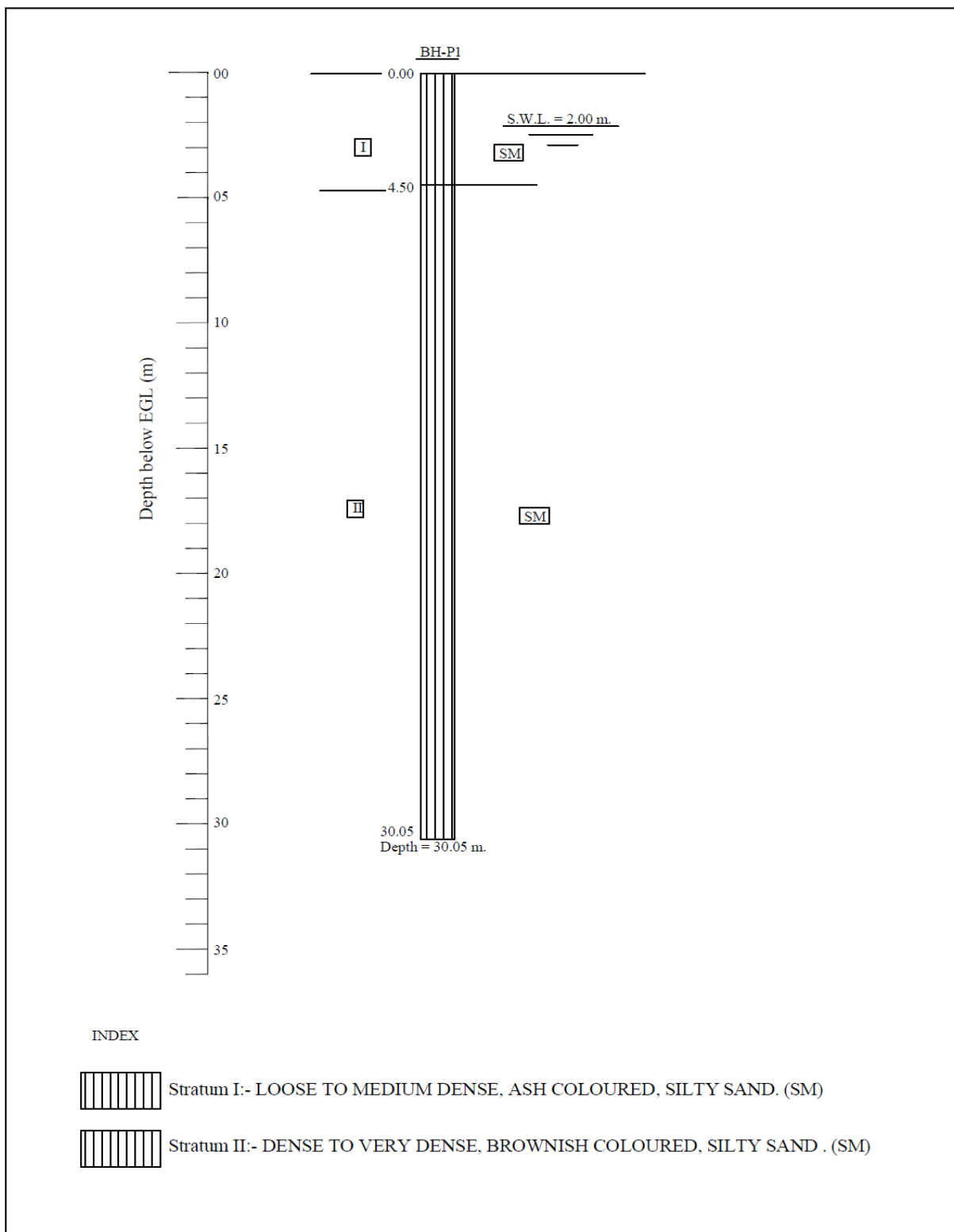


Fig.1 Generalized Sub Soil Profile

6.1 Stratum-I:

The soil in this layer consists of loose to medium dense, ash coloured, and silty sand. 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 (%)	10
Sand (%)	64
Silt+Clay (%)	26
Specific Gravity	2.66
Free Swell Index (%)	NIL
C (kg/cm ²)	0.12
Phi (Degree)	32

Note: Average properties are based on laboratory test results only.

IS Classification: SM

6.2 Stratum-II:

The soil in this layer consists of dense to very dense, brownish coloured, silty sand. Average corrected “N” value of this layer is 40. The soil samples that could be collected from this layer, shows the following average properties of the layer.

Gravel (%)	0
Sand (%)	82
Silt+Clay (%)	18
Specific Gravity	2.61
Free Swell Index (%)	NIL
C (kg/cm ²)	0.04
Phi (Degree)	40

Note: Average properties are based on laboratory test results only.

IS Classification: SM

7.0 FOUNDATIONS

In view of the above sub-soil conditions in mind and the type of structure the aspects regarding bearing capacity of open foundations have been discussed in the following paragraphs. Designer can choose the requirement depending upon the loading & geometry of the structure envisaged.

8.0 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 Standard 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 $CRR/CSR \leq 1.0$ indicates that the soil is prone to liquefaction whereas $CRR/CSR > 1.0$ or corrected $N_1 > 30$ indicates the soil is non liquefiable. Based on the above, the liquefaction resistance of the subsoil is determined and presented below.

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-P1	0.50	8	0.93	0.43	1.70	0.75	0.98	1.05	1.10	0.75	8.7	26	4.39	1.12	14.2	1.00	0.51	0.15	0.525	Liquifiable	
	1.50	12	2.78	1.28	1.70	0.75	0.98	1.05	1.10	0.75	13.0	26	4.39	1.12	19.0	0.99	0.50	0.20	0.711	Liquifiable	
	3.00	19	5.55	2.55	1.70	0.75	0.98	1.05	1.10	0.80	22.0	26	4.39	1.12	29.1	0.98	0.50	0.42	1.468	Non Liquifiable	
	4.50	34	8.33	3.83	1.62	0.75	0.98	1.05	1.20	0.85	43.5	18	3.23	1.07	49.6	(N ₁) _{60CS} >30					Non Liquifiable
	6.00	50	11.10	5.10	1.40	0.75	0.98	1.05	1.20	0.95	61.9	18	3.23	1.07	69.2						Non Liquifiable
	7.50	53	13.88	6.38	1.25	0.75	0.98	1.05	1.20	0.95	58.7	18	3.23	1.07	65.8						Non Liquifiable
	9.00	61	16.65	7.65	1.14	0.75	0.98	1.05	1.20	0.95	61.6	18	3.23	1.07	68.9						Non Liquifiable
	10.50	73	19.43	8.93	1.06	0.75	0.98	1.05	1.20	1.00	71.9	18	3.23	1.07	79.9						Non Liquifiable
	12.00	76	22.20	10.20	0.99	0.75	0.98	1.05	1.20	1.00	71.1	18	3.23	1.07	79.1						Non Liquifiable
	13.50	81	24.98	11.48	0.93	0.75	0.98	1.05	1.20	1.00	71.5	18	3.23	1.07	79.4						Non Liquifiable
	15.00	91	27.75	12.75	0.89	0.75	0.98	1.05	1.20	1.00	76.2	18	3.23	1.07	84.4						Non Liquifiable
	16.50	88	30.53	14.03	0.84	0.75	0.98	1.05	1.20	1.00	70.2	18	3.23	1.07	78.1						Non Liquifiable
	18.00	94	33.30	15.30	0.81	0.75	0.98	1.05	1.20	1.00	71.8	18	3.23	1.07	79.8						Non Liquifiable
	19.50	92	36.08	16.58	0.78	0.75	0.98	1.05	1.20	1.00	67.5	18	3.23	1.07	75.2						Non Liquifiable
	21.00	95	38.85	17.85	0.75	0.75	0.98	1.05	1.20	1.00	67.2	18	3.23	1.07	74.9						Non Liquifiable
	22.50	93	41.63	19.13	0.72	0.75	0.98	1.05	1.20	1.00	63.5	18	3.23	1.07	71.0						Non Liquifiable
	24.00	96	44.40	20.40	0.70	0.75	0.98	1.05	1.20	1.00	63.5	18	3.23	1.07	71.0						Non Liquifiable
	25.50	95	47.18	21.68	0.68	1.00	0.98	1.05	1.20	1.00	81.3	18	3.23	1.07	89.9						Non Liquifiable
	27.00	97	49.95	22.95	0.66	1.00	0.98	1.05	1.20	1.00	80.7	18	3.23	1.07	89.3						Non Liquifiable
	28.50	100	52.73	24.23	0.64	1.00	0.98	1.05	1.20	1.00	81.0	18	3.23	1.07	89.6						Non Liquifiable
	30.00	100	55.50	25.50	0.63	1.00	0.98	1.05	1.20	1.00	78.9	18	3.23	1.07	87.4						Non Liquifiable

Considering above conditions, it can be concluded that the soil at the present site is liquefiable up to 3.00 m depth below EGL.

9.0 CAPACITY CALCULATION:

9.1 USE OF OPEN/SHALLOW FOUNDATION:

BEARING CAPACITY OF OPEN/SHALLOW FOUNDATION

Assumed foundation size = 10.00 m x 10.00 m.

Assumed Depth of foundation = 3.50 m below EGL.

Design corrected “N” = 35. Corresponding $\Phi = 37^\circ$ [Ref: Fig. No. 1.of IS 6403]

Use, $\Phi = 28 + 15 \times D_r$. [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 $D_r = 0.65$. $\Phi = 37.75^\circ$.

From laboratory test results, $C = 0.04 \text{ kg/cm}^2$ & $\Phi = 40^\circ$.

Considering the subsoil, laboratory test results and to be on the safer side,
 Use, design $C = 0.00 \text{ kg/cm}^2$ & $\Phi = 33^\circ$.
 Relative density = 33.33 %.
 Method of analysis = Interpolate between local shear and general shear.

DETERMINATION OF BEARING CAPACITY:

The Net Ultimate Bearing Capacity is given as (As per IS 6403):

$$q_d = C N_c S_c d_c + q (N_q - 1) S_q d_q + 0.5 B \gamma N_\gamma S_\gamma d_\gamma W'$$

Where,

N_c , N_q and N_γ are bearing capacity factors,

S_c , S_q and S_γ are shape factors,

d_c , d_q and d_γ are depth factors,

And

C = Cohesion

q = Effective surcharge at the base level of foundation,

B = Width of foundation,

γ = Bulk Density below foundation.

W' = Correction factor for location of water table.

Considering General Shear:

Relative density = 70.00 %. Corresponding $\Phi = 39^\circ$

Cohesion, C =	0.00	kg/cm^2			
Friction Angle, Φ =	39	$^\circ$			
Depth of Foundation, D_f =	3.50	m			
Width of Foundation, B =	10.00	m			
Submerged Density, γ =	0.90	t/m^3			
Water table correction =	0.50				
L/B ratio =	1				
FOS =	2.50				
N_c =	69.47		N_q =	58.02	N_γ = 97.13
D_c =	1.15		D_q =	1.07	D_γ = 1.07
S_c =	1.30		S_q =	1.20	S_γ = 0.80
Computed Net Ultimate Bearing Capacity =		617.12	t/m^2		
So, we have Net Safe Bearing Capacity =		246.85	t/m^2		

Considering Local Shear:

Relative density = 20.00 %.Corresponding $\Phi = 31^\circ$

Using $\Phi = \tan^{-1}\{(2/3) \times \tan 31\} = 22^\circ$,

Cohesion, C =	0.00	kg/cm ²			
Friction Angle, $\Phi =$	22	°			
Depth of Foundation, $D_f =$	3.50	m			
Width of Foundation., B =	10.00	m			
Submerged Density, $\gamma =$	0.90	t/m ³			
Water table correction =	0.50				
L/B ratio =	1				
FOS =	2.50				
$N_c =$	17.19		$N_q =$	8.10	$N_\gamma =$ 7.59
$D_c =$	1.10		$D_q =$	1.05	$D_\gamma =$ 1.05
$S_c =$	1.30		$S_q =$	1.20	$S_\gamma =$ 0.80
Computed Net Ultimate Bearing Capacity =		57.77	t/m ²		
So, we have Net Safe Bearing Capacity =		23.11	t/m ²		

Interpolated net safe bearing capacity = $23.11 + \{[(246.85 - 23.11)/(39 - 31)] \times (33 - 31)\}$
= 79.04 t/m².

The above bearing capacity should be checked against settlement criteria.

SETTLEMENT CALCULATION

At 10 t/m² pressure, settlement for foundation of 10.00 m width founding on sandy soil with design N = 35 is 8.00 mm. [Ref: Fig-9 of IS: 8009 (Part-I)]

Water table correction = 0.50. (Considering ground water level at ground level).

So the corrected settlement = $8.00 / 0.50 = 16.00$ mm.

So for 79.04 t/m², total settlement = $S_t = (79.04/10) \times 16 = 126.46$ mm.

Applying correction for depth factor,

Corrected $S_t = 126.46 \times 0.90 = \underline{113.81 \text{ mm.}}$

$D/(L \times B)^{0.5} = 0.35$, Corresponding depth factor for L/B ratio = 1.00 is 0.90.

[Ref: Fig 12 of IS 8009 (Part I)]

Considering permissible settlement 75 mm, **allowable Bearing Capacity is 45.00 t/m**

Allowable bearing capacity value on improved soil at borehole location is as follows:

Location	Foundation size (m x m)	Depth of foundation below EGL (m)	Net Ultimate bearing capacity (t/m ²)	Net Safe bearing capacity (t/m ²)	Anticipated Settlement (mm)	Allowable bearing capacity (t/m ²)
BH-P1	10 x 10	3.50	197.63	79.05	113.81	45.00
		4.00	210.81	84.32	118.72	48.00
		4.50	224.22	89.69	123.41	50.00

Note: 1. EGL means Existing Ground Level. 2. Permissible Settlement 75 mm for isolated footing.

[Refer IS 1904, Table-1]

10.0 BORE LOG DATA SHEET:

BORE LOG DATA SHEET																			
Bore Hole No. :- BH-3 (P1)					Borehole Diameter :- 150 mm Drill Hole:- NX.			Field Tests		Nos	Samples	Nos							
Location :- 142+295					Northing :- -			S		21	U	-							
					Easting :- 556300.213														
					R.L. Value :- 2871128.888			Vane		-	W	1							
					Termination Depth (m):- 30.05			C		-	D	-							
Water Level : 2.00 m below GL																			
Depth (m.)	Penetration			N value	Strata Description	Symbol	Thickness (m)	Samples			TCR (%)	RQD (%)	Remarks						
	15 cm	15 cm	15 cm					Type	No.	Depth (m.)									
0.00	2	3	5	<u>8</u>	LOOSE ,CLAY SILTY SAND. WITH GRAVAL		4.50	S	1	0.50-0.95									
	4	5	7	<u>12</u>				S	2	1.50-1.95									
	6	8	11	<u>19</u>				S	3	3.00-3.15									
4.50																			
4.50	9	14	20	<u>34</u>	Dense to very dense, light grey, silty fine to medium sand with mica and gravels.			S	4	4.50-4.55									
	12	20	30	<u>50</u>				S	5	6.00-6.45									
	15	21	32	<u>53</u>				S	6	7.50-7.95									
	17	25	36	<u>61</u>				S	7	9.00-9.45									
	20	31	42	<u>73</u>				S	8	10.50-10.95									
	25	32	44	<u>76</u>				S	9	12.00-12.45									
	26	40	41	<u>81</u>				S	10	13.50-13.95									
	29	39	52	<u>91</u>				S	11	15.00-15.45									
	36	44	44	<u>88</u>				S	12	16.50-16.95									
	35	43	51	<u>94</u>				S	13	18.00-18.45									
	36	40	52	<u>92</u>				S	14	19.50-19.95									
	32	44	51	<u>95</u>				S	15	21.00-21.45									
	39	43	50	<u>93</u>				S	16	22.50-22.95									
	40	41	55	<u>96</u>				S	17	24.00-24.45									
	32	42	53	<u>95</u>				S	18	25.50-25.95									
	36	46	51	<u>97</u>				S	19	27.00-27.45									
	42	44	56	<u>100</u>				S	20	28.50-28.95									
30.05	5 cm pent. for 69 blows			<u>>100</u>										S	21	30.00-30.05			
" S " - 'SPT'		" U " - 'UNDISTURBED SAMPLE'		" D " - 'DISTURBED SAMPLE'				" C " - 'CORE'		" W " - 'WATER SAMPLE'				" TCR " - 'TOTAL CORE RECOVERY'		" RQD " - 'ROCK QUALITY DESIGNATION'			