

राष्ट्रीय राजमार्ग एवं अवसंरचना विकास निगम लिमिटेड

NATIONAL HIGHWAYS & INFRASTRUCTURE DEVELOPMENT CORPORATION LTD.

FINAL DETAILED PROJECT REPORT

APRIL 2020

CONSULTANCY SERVICES FOR PREPERATION OF DETAILED PROJECT REPORT AND PROVIDING PRE-CONSTRUCTION ACTIVITIES IN RESPECT OF THE FOLLOWING STRETCH ON NH-244 (OLD NH-1B) IN THE STATE OF JAMMU AND KASHMIR.

- (1) SUDHMAHADEV- DRANGA TUNNEL OF APPROX. LENGTH 4.5 KM AND ITS APPROACH ROAD ON CHENANI - SUDHMAHADEV-GOHA ROAD PORTION.
- (2) VAILOO TUNNEL OF APPROX. LENGTH 10.0 KM UNDER SINTHAN PASS AND ITS APPROACH ROAD ON GOHA-KHELLANI- KHANABAL ROAD PORTION.
- (3) ROAD PORTION FROM 82.675 TO 82.925 AT KM 83 ON BATOTE-KISHTWAR ROAD SECTION OF NH-244.
- (4) EXTENDED ROAD SECTION FROM GOHA TO KHELLANI OF 30 KM LENGTH



GOHA - KHELLANI ROAD

PACKAGE-IA (KM 12.850 to KM 20.300)

VOLUME - II - DESIGN REPORT (STRUCTURE)

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DESIGN INDEX

Consultancy Services for Preparation of Detailed Project Report and Providing Pre-Construction activities in Respect of the Following Stretches on NH-244 (Old NH-1B) in the State of Jammu & Kashmir (i) Sudhmahadev-Dranga Tunnel of approx. length 4.5 Km and its Approach Road on Chenani-Sudhmahadev-Goha road Portion. (ii) Vailoo Tunnel of Approx. length 10.00 Km under Sinthan Pass and Its approach Roads on Goha-Khellani-Khanabal Road Portion & the Extended Road Section from Goha Side approach of Sudhmahadev Dranaga Tunnel Upto Khellani Town

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DESIGN OF ABUTMENT(A1) WITH
OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
13+025

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 0 m
Design speed of vehicle = 0 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 18.750m	18.750m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side) =	0.600m	0.600m
Slab projection Beyond C/L of Bearing (Span Side) =	0.600m	0.600m
Span -c/c of E.J.	= 20.000m	20.00m
Type of Superstructure	= Precast RCC Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 11.500m	
Projection beyond crash barrier	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 0.000m	
Railing/kerb on footpath edge	= 0.000m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 4
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 350 kN/m² (as per geotechnical report)
SBC of soil-Normal Case = 449 kN/m²
SBC of soil-Seismic Case = 561 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1 Normal Case
= 1 Seismic Case
Permissible FOS against Overturning = 1 Normal Case
= 1 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 0.900m	
Height of Trapering portion	= 1.193m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 1.950m	1.950m
Width of Abutmentcap at bottom of Tapering Portion	= 1.000m	1.000m
Projection of Abutment Cap (Span Side)	= 0.500m	0.500m
Projection of Abutment Cap Back Side	= 0.450m	0.450m
Abutmentcap thickness (Uniform portion)	= 0.300m	

Abutment cap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	0.974m	0.974m
Thickness of Abutment shaft at Bottom	=	1.200m	1.200m

Solid Return Wall

Length of Return wall	=	5.800m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	10.500m
abutment pedestal width	=	1.200m
abutment pedestal Height	=	0.000m
Width of Toe Slab	=	3.500m
Width of Heel Slab	=	5.800m
Thickness of Toe slab at tip	=	0.500m
Thickness of Toe slab near shaft	=	1.400m
Thickness of heel slab at tip	=	0.500m
Thickness of heel slab near shaft	=	1.400m
Width of backfill on heel slab	=	5.800m
Thickness of heel slab at back fill edge	=	1.400m
Height of back fill at bottom edge of heel slab	=	11.134m
Height of back fill at back fill edge of heel slab	=	10.234m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1397.701m	Cross Slope (Bi-directional)	=	2.500%	
Deck level at Outer Edge =	1397.557m	Height of Superstructure	=	1.600m	
Deck level at center line =	1397.701m	Min. Height of Footpath Side Pedestal (1)	=	0.300m	
Soffit Level at center of bridge =	1396.036m	Height of Pedestal (2)	=	0.375m	
Abutment cap top level =	1395.536m	Height of Pedestal (3)	=	0.450m	
Abutment cap bottom lvl (uniform portion ends)	1395.236m	Height of Pedestal (4)	=	0.525m	
Abutment cap bottom lvl (corbel portion ends)	1394.936m	Height of Pedestal (5)	=	0.000m	
Abutment shaft top level =	1394.936m	Height of Pedestal (6)	=	0.000m	
Ground level/LBL =	1395.905m	Distance of nearest girder to c.l. of deck	=	1.500m	
Abutment shaft bottom level =	1387.395m	Height (Avg.) of Dirt Wall	=	2.093m	
Foundation level =	1385.995m	Abutment shaft Above G.L	=	0.000m	
HFL	=	1395.905m	Abutment Shaft below G.L	=	8.510m
			Height of abutment shaft	=	7.541m
			MSL	=	1395.905m
			Wedge over girder flange	=	0.0250m

Material Specification

Concrete Grade	=	M 35
Characteristic compressive strength of concrete, f _{ck}	=	35.00 Mpa at 28 days
Design Compressive strength of Concrete, f _{cd}	=	15.63 Mpa at 28 (0.67/1.5 * f _{ck})
Tensile strength of concrete , f _{ctm}	=	2.77 MPa

Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E _{cm}	=	32308.250	N/mm ²
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f _y or f _{yk}	=	500	Mpa
Design Yield Strength of Reinforcement, f _{yd}	=	434.78	Mpa (1/1.15 * f _y)
Modulus of Elasticity of Steel (E _s)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48	f _{ck}
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm
<u>Seismic Data:</u>			
Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction
<u>Properties of backfill material :</u>			
c	=	0	
φ	=	35	
θ	=	90	
β	=	0	
δ	=	22.5	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.000m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

		eT
DL Reaction on Bearing No.-1	=	450.94 4.50
DL Reaction on Bearing No.-2	=	432.46 1.50
DL Reaction on Bearing No.-3	=	432.46 -1.50
DL Reaction on Bearing No.-4	=	450.94 -4.50
DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		1766.80 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	72.000 4.50
SIDL Reaction on Bearing No.-2	=	8.000 1.50
SIDL Reaction on Bearing No.-3	=	8.000 -1.50
SIDL Reaction on Bearing No.-4	=	72.000 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		160.00 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	60.954 4.50
SIDL Reaction on Bearing No.-2	=	66.495 1.50
SIDL Reaction on Bearing No.-3	=	66.495 -1.50
SIDL Reaction on Bearing No.-4	=	60.954 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		254.90 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 2-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	981.82	KN	Corr. Transv	=	932.730 kNm	ecc.	0.95
min	=	368.18	KN	Corr. Transv	=	349.770 kNm	ecc.	0.95

SV Loading

max	=	2003.04	KN	Corr. Transv	=	600.912 kNm	ecc.	0.30
min	=	1416.96	KN	Corr. Transv	=	425.088 kNm	ecc.	0.30

**MAXIMUM TRASVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	758.73	KN	Corr. Transv	=	2393.795 kNm	ecc.	3.16
min	=	241.27	KN	Corr. Transv	=	761.205 kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.300 x	0.800 x	0.800 =	0.1920 m ³	4.500
Pedestal 2	=	0.375 x	0.800 x	0.800 =	0.2400 m ³	1.500
Pedestal 3	=	0.450 x	0.800 x	0.800 =	0.2880 m ³	-1.500
Pedestal 4	=	0.525 x	0.800 x	0.800 =	0.3360 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					1.0560 m ³	-0.682

VOLUME CALCULATION

C.G. Of Footing = 5.250 m
 C.G. Of shaft from toe tip = 4.100 m
 Distance between c.g. of shaft and footing = 1.150 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	0.900	3.375	-0.800	0.350	-4.900	0.000
-Trapering portion	1	12.50	0.300	1.193	4.474	-0.800	0.350	-4.900	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.100	0.050	-5.200	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-1.050	0.100	-5.150	0.000
Cap (uniform portion)	1	12.50	1.950	0.300	7.313	0.025	1.175	-4.075	0.000
Cap (Corbel Portion)	1	12.50	1.950	0.300	5.433	0.025	1.175	-4.075	0.000
		12.50	1.000						
Shaft above HFL	1	12.50	0.987	0.000	0.000	0.106	1.256	-3.994	0.000
Shaft below HFL	1	12.50	1.087	7.541	102.478	0.054	1.204	-4.046	0.000
Solid Return Wall	1	5.80	0.500	10.756	31.192	-3.500	-2.350	-7.600	6.000
Cantilever Return wall(Rectangular portion)	1	0.00	0.500	0.000	0.000	-0.600	0.550	-4.700	6.000
Cantilever Return wall(Traingular portion)	1	0.00	0.500	0.000	0.000	-0.600	0.550	-4.700	6.000
Footing									
Heel Slab	1	12.50	5.800	0.950	68.875		-1.892	-7.142	0.000
Toe Slab	1	12.50	3.500	0.950	41.563		3.224	-2.026	0.000
Portion between Heel and Toe	1	12.50	1.200	1.400	21.000		1.150	-4.100	0.000
Back filling above HFL over Heel Slab	1	12.50	5.800	1.796	130.210		-2.350	-7.600	0.000
Back filling below HFL over Heel Slab	1	12.50	5.800	8.960	649.600		-2.399	-7.649	0.000
Backfill above Heel slab	1	12.50	5.800	10.684	774.599		-2.391	-7.641	0.000
Front Filling over Toe Slab	1	12.50	3.500	8.960	392.000		3.471	-1.779	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	3.500	8.960	392.000		2.321	-2.929	0.000
Side filling between heel and toe	1	0.00	1.200	8.960	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-1.100	0.050	-5.200	0.000
Back fill above HFL on flared portion of stem	1	12.50	-0.026	1.796	-0.577		0.657	-4.593	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.226	8.960	25.278		0.625	-4.625	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barr	2	8 kN/m	1.750	28.00kN		-0.800	0.350	-4.900

SECTIONAL PROPERTIES

Width of Footing (B) = 10.500 m
 Length of Footing (L) = 12.500 m

A = 10.500 x 12.500 = 131.250 m²
 ZL = 12.500 x 18.375 = 229.688 m³
 ZT = IT1 + IT2

distance of extreme point from centre

IT1 = 10.500 x 162.760 = 1708.98 m⁴
 IT2 (moment of inertia of triangle) = 10.500 x 0.000 + 0.500 x 10.500 x 0.000 x 39.063
 from centre of footing = 0.000 m⁴

Moment of inertia of two triangle = 0.000 m⁴

Total moment of inertia = 1708.98 m⁴

Distance of extreme point from centre of footing = 6.250 + 0.000 = 6.250 m

Total Section modulus (ZT) = 273.438 m³

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.000	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.500
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Servicibility Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.00	1.00	1.00
wearing course	1.20	1.20	1.20
Back Filling Weight	1.00	1.00	1.00
Shrinkage Creep Effect	1.00	1.00	1.00
Earth Pressure due to back filling	1.00	1.00	1.00
CWLL and Associate load and FPLL	1.00	0.75	0.00
Live Load Surcharge	0.80	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.500
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

Horizontal Seismic Force For Zone

4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.)
or horizontal vibrations.

$$= 2.0 \frac{D}{1000F}^{1/2}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 9.841 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 11.284 m

Abutment Cap Top Level - Foundation Level

= 9.541 m

Dimensions of Abutment Shaft

Length = 12.50 m

Width = 1.10 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 1.040 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 179.036 m^4

E_{cm} = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 110.817 KN
 D = 2181.70 KN
 T = 0.2806 sec

Transverse Direction

Force = 15681.864 KN
 D = 2451.697 KN
 T = 0.0250 sec

For Hard or Rocky Strata, with $N > 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.375

For Medium Strata, with $N > 10 \& N < 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.375

For Soft Soil Strata, with $N < 10$

$\frac{S_a}{g}$ = 2.5

$\frac{S_a}{g}$ = 1.375

Medium Soil Strata

S_a/g = 2.5

S_a/g = 1.375

Seismic Coeff. In Longitudinal Direction = 0.12

Seismic Coeff. In Transverse Direction = 0.198

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.198	In Transverse direction
Av	=	0.132	In Vertical direction

For Design of Foundation

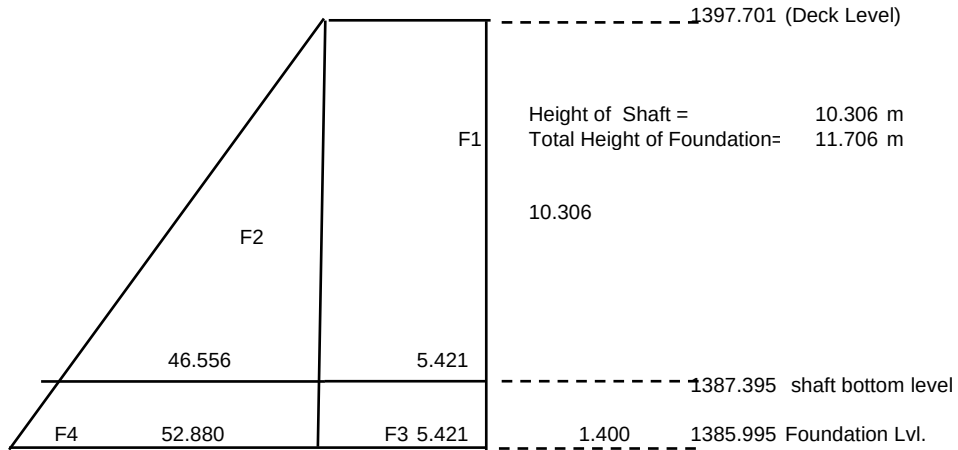
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.162	In Longitudinal direction
Ah	=	0.198	In Transverse direction
Av	=	0.132	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = 1.2 m
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1387.395 m (at Shaft Base)			
@ RL					1385.995 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	10.306	x	12.500	=	698.335 kN
M1	=	698.34	x	5.15	=	3598.523 kN.m		at Shaft Bottom
F3	=	5.421	x	11.706	x	12.500	=	793.200 kN
M3	=	793.200	x	5.853	=	4642.597 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	10.306	= 46.556 kN/m ²
F2	=	0.5	x	46.56	x	10.306	x 12.50
	=	2998.769 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	2998.77	x	4.33	=	12980.231 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	11.706	= 52.880 kN/m ²
F4	=	0.5	x	52.88	x	11.706	x 12.50
	=	3868.831 kN					
M4	=	3868.83	x	4.92	=	19021.185 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m ³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	10.306	=	49.469 kN/m ²	
F	=	0.5	x	49.469	x	10.306	x 12.500
	=	3186.409 kN					

$$M = 3186.41 \times 3.435 = 10946.377 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 11.706 = 56.19 \text{ kN/m}^2$$

$$F = 0.5 \times 56.189 \times 11.71 \times 12.50 = 4110.913 \text{ kN}$$

$$M = 4110.91 \times 3.902 = 16040.783 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50$$

$$= 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

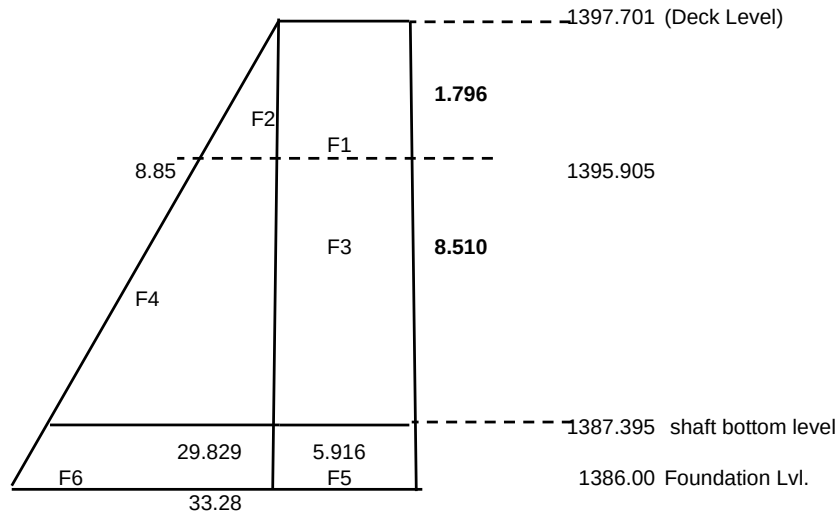
$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	12980.231	19021.185	2998.769	3868.831
Due to Minimum Fluid Pressure	10946.377	16040.783	3186.409	4110.913
Governing of Two	12980.231	19021.185	3186.409	4110.913
Due to Live Load Surcharge	3598.523	4642.597	698.335	793.200
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	11.3 degree	0.196 radians
	Ka'	=	0.246 active component	
	Kp'	=	5.249 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	
Assuming				

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1387.4 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m ²
F1	=	5.916	x	10.306	x	12.500	=	762.068 kN
M1	=	762.07	x	5.15	=	3926.935 kN.m		at Shaft Bottom
F3	=	5.916	x	11.706	x	12.500	=	865.589 kN
M3	=	865.59	x	5.85	=	5066.295 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	1.796	=	8.854 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	8.510	=	20.975 kN/m ²
F2	=	0.5	x	8.85	x	1.796	x	12.50
	=	99.381 kN						
F4	=	$\left(\frac{8.85 + 29.83}{2} \right)$	x		x	8.51	x	12.50
	=	2057.431 kN						
Total Force	=	2156.813 kN						
M2	=	99.38	x	9.26	=	920.700 kN.m		
M4	=	2057.43	x	3.49	=	7172.031 kN.m		
Total Moment	=	8092.73 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 1.796 = 8.854 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 9.910 = 24.426 \text{ kN/m}^2$$

$$F2 = 0.5 \times 8.85 \times 1.796 \times 12.50 = 99.381 \text{ kN}$$

$$F6 = \left(\frac{8.85 + 33.28}{2} \right) \times 9.91 \times 12.50 = 2609.633 \text{ kN}$$

$$\text{Total Force} = 2709.015 \text{ kN}$$

$$M2 = 99.38 \times 10.66 = 1059.834 \text{ kN.m}$$

$$M6 = 2609.63 \times 4.00 = 10431.932 \text{ kN.m}$$

$$\text{Total Moment} = 11491.77 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

$$= 5.249 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	8092.732	11491.766	2156.813	2709.015
Due to Minimum Fluid Pressure	10946.377	16040.783	3186.409	4110.913
Governing of Two	10946.377	16040.783	3186.409	4110.913
Due to Live Load Surcharge	3926.935	5066.295	762.068	865.589
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

αh	=	0.120	
αv	=	0.132	
ϕ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

αh	=	HORIZONTAL SEISMIC COEFFICIENT
αv	=	VERTICAL SEISMIC COEFFICIENT
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

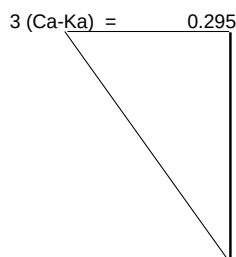
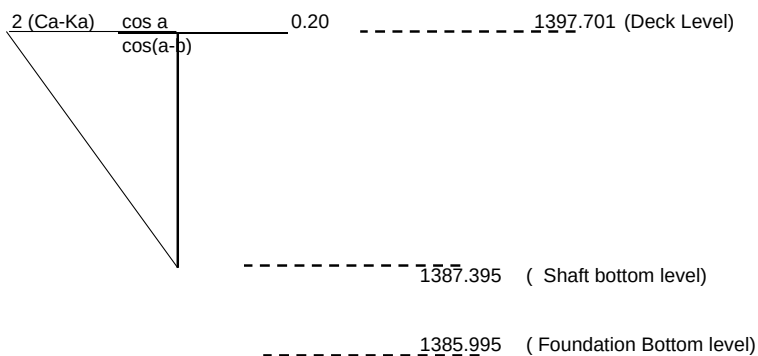
$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.106 \\ 0.137 \end{matrix}$$

$$C_a = \begin{matrix} 1 \\ 0.324 \end{matrix} \begin{matrix} 2 \\ 0.267 \end{matrix}$$

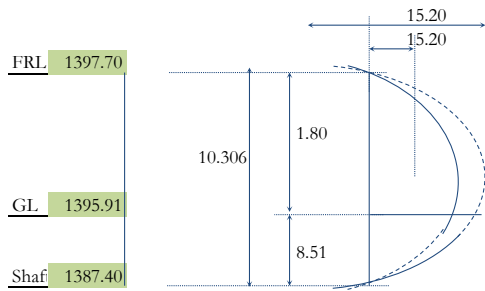
Ca	=	0.324	
Ka	=	0.226	
Dynamic Increment	=	0.324	-0.226 0.098

3 Earth Pressure : DRY CASE (Seismic case)

Equivalent Live Load Surcharge height	=	1.2 m
Assuming	γ_{dry}	= 20 kN/m ³
	γ_{water}	= 10.00 kN/m ³

Active Earth Pressure**Surcharge Earth Pressure****Earth Pressure Diagram for Dynamic Increment****Due to Dynamic Live Load Surcharge**

	=	0.197	x	20.00	x	1.2	=	4.718 kN/m ²
at Shaft Bottom Level								
E1	=	0.50	x	4.718	x	10.306	x	12.500
	=	303.913	kN					
M1	=	303.913	x	6.905			=	2098.527 kN.m
at Foundation Bottom Level								
E2	=	0.50	x	4.718	x	10.306	x	12.500
	=	303.913	kN					
M2	=	303.913	x	8.305			=	2524.005 kN.m

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	15.20 t/m ²
h	=	10.306 m
y	=	-3.357 m
L	=	12.500 m

Parabola below Water Level

p_mid_height	=	15.20 t/m ²
h	=	10.306 m
y	=	3.357 m
L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	105.1	9.1
Parabola below Water Level	1200.0	4.8
Total Dynamic Earth Pressure	1305.1	5.2

$$E3 = 1305.054 \text{ kN}$$

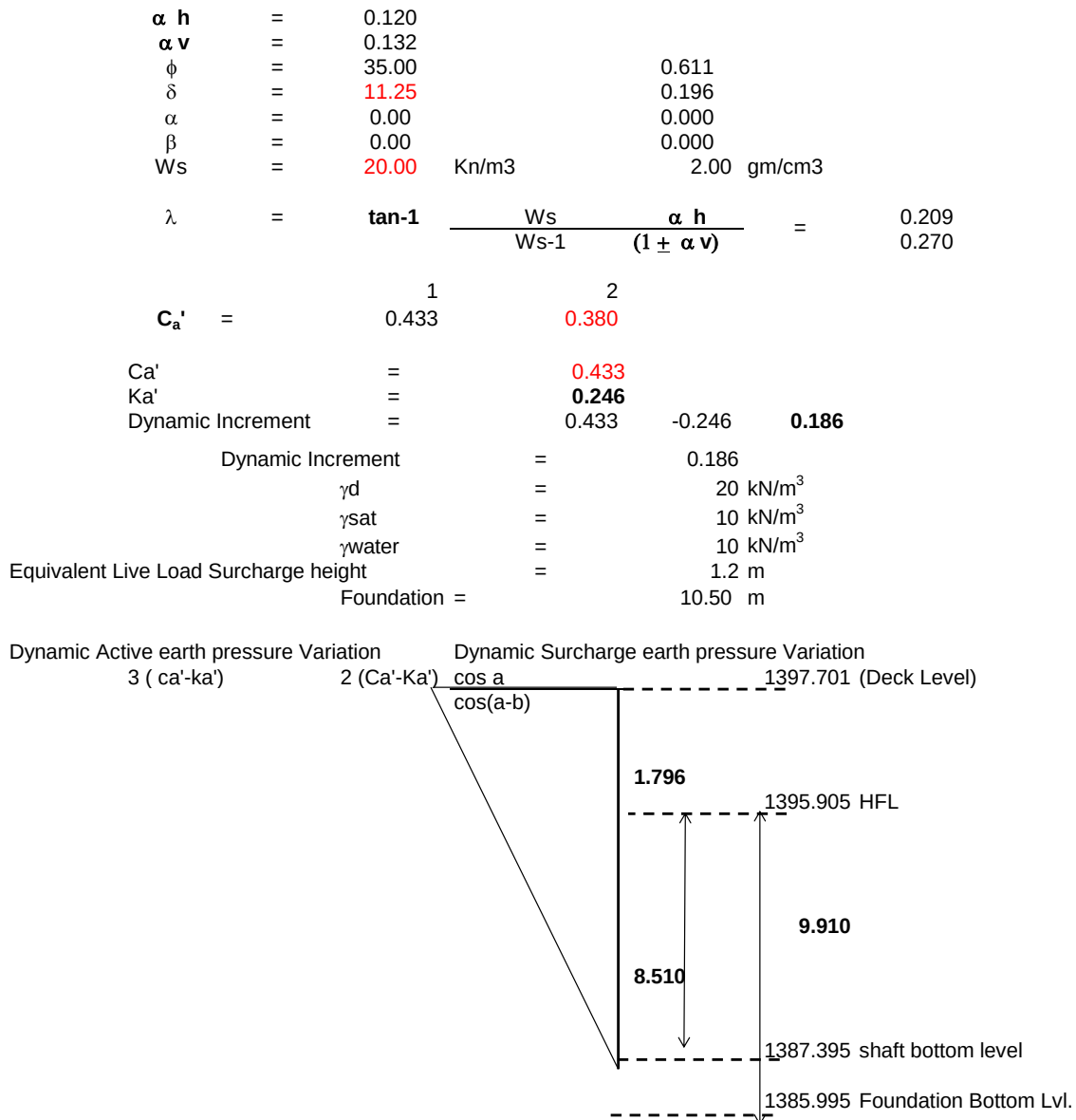
$$E4 = 1305.054 \text{ kN}$$

$$M3 = 1305.05 \times 5.15 = 6724.943 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 1305.05 \times 6.55 = 8552.019 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	12980.231	19021.185	2998.769	3868.831
Due to active Earth Pressure (dynamic Increment)	6724.943	8552.019	1305.054	1305.054
Total Earth Pressure	19705.174	27573.203	4303.823	5173.885
Due to Minimum Fluid Pressure	10946.377	16040.783	3186.409	4110.913
Governing of Two	12980.231	27573.203	4303.823	5173.885
Due to Live Load Surcharge (Static)	3598.523	4642.597	698.335	793.200
Due to Live Load Surcharge(Dynamic)	2098.527	2524.005	303.913	303.913
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case**Earth Pressure Diagram**

h'	=	8.510
h	=	10.306

Dynamic active earth pressure

$3 (ca'-ka')$	=	0.295 at deck level
$3 (ca'-ka')$	=	0.559 at deck level
$3 (ca'-ka')h'/h$	=	0.462 at HFL level

Dynamic Surcharge earth pressure

$2 (Ca'-Ka') \frac{\cos a}{\cos(a-b)}$	=	0.373 at deck level	Pressure	=	8.94 kN/m ²
	=	0.308 at HFL level	Pressure	=	7.39 kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	4.718 kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	7.39 kN/m ²

at Shaft Bottom Level

F1	=	6.052	x	1.796	x	12.500	=	135.871 kN
M1	=	135.87	x	9.41	=	1278.271 kN.m		
F2	=	0.50	x	7.386	x	8.510	x	12.500
	=	392.846 kN						

$$M2 = 392.85 \times 5.70 = 2239.890 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 528.717 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 3518.162 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

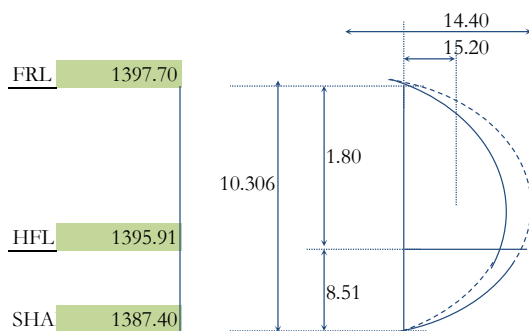
$$F3 = 6.052 \times 1.796 \times 12.500 = 135.871 \text{ kN}$$

$$M3 = 135.87 \times 10.81 = 1468.490 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 7.386 \times 8.510 \times 12.500 \\ &= 392.846 \text{ kN} \end{aligned}$$

$$M4 = 392.85 \times 7.10 = 2789.875 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 528.717 \text{ kN} \\ \text{Total Moment at Founding level} &= 4258.365 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 15.20 \text{ t/m}^2 \\ h &= 10.306 \text{ m} \\ y &= -3.357 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 14.40 \text{ t/m}^2 \\ h &= 10.306 \text{ m} \\ y &= 3.357 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	105.1	9.1
Parabola below Water Level	1137.5	4.8
Total Dynamic Earth Pressure	1242.5	5.2

Total Force (F1 + F2)	=	1242.539 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	1242.539 kN	at Foundation Bottom Level

$$M1 = 1242.54 \times 5.17 = 6424.569 \text{ kN.m}$$

$$\text{Total Mome} = 6424.569 \text{ kN.m} \quad \text{at Shaft Bottom}$$

$$M1 = 1242.54 \times 6.57 = 8164.125 \text{ kN.m}$$

$$\text{Total Mome} = 8164.125 \text{ kN.m} \quad \text{at Foundation Bottom Level}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Bottom	At Shaft Bottom Lvl	At Foundation Bottom Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure(Static)	8092.732	11491.766	2156.813	2709.015
Due to active Earth Pressure (Dynamic Increment)	6424.569	8164.125	1242.539	1242.539
Total Earth Pressure	14517.301	19655.891	3399.352	3951.554
Due to Minimum Fluid Pressure	10946.377	16040.783	3186.409	4110.913
Governing of Two	14517.301	19655.891	3399.352	4110.913
Due to Live Load Surcharge(Static)	3926.935	5066.295	762.068	865.589
Due to Live Load Surcharge (Dynamic Increment)	3518.162	4258.365	528.717	528.717
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - SPHERICAL BEARING

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1766.80	1.35	1.35	2385.18	2385.18	
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00	
WC	=	254.90	1.75	1.75	446.07	446.07	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax- Reaction case	=	981.82	1.5	0.20	1472.73	196.36	(1-70RW + 2-CLASS A)
CWLLmax- Reaction case	=	2003.04	1	0.20	2003.04	400.61	SV Loading
CWLLmin	=	368.18	1.5	0.20	552.27	73.64	(1-70RW + 2-CLASS A)
CWLLmin	=	1416.96	1	0.20	1416.96	283.39	SV Loading
CWLLmax- Transv. Moment Case		758.73	1.5	0.20	1138.10	151.75	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 1108 = 255.4 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3047.25	0	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.1	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.1	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.1	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	785.41	152.362	545.068	633.048	785.411	
DL+SIDL+LL-Max Reaction case		3120.89	836.49	156.044	574.290	680.447	836.491	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	836.49	162.181	580.426	674.310	836.491	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3199.00	836.49	159.950	578.195	676.541	836.491	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	235.623	152.362	270.174	83.261	270.174	
DL+SIDL+LL-Max Reaction case		3120.89	286.703	156.044	299.396	130.659	299.396	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	286.703	162.181	305.532	124.523	305.532	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3199.00	286.703	159.950	303.301	126.753	303.301

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.35	1.35	2385.18	2385.18
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	254.90	1.75	1.75	446.07	446.07
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	981.82	1.5	0.75	1472.73	736.37
CWLLmax-Transv. Moment Case	=	758.73	1.5	0.75	1138.10	569.05
CWLLmin	=	368.18	1.5	0.75	552.27	276.13

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	3047.25	0.000	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.100	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0.000	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.100	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0.000	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.100	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	1060.30	152.362	682.515	907.942	1060.305	
DL+SIDL+LL-Max Reaction case		3323.38	1111.38	166.169	721.862	945.216	1111.385	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	1111.38	189.181	744.873	922.204	1111.385	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	1111.38	180.815	736.507	930.570	1111.385	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	318.091	152.362	311.408	165.729	318.091	
DL+SIDL+LL-Max Reaction case		3323.38	369.171	166.169	350.755	203.002	369.171	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	369.171	189.181	373.766	179.991	373.766	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	369.171	180.815	365.401	188.357	369.171	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1	1.00	1766.80	1766.80
SIDL except wc	=	160.00	1	1.00	160.00	160.00
WC	=	254.90	1	1.00	254.90	254.90
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1	0.20	981.82	196.36
CWLLmax- Transv. Moment Case		758.73	1	0.20	758.73	151.75
CWLLmin	=	368.18	1	0.20	368.18	73.64

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	0.000	109.085	109.085	-109.085	109.085	
DL+SIDL+LL-Max Reaction case	2549.88	255.400	127.494	255.194	127.906	255.194	(1-70RW + 2-CLASS A)
	3598.66	0.000	179.933	179.933	-179.933	179.933	SV Loading
DL+SIDL+LL-Min Reaction case	3163.52	255.400	158.176	285.876	97.224	285.876	(1-70RW + 2-CLASS A)
	4184.74	0.000	209.237	209.237	-209.237	209.237	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2940.43	255.400	147.021	274.721	108.379	274.721	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	706.870	109.085	462.520	597.785	706.870	
DL+SIDL+LL-Max Reaction case	2255.33	757.950	112.767	491.742	645.183	757.950	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	757.950	118.903	497.878	639.047	757.950	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	757.950	116.672	495.647	641.278	757.950	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	212.061	109.085	215.115	102.976	215.115	
DL+SIDL+LL-Max Reaction case	2255.33	263.141	112.767	244.337	150.374	263.141	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	263.141	118.903	250.474	144.238	263.141	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	263.141	116.672	248.243	146.469	263.141	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.1	1.10	1943.48	1943.48
SIDL except wc	=	160.00	1.1	1.10	176.00	176.00
WC	=	254.90	1.35	1.35	344.11	344.11
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1.5	0.00	1472.73	0.00
CWLLmax- Transv. Moment Case		758.73	1.5	0.00	1138.10	0.00
CWLLmin	=	368.18	1.5	0.00	552.27	0.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2463.59	0.000	123.180	123.180	-123.180	123.180	
DL+SIDL+LL-Max Reaction case	3015.86	383.100	150.793	342.343	232.307	342.343	(1-70RW + 2- CLASS A)
	3880.55	0.000	194.028	194.028	-194.028	194.028	SV Loading
DL+SIDL+LL-Min Reaction case	3936.32	383.100	196.816	388.366	186.284	388.366	(1-70RW + 2- CLASS A)
	4466.63	0.000	223.332	223.332	-223.332	223.332	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3601.69	383.100	180.084	371.634	203.016	371.634	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	1060.30	123.180	653.332	937.125	1060.305	
DL+SIDL+LL-Max Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	1060.30	123.180	653.332	937.125	1060.305	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	318.091	123.180	282.225	194.912	318.091	
DL+SIDL+LL-Max Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	318.091	123.180	282.225	194.912	318.091	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1766.80	1	1	1	1766.80	1766.80	1766.80
SIDL except wc	=	160.00	1	1	1	160.00	160.00	160.00
WC	=	254.90	1.20	1.20	1.20	305.88	305.88	305.88
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	981.82	1	0.75	0	981.82	736.37	0.00
CWLLmax-Transv. Moment Case	=	758.73	1	0.75	0	758.73	569.05	0.00
CWLLmin	=	368.18	1	0.75	0	368.18	276.13	0.00

Braking Force = 255.4 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2600.86	255.400	130.043	257.743	125.357	257.743	(1-70RW + 2-CLASS A)
	3649.64	0.000	182.482	182.482	-182.482	182.482	SV Loading
DL+SIDL+LL-Min Reaction case	3214.50	255.400	160.725	288.425	94.675	288.425	(1-70RW + 2-CLASS A)
	4235.72	0.000	211.786	211.786	-211.786	211.786	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2991.41	255.400	149.570	277.270	105.830	277.270	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2508.81	191.550	125.441	221.216	66.109	221.216	Dry Case
DL+SIDL+LL-Min Reaction case	2969.04	191.550	148.452	244.227	43.098	244.227	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2801.72	191.550	140.086	235.861	51.464	235.861	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	Dry Case
DL+SIDL+LL-Min Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2232.68	0.000	111.634	111.634	-111.634	111.634	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	0.00	kmph	0.00	kmph
Live Load	W	=	981.82	kN	981.82	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

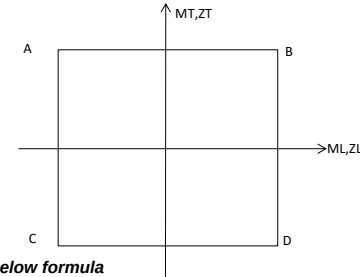
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1385.995 m

Properties of Footing Base:

A	=	131.250	m ²
ZL	=	229.688	m ³
ZT	=	273.438	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1766.799	1.150	2031.819	0.000	0.000
SIDL except Wearing Course	1			160.000	1.150	184.000	0.000	0.000
Wearing Course	1			254.898	1.150	293.132	0.000	0.000
Bearing Pedestal	1	25	1.056	26.400	1.150	30.360	-0.682	-18.000
				2208.097		2539.312		-18.000
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	3.375	84.375	0.350	29.531	0.000	0.000
Dirt Wall-Tapered portion	1	25	4.474	111.855	0.350	39.149	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	0.050	1.406	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	0.100	1.406	0.000	0.000
Cap - (uniform portion)	1	25	7.313	182.813	1.175	214.805	0.000	0.000
Cap - (corbel portion)	1	25	5.433	135.826	1.175	159.595	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.550	0.000	6.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.550	0.000	6.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.350	9.800	0.000	0.000
Approach Slab	1	25	6.563	164.063	0.050	8.203	0.000	0.000
				749.119		463.896		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	31.192	779.810	-2.350	-1832.554	6.000	4678.860
Abutment Shaft	1	25	102.478	2561.938	1.204	3085.783	0.000	0.000
Back filling over heel slab	1	20	774.599	15491.981	-2.391	-37036.906	0.000	0.000
Front Filling over toe slab	1	20	392.000	7840.000	3.471	27210.313	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	68.875	1721.875	-1.892	-3257.969	0.000	0.000
Toe slab	1	25	41.563	1039.063	3.224	3349.609	0.000	0.000
portion between heel & toe	1	25	21.000	525.000	1.150	603.750	0.000	0.000
Vertical Components of active earth pressure	1			1702.796	-5.250	-8939.679	0.000	0.000
				32156.490		-16501.032		4678.860
Total				35113.706		-13497.825		4660.860

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.900	1590.120	-4.100	-6519.490
0.900	144.000	-4.100	-590.400
1.000	254.898	-4.100	-1045.080
0.900	23.760	-4.100	-97.416
	2012.777		-8252.386
0.900	75.938	-4.900	-372.094
0.900	100.670	-4.900	-493.283
0.900	25.313	-5.200	-131.625
0.900	12.656	-5.150	-65.180
0.900	164.531	-4.075	-670.465
0.900	122.243	-4.075	-498.141
0.900	0.000	-4.700	0.000
0.900	0.000	-4.700	0.000
0.900	25.200	-4.900	-123.480
0.900	147.656	-5.200	-767.813
	674.207		-3122.079
0.900	701.829	-7.600	-5333.900
0.900	2305.744	-4.046	-9327.950
0.900	13942.783	-7.641	-106532.827
0.900	7056.000	-1.779	-12554.719
0.900	0.000	0.000	0.000
0.900	1549.688	-7.142	-11068.031
0.900	935.156	-2.026	-1894.922
0.900	472.500	-4.100	-1937.250
0.900	1532.516	-10.500	-16091.422
	28940.841		-166783.382
	31627.825		-178157.847

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1396.036	1095.321
due to Earth pressure	1	4110.913		19021.185

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1396.036	1236.846
due to Earth pressure	1.5	6166.370		28531.777
		6289.549		29768.623

Summary of Forces For SBC

P	35113.706	kN
ML	6618.681	kNm
MT	4660.860	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		35113.706		-13497.825		4660.860
CWLL-Max. Reaction case	1	981.821	1.150	1129.094	0.950	932.730
Vertical Components of LL Surcharge	1	328.554	-5.250	-1724.909	0.000	0.000
Total		36424.081		-14093.639		5593.590

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	255.194	1396.036	2562.401
due to Earth pressure	1	4110.913		19021.185
due to Live load surcharge	1	793.200		4642.597

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		342.343	1396.036	3437.466
due to Earth pressure	1.5	6166.370		28531.777
due to Live load surcharge	1.2	951.839		5571.117
		7460.552		37540.359

Summary of Forces For SBC

P	36424.081	kN
ML	12132.543	kNm
MT	5593.590	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	1095.32	0.00	0.00
4110.91	19021.18	0.00	0.00
4219.998	20116.506	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	1236.85	0.00	0.00
6166.37	28531.78	0.00	0.00
6289.549	29768.623	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	31627.8249		-178157.85
0.000		-4.100	0.00
0.900		-10.500	-3104.84
	31627.825		-181262.68

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
255.19	2562.40	0.00	0.00
4110.91	19021.18	0.00	0.00
793.200	4642.597	0.000	0.000
5159.31	26226.18	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
342.34	3437.47	0.00	0.00
6166.37	28531.78	0.00	0.00
951.84	5571.12	0.00	0.00
7460.552	37540.359	0.000	0.000

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				2208.097		2539.312		-18.000
Substructure & Foundation -Portion 1				749.119		463.896		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	31.192	779.810	-2.350	-1832.554	6.000	4678.860
Shaft above HFL	1	25	0.000	0.000	1.256	0.000	0.000	0.000
Shaft below HFL	1	15	102.478	1537.163	1.204	1851.470	0.000	0.000
Back filling above HFL over heel slab	1	20	130.210	2604.200	-2.350	-6119.870	0.000	0.000
Back filling below HFL over heel slab	1	10	649.600	6496.000	-2.399	-15580.975	0.000	0.000
Front Filling over toe slab	1	10	392.000	3920.000	2.321	9097.156	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	68.875	1033.125	-1.892	-1954.781	0.000	0.000
Toe slab	1	15	41.563	623.438	3.224	2009.766	0.000	0.000
Portion between Heel & Toe	1	15	21.000	315.000	1.150	362.250	0.000	0.000
Vertical Components of active earth pressure	1			1702.796	-5.250	-8939.679	0.000	0.000
				19252.775		-20956.744		4678.860
Total				22209.991		-17953.536		4660.860

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	2012.777		-8252.386
	674.207		-3122.079
0.900	701.829	-7.600	-5333.900
0.900	0.000	-3.994	0.000
0.900	1383.446	-4.046	-5596.770
0.900	2343.780	-7.600	-17812.728
0.900	5846.400	-7.649	-44716.478
0.900	3528.000	-2.929	-10334.559
0.900	0.000	0.000	0.000
0.900	929.813	-7.142	-6640.819
0.900	561.094	-2.026	-1136.953
0.900	283.500	-4.100	-1162.350
0.900	1532.516	-10.500	-16091.422
	17327.498		-109830.433
	20014.482		-121204.898

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1396.036	1095.321
due to Earth pressure	1	4110.913		16040.783

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	1095.32	0.00	0.00
4110.91	16040.78	0.00	0.00
4220.00	17136.10	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1396.036	1236.846
due to Earth pressure	1.5	6166.370		24061.174
		6289.549		25298.020

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	1236.85	0.00	0.00
6166.37	24061.17	0.00	0.00
6289.55	25298.02	0.00	0.00

Summary of Forces For SBC

P	22209.991	kN
ML	-817.433	kNm
MT	4660.860	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		22209.991		-17953.536		4660.860
CWLL-Min. Reaction case	1	368.179	1.150	423.406	0.950	349.770
Vertical Components of LL Surcharge	1	358.539	-5.250	-1882.329	0.000	0.000
Total		22936.709		-19412.459		5010.630

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	20014.482		-121204.898
0.000	0.000	-4.100	0
0.900	322.685	-10.500	-3388.192
	20337.167		-124593.090

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	285.876	1396.036	2870.480
due to Earth pressure	1	4110.913		16040.783
due to Live load surcharge	1	865.589		5066.295

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
285.88	2870.48	0.00	0.00
4110.91	16040.78	0.00	0.00
865.589	5066.295	0.000	0.000
5262.38	23977.56	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		388.366	1396.036	3899.584
due to Earth pressure	1.5	6166.370		24061.174
due to live load surcharge	1.2	1038.707		6079.553
		7593.443		34040.312

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
388.37	3899.58	0.00	0.00
6166.37	24061.17	0.00	0.00
1038.707	6079.553	0.000	0.000
7593.44	34040.31	0.00	0.00

Summary of Forces For SBC

P	22936.709	kN
ML	4565.098	kNm
MT	5010.630	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1

ah= 0.162 In Longitudinal direction
ah= 0.198 In Transverse direction
av= 0.132 In Vertical direction

Weight of shaft below Ground level

= 2891.140 KN

Weight of back fill below Ground level

= 12339.500 KN

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1766.799		104.956	69.971	1.150	2031.819	80.466	1397.103		0.000	0.000	1165.817
SIDL except Wearing Course	1			160.000		9.505	6.336	1.150	184.000	7.287	1398.150		0.000	0.000	115.529
Wearing Course	1			254.898		15.142	10.095	1.150	293.132	11.609	1397.701		0.000	0.000	177.253
Bearing Pedestal	1	25	1.056	26.400		1.568	1.046	1.150	30.360	1.202			-0.682	-18.000	
				2208.097		131.171	87.447		2539.312	100.565				-18.000	1458.600
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	3.375	84.375	13.669	5.012	3.342	0.350	29.531	1.170	1397.251	153.855	0.000	0.000	56.418
Dirt Wall-Tapered portion	1	25	4.474	111.855	18.121	6.645	4.430	0.350	39.149	1.550	1396.204	185.001	0.000	0.000	67.839
Bracket - Uniform portion	1	25	1.125	28.125				0.050	1.406						
Bracket - Tapered portion	1	25	0.563	14.063				0.100	1.406						
Cap - (uniform portion)	1	25	7.313	182.813	29.616	10.860	7.240	1.175	214.805	8.507	1395.386	278.120	0.000	0.000	101.985
Cap - (corbel portion)	1	25	5.433	135.826	22.004	8.069	5.379	1.175	159.595	6.320	1395.086	200.036	0.000	0.000	73.352
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1397.701	0.000	6.000	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1397.701	0.000	6.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.350	9.800				0.000	0.000	
Approach Slab	1	25	6.563	164.063				0.050	8.203				0.000	0.000	

				749.119	83.409	30.586	20.390		463.896	17.547		817.013		0.000	299.595
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	31.192	779.810	126.329	46.324	30.883	-2.350	-1832.554	-72.575	1392.359	803.951	6.000	4678.860	294.805
Abutment Shaft	1	25	102.478	2561.938	-53.331	-19.556	-13.037	1.204	3085.783	-15.703	1395.421	-502.670	0.000	0.000	-184.327
Back filling over heel slab	1	20	774.599	15491.981	0.000	0.000	0.000	-2.391	-37036.906	0.000	1392.359	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	392.000	7840.000				3.471	27210.313				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	68.875	1721.875				-1.892	-3257.969				0.000	0.000	
Toe slab	1	25	41.563	1039.063				3.224	3349.609				0.000	0.000	
portion between heel & toe	1	25	21.000	525.000				1.150	603.750				0.000	0.000	
Vertical component of active earth pressure	1			1602.522				-5.250	-8413.242				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			540.571				-5.250	-2837.998				0.000	0.000	
				32596.787	72.998	26.768	17.845		-18812.593	-88.278		301.281		4678.860	110.479
Total =				35554.003	156.407	188.525	125.683		-15809.386	29.834		1118.295		4660.860	1868.673
				-125,683											
				-29,834											

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	131.171	1396.036	7097.680	1458.600
due to Substructure	1	156.407	57.354		1118.295	410.073
due to Earth pressure	1	5173.885			27573.203	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1396.036	10646.52
due to Substructure	1.5	156.550		941.68
due to Active Earth pressure	1	3868.831		19021.18
due to dynamic Earth pressure	1.5	1957.581		12828.03
		7043.266		43437.412

Summary of Forces For SBC

	Downward	Upward	
P	35679.686	35428.320	kN
ML	20009.626	19949.958	kNm
MT	6529.533	6529.533	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	7097.68	0.00	0.00	0.00	0.00	131.17	1458.60
156.41	1118.29	0.00	0.00	0.00	0.00	57.35	410.07
5173.88	27573.20						
6037.16	35789.18	0.00	0.00	0.00	0.00	188.52	1868.67

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	10646.52	0.00	0.00
156.55	941.68	0.00	0.00
3868.831	19021.185	0.000	0.000
1957.581	12828.028	0.000	0.000
7043.27	43437.41	0.00	0.00

Summary of Restoring Forces

Vertical Load	32012.605	kN
Moment	-183590.402	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				2208.097			87.447		2539.312	100.565				-18.000	1458.600
Forces from Substructure				33345.906	156.407	57.354	38.236		-18348.697	-70.731		1118.295		4678.860	410.073
CWLL-Max. Reaction case	0.20			196.36		11.665	7.777	1.150	225.819	8.943	1398.901		0.950	186.546	150.548
Vertical component of LL Surcharge	0.20			65.71				-5.250	-344.982				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			25.18				-5.250	-132.179				0.000	0.000	
Total =				35841.255	156.407	200.190	133.460		-16060.728	38.777		1118.295		4847.406	2019.221
							-133.460			-38.777					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Forces from Superstructure				2012.777	78.771	0.000	-8252.39	-322.962
Forces from Substructure				30101.562	22.962	0.000	-174817.05	-198.001
CWLL-Max. Reaction case	0.00			0.00	0.000	-4.100	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-10.500	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-10.500	0.00	
Total =				32114.339	101.734		-183069.44	-520.964
					-101.734			520.964

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	757.950	142.836	1396.036	7610.574	1609.148
due to Substructure	1	156.407	57.354		1118.295	410.073
due to Earth pressure	1	5173.885			27573.203	
due to Live load surcharge	0.20	219.423			1433.320	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1396.036	10646.520
due to Substructure	1.5	156.550		941.680
due to Active Earth pressure	1	3868.831		19021.185
due to dynamic Earth pressure	1.5	1957.581		12828.0281
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		7043.266		43437.412

Summary of Forces For SBC

	Downward	Upward	
P	35974.715	35707.795	kN
ML	21713.442	21635.887	kNm
MT	6866.627	6866.627	kNm

Summary of Restoring Forces

Vertical Load	32012.605	kN
Moment	-183590.402	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
757.95	7610.57	0.00	0.00	0.00	0.00	142.84	1609.15
156.41	1118.29	0.00	0.00	0.00	0.00	57.35	410.07
5173.88	27573.20						
219.42	1433.32						
6307.66	37735.39	0.00	0.00	0.00	0.00	200.19	2019.22

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	10646.52	0.00	0.00
156.55	941.68	0.00	0.00
3868.831	19021.185		
1957.581	12828.028		
0.000	0.000		
0.000	0.000		
7043.27	43437.41	0.00	0.00

Case 7 : DL+SIDL-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2208.097		131.171	87.447		2539.312	100.565				-18.000	1458.600
Substructure & Foundation -Portion 1				749.119	83.409	30.586	20.390		463.896	17.547		817.013		0.000	299.595
Substructure & Foundation -Portion 2															
Solid Return wall	1		31.19	779.81	126.329	46.324	30.883	-2.35	-1832.554	-72.575	1392.36	803.951	6.00	4678.860	294.805
Shaft above HFL	1	25	0.000	0.000	0.000	0.000	0.000	1.256	0.000	0.000	1395.421	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	102.478	1537.163	-53.331	-19.556	-13.037	1.204	1851.470	-15.703	1395.905	-528.509	0.000	0.000	-193.802
Back filling above HFL over heel slab	1	20	130.210	2604.200	0.000	0.000	0.000	-2.350	-6119.870	0.000	1396.803	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	649.600	6496.000	0.000	0.000	0.000	-2.399	-15580.975	0.000	1391.300	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	392.000	3920.000				2.321	9097.156				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	68.875	1033.125				-1.892	-1954.781				0.000	0.000	
Toe slab	1	15	41.563	623.438				3.224	2009.766				0.000	0.000	
portion between heel & toe	1	15	21.000	315.000				1.150	362.250				0.000	0.000	
Vertical component of active earth pressure	1			1122.111				-5.250	-5891.081				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			514.677				-5.250	-2702.053				0.000	0.000	
				19186.767	112.080	41.099	27.399		-20760.672	-88.278		475.068		4678.860	174.205
Total =				22143.982	195.489	202.856	135.237		-17757.464	29.834		1292.081		4660.860	1932.400

-135.237

-29.834

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLS due to Fv
Superstructure				2012.777	78.771		-8252.386	-322.962
Substructure & Foundation -Portion 1				674.207	18.351		-2925.275	-80.552
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	31.1924	701.829	27.795	-7.60	-5333.900	-211.239
Shaft above HFL	0.9	25	0.000	0.000	0.000	-3.994	0.000	0.000
Shaft below HFL	0.9	15	102.478	1383.446	-23.184	-4.046	-5596.770	93.790
Back filling above HFL over heel slab	0.9	20	130.210	2343.780	0.000	-7.600	-17812.728	0.000
Back filling below HFL over heel slab	0.9	10	649.600	5846.400	0.000	-7.649	-44716.478	0.000
Front Filling ovetoe	0.9	10	392.000	3528.000		-2.929	-10334.559	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	68.875	929.813		-7.142	-6640.819	
Toe slab	0.9	15	41.563	561.094		-2.026	-1136.953	
portion between heel & toe	0.9	15	21.000	283.500		-4.100	-1162.350	
Vertical component of active earth pressure	0.9			1009.900		-10.500	-10603.946	
Vertical component of dynamic increment of earth pressure	0.9			463.209		-10.500	-4863.695	
				17268.090	13.210		-109206.65	-157.229
Total =				19955.074	110.332		-120384.31	-560.743

-110.332

560.743

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLS due to FL
	83.409		817.013
1.0	126.329	1392.359	803.951
1.0	0.000	1395.421	0.000
1.0	-105.371	1395.905	-1044.230
1.0	0.000	1396.803	0.000
1.0	0.000	1391.300	0.000
	20.958		-240.279
	104.367		576.734

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	131.171	1396.036	7097.680	1458.600
due to Substructure	1	195.489	71.685		1292.081	473.800
due to Earth pressure	1	4110.913			19655.891	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1396.036	10646.520
due to Substructure	1.5	156.550		865.101
due to Active Earth pressure	1	2709.015		11491.766
due to dynamic Earth pressure	1.5	1863.809		12246.187
		5789.679		35249.574

Summary of Forces For SBC

	Downward	Upward	
P	22279.220	22008.745	kN
ML	10318.022	10258.354	kNm
MT	6593.260	6593.260	kNm

Forces along Long. Axis

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	7097.68	0.00	0.00	0.00	0.00	131.17	1458.60
195.49	1292.08	0.00	0.00	0.00	0.00	71.68	473.80
4110.91	19655.89						
5013.27	28045.65	0.00	0.00	0.00	0.00	202.86	1932.40

Forces along Long. Axis

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	10646.52	0.00	0.00
156.55	865.10	0.00	0.00
2709.015	11491.766		
1863.809	12246.187		
5789.68	35249.57	0.00	0.00

Summary of Restoring Forces

Vertical Load	19844.742	kN
Moment	-120945.055	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha h \times P$ (kN)	FT = $0.3 \times \alpha h \times P$ (kN)	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				2208.097		131.171	87.447		2539.312	100.565				-18.000	1458.600
Forces from Substructure				19935.885	195.489	71.685	47.790		-20296.775	-70.731		1292.081		4678.860	473.800
CWLL-Max. Reaction case	0.20			73.64		4.374	2.916	1.150	84.681	3.354	1398.901		0.950	69.954	56.455
Vertical component of LL Surcharge	0.20			71.71				-5.250	-376.466				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			43.80				-5.250	-229.952				0.000	0.000	
Total =				22333.126	195.489	207.230	138.154		-18279.200	33.188		1292.081		4730.814	1988.855
								-138.154		-33.188					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				2012.777	78.771		-8252.386	-322.962
Forces from Substructure				17942.297	31.561		-112131.93	-237.781
CWLL-Max. Reaction case	0.00			0.00	0.00		-4.10	0.00
Vertical component of LL Surcharge	0.00			0.00		-4.100	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-10.500	0.000	0.000
Total =				19955.074	110.332		-120388.41	-560.743
						-110.332		560.743

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	757.950	135.546	1396.036	7610.574	1515.055
due to Substructure	1	195.489	71.685		1292.081	473.800
due to Earth pressure	1	4110.913			19655.891	
due to Live load surcharge	0.20	278.861			1864.932	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
757.95	7610.57	0.00	0.00	0.00	0.00	135.55	1515.05
195.49	1292.08	0.00	0.00	0.00	0.00	71.68	473.80
4110.91	19655.89						
278.86	1864.93						
5343.21	30423.48	0.00	0.00	0.00	0.00	207.23	1988.86

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1396.036	10646.520
due to Substructure	1.5	156.550		865.101245
due to Active Earth pressure	1	2709.015		11491.766
due to dynamic Earth pressure	1.5	1863.809		12246.187
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		5789.679		35249.574

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
1060.30	10646.52	0.00	0.00
156.55	865.10	0.00	0.00
2709.015	11491.766		
1863.809	12246.187		
0.000	0.000		
0.000	0.000		
5789.68	35249.57	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	22471.280	22194.973	kN
ML	12177.465	12111.090	kNm

Summary of Restoring Forces

Vertical Load	19844.742	kN
Moment	-120949.155	kNm

MT	6719.669	6719.669	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x $\alpha_h \times P$ (kN)	FT = $\alpha_h \times P$ (kN)	Fv = 0.3 x α_v $\times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				2208.097		437.237	87.447		2539.312	100.565				-18.000	4862.000
Substructure & Foundation -Portion 1				749.119	25.023	101.952	20.390		463.896	17.547		245.104		0.000	998.650
Substructure & Foundation -Portion 2				32596.787	21.900	89.227	17.845		-18812.593	-88.278		90.384		4678.860	368.262
Total =				35554.003	46.922	628.417	125.683		-15809.386	29.834		335.488		4660.860	6228.912
							-125.683					-29.834			

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x $\alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				2012.777	78.771		-8252.39	-322.962
Substructure & Foundation -Portion 1				674.207	18.351		-2925.27	-80.552
Substructure & Foundation -Portion 2				29427.355	4.611		-171891.78	-117.449
Total =				32114.339	101.734		-183069.44	-520.964
							-101.734	520.964

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x α_h $\times P$ (kN)	C.g. of Force (m)	MLs due to FL
	83.409		817.013
	20.958		-189.226
	104.367		627.787

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.115	437.237	1396.036	2159.973	4862.000
due to Substructure	1	46.922	191.179		335.488	1366.911
due to Earth pressure	1	5173.885			27573.203	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	2159.97	0.00	0.00	0.00	0.00	437.24	4862.00
46.92	335.49	0.00	0.00	0.00	0.00	191.18	1366.91
5173.88	27573.20						
5435.92	30068.66	0.00	0.00	0.00	0.00	628.42	6228.91

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1396.036	3193.956
due to Substructure	1.5	156.550		941.679879
due to Active Earth pressure	1	3868.831		19021.1845
due to dynamic Earth pressure	1.5	587.274		3848.40842
		4930.747		27005.229

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	3193.96	0.00	0.00
156.55	941.68	0.00	0.00
3868.831	19021.185		
587.274	3848.408		
4930.75	27005.23	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	35679.686	35428.320	kN
ML	14289.113	14229.445	kNm
MT	10889.772	10889.772	kNm

Summary of Restoring Forces

Vertical Load	32012.605	kN
Moment	-183590.402	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Total =	35841.255	133.460	-16060.73	38.777	4847.406
		-133.460		-38.777	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	476.121	1396.036	2642.198	5363.826
due to Substructure	1	46.922	191.179		335.488	1366.911
due to Earth pressure	1	5173.885			27573.203	
due to Live load surcharge	0.2	219.423			1433.320	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1396.036	3193.956
due to Substructure	1.5	46.96		282.504
due to Active Earth pressure	1	3868.83		19021.185
due to dynamic Earth pressure	1.5	587.27		3848.408
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		4821.16		26346.053

Summary of Forces For SBC

	Downward	Upward	
P	35974.715	35707.795	kN
ML	15962.260	15884.705	kNm
MT	11578.143	11578.143	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Superstructure	2208.097	87.447	2539.312	100.565	-18.000
Substructure & Foundation -Portion 1	749.119	20.390	463.896	17.547	0.000
Substructure & Foundation -Portion 2	19186.767	27.399	-20760.672	-88.278	4678.860
Total =	22143.982	135.237	-17757.464	29.834	4660.860
		-135.237		-29.834	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L2}$	MLs due to F_v
Total =	32114.339	101.734	-183069.44	-520.964
		-101.734		520.964

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.14	2642.20	0.00	0.00	0.00	0.00	476.12	5363.83
46.92	335.49	0.00	0.00	0.00	0.00	191.18	1366.91
5173.88	27573.20						
219.42	1433.32						
5703.37	31984.21	0.00	0.00	0.00	0.00	667.30	6730.74

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	3193.96	0.00	0.00
46.96	282.50	0.00	0.00
3868.831	19021.185		
587.274	3848.408		
0.000	0.000		
0.000	0.000		
4821.16	26346.05	0.00	0.00

Summary of Restoring Forces

Vertical Load	32012.605	kN
Moment	-183590.402	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	Long. Ecc. (e_{L2}) @ Toe (m)	ML = $P \times e_{L2}$	MLs due to F_v
Superstructure	2012.777	78.771		-8252.386	-322.962
Substructure & Foundation	674.207	18.351		-2925.275	-80.552
Substructure & Foundation	17268.090	13.210		-109206.652	-157.229
Total =	19955.074	110.332		-120384.312	-560.743
		-110.332			560.743

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.12	437.24	1396.04	2159.97	4862.00
due to Substructure	1	58.65	238.95		387.62	1579.33
due to Earth pressure	1	4110.91			19655.89	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1396.036	3193.96
due to Substructure	1.5	46.965		259.53
due to Active Earth pressure	1	2709.015		11491.77
due to dynamic Earth pressure	1.5	559.143		3673.86
		3633.214		18619.11

Summary of Forces For SBC

	Downward	Upward	
P	22279.220	22008.745	kN
ML	4475.858	4416.190	kNm
MT	11102.194	11102.194	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	$ML = P \times eL1$	MLs due to F_v	$MT = P \times eT$
Total =	22333.126	138.154	-18279.20	33.188	4730.814
		-138.154		-33.188	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	451.81844	1396.036	2642.198	5050.18316
due to Substructure	1	58.647	238.94933		387.624	1579.33363
due to Earth pressure	1	4110.913			19655.891	
due to Live load surcharge	0.2	278.861			1864.932	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1396.036	3193.96
due to Substructure	1.5	46.96		259.53
due to Active Earth pressure	1	2709.01		11491.77
due to dynamic Earth pressure	1.5	559.14		3673.86
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		3633.21		18619.11

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	2159.97	0.00	0.00	0.00	0.00	437.24	4862.00
58.65	387.62	0.00	0.00	0.00	0.00	238.95	1579.33
4110.91	19655.89						
4384.68	22203.49	0.00	0.00	0.00	0.00	676.19	6441.33

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	3193.96	0.00	0.00
46.96	259.53	0.00	0.00
2709.015	11491.766		
559.143	3673.856		
3633.21	18619.11	0.00	0.00

Summary of Restoring Forces

Vertical Load	19844.742	kN
Moment	-120945.055	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	$ML = P \times eL2$	MLs due to F_v
Total =	19955.074	110.332		-120388.41	-560.743
		-110.332			560.743

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.14	2642.20	0.00	0.00	0.00	0.00	451.82	5050.18
58.65	387.62	0.00	0.00	0.00	0.00	238.95	1579.33
4110.91	19655.89						
278.86	1864.93						
4711.56	24550.65	0.00	0.00	0.00	0.00	690.77	6629.52

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	3193.96	0.00	0.00
46.96	259.53	0.00	0.00
2709.015	11491.766		
559.143	3673.856		
0.000	0.000		
0.000	0.000		
3633.21	18619.11	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	22471.280	22194.973	kN
ML	6304.633	6238.258	kNm
MT	11360.331	11360.331	kNm

Summary of Restoring Forces

Vertical Load	19844.742	kN
Moment	-120949.155	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1398.901 -	1385.995)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1398.901 -	1385.995)	=	0.000 kNm

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

LOAD CASES	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Normal Dry Case															
Case 1 : DL+SIDL-Normal Dry Case	35113.706	6618.681	4660.860	255.762	313.394	221.672	279.304	313.394	221.672	6289.549	22139.477	3.52	29768.623	178157.85	5.98
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	36424.081	12132.543	5593.590	245.151	350.795	204.238	309.882	350.795	204.238	7460.552	22139.477	2.97	37540.359	181262.68	4.83
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	22209.991	-817.433	4660.860	189.823	182.706	155.732	148.615	189.823	148.615	6289.549	14010.137	2.23	25298.020	121204.90	4.79
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	22936.709	4565.098	5010.630	173.205	212.956	136.556	176.307	212.956	136.556	7593.443	14236.017	1.87	34040.312	124593.09	3.66
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	35679.686	20009.626	6529.533	208.608	382.841	160.849	335.083	382.841	160.849	7043.266	22408.824	3.18	43437.412	183590.40	4.23
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	35974.715	21713.442	6866.627	204.671	393.740	154.446	343.516	393.740	154.446	7043.266	22408.824	3.18	43437.412	183590.40	4.23
								SAFE	SAFE			SAFE			SAFE

Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	22279.220	10318.022	6593.260	148.937	238.781	100.712	190.556	238.781	100.712	5789.679	13891.319	2.40	35249.574	120945.06	3.43
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	22471.280	12177.465	6719.669	142.767	248.802	93.617	199.652	248.802	93.617	5789.679	13891.319	2.40	35249.574	120949.16	3.43
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	35679.686	14289.113	10889.772	249.460	373.882	169.809	294.231	373.882	169.809	4930.747	22408.824	4.54	27005.229	183590.40	6.80
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	35974.715	15962.260	11578.143	246.940	385.932	162.255	301.246	385.932	162.255	4821.162	22408.824	4.65	26346.053	183590.40	6.97
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	22279.220	4475.858	11102.194	190.862	229.835	109.657	148.631	229.835	109.657	3633.214	13891.319	3.82	18619.108	120945.06	6.50
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	22471.280	6304.633	11360.331	185.307	240.205	102.215	157.112	240.205	102.215	3633.214	13891.319	3.82	18619.108	120949.16	6.50
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

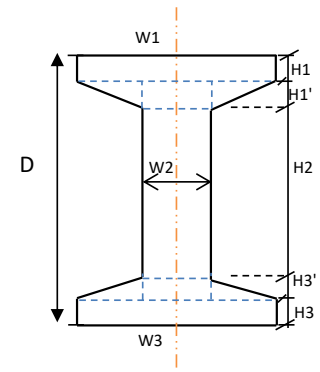
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	325
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	675	100	33750
1' (Rect.)	325	100	32500
2 (Rect.)	325	950	308750
3' (Tri.)	300	150	22500
3' (Rect.)	325	150	48750
3 (Rect.)	625	250	156250
			752500



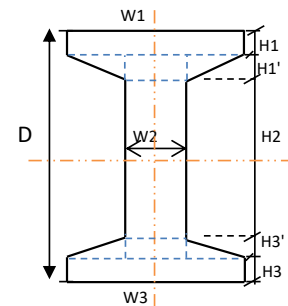
Total Area of Section A = 0.753 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	78
H2	=	1122
H3'	=	0
H3	=	250
W1	=	1000
W2	=	475
W3	=	475

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	525	78	20417
1' (Rect.)	475	78	36944
2 (Rect.)	475	1122	533056
3' (Tri.)	0	0	0
3' (Rect.)	475	0	0
3 (Rect.)	475	250	118750
			859167



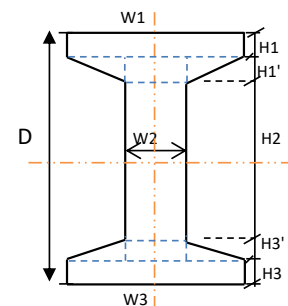
Total Area of Section A = 0.859 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

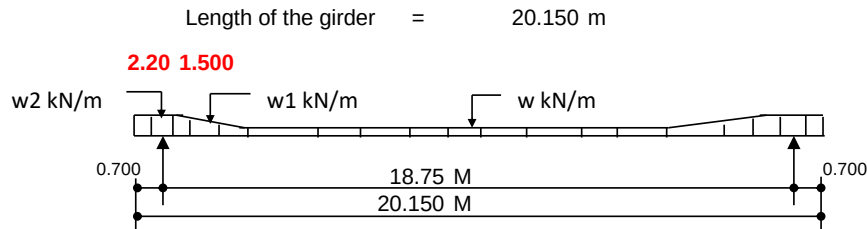
D	=	1600
H1	=	150
H1'	=	56
H2	=	1394
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	56	10417
1' (Rect.)	625	56	34722
2 (Rect.)	625	1394	871528
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066667



Total Area of Section A = 1.067 m²

1.0 Self weight of Inner Girder & Outer Girder



UDL , w	=	18.81 kN/m
Avg. , w ₁	=	21.48 kN/m
UDL , w ₂	=	26.67 kN/m
Supp. Reaction , R	=	210.82 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	
UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder
Supp. Reaction , R	=	180.091 kN	For Outer Girder
	=	166.238 kN	For Inner Girder
weight of deck slab has been Increase by 5 %			
Supp. Reaction , R	=	189.095 kN	For Outer Girder
	=	174.549 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

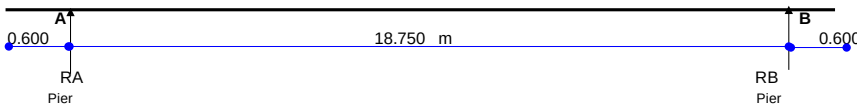
4.0 Weight due to wearing coat

Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	
UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder
Supp. Reaction , R	=	60.954 kN	For Outer Girder
	=	66.495 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)

Maximum Live Load Reaction Case**Due to Class A 1-Lane , 2-Lanes, 3-Lanes & 4-Lanes**Considering **114.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**

Nos. of Lane for design purpose = **3 LANE**

Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

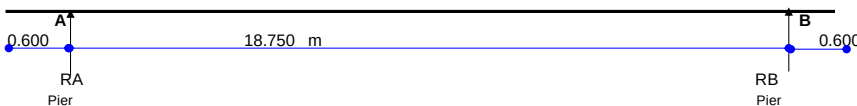
1st Train	Total Load	500.0kN				
					363.637	136.363
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	363.637	136.363

Due to Class 70R WheeledConsidering **170.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

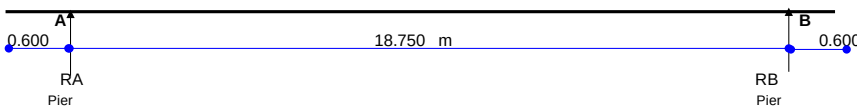
1st Train	Total Load	1000.0kN				
					758.731	241.269
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	758.731	241.269

Due to Class A 1-Lane/2-Lane (along with Class 70R)Considering **68.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

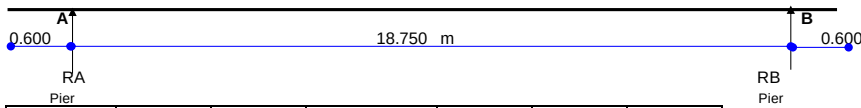
1st Train	Total Load	554.0kN				
					284.864	269.136
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
27.000	1.100	18.800	18.200	491.400		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	284.864	269.136

Due to Class SV LoadingConsidering **180.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = 18.75 m
 Projection beyond c.L. of bearing at Abut. end = 0.600 m
 Projection beyond c.L. of bearing at far end = 0.600 m
 Total length including Overhangs = 19.95 m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	3420.0kN				
180.000		0.000	-0.600	-108.000	2003.040	1416.960
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		
180.000	1.500	9.000	8.400	1512.000		
180.000	1.500	10.500	9.900	1782.000		
180.000	1.500	12.000	11.400	2052.000		
180.000	1.500	13.500	12.900	2322.000		
180.000	1.500	15.000	14.400	2592.000		
180.000	1.500	16.500	15.900	2862.000		
180.000	1.500	18.000	17.400	3132.000		
180.000	1.500	19.500	18.900	3402.000		
0.000	1.500	0.000	-0.600	0.000		
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	2003.040	1416.960

Deck Width = 12.500 m

transverse eccentricity of superstructure about c.g of abutment and foundation = 0 m

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	2003.040 KN	Corr. MT	= 600.912 kNm
Minimum Reaction (RB)	=	1416.960 KN	Corr. MT	= 425.088 kNm

1-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-0.9	0	0	=	4.45 m
Maximum Reaction (RA)	=	363.637 KN		Corr. MT	=	1618.186 kNm			
Minimum Reaction (RB)	=	136.363 KN		Corr. MT	=	606.814 kNm			

2-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-0.85	0	=	2.70 m
Maximum Reaction (RA)	=	727.275	KN	Corr. MT	=	1963.642	kNm		
Minimum Reaction (RB)	=	272.725	KN	Corr. MT	=	736.358	kNm		

3-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	0.95 m
Maximum Reaction (RA)	=	1090.912 KN	Corr. MT	=	1036.366 kNm				
Minimum Reaction (RB)	=	409.088 KN	Corr. MT	=	388.634 kNm				

4-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-0.8 m
Maximum Reaction (RA)	=	1454.549 KN		Corr. MT	=	-1163.639 kNm				
Minimum Reaction (RB)	=	545.451 KN		Corr. MT	=	-436.361 kNm				

1- 70RW

Transverse Ecc.	=	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m
Maximum Reaction (RA)	=	758.731 KN		Corr. MT	=	2394			
Minimum Reaction (RB)	=	241.269 KN		Corr. MT	=	761			

1-CLASS A + 1-70RW

[illegible]

2-CLASS A + 1-70RW

FORCES AT JOINT											
Transverse Ecc.	=	for 1st Class A for 2nd Class A for 70RW	= = =	4.45 m 4.45 -0.90 2.7 -0.85	-0.85	=	0 -1.8 -0.25	2.7 m -1.2	-1.395	=	-2.795
Maximum Reaction (RA)	=	1328.459 KN	Corr. MT	=	284.86 x	4.45 + +	284.86 x 758.73 x	2.700 -2.795	=	-84	
Minimum Reaction (RB)	=	779.541 KN	Corr. MT	=	269.136	4.45 + +	269.14 x 241.27 x	2.700 -2.795	=	1250	

1- 70RW + 1-CLASS A

Transverse Ecc.	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155	m
	for Class A	=	6.25	-0.5	-7.25	-0.25	= -0.9	=	-2.650 m
Maximum Reaction (RA)	=	1043.595 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650	
Minimum Reaction (RB)	=	510.405 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650	

1- 70RW + 2-CLASS A

Transverse Ecc.	for 70RW	=	3.155 m							
	for 1st Class	=	-2.650 m							
	for 2nd Class	=	-2.650	-0.9	-0.85	=		-4.400 m		
Maximum Reaction (RA)	=	1328.459 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650	=	386
					+		284.86 x	-4.400		
Minimum Reaction (RB)	=	779.541 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650	=	-1136
					+		269.14 x	-4.400		

2- 70RW

Transverse Ecc.	for 1-70RW for 2-70RW	= =	3.16 m 3.155	-1.395	-1.2	-1.40	=	-0.835	m	
Maximum Reaction (RA)	=	1517.461 KN	Corr. MT	=	758.73 x	3.155 +	758.73 x	-0.835	=	1760
Minimum Reaction (RB)	=	482.539 KN	Corr. MT	=	241.269	3.155 +	241.269	-0.835	=	560

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	363.637	1618.186
2-CLASS A	727.275	1963.642
3-CLASS A	1090.912	1036.366
4-CLASS A	0.000	0.000
1- 70RW	758.731	2393.795
1-CLASS A + 1-70RW	1043.595	1347.312
1- 70RW + 1-CLASS A	1043.595	1638.906
2-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 2-CLASS A	0.000	0.000
2- 70RW	0.000	0.000

Reaction RB, , (kN)	Corr. MT (kNm)
136.363	606.814
272.725	736.358
409.088	388.634
0.000	0.000
241.269	761.205
510.405	1222.988
510.405	47.994
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 1090.912 KN
 Reaction RB = 409.088 KN

& Corresponding moment = 1036.366 kNm
 & Corresponding moment = 388.634 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 758.731 KN
 Reaction RB = 241.269 KN

& Max moment = 2393.795 kNm
 & Max moment = 761.205 kNm

Stability Check Summary

Description	P _{max} (kN/m ²)	P _{min} (kN/m ²)	Sliding	Overturnin g	Shear (Heel slab)	Shear (Toe slab)
Normal Dry Case						
Case 1 : DL+SIDL-Normal Dry Case	313.39	221.67	3.52	5.98	0.009	0.987
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	350.80	204.24	2.97	4.83		
Permissible	449	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Normal HFLCase						
Case 3 : DL+SIDL-Normal HFL Case	189.82	148.61	2.23	4.79		
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	212.96	136.56	1.87	3.66		
Permissible	449	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic Dry Case						
Case 5 : DL+SIDL-Long. Seismic Dry Case	382.84	160.85	3.18	4.23		
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	393.74	154.45	3.18	4.23		
Permissible	561	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic HFL Case						
Case 7 : DL+SIDL-Long. Seismic HFL Case	238.781	100.712	2.40	3.43		
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	248.802	93.617	2.40	3.43		
Permissible	561	0	1.00	1.00		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic Dry Case						
Case 9 : DL+SIDL-Trans. Seismic Dry Case	373.882	169.809	4.54	6.80		
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	385.932	162.255	4.65	6.97		
Permissible	561	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic HFL Case						
Case 11 : DL+SIDL-Trans. Seismic HFL Case	229.835	109.657	3.82	6.50		
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	240.205	102.215	3.82	6.50		
Permissible	561	0	1.00	1.00	4.192	4.192
Remarks	O.K	O.K	O.K	O.K	O.K	O.K

Reinforcement summary

Type of reinforcement	Area of steel required	Area of steel provided						
Heel Slab								
Main steel at bottom face	1895	20	mm bar @	200	mm c/c (i.e.)	3142	mm ²	(O.K)
		20	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	628	12	mm bar @	150	mm c/c (i.e.)	754	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 20 Legged bar @	150	mm c/c (i.e.)			
Toe Slab								
Main steel at bottom face	6094	32	mm bar @	200	mm c/c (i.e.)	8042	mm ²	(O.K)
		32	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	1608	20	mm bar @	150	mm c/c (i.e.)	2094	mm ²	(O.K)
Shear Reinforcement	Provide Shear Reinf.	10	mm 22 Legged bar @	150	mm c/c (i.e.)			

Abutment Shaft								
Vertical steel at earth face	8621	32	mm bar @	100	mm c/c (i.e.)	11184	mm ²	(O.K)
		20	mm bar @	100	mm c/c (i.e.)			
Vertical steel at non-earth face	1440	16	mm bar @	100	mm c/c (i.e.)	2011	mm ²	(O.K)
Distribution steel	3299	16	mm 2 Legged bar @	120	mm c/c (i.e.)	3351	mm ²	(O.K)
Abutment Cap								
Top Longitudinal Reinf.	1106	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Bottom Longitudinal Reinf.	1106	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Side Face Longitudinal Reinf.		12	mm dia bar	200	mm c/c (i.e.)			
Transverse Reinf.	2.19E+06	12	mm 2 Legged bar @	150	mm c/c (i.e.)	2.77E+06	mm ³ /m	(O.K)
Corbel Reinforcement								
Primary tensile reinf.	1210	20	mm bar @	125	mm c/c (i.e.)	2513	mm ²	(O.K)
Horizontal Stirrups	286.17	12	mm 2 Legged bar @	in 3 Vertical Layers		679	mm ² /m	(O.K)
Vertical Stirrups	302.40	10	mm 2 Legged bar @	200	mm c/c (i.e.)	785	mm ³ /m	(O.K)

***DESIGN OF PIER P1 AND
ITS FOUNDATION
FOR MINOR BRIDGE AT CH- 13+025***

This Document Contains Design Calculations of Pier with Open Foundation

1. SALIENT FEATURES OF THE BRIDGE

Skew angle : 0 degree 0 Radian

<u>SPAN DETAILS</u>	<u>Left span</u>		<u>Right span</u>		<u>Unit</u>
	<u>Right</u>	<u>Skew</u>	<u>Right</u>	<u>Skew</u>	
C/C Expansion joint	= 20.000	20.000	20.000	20.00	m
C/C Expansion joint to C.L. brg	= 0.625	0.625	0.625	0.63	m
Expansion gap	= 50	50	50	50	mm
C/C bearing	= 18.750	18.750	18.750	18.750	m

SUPERSTRUCTURE DETAILS

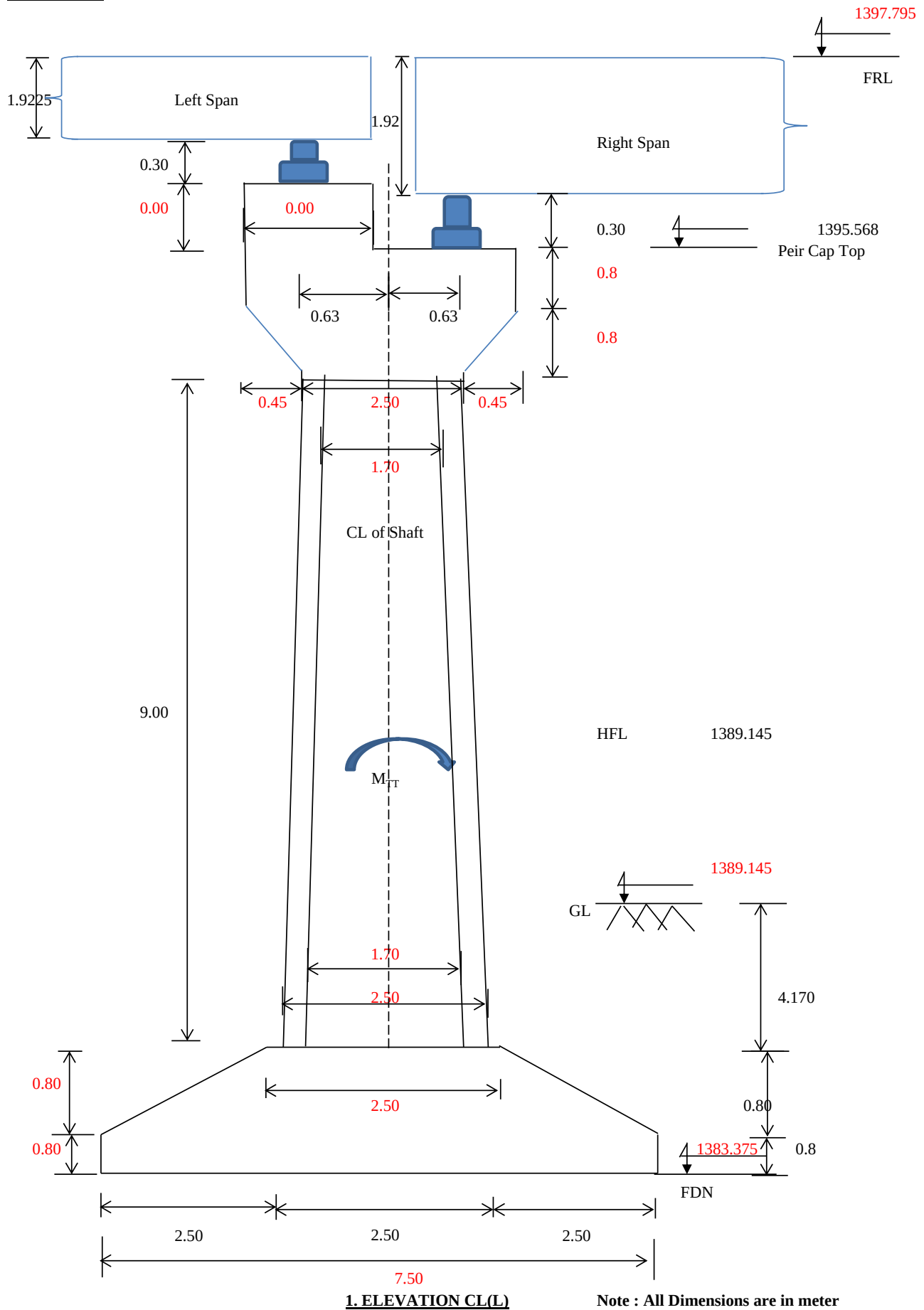
	Left Span	Right Span
Overall deck width of bridge	= 12.50 m	12.50 m
Carriage way width	= 11.50 m	11.50 m
Crash barrier width	= 0.45 m	0.45 m
Height of crash barrier	= 1.00 m	1.00 m
C.g of Crash barrier from bottom	= 0.40 m	0.40 m
Cross-slope	= 2.50 %	2.50 %
Wearing coat thickness	= 65 mm	65 mm
Nos. of Bearing	= 4	4
Spacing of Bearing	= 3.00 m	3.00 m
Depth of Girder	= 1.600 m	1.600 m
Depth of slab	= 0.22 m	0.22 m
Depth of superstructure	= 1.820 m	1.820 m
Bearing + Pedestal height	= 0.30 m	0.30 m
Width of main girder at top	= 0.40 m	0.40 m
Width of cross girder	= 0.40 m	0.40 m
Nos. of cross-girders	= 2	2
FRL to Soffit	= 1.92 m	1.92 m
Type of Bearing	OR SPHERICAL BR	

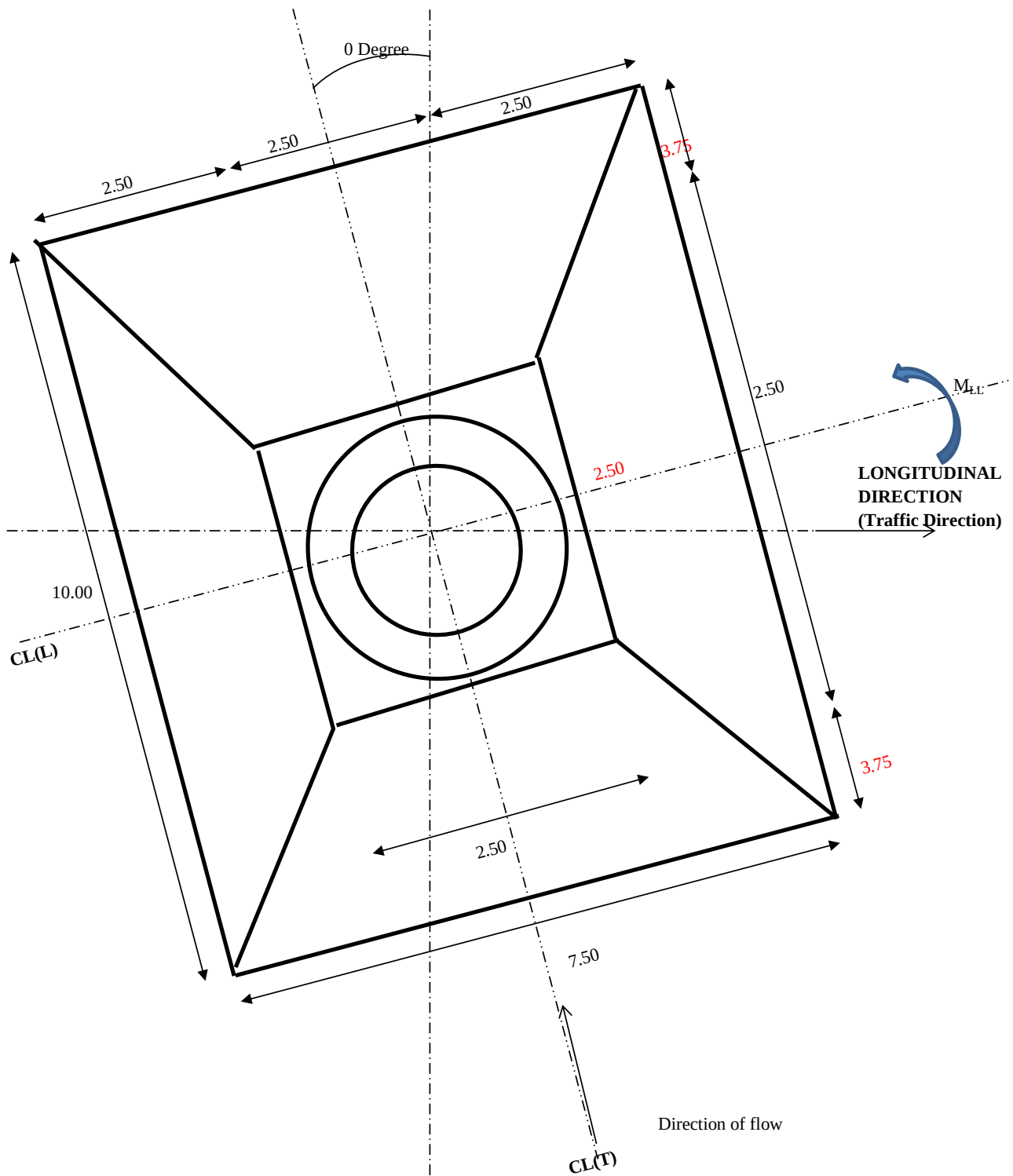
LEVEL DETAILS

Formation level	= 1397.795 m
Pier cap top level	= 1395.568 m
HFL	= 1389.145 m
Ground level	= 1389.145 m
MSL (Normal)/Rock Level	= 1389.145 m
MSL (Seismic)/Rock Level	= 1389.145 m
Pier shaft bottom/Footing top level	= 1384.970 m
Founding level	= 1383.375 m

DIMENSION DETAILS

Pier cap:	Right	Skew
Width of pier cap at top	= 3.40 m	3.40 m
Width of pier cap at bottom	= 2.50 m	2.50 m
Length of pier cap at top	= 10.50 m	10.50 m
Length of pier cap at bottom	= 2.50 m	2.50 m
Thickness of uniform portion	= 0.80 m	0.80 m
Thickness of tapered portion	= 0.80 m	0.80 m
Pier shaft:		
Width of pier at top	= 2.50 m	2.50 m
Width of pier at bottom	= 2.50 m	2.50 m
Length of pier (rectangle portion)	= 2.50 m	2.50 m
Length of pier(end circular portion Top)	= 1.25 m	1.25 m
Length of pier (end circular portion Bottom)	= 1.25 m	1.25 m
Total length of pier shaft at top	= 5.00 m	5.00 m
Total length of pier shaft at bottom	= 5.00 m	5.00 m
Footing:		
Width of footing at top	= 2.50 m	2.50 m
Width of footing at bottom	= 7.50 m	7.50 m
Length of footing	= 10.00 m	10.00 m
Thickness at tip	= 0.80 m	0.80 m
Maximum thickness	= 1.60 m	1.60 m

DIAGRAMS:



Note : All Dimensions are in meter

2. PLAN SHOWING FOOTING & PIER SHAFT



2. MATERIALS DETAILS**CONCRETE:**

Grade of concrete	fck	=	M 35	
Mean value of concrete compressive strength, fcm		=	M 45 (28 days strength from table 6.5 of IRC:112)	
Design concrete compressive strength, fcd		=	0.447 x fck	(Basic and Seismic combination)
		=	15.633 Mpa	
Secant modulus of elasticity, Ecm		=	32000 Mpa	(from table 6.5 of IRC:112)
Mean axial tensile strength fctm		=	2.8 Mpa	(from table 6.5 of IRC:112)
Density of concrete		=	2.5 t/m ³	

SOIL :

Density dry		=	2 t/m ³	
Density Saturated		=	2.3 t/m ³	
Soil density (submerged)		=	2 t/m ³	
Angle of Internal friction ϕ		=	35	
Cohesion c		=	0	
Gross bearing capacity		=	40.00 t/m ²	(Normal case)
		=	50.00 t/m ²	(Seismic case)

CREEP COEFFICIENT:

(Table 6.5 of IRC:112)

Pier shaft cross-sectional area	Ac	=	11.159 m ²	
Perimeter in contact with atmosphere, u		=	12.854 m	
Notional size, ho = 2Ac/u		=	1736 mm	
Age of concrete at the time of loading to		=	90 days	
	ta	=	25550 days	
	f(α 90)	=	1.2 (Table 6.9 IRC 112)	
Hence creep co-efficient for pier shaft		=	1.32 (increased by 10%)	
Creep co-efficient for foundation		=	1.0 (as ho = α for foundation)	
Hence modulus of elasticity for long term loading, Ecm'		=	13793 Mpa	Ecm' = Ecm / (1 + f)

REINFORCING STEEL:

Grade of reinforcement	fy	=	Fe 500 Mpa	
Modulus of elasticity	Es	=	200000 Mpa	

3. PERMISSIBLE STRESSES**MAXIMUM PERMISSIBLE STRESS IN CONCRETE:**

Rare combination		=	0.48 x fck	
		=	16.8 Mpa	
Quasi permanent combination		=	0.36 x fck	
		=	12.6 Mpa	

MAXIMUM PERMISSIBLE STRESS IN STEEL:

Rare Combination		=	300 Mpa	
Permissible crack width	Wk,max	=	0.3 mm	

4. SEISMIC PARAMETERS

Seismic zone		=	IV	
Type of soil		=	Type2(Medium)	
Zone factor	Z	=	0.24	
Importance factor	I	=	1.20	
Response Reduction Factor(Long)	R	=	3	
Response Reduction Factor(Trans)	R	=	3	

5. ENVIRONMENTAL PARAMETERS

Relative humidity		=	80%	
Condition of exposure		=	Moderate	

6.OTHER PARAMETERS

Mean velocity of water current flow		=	0.00 m/s	
Design Speed	V	=	40 Kmph	
Radius of Curvature	R	=	0 m	

7.LOADS FROM SUPERSTRUCTURES

Angle of Skew	=	0 °	=	0.000 Radian
No. of Bearing	=		4.00	
		Right		Skew
c/c of bearing in Transverse Direction	=	3.000	m	3.000 m
c/c bearing on Longitudinal Direction	=	1.250	m	1.250 m
Impact Factor for Design of Pier Cap				= 1.28

Left Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	1766.80	160.00								
Total Load	1926.80		254.90	0.000	0.00	0.00	0.00	0.00	0.00	0.00
Resultant Transverse Ecc.	0.000		0.000	0.000	0.950	1.210	1.000	1.000	0.300	0.300
Right Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	1766.80	160.00								
Total Load	1926.80		254.90	0.00	981.82	368.18	0.00	0.00	2003.04	1416.96
Resultant Transverse Ecc.	0.000		0.000	0.000	0.950	0.950	1.000	1.000	0.300	0.300

8. VOLUME OF SUBSTRUCTURE**PIER CAP****Uniform portion**

No.		L(m)	B(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.500	0.000	0.000	0.000	1.700	0.000	10.5975	12.20
2	1	10.500	3.400	0.800	28.560	0.000	0.000	10.1975	11.80

Tapered portion

2a	Rectangle	2.500	3.400	0.800	6.800	0.000	0.000	9.398	10.998
	Triangle	4.000	3.400	0.800	10.880	0.000	0.000	9.531	11.131
Total				Inc. 10%	50.864	0.000	0.000	9.021	10.475

PIER SHAFT**Above HFL****Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	4.828	12.739	0.000	0.000	6.584	8.184
2	0	2.500	0.000	4.828	0.000	-1.250	0.000	5.779	7.379
3	0	2.500	0.000	4.828	0.000	1.250	0.000	5.779	7.379

Cylinder/Conical Frustum portion

4	0.000	0.850	1.25	4.828	0.000	0.000	0.000	6.584	8.184
Total					12.739	0.000	0.000	6.584	8.184

Below HFL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	4.170	11.004	0.000	0.000	2.085	3.685
2	0	2.500	0.000	4.170	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	4.170	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.250	0.850	4.170	0.000	0.000	0.000	2.085	3.685
Total					11.004	0.000	0.000	2.085	3.685

Above LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	4.828	12.739	0.000	0.000	6.584	8.184
2	0.0	2.500	0.000	4.828	0.000	-0.850	0.000	5.779	7.379
3	0.0	2.500	0.000	4.828	0.000	0.850	0.000	5.779	7.379

Cylinder/Conical Frustum portion

4	0.000	0.850	0.850	4.828	0.000	0.000	0.000	6.450	8.050
Total					12.739	0.000	0.000	6.584	8.184

Below LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	4.17	11.004	0.000	0.000	2.085	3.685
2	0	2.500	0.000	4.17	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	4.17	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.25	0.850	4.17	0.000	0.000	0.000	1.756	3.356
Total					11.004	0.000	0.000	2.085	3.685

FOOTING

No.		L	B	H	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	2.500	1.60	40.000	0.000	0.000	-0.800	0.800
2	1	10.000	2.5	1.200	30.000	-2.361	0.000	-0.978	0.622
3	1	10.000	2.500	1.200	30.000	2.361	0.000	-0.978	0.622
Total					100	0.000	0.000	-0.907	0.693

EARTH ABOVE FOOTING

No.		L	B	H	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	7.500	5.770	432.750	0.000	0.000	1.285	2.885
2	-1	Shaft Below LBL			-11.004	0.000	0.000	2.085	3.685
3	-1	Footing			-100.000	0.000	0.000	-0.907	0.693
Total					321.746	0.000	0.000	1.939	3.539

9. CALCULATION OF SEISMIC FORCE

$$\text{Time Period (T)} = 2 \sqrt{\frac{D}{1000 F}}$$

$$\begin{aligned} D &= \text{Approximate DL of super-structure \& LL in Tonne (DL+20\% LL)} \\ &= 4363.39 \text{ KN} + 196.364 \text{ KN} \\ &= 4560 \text{ KN} \end{aligned}$$

$$F =$$

Horizontal force in KN required to be applied at center of mass of super-structure for 1 mm deflection at the top of pier / abutment for the earthquake in the transverse direction; and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

$$F = \frac{6 E I}{x^2(3L-x)} \quad \text{for 1 mm deflection at } x$$

$$E = 32000 \text{ N/mm}^2$$

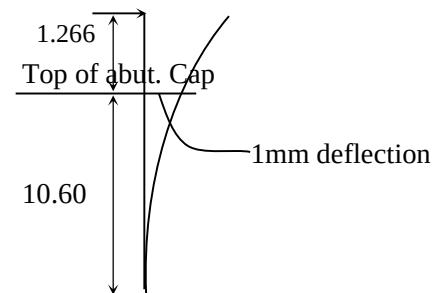
Column Cross-Section

$$D1 = 2.50 \text{ m (Average Outer Dia)}$$

$$D2 = 1.70 \text{ m (Average Inner Dia)}$$

$$I_{TT} = 1.13E+12 \text{ mm}^4 \quad 1.5E+12$$

$$I_{LL} = 1.13E+12 \text{ mm}^4$$



$$\text{Force required in Long. Dir. } F_L = 91 \text{ KN}$$

$$\text{Force required in Trans. Dir. } F_T = 77 \text{ KN}$$

$$\text{Time Period -Longitudinal Seismic Case} = 0.447 \text{ seconds}$$

$$\text{Time Period -Transverse Seismic Case} = 0.486 \text{ seconds}$$

$$S_a/g \text{ -Longitudinal Seismic Case} = 2.500$$

$$S_a/g \text{ -Transverse Seismic Case} = 2.500$$

$$\text{Seismic Zone} = \text{IV}$$

$$\text{Zone Factor } Z = 0.24$$

$$\text{Importance Factor } I = 1.20$$

$$\begin{aligned} \text{Response reduction Factor } R &= 3.00 \quad \text{Longitudinal} \\ R &= 3.00 \quad \text{Transverse} \end{aligned}$$

$$\text{Avg response acceleration } S_a/g = 2.500 \quad \text{Longitudinal}$$

$$\text{Avg response acceleration } S_a/g = 2.500 \quad \text{Transverse}$$

$$\text{Elastic Seismic Coefficient} = \frac{Z}{2} \times (S_a/g) \times (I/R)$$

$$\text{In longitudinal direction} = 0.360 / R$$

$$\text{In Transverse direction} = 0.360 / R$$

			Substructure	Foundation
Enhancement factor				1.35
Design Seismic coefficient	Long. Ah		0.120	0.162
	Trans. At		0.120	0.162
	Vert. Av		0.080	0.108

CALCULATION OF SEISMIC FORCES

SEISMIC COMPONENT OF SUPER-STRUCTURE DL, SIDL, WC & FPLL :

Loads & Their Lever arm

Description	LEFT			RIGHT		
	W	eVS	eVF	W	eVS	eVF
	Tonnes	(m)	(m)		(m)	(m)
Total Super-Structure DL	353.36	12.18	13.78	353.36	12.18	13.78
Total Super-Structure SIDL	32.00	13.22	14.82	32.00	13.22	14.82
Total Super-Structure WC	50.98	12.85	14.45	50.98	12.85	14.45
Total Super-Structure FPLL	0.00	12.85	14.45	0.00	12.85	14.45
Total	436.34	12.33	13.93	436.34	11.36	12.85

* Bearing level	10.90	12.50		10.90	12.50

SEISMIC COMPONENT OF SUB-STRUCTURE :

Description	W	eVS	eVF
	Tonnes	(m)	(m)
Total weight of shaft and pier cap	186.52	7.58	9.08
Total weight of foundation	250.00	-0.91	0.69

SEISMIC COMPONENT OF LIVE LOAD(70R+Class A) :

Description	W	W	eVS	eVF
	Tonnes	Tonnes	(m)	(m)
	70R + Class A	70R + Class A		
Maximum Live Load	98.18	0.00	14.02	15.62
Minimum Live Load	36.82	0.00	14.02	15.62

Summary of Seismic force at shaft and foundation

Super structure

Dead load - Super structure about shaft	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		52.36		570.601	
Seismic Transverse direction			52.36		620.44
Seismic Vertical Direction	34.91				

Dead load - Super structure about foundation	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		70.69		883.41	
Seismic Transverse direction			70.687		946.55
Seismic Vertical Direction	47.12				

Dead load Left span Dislodged- about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		35.34		441.705	
Seismic Transverse direction			35.34		454.07
Seismic Vertical Direction	23.56				

Sub structure

Dead load - Sub structure about Shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		22.38		169.70	
Seismic Transverse direction			22.38		169.70
Seismic Vertical Direction	14.92				

Dead load - Sub structure about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		30.2162		274.44	
Seismic Transverse direction			30.22		274.44
Seismic Vertical Direction	20.14				

Live Load - Max**70R + Class A**

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.36		33.04
Seismic Vertical Direction	1.57				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			3.18		49.69
Seismic Vertical Direction	2.12				

70R

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.36		33.04
Seismic Vertical Direction	1.57				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			3.18		49.69
Seismic Vertical Direction	2.12				

Live Load - Min**70R + Class A**

Live load - Min shaft	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.88		12.39
Seismic Vertical Direction		0.59				

Live load - Min foundation	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				1.19		18.63
Seismic Vertical Direction		0.80				

70R

Live load - Min shaft	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.88		12.39
Seismic Vertical Direction		0.59				

Live load - Min foundation	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				1.19		18.63
Seismic Vertical Direction		0.80				

10. HORIZONTAL FORCES

Consider Left span free and Right Span fix on pier

BOTH SPAN POSITION CONDITION(70R + Class A)

1. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (176.680 + 41.49 + 0.00) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= 0.2 \times 100 + 0.05 \times 54 \\ &+ 0.1 \times 100 + 0.05 \times 54 \\ F_h &= 35.4 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (176.680 + 41.49 + 36.82) \\ &= 12.75 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 30.45 \text{ Ton} \\ F_{hr} &= 30.45 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 30.45 - 10.91 \\ &= 19.54 \text{ Ton} \end{aligned}$$

2. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (176.680 + 41.49 + 0.00) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= 0.2 \times 100 + 0.05 \times 54 \\ F_h &= 22.7 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (176.680 + 41.49 + 98.18) \\ &= 15.82 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 27.17 \text{ Ton} \\ F_{hr} &= 27.17 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 27.17 - 10.91 \\
 &= 16.26 \text{ Ton}
 \end{aligned}$$

3. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 10.91 \text{ Ton} \\
 F_{hl} &= 10.91 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= 0.2 \times \frac{\text{Class 70R on First two Lane}}{20} + 0.05 \times \frac{\text{Class A on third Lane}}{10.8} \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force (Shaft)} &= 26.1803638 \text{ Ton} \\
 \text{Seismic Horizontal force (foundation)} &= 35.3434911 \text{ Ton} \\
 \text{Total Horizontal force for Shaft (Fh)} &= 30.7203638 \text{ Ton} \\
 \text{Total Horizontal force for foundation (Fh)} &= 39.8834911 \text{ Ton} \\
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{7.36} \right) \\
 &= 11.276664 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \right) \text{ and } F_h - \mu (R_g + R_q)$$

$$\begin{aligned}
 F_{hr} \text{ for shaft} &= 28.91 \text{ Ton} \\
 F_{hr} \text{ for foundation} &= 33.49 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 28.91 - 10.91 \\
 &= 18.00 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 33.49 - 10.91 \\
 &= 22.58 \text{ Ton}
 \end{aligned}$$

4. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 10.91 \text{ Ton} \\
 F_{hl} &= 10.91 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= 0.2 \times \frac{\text{Class 70R on First two Lane}}{20} + 0.05 \times \frac{\text{Class A on third Lane}}{10.8} \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force (Shaft)} &= 26.1803638 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\
 \text{Seismic Horizontal force (foundation)} &= 35.3434911 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \%})
 \end{aligned}$$

$$\begin{aligned}
 \text{Total Horizontal force for Shaft}(F_h) &= 30.7203638 \text{ Ton} \quad \text{inc.)} \\
 \text{Total Horizontal force for foundation}(F_h) &= 39.8834911 \text{ Ton} \\
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{19.64} \right) \\
 &= 11.890306 \text{ Ton} \\
 \text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q)) & \\
 F_{hr} \text{ for shaft} &= 29.52 \text{ Ton} \\
 F_{hr} \text{ for foundation} &= 34.10 \text{ Ton} \\
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 29.52 - 10.91 \\
 &= 18.61 \text{ Ton} \\
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 34.10 - 10.91 \\
 &= 23.19 \text{ Ton}
 \end{aligned}$$

BOTH SPAN POSITION CONDITION(70R)

5. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 10.91 \text{ Ton} \\
 F_{hl} &= 10.91 \text{ Ton}
 \end{aligned}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 100 + \frac{\text{Class A on third Lane}}{0.05} \times 0.00$$

$$F_h = 20 \text{ Ton}$$

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{36.82} \right) \\
 &= 12.75 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned}
 F_{hr} &= 22.75 \text{ Ton} \\
 F_{hr} &= 22.75 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 22.75 - 10.91 \\
 &= 11.84 \text{ Ton}
 \end{aligned}$$

6. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 10.91 \text{ Ton}
 \end{aligned}$$

$$F_{hl} = 10.91 \text{ Ton}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 100 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 20 \text{ Ton}$$

$$\mu (R_g + R_q) = 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{98.18} \right) = 15.82 \text{ Ton}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$= 25.82 \text{ Ton}$$

$$F_{hr} = 25.82 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 25.82 - 10.91 \\ &= 14.91 \text{ Ton} \end{aligned}$$

7. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 20 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 4 \text{ Ton}$$

$$\text{Seismic Horizontal force (Shaft)} = 26.1803638 \text{ Ton}$$

$$\text{Seismic Horizontal force (foundation)} = 35.3434911 \text{ Ton}$$

$$\text{Total Horizontal force for Shaft (Fh)} = 30.1803638 \text{ Ton}$$

$$\text{Total Horizontal force for foundation (Fh)} = 39.3434911 \text{ Ton}$$

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{7.36} \right) \\ &= 11.276664 \text{ Ton} \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$F_{hr} \text{ for shaft} = 28.37 \text{ Ton}$$

$$F_{hr} \text{ for foundation} = 32.95 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 28.37 - 10.91 \\ &= 17.46 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 32.95 - 10.91 \end{aligned}$$

$$= 22.04 \text{ Ton}$$

8. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= 0.2 \times \frac{\text{Class 70R on First two Lane}}{20} + 0.05 \times \frac{\text{Class A on third Lane}}{0} \\ F_h &= 4 \text{ Ton} \\ \text{Seismic Horizontal force(Shaft)} &= 26.1803638 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\ \text{Seismic Horizontal force(foundation)} &= 35.3434911 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.}) \\ \text{Total Horizontal force for Shaft}(F_h) &= 30.1803638 \text{ Ton} \\ \text{Total Horizontal force for foundation}(F_h) &= 39.3434911 \text{ Ton} \\ \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{19.64} \right) \\ &= 11.890306 \text{ Ton} \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned} F_{hr} \text{ for shaft} &= 28.98 \text{ Ton} \\ F_{hr} \text{ for foundation} &= 33.56 \text{ Ton} \\ \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 28.98 - 10.91 \\ &= 18.07 \text{ Ton} \\ \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 33.56 - 10.91 \\ &= 22.65 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(Special Vehicle)

9. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{0.00} \right) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \frac{\text{CWLL}}{141.70} \right) \\ &= 17.99 \text{ Ton} \\ F_{hr} &= 17.99 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 17.99 - 10.91 \\ &= 7.08 \text{ Ton} \end{aligned}$$

10. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \text{CWLL} \right) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \text{CWLL} \right) \\ &= 20.92 \text{ Ton} \\ F_{hr} &= 20.92 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 20.92 - 10.91 \\ &= 10.02 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(NO LIVE LOAD)

11. Normal No Reaction

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \text{CWLL} \right) \\ &= 10.91 \text{ Ton} \\ F_{hl} &= 10.91 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \text{CWLL} \right) \\ &= 10.91 \text{ Ton} \\ F_{hr} &= 10.91 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 10.91 - 10.91 \\ &= 0.00 \text{ Ton} \end{aligned}$$

ONE SPAN DISLOGED CONDITION

12. Normal Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{176.680} + \frac{\text{Total SIDL load}}{41.49} + \text{CWLL} \right) \\ &= 10.908485 \text{ Ton} \\ F_{hr} &= 10.90848 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 10.90848 - 0.000 \\
 &= 10.90848 \text{ Ton}
 \end{aligned}$$

13. Seismic Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= \text{Class 70R on First two Lane} + \text{Class A on third Lane} \\
 &= 0.2 \times 0 + 0.05 \times 0
 \end{aligned}$$

$$F_h = 0 \text{ Ton}$$

$$\begin{aligned}
 \text{Seismic Horizontal force(Shaft)} &= 26.1803638 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\
 \text{Seismic Horizontal force(foundation)} &= 35.3434911 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.}) \\
 \text{Total Horizontal force for Shaft}(F_h) &= 26.1803638 \text{ Ton} \\
 \text{Total Horizontal force for foundation}(F_h) &= 35.3434911 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times (\text{Total DL} + \text{Total SIDL load} + \text{CWLL}) \\
 &= 0.05 \times (176.680 + 41.49 + 0.00) \\
 &= 10.908485 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned}
 F_{hr} \text{ for shaft} &= 24.00 \text{ Ton} \\
 F_{hr} \text{ for foundation} &= 28.58 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 24.00 - 0.00 \\
 &= 24.00 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 28.58 - 0.00 \\
 &= 28.58 \text{ Ton}
 \end{aligned}$$

Summary of forces			
Cases		Horizontal force(Ton)	
		Shaft	Foundation
BOTH SPAN POSITION CONDITION(70R + Class A)			
	1. Normal Max Reaction on Fix End	19.54	19.54
	2. Normal Min Reaction on Fix End	16.26	16.26
	3. Seismic Max Reaction on Fix End	18.00	22.58
	4. Seismic Min Reaction on Fix End	18.61	23.19
BOTH SPAN POSITION CONDITION(70R)			
	5. Normal Max Reaction on Fix End	11.84	11.84
	6. Normal Min Reaction on Fix End	14.91	14.91
	7. Seismic Max Reaction on Fix End	17.46	22.04
	8. Seismic Min Reaction on Fix End	18.07	22.65
BOTH SPAN POSITION CONDITION(Special Vehicle)			
	9. Normal Max Reaction on Fix End	7.08	7.08
	10. Normal Min Reaction on Fix End	10.02	10.02
BOTH SPAN POSITION CONDITION(NO LIVE LOAD)			
	11. Normal No Reaction	0.00	0.00
ONE SPAN DISLOGED CONDITION			
	12. Normal Span Dislodge Condition	10.91	10.91
	13. Seismic Span Dislodge Condition	24.00	28.58

11.A CALCULATION OF CENTRIFUGAL FORCE

As per clause 212 of IRC:6-2014

$$C = \frac{WV^2}{127R}$$

70R + Class A

Maxium Reaction				
Live Load	W	=	98.18	t
Design Speed	V	=	40	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0.00	14.02	15.62

Minimum Reaction				
Live Load	W	=	0.00	t
Design Speed	V	=	40	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.02	15.62

70R

Maxium Reaction				
Live Load	W	=	98.18	t
Design Speed	V	=	40	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.02	15.62

Minimum Reaction				
Live Load	W	=	36.82	t
Design Speed	V	=	40	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.02	15.62

Special Vehicle**Maxium Reaction**

Live Load	W	=	200.30	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.02	15.62

Minimum Reaction

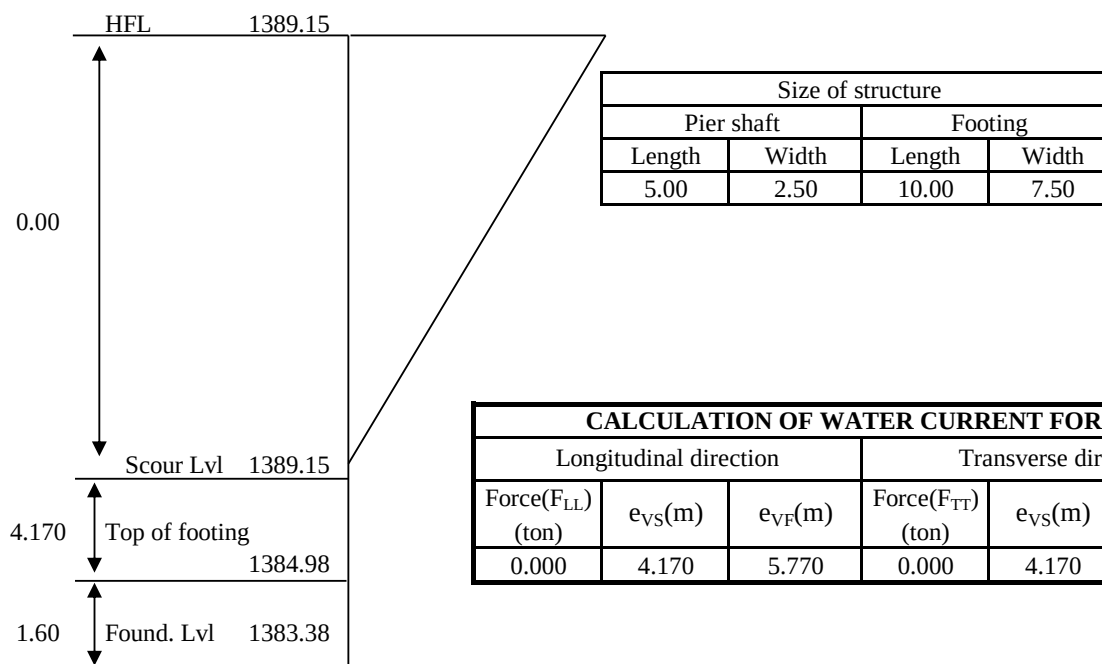
Live Load	W	=	141.70	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

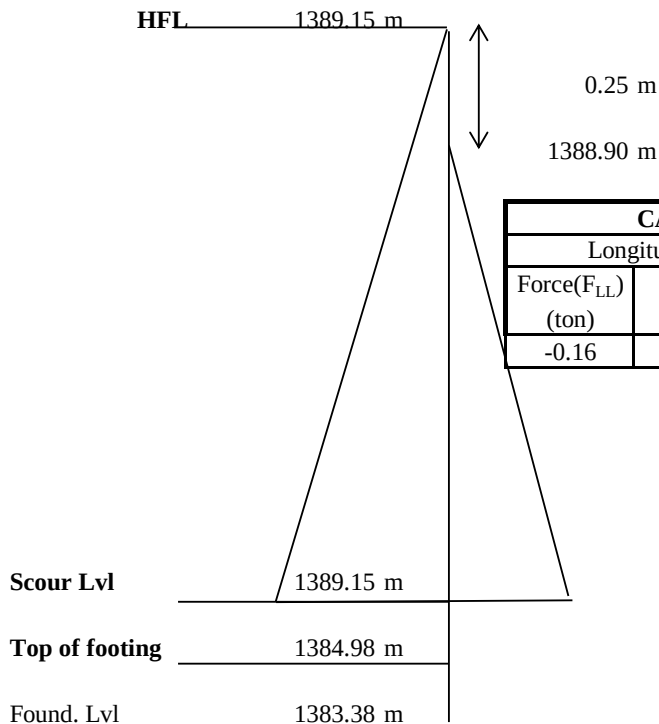
$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.02	15.62

12. WATER CURRENT FORCES**NORMAL CASE****Water current forces**

HFL	=	1389.145 m	
MSL (NORMAL)	=	1389.145 m	
Water current pressure	P	=	52 KV ² Kg/m ²
For piers with square ends	K	=	1.5
For piers with semicircular ends	K	=	0.66
Mean velocity of water current	V _{mean}	=	0.00 m/s
Max. velocity of water current	V	=	$\sqrt{2}$ x 0.00
		=	0.00 m/s
Angle of variation		=	20 degree
Water current pressure at HFL	(Trans direction)	=	0.00 t/m ² 52 K (vcosq) ²
Water current pressure at HFL	(Long direction)	=	0.00 t/m ² 53 K (vsinq) ²

**Pressure diagram (Normal case)**

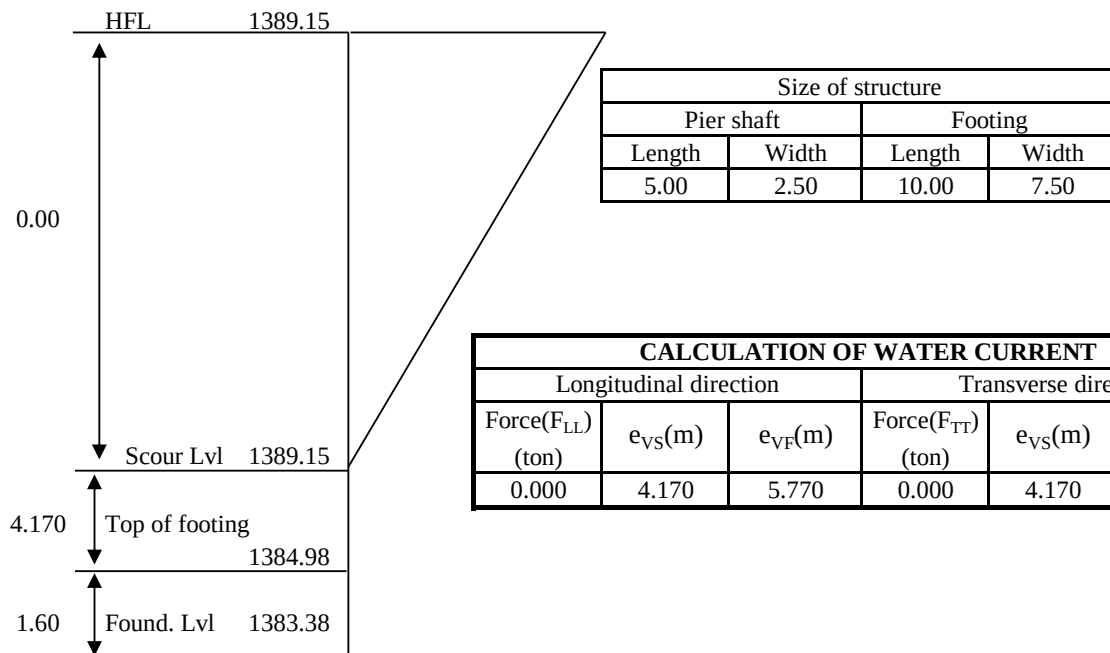
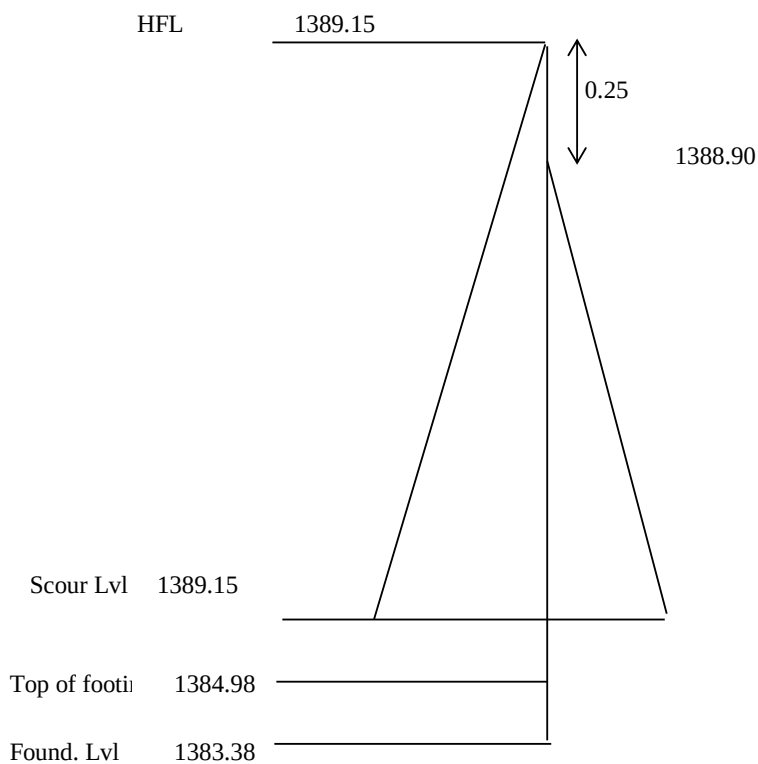
Static Force

CALCULATION OF HYDROSTATIC FORCES					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
-0.16	4.09	5.69	-0.08	4.09	5.69

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	4.170	5.770	0.000	4.170	5.770

SEISMIC CASE

HFL	=	1389.145 m		
MSL (SEISMIC)	=	1389.15 m		
Water current pressure	P	=	52 KV ²	Kg/m ²
For piers with square ends	K	=	1.5	
For piers with semicircular ends	K	=	0.66	
Mean velocity of water current	V_{mean}	=	0.00 m/s	
Max. velocity of water current	V	=	$\sqrt{2}$ x 0.00	
		=	0.00 m/s	
Angle of variation		=	20 degree	
Water current pressure at HFL	(Trans direction)	=	0.00 t/m ²	52 K (vcosq) ²
Water current pressure at HFL	(Long direction)	=	0.00 t/m ²	53 K (vsinq) ²

**Pressure diagram (seismic case)****Static Force**

CALCULATION OF HYDROSTATIC FORCES					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
-0.16	4.09	5.69	-0.08	4.09	5.69

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	4.170	5.770	0.000	4.170	5.770

13. LOAD COMBINATION DEFINATION

PIER SHAFT DESIGN

ULS

LC -1 ULS NORMAL LWL MAX REACTION
 LC -2 ULS NORMAL LWL SPAN DISLOGED

LC - 5 ULS MAX REACTION $1X + 0.3Y + 0.3Z$
 LC - 6 ULS MAX REACTION $0.3X + 1Y + 0.3Z$

LC - 9 ULS NORMAL SV MAX REACTION

SLS

LC -1 RARE NORMAL MAX REACTION
 LC -2 RARE NORMAL SPAN DISLOGED

LC -5 QP NORMAL

LC -7 QP NORMAL SPAN DISLOGED

LC -9 RARE NORMAL SV MAX REACTION

Footing design

LC - 1 ULS NORMAL MAX REACTION(70R +Class A)
 LC - 2 ULS NORMAL SPAN DISLOGED

LC - 5 ULS NORMAL MAX REACTION(70R)

LC - 7 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R + \text{Class A})$
 LC - 8 ULS SPAN DISLOGED $1X + 0.3Y + 0.3Z$
 LC - 9 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{Class A})$
 LC - 10 ULS SPAN DISLOGED $0.3X + 1Y + 0.3Z$

LC - 15 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R)$
 LC - 16 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 19 ULS NORMAL LWL SV MAX REACTION

BASE PRESSRE

LC - 1 NORMAL MAX REACTION(70R+Class A)

LC - 2 NORMAL SPAN DISLOGED

LC - 5 0

LC - 6 SEISMIC SPAN DISLODGED $1X + 0.3Y + 0.3Z$

LC - 7 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{class A})$

LC - 8 SEISMIC LWL SPAN DISLODGED $0.3X + 1Y + 0.3Z$

LC - 13 SEISMIC NORMAL MAX REACTION(70R)

LC - 15 SEISMIC MAX REACTION $1X + 0.3Y + 0.3Z(70R)$

LC - 16 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 13 NORMAL LWL SV MAX REACTION

16. DESIGN FORCES FOR BASE PRESSURE**LC -1 NORMAL MAX REACTION(70R+Class A)**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			192.680		-0.625	0.000		-120.42	0.000	1	
2	Surfacing			25.490		-0.625	0.000		-15.931	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			98.182		0.625	0.950		61.3638	93.273	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			15.620	0.0	0.000	1	
12	Pier cap	50.864	2.5	127.160		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	12.739	2.5	31.849		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	11.004	2.5	27.511		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.000		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				19.541			12.498	244.21		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.5	250.000		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	321.75	2	643.491		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1604.71	299.44	83.95

LC -2 NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.950		0.000	0.000	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			15.620	0.000	0.000	1	
12	Pier cap	50.864	2.5	127.160		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	12.739	2.5	31.849		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	11.004	2.5	27.511		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.000		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				10.908			12.498	136.329		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.5	250.000		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	321.75	2	643.491		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1298.18	272.68	0.00

272.685 0.000

LC - 6 SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.950		0.000	0.000	0	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			15.620	0.00	0.000	0	
12	Pier cap	50.86	2.50	127.16		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	12.74	2.50	31.85		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	11.00	2.50	27.51		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.00		1	

	Water Current Force(F_{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F_{LL})				28.580			12.498	357.18		1	
	Horizontal Force(F_{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.50	250.00		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	321.75	2.00	643.49		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				35.343				441.71		1	
	Transverse				26.180					336.34717	0.3	
	Vertical			17.454							0.3	
	Sub structure											
	Longitudinal				30.216				274.4		1	
	Transverse				30.216					274.44	0.3	
	Vertical			20.144							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M_{TT} (tm)	M_{LL} (tm)
1309.460	1089.26	183.24

LC - 8 SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0.2	
	Right Span											
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.950		0.000	0.000	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0.2	
11	Centifugal force				0.000			15.620	0.00	0.000	0.2	
12	Pier cap	50.86	2.50	127.16		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	12.74	2.50	31.85		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	11.00	2.50	27.51		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.00		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				28.580			12.498	357.18		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.50	250.00		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	321.75	2.00	643.49		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				0.000			0.00			0.3	
	Transverse				26.180					336.34717	1	
	Vertical			17.454							0.3	
	Sub structure											
	Longitudinal				30.216				274.4		0.3	
	Transverse				30.216					274.44085	1	
	Vertical			20.144							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1309.460	455.44	610.79

LC -13 SEISMIC NORMAL MAX REACTION(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			192.680		-0.625	0.000		-120.42	0.000	1	
2	Surfacing			25.490		-0.625	0.000		-15.931	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			98.182		0.625	0.950		61.3638	93.273	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centrifugal force				0.000			15.620	0.0	0.000	1	
12	Pier cap	50.864	2.5	127.160		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	12.739	2.5	31.849		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	11.004	2.5	27.511		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.000		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				11.841			12.498	147.98		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.5	250.000		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	321.75	2	643.491		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1604.71	203.21	83.95

LC - 15 SEISMIC MAX REACTION 1X + 0.3Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
Left Span												
1	DL (superstructure)			192.680		-0.625	0.000		-120.42	0.000	1	
2	Surfacing			25.490		-0.625	0.000		-15.931	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
Right Span												
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			98.182		0.625	0.950		61.364	93.273	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			15.620	0.0	0.000	0.2	
12	Pier cap	50.864	2.500	127.160		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	12.739	2.500	31.849		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	11.004	2.500	27.511		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1.000	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.000		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				22.040			12.498	275.44		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.00	1	
18	Footing	100.00	2.50	250.00		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	321.75	2.00	643.49		0.00	0.00		0.00	0.00	1	
20	Seismic											
Super structure												
	Longitudinal				70.687				883.4		1	
	Transverse				70.687					946.6	0.3	
	Vertical			47.125							0.3	
Sub structure												
	Longitudinal				30.216				274.4		1	
	Transverse				30.216					274.44	0.3	
	Vertical			20.14							0.3	
a)	Max Live Load											
	Longitudinal										1	
	Transverse				3.181					49.69	0.3	
	Vertical			2.121							0.3	

b)	Min Live Load											
	Longitudinal										1.5	1
	Transverse				1.193					18.633107	0.45	0.3
	Vertical			0.795							0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1556.911	1445.57	402.37

LC - 16 SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			192.680		-0.625	0.000		-120.42	0.000	1	
2	Surfacing			25.490		-0.625	0.000		-15.931	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
7	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			98.182		0.625	0.950		61.3638	93.273	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			15.620	0.0	0.000	0.2	
12	Pier cap	50.86	2.50	127.16		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	12.74	2.50	31.85		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	11.00	2.50	27.51		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			5.770	0.00		1	
	Water Current Force(F _{TT})				0.000			5.770		0.000	1	
17	Horizontal Force(F _{LL})				22.040			12.498	275.44		1	
	Horizontal Force(F _{TT})				0.000			12.498		0.000	1	
18	Footing	100.00	2.50	250.00		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	321.75	2.00	643.49		0.00	0.00		0.00	0.00	1	
20	Seismic											

	Super structure											
	Longitudinal				70.687				883.4		0.3	
	Transverse				70.687				946.6		1	
	Vertical		47.125								0.3	
	Sub structure											
	Longitudinal				30.216				274.4		0.3	
	Transverse				30.216				274.44		1	
	Vertical		20.144								0.3	
a)	Max Live Load											
	Longitudinal										0.3	
	Transverse				3.181				49.69		1	
	Vertical		2.121								0.3	
b)	Min Live Load											
	Longitudinal										0.45	0.3
	Transverse				1.193				18.6		1.5	1
	Vertical		0.795								0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1556.911	635.07	1317.28

LC - 18 NORMAL SV MAX REACTION

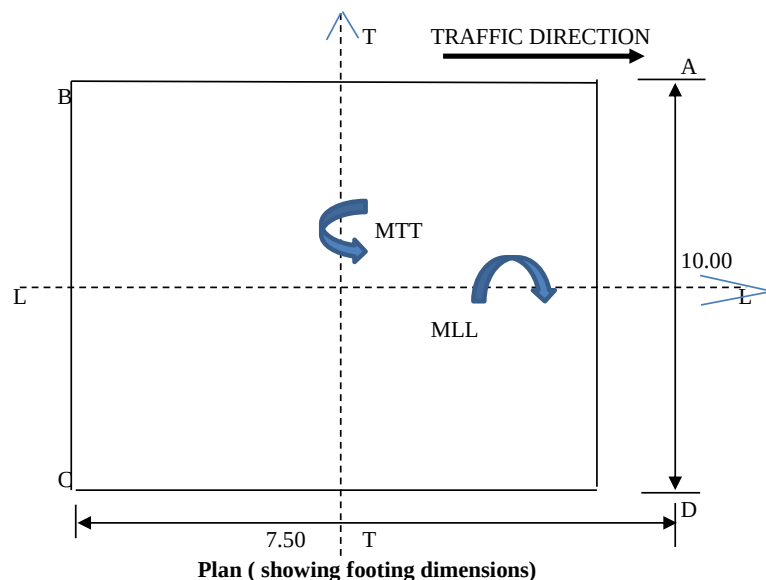
Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			192.680		-0.625	0.000		-120.42	0.000	1	
2	Surfacing			25.490		-0.625	0.000		-15.931	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
	Right Span											
5	DL (superstructure)			192.680		0.625	0.000		120.42	0.000	1	
6	Surfacing			25.490		0.625	0.000		15.931	0.000	1	
7	FLL			0.000		0.625	0.000		0.000	0.000	1	
8	LL			200.304		0.625	0.300		125.19	60.091	1	1
9	Centifugal force				0.000			15.620	0.0	0.000	1	
10	Pier cap	50.864	2.5	127.160		0.000	0.000		0.000	0.000	1	
11	Pier (above LBL)	12.739	2.5	31.849		0.000	0.000		0.000	0.000	1	

12	Pier (below LBL)	11.004	2.5	27.511		0.000	0.000		0.000	0.000	1	
13	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
14	Water Current Force(F_{LL})				0.000			5.770	0.000		1	
	Water Current Force(F_{TT})				0.000			5.770		0.000	1	
15	Horizontal Force(F_{LL})				7.085			12.498	88.54		1	
	Horizontal Force(F_{TT})				0.000			12.498		0.000	1	
16	Footing	100.00	2.5	250.000		0.000	0.000		0.000	0.000	1	
17	Earth Over Footing	321.75	2	643.491		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M_{TT} (tm)	M_{LL} (tm)
1716.65	213.73	60.09

20. CHECK FOR BASE PRESSURE



PROPERTIES OF BASE

Area	=	7.50	x	10.00	=	75.00	m ²
I _{LL}	=	7.50	$\times \frac{10.00^3}{12}$	=	625.00	m ⁴	
I _{TT}	=	10.00	$\times \frac{7.50^3}{12}$	=	351.56	m ⁴	

COORDINATES OF BASE CORNERS

Edges	L(m)	T(m)
A	3.75	5.00
B	-3.75	5.00
C	-3.75	-5.00
D	3.75	-5.00

SUMMARY OF FORCES

$$\text{Base pressure} = \frac{P}{A} + \frac{M_{LL} \cdot x}{I_{LL}} + \frac{M_{TT} \cdot z}{I_{TT}}$$

Sr. No.	Description	Forces at founding levels			Base pressure at footing corners in T/m ²			
		V(Tonne)	M _{TT} (Tm)	M _{LL} (Tm)	A	B	C	D
LC -1	NORMAL MAX REACTION(70R+Class A)	1604.71	299.44	83.95	25.26	18.87	17.53	23.92
LC -2	NORMAL SPAN DISLOGED	1298.18	272.68	0.00	20.22	14.40	14.40	20.22
LC -6	SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z	1309.46	1089.26	183.24	30.54	7.31	4.37	27.61
LC -7	SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R + class A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LC -8	SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z	1309.46	455.44	610.79	27.20	17.49	7.72	17.43
LC -13	SEISMIC NORMAL MAX REACTION(70R)	1604.71	203.21	83.95	24.24	19.90	18.56	22.89

LC - 15	SEISMIC MAX REACTION $1X + 0.3Y + 0.3Z(70R)$	1556.91	1445.57	402.37	39.40	8.56	2.12	32.96
LC - 16	SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R)$	1556.91	635.07	1317.28	38.07	24.52	3.45	16.99
LC - 18	NORMAL SV MAX REACTION	1716.65	213.73	60.09	25.65	21.09	20.13	24.69

Sr. No.	Average pressure along BC	Average pressure along AD	Average pressure along AB	Average pressure along CD	Effective Length along TT	Effective Width along LL	Effective Area	I_{LL}	I_{TT}
	T/m ²	T/m ²	T/m ²	T/m ²	m	m	m ²	m ⁴	m ⁴
LC -1	18.20	24.59	22.07	20.72	10.00	7.50	75.00	625.00	351.56
LC -2	14.40	20.22	17.31	17.31	10.00	7.50	75.00	625.00	351.56
LC -6	5.84	29.08	18.93	15.99	10.00	7.50	75.00	625.00	351.56
LC -8	12.60	22.32	22.35	12.57	10.00	7.50	75.00	625.00	351.56
LC -13	19.23	23.56	22.07	20.72	10.00	7.50	75.00	625.00	351.56
LC -15	5.34	36.18	23.98	17.54	10.00	7.50	75.00	625.00	351.56
LC -16	13.98	27.53	31.30	10.22	10.00	7.50	75.00	625.00	351.56
LC -18	20.61	25.17	23.37	22.41	10.00	7.50	75.00	625.00	351.56

Sr. No.	M _{TT}	M _{LL}	A'		B'		C'		D'		Base Pressure(T/m ²)			
	Tm	Tm	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	T/m ²	T/m ²	T/m ²	T/m ²
LC -1	299.44	83.95	3.75	5	-3.75	5	-3.75	-5	3.75	-5.00	25.26	18.87	17.53	23.92
LC -2	272.68	0.00	3.75	5	-3.75	5	-3.75	-5	3.75	-5.00	20.22	14.40	14.40	20.22
LC -6	1089.26	183.24	3.75	5	-3.75	5	-3.75	-5	3.75	-5.00	30.54	7.31	4.37	27.61
LC -8	455.44	610.79	3.75	5.00	-3.75	5.00	-3.75	-5.00	3.75	-5.00	27.20	17.49	7.72	17.43
LC -13	203.21	83.95	3.75	5.00	-3.75	5.00	-3.75	-5.00	3.75	-5.00	24.24	19.90	18.56	22.89
LC -15	1445.57	402.37	3.75	5.00	-3.75	5.00	-3.75	-5.00	3.75	-5.00	39.40	8.56	2.12	32.96
LC -16	635.07	1317.28	3.75	5.00	-3.75	5.00	-3.75	-5.00	3.75	-5.00	38.07	24.52	3.45	16.99
LC -18	213.73	60.09	3.75	5.00	-3.75	5.00	-3.75	-5.00	3.75	-5.00	25.65	21.09	20.13	24.69

Minimum Effective Area Allowed Normal case = 0.8 x 75 = 60.00 m²

Minimum Effective Area Allowed Seismic case = 0.67 x 75 = 50.250 m²

Minimum Effective Area Normal Case = 75.00 > 60.00 m²

Minimum Effective Area Seismic Case = 75.00 > 50.25 m²

SUMMARY OF BASE PRESSURES

NORMAL CASE

Maximum base pressure = 25.65 t/m² < 40 t/m² **Ok**

Minimum base pressure = 14.40 t/m² > 0 t/m² **Ok**

SEISMIC CASE

Maximum base pressure = 39.40 t/m² < 50 t/m² **Ok**

Minimum base pressure = 2.12 t/m² > 0 t/m² **Ok**

***DESIGN OF PRECAST RCC GIRDER AND
CAST IN SITU SLAB TYPE
SUPERSTRUCTURE***

SPAN = 20m c/c of expansion joint

(OVERALL DECK WIDTH – 12.5 m)

INTRODUCTION

The following report presents the design calculations of RCC Precast Girder. The RCC Girder is designed for Limit State of Collapse, Limit state of Serviceability. All applicable loads and Load combinations are calculated using IRC 6:2010 and IRC 112:2011. It includes Design of Precast RCC Girder, RCC Deck Slab, RCC End Diaphragm and Bearings.

DESIGN CONSIDERATIONS**Grade of Materials**

Grade of Concrete for RCC Slab	M	35
Grade of Concrete For Precast girder	M	35
Grade of Concrete For Diaphragm	M	35
Grade of Reinforcement	Fe	500D

Elastic Moduli of Material

Elastic Modulus of Concrete	E_{cm}	32000 N/mm ²	As per Table 6.5, IRC: 112:2011
Elastic Modulus of Steel	E_s	200000 N/mm ²	As per Clause 6.3.5, IRC: 112-2011
Coefficient of Friction	μ	0.05	As per Clause 706.3.4, IRC: 78-2000
Density of RCC		25 KN/m ³	As per Clause 203, IRC: 6-2010
Density of Wearing Coat		22 KN/m ³	

SUPERSTRUCTURE and SPAN DETAILS

Type of Superstructure	-	RCC Precast Girder			
Type of Bearing	-	Pot PTFE Girder			
No of Spans	-	1			
Skew Angle	-	0 degree	contemporary angle	-	90
Span effective from E/J to E/J (SK)	-	20.000 m			
Span effective from Brg. to Brg. (SQ)	-	19.000 m			
Span effective from Brg. to Brg. (SK)	-	19.000 m			
Spacing of girders	-	3.000 m	number of girder	-	4
Slab overhang from girder centre	-	1.750 m			

To ensure lateral stability, a simply supported or continuous beam should be proportioned that the clear distance between lateral restraints does not exceed $60b$ or $250b^2/h$

Thickness of Deck Slab	-	220 mm	railing	-	0.5
Overall Deck Width	-	12.5 m	footpath	-	1.5
Thickness of Wearing Coat	-	0.065 m	crash barrier	-	0.5
Expansion Gap in Abutment	-	0.04 m			
Clear Cover Provided	-	50 mm			

SEISMIC DESIGN

Seismic Zone	-	IV
Seismic Zone factor	-	0.24
Importance factor	-	1.2

CARRIAGEWAY LIVE LOAD (CWLL)

Superstructure meant for 3-lane carriageway has been designed for governing effect of CWLL loading as specified in IRC:6-2010 with appropriate lane reduction factor and suitable allowances for impact (as mentioned below) so placed on the span to produce maximum stresses.

Impact Factor	=	1.180
Lane Reduction Factor	=	0.900

The following CWLL combination have been considered in the design. The design is then carried out for the most critical CWLL Combination.

- 1 One Lane Class A + One Lane of class 70R Most Eccentric
- 2 Three Lane of Class A Placed Most Eccentrically
- 3 One Wheel of Class 70 R Placed over Girder + 1 Lane Class A
- 4 Class 70 R Placed concentric to the Girder + One Lane of Class 70R
- 5 SV Load

TEMPERATURE EFFECT

The climate condition is assumed to be "MODERATE".

The temperature variation of +/- 25 °C shall be considered for the design of bearings and expansion joints.

Temperature gradients given in IRC 6:2014 has also been considered to evaluated the effect on structure due to daily changes in temperature.

Coefficient of thermal expansion as per IRC: 6-2014

= **1.20E-05**

Poisson's Ratio = 0.15

HUMIDITY

The bridge is located in humid atmospheric conditions. The ambient relative humidity (RH) is assumed to be

75 %

DESIGN CONSTANTS

f_y	=	500 N/mm ²	
f_{yd}	=	435 N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	=	35 N/mm ²	
f_{ctm}	=	2.800 N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	=	15.63 N/mm ²	

Unit Weight of Materials

Unit Weight of Precast Concrete Girder	=	25	kN/m ³
Unit Weight of Reinforced Concrete	=	25	kN/m ³
Unit Weight of Green Concrete	=	26	kN/m ³

CONSTRUCTION SEQUENCE

Casting of Precast Girders and curing	=	7	day
Placing the Girders on the Bearing and Position the I-Girder on permanent bearings. Place shuttering on girder top for casting of RCC deck slab Concrete	=	14	day
Placing of Deck Slab & Diaphragm Conc. And removal of shuttering	=	28	day
Casting of Crash Barriers, Wearing Coat and othe deck furnitures	=	56	days
open to traffic	assumed	=	84 days

Concrete Compressive Strength Development with Time is given by Eq. 6.3 of IRC:112-2011

$f_{cm}(t) = \beta_{cc}(t) * f_{cm}$, where $f_{cm} = f_{ck} + 10$ & $\beta_{cc}(t) = \exp \{ s [1 - (28/(t/t_1))^{1/2}] \}$ for

ordinary portland concrete $S = 0.25$

Characteristic compressive strength with time, $f_{ck}(t) = f_{cm}(t) - 10$

$f_{ck} = 35$ Mpa $f_{cm} = 45$ Mpa

Age of concrete (t) in days	$\beta_{cc}(t)$	$f_{cm}(t)$	$f_{ck}(t)$	f_{ctm}	$\beta_{cc}(t)^a$	$f_{ctm}(t)$
7	0.779	35.046	25.046	2.771	0.779	2.16
14	0.902	40.573	30.573		0.902	2.50
21	0.962	43.293	33.293		0.962	2.67
28	1.000	45.000	35.000		1.000	2.77
beyond 28 days	1.000	45.000	35.000		1.000	2.77

STRUCTURAL ANALYSIS**SECTION PROPERTIES**

Uncracked sectional properties have been used for analysis. The effective flange widths for sectional properties for the longitudinal girders and cross diaphragms have been determined in accordance with IRC:112-2011.

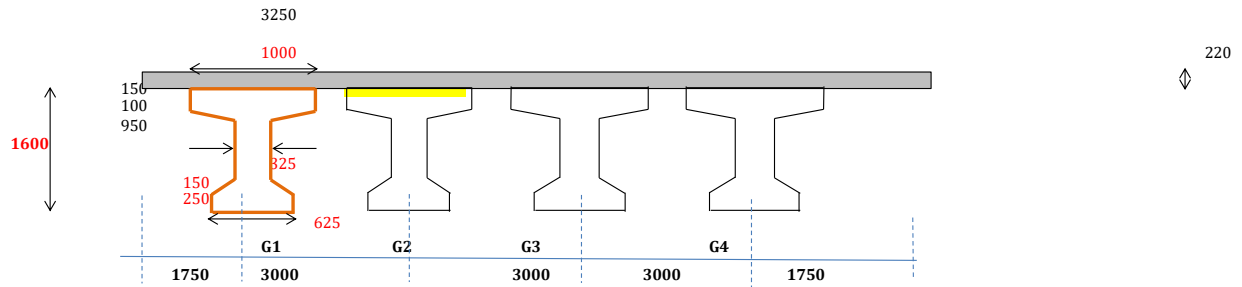
LONGITUDINAL ANALYSIS OF THE DECK FOR SIDL & CWLL

Superstructure has been modeled as an equivalent grillage mesh consisting of longitudinal and transverse members. Analysis of Self weight of girder and green concrete of slab has been carried out by single line model and the analysis of SIDL and CWLL has been carried out by grillage model. STAAD-Pro Software, which is widely accepted for structural analysis, has been used for analysis. For the design of longitudinal girders, moments and shear forces have been determined at the critical sections. The design of various components has been carried out by design excel sheets.

Diaphragms have been designed for DL+SIDL load in Jack up condition.

ANALYSIS OF DECK SLAB

Deck slab has been analysed along transverse direction as a continuous beam with overhangs supported on longitudinal girders and diaphragms. Live load effects shall be taken based on effective width method.

SECTION PROPERTIES (OUTER GIRDER)**AT MID SPAN****Moment of Inertia - I_z**

Parameter	Self	Composite
Depth	1600 mm	1820 mm
Area	752500 mm ²	1467500 mm ²
I_z	2.16E+11 mm ⁴	4.95E+11 mm ⁴
Y_b	842.98 mm	1265.4 mm
Y_t	757.02 mm	554.6 mm
Z_b	2.563E+08 mm ³	3.908E+08 mm ³
Z_t	2.854E+08 mm ³	8.917E+08 mm ³
u	4.825E+00 m	1.077E+01 m

Deck slab 220
Eff. Width 3250

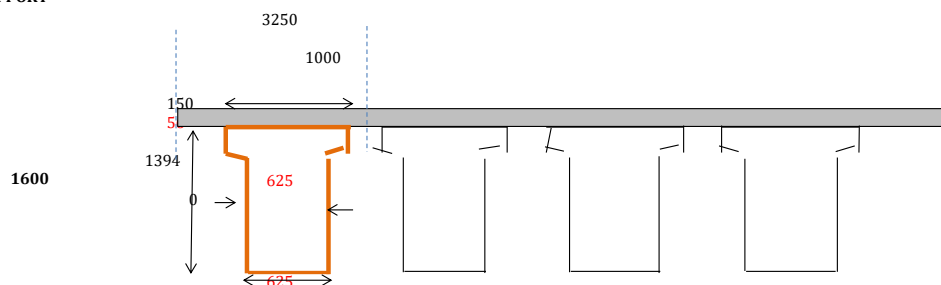
Moment of Inertia - I_y

$I_{self} = bd^3/12$	2.13E+10 mm ⁴
A_z^2	2.55E+09 mm ⁴
I_y	2.38E+10 mm ⁴
Z_{ly}	1689.1 mm
$I_y(\text{slab})$	6.29E+11 mm ⁴
$I_y(\text{comp})$	6.53E+11 mm ⁴

Moment of Inertia - I_x

T_{av} 200.0 mm
 B_{av} 325.0 mm

Portion no.	b	d	$\frac{3b^3 \cdot d^3}{10(b^2 + d^2)}$
Slab	3250	220	5.77E+09
Girder portion	1000	200	2.31E+09
	325	1075	1.01E+10
	625	325	5.07E+09
$I_x \text{ composite(mm}^4\text{)}$			2.33E+10
$I_x \text{ Girder(mm}^4\text{)}$			= 1.75E+10

AT SUPPORT**Moment of Inertia - I_z**

Parameter	Self	Composite
Depth	1600 mm	1820 mm
Area	1066750 mm ²	1781750 mm ²
I_z	2.5E+11 mm ⁴	5.8E+11 mm ⁴
Y_b	846.10 mm	1192.8 mm
Y_t	753.90 mm	627.2 mm
Z_b	3.0E+08 mm ³	4.8E+08 mm ³
Z_t	3.4E+08 mm ³	9.2E+08 mm ³
U		9.8E+00 m

Moment of Inertia - I_y

$I_{self} = bd^3/12$	4.202E+10 mm ⁴
A_z^2	1.477E+09 mm ⁴
I_y	4.350E+10 mm ⁴
Z_{ly}	1625.0 mm

I _y (slab)	6.293E+11	mm ⁴
I _y (comp)	6.72846E+11	mm ⁴

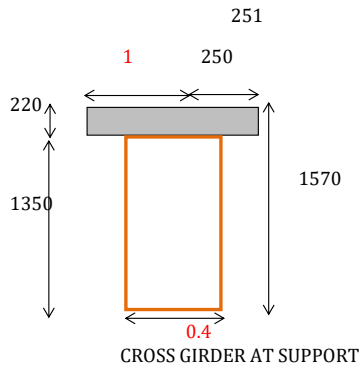
Moment of Inertia - I_x

T _{av}	178.0	mm
B _{av}	0.0	mm

Portion no.	b	d	$\frac{3 \cdot b^3 \cdot d^3}{10(b^2 + d^2)}$
	mm	mm	
1	3250	220	5.77E+09
2	1000	178	1.64E+09
3	625	1422	8.73E+10
4	625	0	0.00E+00
I _{x composite}			= 9.47E+10
I _{x girder}			= 8.89E+10

CROSS MEMBER AT END SUPPORT**Moment of Inertia - I_z**

Area (A _x)	55650	mm ²
y _b	1452	mm
y _t	118	mm
I _{self} = bd ³ /12	3.04E+08	mm ⁴
A _z ²	3.30E+08	mm ⁴
I _z	6.338E+08	mm ⁴

**Moment of Inertia - I_y**

Area (A _x)	55650	mm ²
y _i	124	mm
y _r	126	mm
I _{self} = bd ³ /12	2.88E+08	mm ⁴
A _y ²	8.32E+06	mm ⁴
I _y	2.965E+08	mm ⁴

Moment of Inertia - I_x

Item	b	d	$\frac{3 \cdot b^3 \cdot d^3}{10 \cdot (b^2 + d^2)}$
	mm	mm	
1	250.5	220	4.446E+08
2	0.4	1350	2.592E+01
I _{x,girder}	2.592E+01	mm ⁴	
I _{x,composite}	4.446E+08	mm ⁴	

Effective Width of FlangeFOR **OUTER GIRDER**

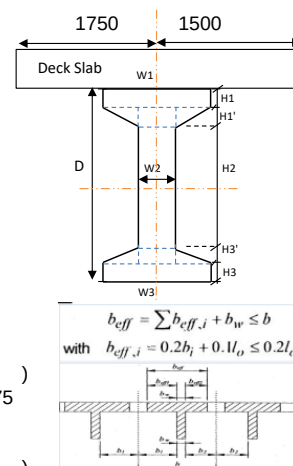
Thickness of Deck Slab = 220 mm

bw	=	b	=	3.250 m
		w2	=	325 mm
		b1	=	1587.5 mm
		b2	=	1337.5 mm
		lo	=	19.00 m

effective width of cantilever on one end = min (0.2 x b1 + 0.1 x lo , 0.2 x lo, b1)
 min (2.2175 , 3.8 , 1.5875)
 beff, 1 = 1.5875

effective width of cantilever on other end = min (0.2 x b2 + 0.1 x lo , 0.2 x lo, b2)
 min (2.1675 , 3.8 , 1.3375)
 beff, 2 = 1.338

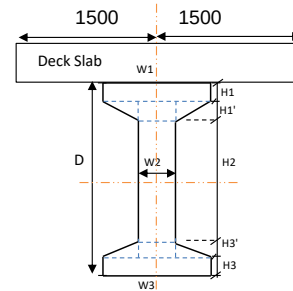
Total effective width of flange beff = 1.588 + 1.3375 + 0.325 = 3.250 m



FOR **INNER GIRDER**

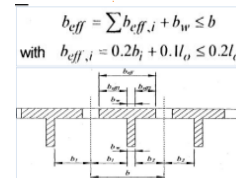
Thickness of Deck Slab = 220 mm

$b_w = b = 3.000 \text{ m}$
 $w_2 = 1689.097104 \text{ mm}$
 $b_1 = 655.451448 \text{ mm}$
 $b_2 = 655.451448 \text{ mm}$
 $l_o = 19.00 \text{ m}$

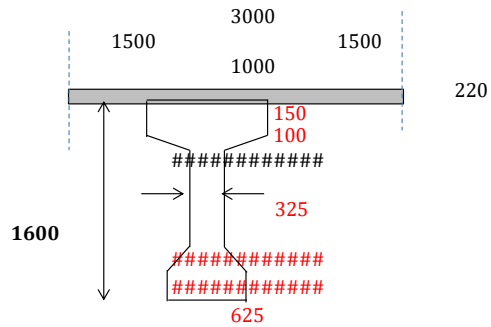


effective width of cantilever on one end = $\min (0.2 \times b_1 + 0.1 \times l_o, 0.2 \times l_o, b_1)$
 $\min (2.03109029, 3.8, 0.655451448)$
 $\text{beff, 1} = 0.655451448$

effective width of cantilever on other end = $\min (0.2 \times b_2 + 0.1 \times l_o, 0.2 \times l_o, b_2)$
 $\min (2.03109029, 3.8, 0.655451448)$
 $\text{beff, 2} = 0.655$



Total effective width of flange $\text{beff} = 0.655 + 0.6554514 + 3.000 \text{ m}$
 $= 4.3104514 \text{ m}$

SECTION PROPERTIES (INNER GIRDER)**AT MID SPAN****Moment of Inertia - I_z**

Parameter	Self	Composite
Depth	1600 mm	1820 mm
Area	752500 mm ²	1412500 mm ²
I_z	2.16E+11 mm ⁴	4.83E+11 mm ⁴
Y_b	842.98 mm	1248.1 mm
Y_t	757.02 mm	571.9 mm
Z_b	2.6E+08 mm ³	3.9E+08 mm ³
Z_t	2.9E+08 mm ³	8.4E+08 mm ³
u	4.8E+00 m	1.0E+01 m

Deck slab
Eff. Width

220 mm
3000 mm wide.

Moment of Inertia - I_y

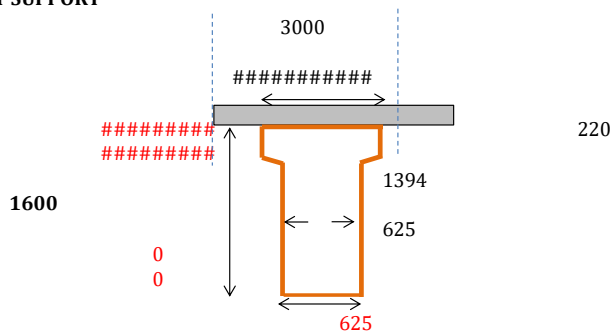
$I_{self} = bd^3/12$	2.13E+10 mm ⁴
A_z^2	3.57E+09 mm ⁴
I_y	2.48E+10 mm ⁴
Z_L	1500.0 mm
$I_y(\text{slab})$	4.95E+11 mm ⁴
$I_y(\text{comp})$	5.20E+11 mm ⁴

Moment of Inertia - I_x

T_{av} 200.0 mm
 B_{av} 325.0 mm

Portion no.	b	d	$\frac{3 \cdot b^3 \cdot d^3}{10(b^2 + d^2)}$
	mm	mm	
1	3000	220	5.32E+09
2	1000	200	2.31E+09
3	325	1075	1.01E+10
4	625	325	5.07E+09

$I_{x \text{ composite}} = 2.28E+10 \text{ mm}^4$
 $I_{x \text{ girder}} = 1.75E+10 \text{ mm}^4$

AT SUPPORT

Moment of Inertia - I_z

Parameter	Self	Composite	
Depth	1600	1820	mm
Area	1066750	1726750	mm ²
I_z	2.5E+11	5.6E+11	mm ⁴
Y_b	846.10	1176.3	mm
Y_t	753.90	643.7	mm
Z_b	3.0E+08	4.8E+08	mm ³
Z_t	3.4E+08	8.7E+08	mm ³
u		8.8E+00	m

Moment of Inertia - I_y

$I_{self} = bd^3/12$	4.202E+10	mm ⁴
A_z^2	1.477E+09	mm ⁴
I_y	4.350E+10	mm ⁴
Z_{ly}	1500.0	mm
$I_y(\text{slab})$	4.950E+11	mm ⁴
$I_y(\text{comp})$	5.38E+11	mm ⁴

Moment of Inertia - I_x

T_{av}	=	178.0	mm
B_{av}	=	0.0	mm

Portion no.	b	d	$\frac{3b^3 \cdot d^3}{10(b^2 + d^2)}$
	mm	mm	
1	3000	220	5.32E+09
2	1000	178	1.64E+09
3	625	1422	8.73E+10
4	625	0	0.00E+00

$I_{x, \text{composite}}$		9.43E+10	mm ⁴
$I_{x, \text{girder}}$	=	8.89E+10	mm ⁴

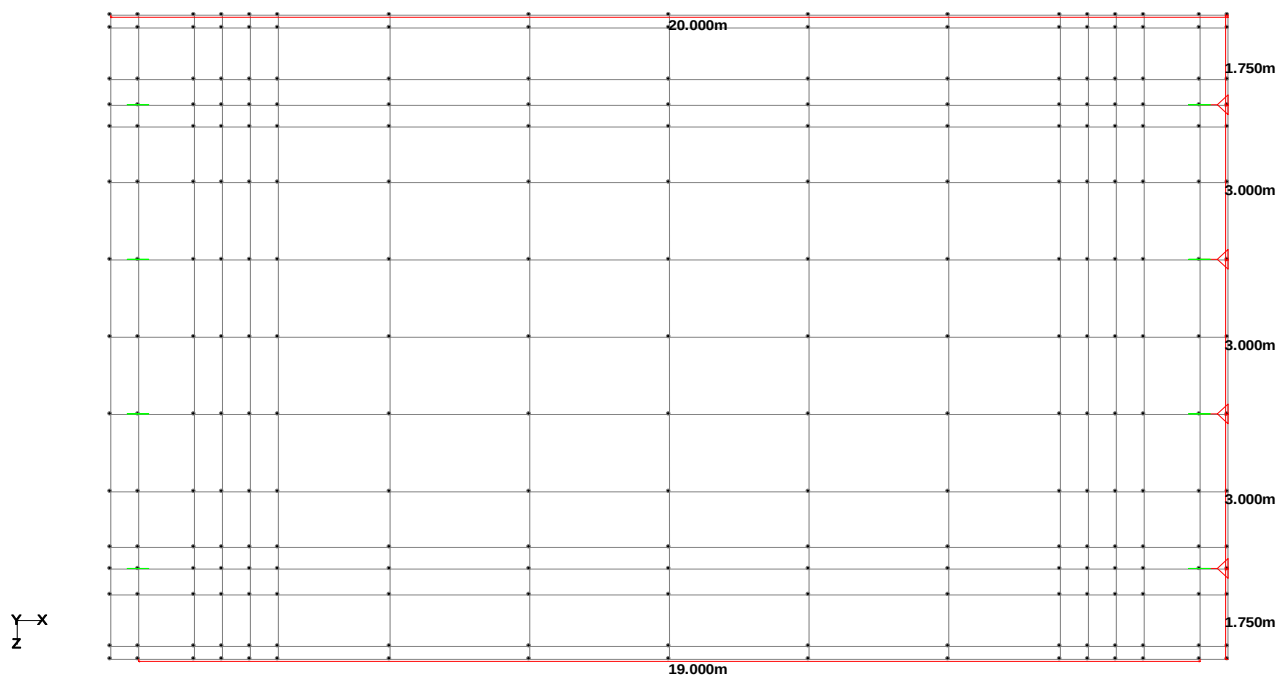
GRILLAGE IDEALISATION

Fig. Showing the Grid Model of Deck Slab with Longitudinal and Cross Girder using Staad-Pro

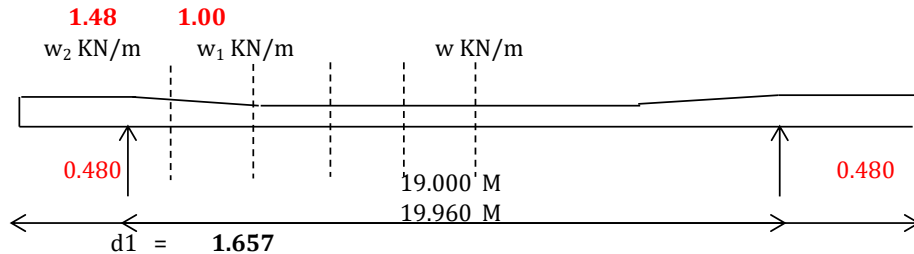
CALCULATION OF MOMENTS AND SHEARS (FOR OUTER GIRDERS)

Analysis for Self wt. of Girder is done by taking it as an individual girder while analysis for SIDL, FPLL and LL are done by GRID analysis using STAAD-Pro.

DEAD LOAD

1. Under self weight of the RCC Outer Girder

Length of the girder = **19.960 m**



Distance Between Support, L = **19 m**
Overhang = **0.480 m**

UDL, w = 18.81 KN/m
Avg., w_1 = 22.74 KN/m
UDL, w_2 = 26.67 KN/m
Supp. Reaction, R = **203.30 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	855.66	0.00
2	2	0.375 L	802.61	44.68
3	3	0.25 L	643.43	89.36
4	4	0.125 L	378.15	134.04
5	At D_{eff}	0.087 L	276.78	148.90
6	At web widening	2.00	326.56	141.09
7	At support	0	-3.072	190.50

2. Due to self-weight of cast-in-situ deck slab

UDL, w = 17.875 KN/m

Avg., w_1 = 17.875 KN/m

UDL, w_2 = 17.875 KN/m

Supp. Reaction, R = **178.393 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	804.55	0.00
2	2	0.375 L	754.14	42.45
3	3	0.25 L	602.90	84.91
4	4	0.125 L	350.83	127.36
5	At D_{eff}	0.087 L	254.73	140.20
6	At web widening	2.00	301.82	134.06
7	At support	0.0	-2.06	169.81

3. 'Additional weight due to green concrete

$$\begin{aligned} \text{Weight of green concrete} &= 26 \text{ kN/m}^3 \\ \text{Additional weight due to green concrete} &= 26 - 25 = 1 \text{ kN/m}^3 \\ &= 0.715 \text{ kN/m of girder} \end{aligned}$$

$$\text{UDL, } w = 0.715 \text{ KN/m (On entire length of girder)}$$

$$\text{Avg., } w_1 = 0.715 \text{ KN/m}$$

$$\text{UDL, } w_2 = 0.715 \text{ KN/m}$$

$$\text{Supp. Reaction, } R = 7.136 \text{ KN}$$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	32.18	0.00
2	2	0.375 L	30.17	1.70
3	3	0.25 L	24.12	3.40
4	4	0.125 L	14.03	5.09
5	At D _{eff}	0.087 L	10.19	5.61
6	At web widening	2.00	12.07	5.36
7	At support	0.0	-0.08	6.79

4. 'Shuttering Load

As per clause 4.2.2.2.2 of IRC:87-1984 superimposed loads

$$= 3.6 \text{ kN/m}^2 = 11.70 \text{ kN/m}$$

$$\text{UDL, } w = 11.700 \text{ KN/m (On entire length of girder)}$$

$$\text{Avg., } w_1 = 11.700$$

$$\text{UDL, } w_2 = 11.700 \text{ KN/m}$$

$$\text{Supp. Reaction, } R = 116.766 \text{ t}$$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	526.61	0.00
2	2	0.375 L	493.62	27.79
3	3	0.25 L	394.62	55.58
4	4	0.125 L	229.64	83.36
5	At D _{eff}	0.087 L	166.74	91.77
6	At web widening	2.00	197.55	87.75
7	At support	0	-1.35	111.15

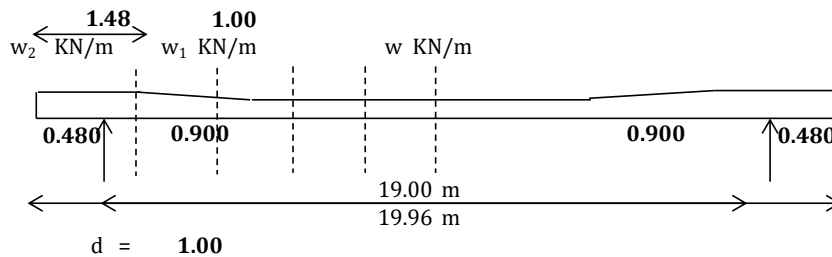
CALCULATION OF MOMENTS AND SHEARS(FOR INTERMEDIATE GIRDERS)

Analysis for Self Wt. of Girder is done by taking it as an individual girder while analysis for SIDL, FPLL and LL are done by GRID analysis using STAAD-Pro.

DEAD LOAD

1. Self weight of the RCC Inner Girder

Length of the girder = **19.960 m**



Distance Between Support, L = **19 m**
Overhang = **0.480 m**

UDL, w = 18.81 KN/m
Avg., w_1 = 22.74 KN/m
UDL, w_2 = 26.67 KN/m
Supp. Reaction, R = **203.30 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	858.63	0.39
2	2	0.375 L	804.64	45.07
3	3	0.25 L	644.53	89.75
4	4	0.125 L	378.31	134.43
5	At D_{eff}	0.087 L	276.78	148.90
6	At web widening	2.00	326.58	141.49
7	At support	0	-3.07	190.50

2. Due to self-weight of cast-in-situ deck slab

UDL, w = 16.500 KN/m (On entire length of girder)

Avg., w_1 = 16.500 KN/m

UDL, w_2 = 16.500 KN/m

Supp. Reaction, R = **164.670 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	742.66	0.00
2	2	0.375 L	696.13	39.19
3	3	0.25 L	556.52	78.38
4	4	0.125 L	323.85	117.56
5	At D_{eff}	0.087 L	235.14	129.42
6	At web widening	2.00	278.60	123.75

7	At support	0.0	-2.1	169.8

3. 'Additional weight due to green concrete

weight of green concrete = 26 kN/m^3
Additional weight due to green concrete = $26 - 25 = 1 \text{ kN/m}^3$
 = 0.66 kN/m of girder
 UDL, $w = 0.660 \text{ KN/m}$ (On entire length of girder)
 Avg., $w_1 = 0.660 \text{ KN/m}$
 UDL, $w_2 = 0.660 \text{ KN/m}$
 Supp. Reaction, $R = 6.587 \text{ KN}$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	29.71	0.00
2	2	0.375 L	27.85	1.57
3	3	0.25 L	22.26	3.14
4	4	0.125 L	12.95	4.70
5	At D_{eff}	0.087 L	9.41	5.18
6	At web widening	2.00	11.14	4.95
7	At support	0	-0.082	6.793

4. 'Shuttering Load

As per clause 4.2.2.2.2 of IRC:87-1984 superimposed loads
 = $3.6 \text{ kN/m}^2 = 10.80 \text{ kN/m}$
 UDL, $w = 10.800 \text{ KN/m}$ (On entire length of girder)
 Avg., $w_1 = 10.800 \text{ KN/m}$
 UDL, $w_2 = 10.800 \text{ KN/m}$
 Supp. Reaction, $R = 107.784 \text{ KN}$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	486.11	0.00
2	2	0.375 L	455.65	25.65
3	3	0.25 L	364.27	51.30
4	4	0.125 L	211.97	76.95
5	At D_{eff}	0.112 L	192.64	79.62
6	At web widening	2.00	182.41	81.00
7	At support	0	-1.35	111.15

Reaction over each Bearing excluding the weight of cross girder

Bearing No.	Force	DL (KN)	SIDL	Surfacing	3 Lane CWLL Reaction Max	3 Lane CWLL Reaction Min	SV Loading Reaction Max	SV Loading Reaction Min
1	FY	381.70	126.017	67.792	398.857	-3.342	0.625	-43.416
2	FY	367.97	-41.017	60.608	538.7	-11.639	493.461	-11.639
3	FY	367.97	-41.017	60.608	600.7	-2.874	904.351	-2.874
4	FY	381.70	126.017	67.792	238.508	-29.069	101.365	-29.069

Summary of BM & Shear Forces (Outer Girder - OG1)

Postion on member	Distance from Start of Girder	DL -Precast Girder		DL Deck Slab		DL -Green Weight of Slab		Constructional Load		SIDL		Surfacing		FPLL		Removal of Constr. Load		Live Load (70-R +CLASS -A)		Live Load(SV Loading)		DL+SIDL+LL	
		Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Max. SF	Max. BM	Max SF	Max BM		
		m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN
L/2	9.500	0.00	855.66	0.00	804.55	0.00	32.18	0.00	526.61	11.72	393.66	6.79	310.63	0.00	0.00	0.00	526.61	71.96	1788.05	27.46	1240.11	90.46	4152.55
3L/8	7.125	44.68	802.61	42.45	754.14	1.70	30.17	27.79	493.62	35.24	367.14	22.44	291.14	0.00	0.00	27.79	493.62	161.21	1604.18	26.86	1172.33	306.02	3819.20
L/4	4.750	89.36	643.43	84.91	602.90	3.40	24.12	55.58	394.62	58.36	284.07	38.04	232.60	0.00	0.00	55.58	394.62	232.64	1168.29	78.43	965.76	503.31	2931.29
L/8	2.375	134.04	378.15	127.36	350.83	5.09	14.03	83.36	229.64	68.02	137.42	48.15	132.64	0.00	0.00	83.36	229.64	230.08	590.16	123.24	637.33	607.64	1589.21
At D _{eff}	2.000	148.90	276.78	140.20	254.73	5.61	10.19	91.77	166.74	71.10	103.36	50.97	108.47	0.00	0.00	91.77	166.74	232.04	476.72	154.88	541.74	643.21	1220.06
At Web widening	1.000	141.09	326.56	134.06	301.82	5.36	12.07	87.75	197.55	73.965	67.679	53.725	82.83	0.00	0.00	87.75	197.55	270.36	229.96	165.40	374.08	673.21	1008.84
At support	0.000	190.50	-3.07	169.81	-2.06	6.79	-0.08	111.15	-1.35	82.505	34.578	58.53	56.19	0.00	0.00	111.15	-1.35	270.36	-0.04	173.58	185.76	771.71	85.64

Summary of Torsion (Outer Girder - OG1)

Postion on member	Distance from Start of Girder	SIDL	Surfacing	FPLL	Live Load(70-R + class- a)	Live Load(SV Loading)
	m	kNm	kNm	kNm	kNm	kNm
L/2	9.500	4.776	0.204	0	8.301	15.864
3L/8	7.125	15.061	0.67	0	9.033	15.963
L/4	4.750	28.905	1.398	0	10.887	46.921
L/8	2.375	40.688	1.997	0	3.209	77.278
At D _{eff}	2.000	46.817	2.31	0	2.633	95.987
At Web widening	1.000	51.949	2.565	0	12.165	112.28
At support	0.000	67.086	3.661	0	0.035	129.98

Summary of BM & Shear Forces (Center Girder - CG2)

Postion on member	Distance from Start of Girder	DL -Precast Girder		DL -Deck Slab		DL -Green Weight of Slab		Constructional Load (Shuttering)		SIDL		Surfacing		FPLL		Removal of Constr. Load		Live Load (class A+ 70-R)		Live Load(SV Loading)		DL+SIDL+LL	
		Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Shear	BM	Max. SF	Max. BM	Max SF	Max BM		
		m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m	KN
L/2	9.500	0.39	858.63	0.00	742.66	0.00	29.71	0.00	486.11	6.88	40.95	4.54	290.18	0.00	0.00	0.00	486.11	6.24	1795.12	12.98	2654.75	58.86	4587.2
3L/8	7.125	45.07	804.64	39.19	696.13	1.57	27.85	25.65	455.65	6.14	40.35	9.79	272.26	0.00	0.00	25.65	455.65	166.15	1574.49	155.67	2488.58	290.08	4302.0
L/4	4.750	89.75	644.53	78.38	556.52	3.14	22.26	51.30	364.27	7.28	38.28	18.88	272.17	0.00	0.00	51.30	364.27	218.90	1091.38	340.72	1986.40	566.01	3497.9
L/8	2.375	134.43	378.31	117.56	323.85	4.70	12.95	76.95	211.97	7.65	35.37	24.13	218.39	0.00	0.00	76.95	211.97	230.56	554.96	484.76	1053.65	796.25	2009.6
At D _{eff}	2.000	148.90	276.78	129.42	235.14	5.18	9.41	79.62	192.64	8.56	17.83	33.29	218.22	0.00	0.00	79.62	192.64	275.24	430.25	536.23	817.13	865.67	1565.1
At Web widening	2.000	141.49	326.58	123.75	278.60	4.95	11.14	81.00	182.41	5.364	9.864	38.541	128.4	0.00	0.00	81.00	182.41	279.29	294.96	535.86	555.35	849.50	1298.8
At support	0.000	190.50	-3.07	169.81	-2.06	6.79	-0.08	111.15	-1.35	5.364	8.45	44.71	127	0.00	0.00	111.15	-1.35	286.00	120.39	622.69	307.92	1036.16	438.3

Summary of Torsion (Center Girder - CG2)

Postion on member	Distance from Start of Girder	SIDL	Surfacing	FPLL	Live Load (class A +70 R)	Live Load(SV Loading)
	m	kNm	kNm	kNm	kNm	kNm
L/2	9.500	0.902	0.018	0	6.046	3.771
3L/8	7.125	2.465	0.018	0	11.013	10.909
L/4	4.750	3.844	0.028	0	12.386	24.439
L/8	2.375	3.844	0.028	0	10.291	39.226
At D _{eff}	2.000	6.798	0.02	0	10.913	41.68
At Web widening	1.000	7.652	0.02	0	8.1	45.723
At support	0.000	9.353	0.002	0	1.695	53.574



ANALYSIS OF LONG GIRDER FOR TEMPERATURE**Effective Bridge Temperature****INPUT DATA**

Location	-			
Maximum Air shade Temperature	-	47.5 °C		
Minimum Air shade Temperature	-	0 °C		
Mean of Max and Min Temp	-	23.75 °C		
Bridge Temperature assumed	-	33.75 °C		
Mean compressive stress	f_{ck}	=	35 N/mm ²	
Elastic Modulus of concrete	E_c	=	32000 N/mm ²	1.60E+07 KN/m ²
Coefficient of Thermal expansion	α	=	1.20E-05	As per Clause 6.4.2.5 Pt.4 of IRC 112:2011

Sectional Properties of Composite Section

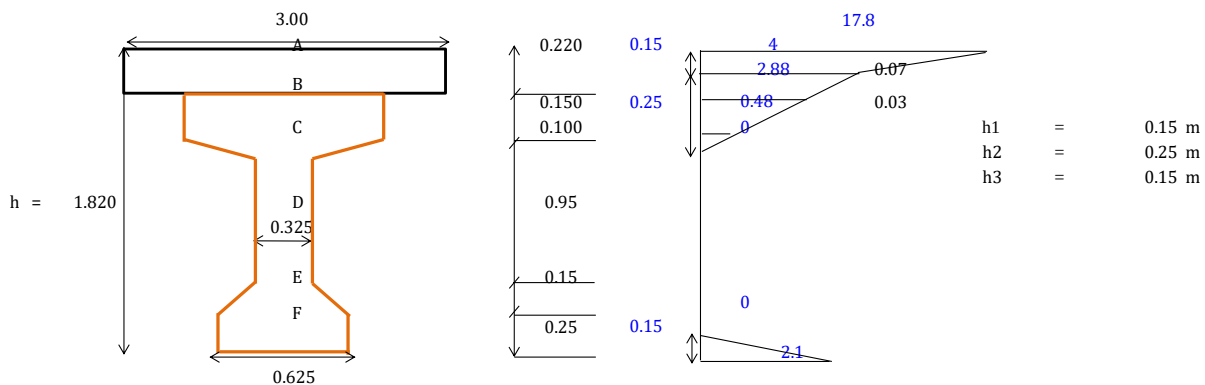
Area of Composite Section	A	=	1.413 m ²
MOI of Composite Section	I_z	=	0.483 m ⁴

CG of the Section

From top of Slab	y_{ts}	=	0.57 m
From top of Girder	y_{tg}	=	0.35 m
From bottom of Girder	y_{bg}	=	1.25 m

Modulii of Section

	Z_{ts}	=	0.845 m ³
	Z_{tg}	=	1.373 m ³
	Z_{bg}	=	0.387 m ³

Rising Gradient at MidSpan

Portion	B (m)	D	A	y from top	A y	A y ²	t	A t	A t y
	m	m	m ²	m	m ³	m ⁴	°C	m ² °C	m ³ °C
A	3.00	0.22	0.660	0.110	0.073	0.008	10.340	6.824	0.751
B	1.00	0.15	0.150	0.295	0.044	0.013	1.680	0.252	0.074
C	0.6625	0.1	0.066	0.403	0.027	0.011	0.240	0.016	0.006
D	0.325	0.95	0.309	0.945	0.292	0.276	0.000	0.000	0.000
E	0.475	0.15	0.071	1.470	0.105	0.154	0.000	0.000	0.000
F	0.625	0.25	0.156	1.745	0.273	0.476	1.050	0.164	0.286
SUM			1.413		0.813	0.937		7.256	1.118

d	=	Depth of respective member
B	=	Width of respective member
A	=	Area of respective zone
y	=	c.g. distance considered from top
t	=	Average temperature as per Temperature gradient of IRC 6
α	=	Coefficient of Thermal expansion as per IRC: 6-2010 Cl 215.4
α	=	1.20E-05

$$\varepsilon^o \cdot \sum A - \theta \cdot \sum A y = \alpha \cdot \sum A t \quad \text{----- Eq.1}$$

$$\varepsilon^o \cdot \sum A y - \theta \cdot \sum A y^2 = \alpha \cdot \sum A y t \quad \text{----- Eq.2}$$

Eigen Stress

$$f_{ci} = E_c (\epsilon^o - (y \cdot \theta + \alpha t))$$

Above Equation Becomes

$$1.413 \epsilon^o - 0.813 \theta = 8.7E-05 \quad \text{Eq.1}$$

$$0.813 \epsilon^o - 0.937 \theta = 1.3E-05 \quad \text{Eq.2}$$

Multiplying eq. 1 by 0.813 & eq. 2 by 1.413 and subtract 1 and 2

$$\begin{array}{rclclcl} 1.148 \epsilon^o & - & 0.661 & \theta & = & 7.1E-05 \\ 1.148 \epsilon^o & - & 1.324 & \theta & = & 1.9E-05 \\ \hline 0 & & 0.663 & \theta & = & 5.2E-05 \end{array}$$

By solving the above two equations

$$\text{Rotation of plane } \theta = 7.81E-05$$

$$\text{Strain at Top Fibre } \epsilon^o = 1.07E-04$$

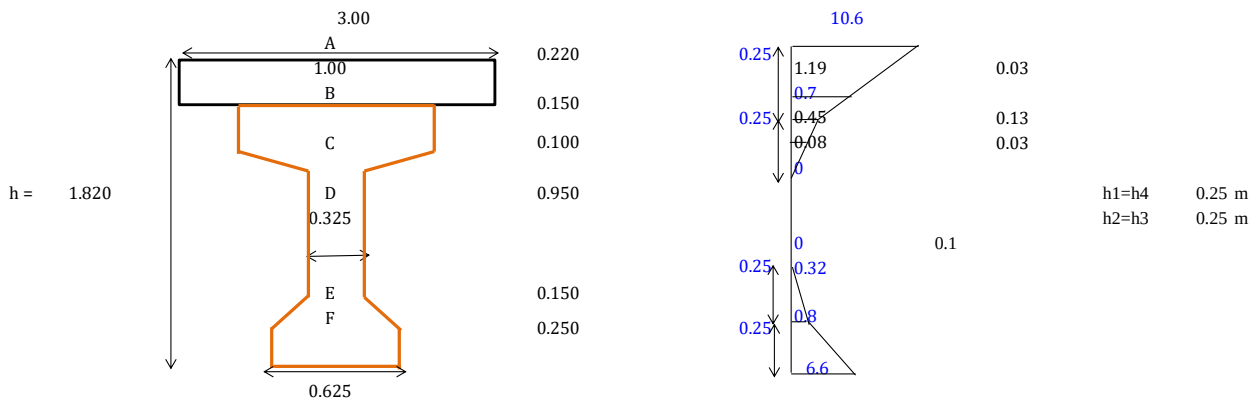
$$f_{ck} = 35 \text{ N/mm}^2$$

$$E_c = 3.20E+04 \text{ N/mm}^2$$

S.No	y	y. θ	t	α. t	f _{ci}
1	0	0.00E+00	17.8	2.14E-04	-3.424
2	0.22	1.72E-05	2.88	3.46E-05	1.755
3	1.82	1.42E-04	2.1	2.52E-05	-0.002

Note : f_{ci} is in Mpa, - ve is Compression

Falling Gradient at Mid Span



Portion	B (m)	D	A	y from top	A y	A y ²	t	A t	A t y
	m	m	m ²	m	m ³	m ⁴	°C	m ² °C	m ³ °C
A	3.00	0.22	0.660	0.110	0.073	0.008	5.894	3.890	0.428
B	1.000	0.15	0.150	0.295	0.044	0.013	0.574	0.086	0.025
C	0.6625	0.1	0.066	0.403	0.027	0.011	0.266	0.018	0.007
D	0.325	0.95	0.309	0.945	0.292	0.276	0.000	0.000	0.000
E	0.475	0.15	0.071	1.470	0.105	0.154	0.560	0.040	0.059
F	0.625	0.25	0.156	1.745	0.273	0.476	3.700	0.578	1.009
SUM			1.413		0.813	0.937		4.612	1.528

Equation 1 and 2 becomes

$$1.413 \epsilon^o - 0.813 \theta = 5.5E-05 \quad \text{Eq.1}$$

$$0.813 \epsilon^o - 0.937 \theta = 1.8E-05 \quad \text{Eq.2}$$

Multiplying eq. 1 by 0.813 & eq. 2 by 1.413 and subtract 1 and 2

$$\begin{array}{rclclcl} 1.148 \epsilon^o & - & 0.661 & \theta & = & 4.5E-05 \\ 1.148 \epsilon^o & - & 1.324 & \theta & = & 2.6E-05 \\ \hline 0 & & 0.663 & \theta & = & 1.9E-05 \end{array}$$

$$\text{Rotation of plane } \theta = 3E-05$$

$$\text{Strain at Top Fibre } \epsilon^o = 6E-05$$

S.No	y	y. θ	t	α. t	f _{ci}
1	0	0.00E+00	10.6	1.27E-04	-3.150
1	0.22	6.33E-06	0.448	5.38E-06	1.409
2	1.82	5.23E-05	6.6	7.92E-05	-2.426

Note : f_{ci} is in Mpa, - ve is Tension

CREEP AND SHRINKAGE FOR COMPOSITE CONCRETE CONSTRUCTION

Shrinkage parameters are calculated for Precast beam and Cast in situ slab to account for the differential shrinkage strain.

The hogging restrain moment M_{cs} due to differential shrinkage is given by

$$F_{cs} = 290.322 \text{ KN}$$

$$M_{cs} = (\varepsilon_{diff})(E_{cf})A_{cf}(\delta_{cent})\alpha = 128.53 \text{ KN-m}$$

$$\begin{aligned} \text{Area of Composite Section} & \quad A = 1.4125 \text{ m}^2 \\ \text{MOI of Composite Section} & \quad I_z = 0.482996 \text{ m}^4 \end{aligned}$$

CG of the Section

$$\begin{aligned} \text{From top of Slab} & \quad y_{ts} = 0.572 \text{ m} \\ \text{From top of Girder} & \quad y_{tg} = 0.352 \text{ m} \\ \text{From bottom of Girder} & \quad y_{bg} = 1.248 \text{ m} \end{aligned}$$

Moduli of Section

$$\begin{aligned} Z_{ts} & = 0.84455 \text{ m}^3 \\ Z_{tg} & = 1.37255 \text{ m}^3 \\ Z_{bg} & = 0.38698 \text{ m}^3 \end{aligned}$$

Stresses

SNO	Location	Stress due to Direct Tensile Force in Deck Slab(N/mm ²)	Stress due to Moments in composite section(N/mm ²)	Stress due Compressive force in composite section(N/mm ²)	Net Stresses (N/mm ²)
1	Top of Slab	-0.4399	0.1522	0.2055	-0.0822
2	Top of Girder	0.0000	0.0936	0.2055	0.2992
3	Bottom of Girder	0.0000	0.3321	0.2055	0.5377

$$\alpha = \frac{I - e^2}{\phi} = 0.37$$

$$\phi = 2.5$$

			Precast Girder	In-situ Slab	
Compressive strength	f_{ck}	=	35	35	N/mm ²
Cross sectional area of concrete	A_c	=	752500	660000	mm ²
Perimeter of the cross section	u	=	4538	2440	mm
Notional size of the cross section(h_o)		=	331.6	541.0	mm
Coefficient depending on notional size	K_h	=	0.74	0.7	
Relative humidity	RH	=	75	75	%
Drying Shrinkage strain		=	2.76E-04	2.76E-04	

Refer Table 6.7 of IRC:112-2011

$$\begin{aligned} \text{Final value of Drying shrinkage strain} & = \varepsilon_{cd, \infty} = 2.04\text{E-}04 \quad 1.93\text{E-}04 \\ \text{Autogenous shrinkage strain} & = \varepsilon_{ca} = 4.50\text{E-}05 \quad 4.50\text{E-}05 \end{aligned}$$

$$\text{Total Shrinkage strain} = \varepsilon_{cd} + \varepsilon_{ca} = 2.49\text{E-}04 \quad 2.38\text{E-}04$$

Time Dependent Shrinkage of Precast Girder

$$\text{Autogenous shrinkage with time } t = 28 \text{ days}$$

$$\varepsilon_{ca}(t) = \beta_{as}(t) \cdot \varepsilon_{ca} = 2.94\text{E-}05$$

$$\beta_{as}(t) = 1 - \exp(-0.2\sqrt{t}) = 0.653$$

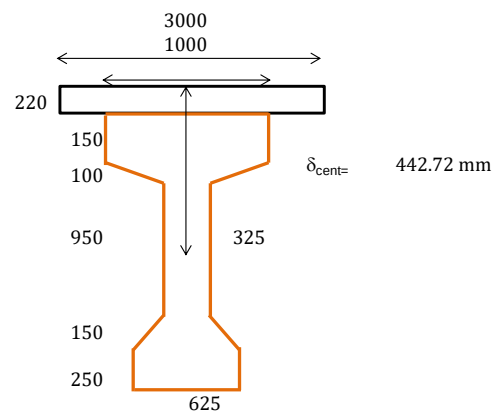
Drying Shrinkage with time

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd} = 1.92\text{E-}05$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04\sqrt{h_o}^3} = 0.094$$

$$\text{Total Shrinkage at time} = 4.85\text{E-}05$$

$$\varepsilon_{diff} = \varepsilon_{slab, \infty} - (\varepsilon_{girder, \infty} - \varepsilon_{girder, t}) = 3.75\text{E-}05$$



DESIGN OF OUTER (OG1) GIRDER

ULS DESIGN OF LONGITUDINAL MEMBERS FOR TORSION

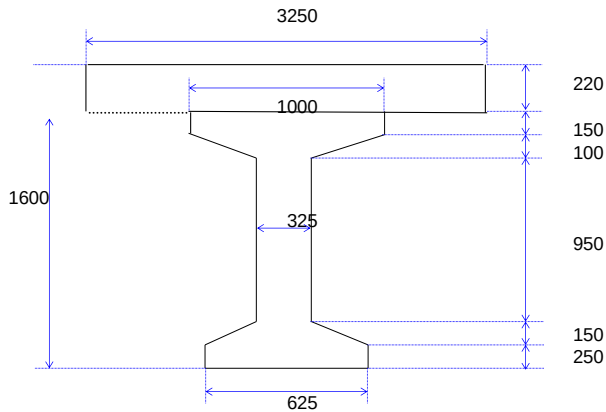
ULTIMATE BENDING MOMENT AND SHEAR FORCE

Bending Moment (kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	855.66	802.61	643.43	378.15	276.78	326.56
DL of Slab	1.35	804.55	754.14	602.90	350.83	254.73	301.82
SIDL	1.35	393.66	367.14	284.07	137.42	103.36	67.68
Surfacing	1.75	310.63	291.14	232.60	132.64	108.47	82.83
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	2109.90	1892.93	1378.59	696.39	562.52	271.35
TOTAL M _u		6481.17	5946.14	4540.96	2446.35	1890.69	1491.65

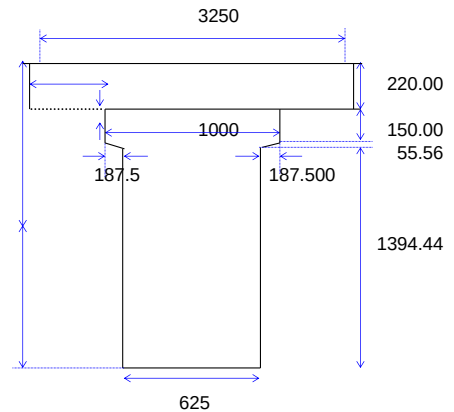
Shear Force (KN)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.00	44.68	89.36	134.04	148.90	141.09
DL of Slab	1.35	0.00	42.45	84.91	127.36	140.20	134.06
SIDL	1.35	11.72	35.24	58.36	68.02	71.10	73.97
Surfacing	1.75	6.79	22.44	38.04	48.15	50.97	53.73
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	84.91	190.23	274.51	0.00	273.81	319.02
TOTAL M _u		155.06	489.81	792.39	528.97	986.18	1043.87

Torsional Moment(kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.00	0.00	0.00	0.00	0.00	0.00
DL of Slab	1.35	0.00	0.00	0.00	0.00	0.00	0.00
SIDL	1.35	4.78	15.06	28.91	40.69	46.82	51.95
Surfacing	1.75	0.20	0.67	1.40	2.00	2.31	2.57
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	9.80	10.66	12.85	3.79	3.11	14.35
TOTAL M _u		21.50	37.49	60.74	64.10	71.91	96.15

Equivalent Section at Mid-Span



Equivalent Section at Support



Calculation of Section at mid span:

	width	depth	Area	perimeter	tef,i	eff.depth	Ak	uk
Deck Slab	3.250	0.220	0.715	6.109	0.136	0.084	0.262	5.565
Top Flange	0.831	0.250	0.208	1.006	0.207	0.043	0.027	0.712
Web	0.325	0.950	0.309	1.900	0.163	0.788	0.128	1.575
Bottom Flange	0.550	0.400	0.220	1.575	0.140	0.260	0.107	0.000
					0.147		0.524	7.851

Total Torsional inertia of section at mid-span = 0.0233 m⁴
 A for composite section = 1.468 m²
 perimeter u for composite section = 10.765 m
 Thickness of web = 0.325 m

Calculation of Section at Support:

	width	depth	area	perimeter	tef,i	eff.depth	Ak	uk
Deck Slab	3.250	0.220	0.715	6.034	0.136	0.084	0.262	5.490
Top Flange	0.906	0.206	0.186	0.692	0.269	0.000	0.000	0.000
Web	0.625	1.394	0.872	3.414	0.255	1.139	0.421	2.648
					0.210		0.683	8.138

Total Torsional inertia of section at support	=	0.095 m ⁴
A for composite section	=	1.782 m ²
perimeter u for composite section	=	9.765 m
Thickness of web	=	0.625 m

Material Stress

fck	=	35 N/mm ²	fcd	=	15.63 N/mm ²
fyk	=	500 N/mm ²	fyd	=	434.78 N/mm ²
fctm	=	2.77 N/mm ²	Es	=	2.00E+05 N/mm ²

ULS TORSIONAL MOMENT CAPACITY OF OF LONGITUDINAL SECTION

		fck =	40.00	N/mm ²	fcd =	17.87	N/mm ²
		fyd =	434.78	N/mm ²			
			L/2	3L/8	L/4	L/8	Web Widening
Ultimate Torsion Moment , T _{Ed}	kNm	21.50	37.49	60.74	64.10	71.91	96.15
effective cover	m	0.163	0.163	0.163	0.142	0.110	0.110
t _{ef,i} = max(A / u, 2*eff. Cover)	m	0.147	0.147	0.147	0.147	0.210	0.210
Area of section , Ak	m ²	0.524	0.524	0.524	0.524	0.603	0.603
Perimeter of section , uk	m	7.851	7.851	7.851	7.851	7.995	7.995
Shear stress due to pure Torsion, τ _{t,i} = T _{Ed} / (2 .Ak. t _{ef,i})	N/mm ²	0.140	0.244	0.395	0.417	0.284	0.380
Length of section , Zi	m	0.325	0.325	0.325	0.325	0.625	0.625
Shear force due to torsion =V _{Ed,i} = τ _{t,i} : t _{ef} . Zi	kN	6.67	11.64	18.85	19.90	37.26	49.82
Mean compr. stress at c.g. sec. σ _{cp} = N _{Ed} / Ac ≤ 0.2 fcd	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000
Coefficient α _{cw} =1+σ _{cp} /fcd		1.000	1.000	1.000	1.000	1.000	1.000
strength reduction factor for section cracked in shear = v = 0.6 [1 - fck/310]		0.52	0.52	0.52	0.52	0.52	0.52
cot θ		2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22
Maximum Torsional Moment T _{Rd,max} = 2 . v . α _{cw} . fcd .Ak. tef. sinθ . cosθ	kNm	495.16	495.16	495.16	495.16	813.95	813.95
Torsional Long. Reinf. A _{sl} = T _{Ed} . cot θ/(2 . Ak. fyd)	mm ²	926.91	1616.62	2618.88	2763.98	2740.25	3664.24
Torsional reinf. In web on each face	mm ²	92.97	162.15	262.67	277.23	477.96	639.13
Torsional reinf. In bottom flange	mm ²	0.00	0.00	0.00	0.00	214.23	286.46
Torsional reinf. In top flange	mm ²	84.02	146.54	237.39	250.54	0.00	0.00
Total torsional steel in girder		176.99	308.68	500.06	527.77	692.19	925.59
Total torsional moment, T _{Rd,max}	kNm	495.16	495.16	495.16	495.16	813.95	813.95
Total shear force due to torsion, V _{Edi}	kN	6.67	11.64	18.85	19.90	37.26	49.82
Design shear force, Vu	kN	155.1	489.8	792.4	529.0	986.2	1043.9
Total shear Force , V _{Ed} =V _u +V _{Edi}	kN	161.73	501.45	811.24	548.87	1023.43	1093.69
Maximum shear capacity V _{Rd,max}	kN	1359.21	1359.21	1359.21	1379.39	2710.90	2710.90
T _{Ed} /T _{Rd,max} + V _{Ed} / V _{Rd,max} ≤ 1.0		0.162	0.445	0.720	0.527	0.466	0.522
Check		OK	OK	OK	OK	OK	OK

Characteristic yield stress of steel , f_{yk}	=	500 N/mm ²	
Design yield strength of shear reinforcement, $f_{ywd} = 0.8 \cdot f_{yk}$	=	400 N/mm ²	Refer clause 10.3.3.2 of IRC:112-2011
Strength reduction factor , v_1	=	0.6	Refer clause 10.3.3.2 of IRC:112-2011
Design Compressive strength of concrete , f_{cd}	=	15.63 N/mm ²	

Overall Depth , D	mm	1600.0	1600.0	1600.0	1600.0	1600.0	1600.0
Effective cover	mm	163.3	163.3	163.3	142.0	110.0	110.0
Effective depth , d	mm	1436.7	1436.7	1436.7	1458.0	1490.0	1490.0
Width of web , b_w	mm	325.00	325.00	325.00	325.00	625.00	625.00
Coefficient , α_{cw} for $\sigma_{cp} = 0$		1.00	1.00	1.00	1.00	1.00	1.00
Lever arm , $Z = 0.9 d$	mm	1293.00	1293.00	1293.00	1312.20	1341.00	1341.00
$1 \geq \cot \theta \leq 2.5$		2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22
$V_{Rd,max}$	KN	1359.21	1359.21	1359.21	1379.39	2710.90	2710.90

ULS DESIGN OF LONGITUDINAL GIRDER**ULS DESIGN****Material Properties**

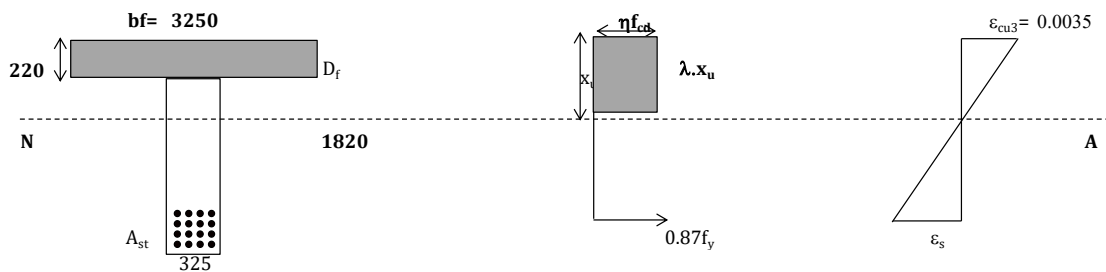
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

LOADINGS**ULS Moments**

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1.35	855.66	802.61	643.43	378.15	276.78	326.56
2	DL of Slab	1.35	804.55	754.14	602.90	350.83	254.73	301.82
3	SIDL	1.35	393.66	367.14	284.07	137.42	103.36	67.68
4	Surfacing	1.75	310.63	291.14	232.60	132.64	108.47	82.828
5	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
6	Live Load with impact	1.5	2109.90	1892.93	1378.59	696.39	562.52	271.35
Total Design Moment M_{ED}			6481.17	5946.14	4540.96	2446.35	1890.69	1491.65

* γ – Partial safety factor**At Midspan(L/2)**

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

Area of reinforcement provided A_{st} = **14476.5 mm²** Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = **163.33 mm**

Therefore, d_{eff} = **1.657 m**

Assuming N.A lies in the flange

$$b_f \cdot \eta f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 123.94 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 154.93 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu3}}\right)} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned}
 M_{ED} &= M_{u,lim} \\
 6.48E+09 &= \eta \cdot f_{cd} \cdot \frac{x_{u,max}}{d} \cdot (1 - 0.4 \cdot \frac{x_{u,max}}{d}) \cdot b_f \cdot d^2 \\
 d_{reqd} &= 523.99 \text{ mm} \quad \text{OK}
 \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot (d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}})$$

$$\begin{aligned}
 \text{due to flexure} \quad A_{st,Required} &= 9212.82 \text{ mm}^2 \\
 \text{min steel required} \quad A_{s,min} &= 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 783.93 \text{ mm}^2 \\
 &= 0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2 \\
 \text{due to torsion} \quad A_{st,torsion} &= 176.99 \text{ mm}^2 \\
 \text{Total Ast required due to flexure+torsion} &= 9389.81 \text{ mm}^2 \\
 b_t &- \text{Mean width of tension Zone} \\
 \text{Moment Resisting capacity } M_R &= 10042.214 \text{ KN-m} \quad \text{Safe}
 \end{aligned}$$

At SECTION 2-2 (3L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

$$\text{Area of reinforcement provided } A_{st} = 14476.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 163.33 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.657 \text{ m}$$

Assuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned}
 \eta &= 1.00 \\
 \lambda x_u &= 123.94 \text{ mm} \quad \text{N.A lies inside the flange} \\
 \lambda &= 0.8 \\
 x_u &= 154.93 \text{ mm}
 \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}})} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned}
 M_{ED} &= M_{u,lim} \\
 5.95E+09 &= \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max}) \\
 d_{reqd} &= 523.2345 \text{ mm} \quad \text{OK}
 \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}} \right)$$

$$A_{st, Required} = 8434.92 \text{ mm}^2$$

$$A_{s, min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 783.93 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, torsion} = 308.68 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 8743.60 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 10042.214 \text{ KN-m} \quad \text{Safe}$$

At SECTION 3-3 (L/4)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

$$\text{Area of reinforcement provided } A_{st} = 14476.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 163.33 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.657 \text{ m}$$

Asssuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\eta = 1.00$$

$$\lambda x_u = 123.94 \text{ mm}$$

N.A lies inside the flange

$$\lambda = 0.8$$

$$x_u = 154.93 \text{ mm}$$

$$x_{u, max} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

$$\frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu, 3}} \right)}$$

Check for Depth Required

$$M_{ED} = M_{u, lim}$$

$$4.54E+09 = \eta f_{cd} b_f x_{u, max} (d - 0.4 x_{u, max})$$

$$d_{reqd} = 496.17 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}} \right)$$

$$A_{st, Required} = 6407.28 \text{ mm}^2$$

$$A_{s, min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 783.93 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, torsion} = 500.06 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 6907.34 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 10042.214 \text{ KN-m} \quad \text{Safe}$$

At SECTION 4-4 (L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	15		

Area of reinforcement provided A_{st} = **12063.7 mm²** Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = **142.00 mm**

Therefore, d_{eff} = **1.678 m**

Asssuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 103.28 \text{ mm} && \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 129.11 \text{ mm} \end{aligned}$$

$$x_{u,max} = 1034.89 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= M_{u,lim} \\ 2.45E+09 &= \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max}) \\ d_{reqd} &= 460.48 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}} \right)$$

$$A_{st,Required} = 3380.65 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 794.03 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 708.96 \text{ mm}^2$$

due to torsion $A_{st,torsion} = 527.77 \text{ mm}^2$

Total A_{st} required due to flexure+torsion = **3908.41 mm²**

Moment Resisting capacity $M_R = 8534.664 \text{ KN-m}$ **Safe**

At Deff

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	0	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	10		

$$\text{Area of reinforcement provided } A_{st} = 8042.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 110.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.710 \text{ m}$$

Asssuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\eta = 1.00$$

$$\lambda \cdot x_u = 68.86 \text{ mm}$$

$$\lambda = 0.8$$

$$x_u = 86.07 \text{ mm}$$

N.A lies inside the flange

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} = 1054.63 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$M_{ED} = M_{u,lim}$$

$$1.89\text{E}+09 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 457.13 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$A_{st,Required} = 2558.14 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 809.17 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 722.48 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 692.19 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 3250.33 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5861.9504 \text{ KN-m} \quad \text{Safe}$$

At Web Widening

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	0	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	10		

$$\text{Area of reinforcement provided } A_{st} = 8042.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 110.00 \text{ mm}$$

$$\text{Therefore, } d_{\text{eff}} = 1.710 \text{ m}$$

Asssuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 68.86 \text{ mm} & \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 86.07 \text{ mm} \end{aligned}$$

$$x_{u,\text{max}} = 1054.63 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{\text{ED}} &= \frac{d_{\text{eff}}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} \\ 1.49\text{E}+09 &= \eta f_{cd} b_f x_{u,\text{max}} (d - 0.4 x_{u,\text{max}}) \\ d_{\text{reqd}} &= 449.69 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$\begin{aligned} A_{\text{st,Required}} &= 2015.48 \text{ mm}^2 \\ A_{s,\text{min}} &= 0.26 \cdot \frac{f_{cm}}{f_{yk}} \cdot b_t \cdot d_{\text{eff}} = 809.17 \text{ mm}^2 \\ &= 0.0013 \cdot b_t \cdot d_{\text{eff}} = 722.48 \text{ mm}^2 \end{aligned}$$

$$\text{due to torsion} \quad A_{\text{st,torsion}} = 925.59 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 2941.07 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5861.9504 \text{ KN-m} \quad \text{Safe}$$

Summary

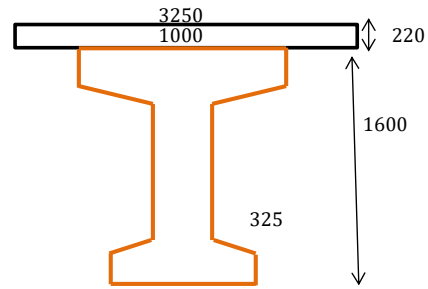
SNO	Description	unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Design Bending Moment M_{ED}	KN-m	6481.17	5946.14	4540.96	2446.35	1890.69	1491.65
2	Area of Steel Required A_{st} (flexure+torsion)	mm ²	9389.81	8743.60	6907.34	3908.41	3250.33	2941.07
3	Area of Steel Provided	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
	Check for steel provided		OK	OK	OK	OK	OK	OK
3	Depth of Neutral Axis x_u	mm	154.93	163.33	163.33	142.00	110.00	86.07
4	Critical Depth of N.A $x_{u,\text{max}}$	mm	1021.73	1021.73	1021.73	1034.89	1054.63	1054.63
5	Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
6	Total Depth required D_{reqd}	mm	663.99	663.23	636.17	600.48	597.13	589.69
7	Total Depth Provided	mm	1820.00	1820.00	1820.00	1820.00	1820.00	1820.00
9	Moment of Resistance M_R	KN-m	10042.21	10042.21	10042.21	8534.66	5861.95	5861.95
	Check for Moment Capacity		OK	OK	OK	OK	OK	OK
10	Additional tensile Force ΔF_d	KN	45.23	153.01	251.65	303.82	321.61	336.60
11	$M_{\text{ED}}/Z + \Delta F_d$	KN	4346.86	3988.019	3045.5792	1640.746	1268.0661	1000.4379
12	M_{RD}/Z	KN	6735.221	6735.221	6735.2209	5724.121	3931.5563	3931.5563
	Check (As per 16.5.1.3)		OK	OK	OK	OK	OK	OK

DESIGN OF LONGITUDINAL GIRDER FOR SHEAR**Material Properties**

Grade of Concrete	-	M	35	
Grade of Reinforcement	-	Fe	500	
Elastic Modulus of Concrete	-		32000	N/mm ²
Elastic Modulus of Steel	-		200000	N/mm ²
f_y	=		500	N/mm ²
f_{yd}	=		435	N/mm ²
f_{ck}	=		35	N/mm ²
f_{ctm}	=		2.8	N/mm ²
f_{cd}	=		15.63	N/mm ²

Refer 10.3.2 -

IRC: 112-2011

* γ - Partial Safety factor

$$\text{Shear force due to flexure + torsion} = 1023.43 \text{ KN}$$

$$\text{Bending Moment at critical section} = 1890.69 \text{ KN-m}$$

$$\tan \beta = 0.00$$

$$\text{Effective Depth } d_{eff} = 1710.00 \text{ m}$$

$$\text{Net shear force } S - M \cdot \tan \beta / d' = 1023.43 \text{ KN}$$

$$\text{Hence, Shear stress} = \frac{1.730 \text{ N/mm}^2}{1730.24 \text{ KN/m}^2}$$

$$\text{Design Shear resistance of the member (without Shear R/F)} \quad V_{Rd,c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp} \right] b_w d_{eff} = 297 \text{ KN}$$

$$\text{Minimum Shear resistance} \quad v_{Rd,c} = (v_{min} + 0.15\sigma_{cp}) b_w d_{eff} = 158 \text{ KN}$$

$$\text{Mean Compressive stress} \quad \sigma_{cp} = 0.2f_{cd} = 3.13 \text{ Mpa} \quad \text{(Limited to this value)}$$

$$N_{Ed}/A_c < 0.2f_{cd} \quad 0.00$$

$$\text{Applied Longitudinal Force} \quad N_{Ed} = 0.00$$

$$\text{Design value of concrete compressive strength} \quad f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_m} = 15.633$$

$$v_{min} = 0.031K^{3/2} \cdot f_{ck}^{1/2}$$

$$K = 1 + \sqrt{\frac{200}{d_{eff}}} = 1.342$$

$$\rho_1 = \frac{A_s}{b_w \cdot d_{eff}} < 0.02$$

$$\% \text{ of reinforcement} = \frac{0.01360}{1.360} \%$$

$$\text{Check for Shear resistance} = V_{Rd,Max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}$$

$$\alpha_{cw} = 1.00 \quad \text{for} \quad \sigma_{cp} = 0$$

$$b_w = 325.00$$

$$z = 1.54$$

$$v_1 = 0.60$$

$$V_{Rd,Max} = 4691.64 \text{ KN}$$

$$\text{Design Shear resistance} = 297.44 \text{ KN}$$

$$\text{Design Shear Force} = 1023.43 \text{ KN}$$

Provide Shear Reinforcement

SHEAR REINFORCEMENT

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot \theta \quad \longrightarrow \quad \frac{A_{sw}}{s} = 0.001662 \text{ mm}^2/\text{m}$$

$$V_{Rd,s} = 1023.43 \text{ KN}$$

$$Z = 1539$$

$$f_{ywd} = 0.8 \cdot f_y = 400 \text{ N/mm}^2$$

$$\cot \theta = 2.5$$

$$S = \text{Spacing of stirrups}$$

$$\text{For combined Shear and Torsion } A_{swt,Reqd} = 0.00166 \text{ mm}^2/\text{mm}$$

$$\text{Using } 2 \text{ L- } 12 @ A_{sw} = 1.81 \text{ mm}^2/\text{mm} \quad \text{OK}$$

$$\text{Spacing to be Provided } S = 125 \text{ mm}$$

Minimum Shear Reinforcement

$$\text{Shear Reinforcement ratio } \rho_w = \frac{A_{sw}}{(s \cdot b_w \cdot \sin \alpha)} = 0.0445 \quad \text{OK}$$

$$\text{Minimum Shear Reinforcement ratio } \rho_{w,min} = \frac{(0.072 \cdot \sqrt{f_{ck}})}{f_{yk}} = 0.00085 \quad \text{Refer 10.3.2 IRC: 112-2011}$$

INTERFACE SHEAR

Refer 10.3.4 IRC: 112-2011

Design Shear at Interface

Transverse Shear	V_{Ed}	=	1023.43 KN	
Inner lever arm of composite section	Z	=	1539 mm	
Interface width	b_i	=	325.00 mm	
Interface area		=	325000 mm ²	
Proportion of Longitudinal tensile force carried in top sl	β	=	1.00	
Interface Shear Stress	V_{Edi}	=	$\frac{\beta \cdot V_{ED}}{Z \cdot b_i}$	= 2.046 N/mm ²

Design Shear Resistance at Interface

Interface shear capacity at section	V_{Rdi}	=	$\mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot [\mu \sin \alpha + \cos \alpha]$	≥	$0.5 \cdot v \cdot f_{cd}$	
Factor dependent on interface roughness	μ	=	0.6			
Stress resulting from forces normal to interface	σ_n	=	0.00 Mpa			
Angle of reinforcement to interface	α	=	90 degree			
Strength reduction factor for concrete cracked in shear	v	=	0.5			
	V_{Rdi}	=	4.03 N/mm ²			OK

Minimum reinforcement	A_{ci}	=	487.500 mm ²
Provide	10 @	125	628.3 mm ²

SHEAR IN FLANGE PORTION

Refer 10.3.5 IRC: 112-2011

S.No	Type of Loading	γ^*	At support	At L/8	At L/4	At 3L/8	At L/2
			BM(KN-m)				
1	Dead Load-Girder	1.35	3.07	378.15	643.43	802.61	855.66
2	Dead Load-Slab	1.35	2.06	350.83	602.90	754.14	804.55
3	SIDL	1.35	393.66	367.14	284.07	137.42	103.36
4	Surfacing	1.75	56.19	132.64	232.60	291.14	310.63
5	Live Load	1.5	-0.04	590.16	1168.29	1604.18	1788.05
TOTAL			636.63	2597.13	4225.52	5202.89	5606.50

* γ - Partial Safety factor

Longitudinal shear stress across junction of flange with web V_{EDf} = $\frac{\Delta Fd}{h_f \cdot \Delta x}$

Z	-	Inner lever arm of composite section	=	1.539 m
h_f	-	Thickness of Flange	=	220 mm
ΔFd	-	Change in Normal Force (unbalanced in plane SF at interface Section)	=	3588.89 KN
ν	-	Strength reduction factor	=	0.52
f_{ctd}	-		=	1.87

Description	Support - L/8	L/8 - L/4	L/4 - 3L/8	3L/8 - L/2
ΔFd	1273.88	1058.08	635.07	262.25
Δx	2.375	2.375	2.375	2.375
$V_{edf} = \frac{\Delta Fd}{h_f \cdot \Delta x}$	2.43804	2.02504	1.21544	0.50192
$\cot \theta_f$	2.00	2.00	2.00	2.00
$\frac{A_{sw}}{s} \cdot \frac{V_{EDf} \cdot h_f}{f_{yd} \cdot \cot \theta_f}$	616.52	512.08	307.35	126.92
Provide	2L- 12 Dia	2L- 12 Dia	2L- 12 Dia	2L- 12 Dia
A_{sf}	1809.557	1809.557	1507.964	1507.964
S_f	125.00	125.00	150.00	150.00
$VEd \leq \nu \cdot f_{ctd} \cdot \sin \theta_f \cdot \cos \theta_f$	3.23 OK	3.23 OK	3.23 OK	3.23 OK
$0.4f_{ctd}$	0.75	0.75	0.75	0.75

STRESS IN GIRDER DUE TO DEAD WEIGHT OF SLAB**SLS CHECK AT EACH STAGE**

IRC:112-2011 7.7.1

Material Properties

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

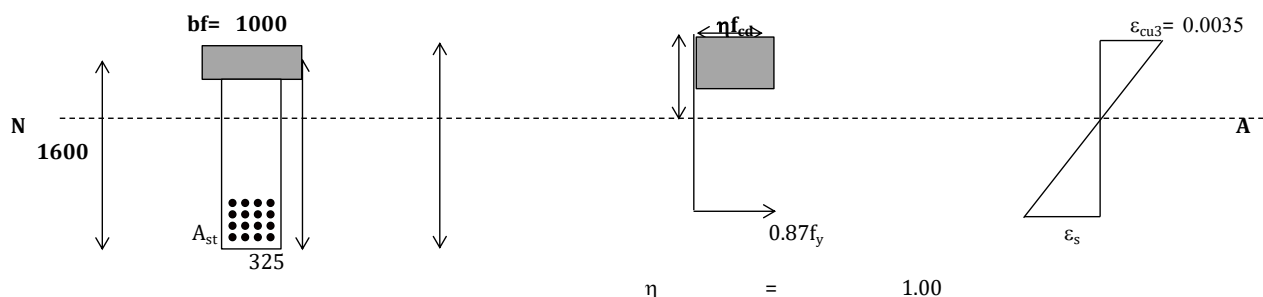
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	30.57	N/mm ²	at 14 days
f_{cm}	-	40.57	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1.35	855.66	802.61	643.43	378.15	276.78	326.56
2	DL of Slab	1.35	804.55	754.14	602.90	350.83	254.73	301.82
Total Design Moment M_{ED}			2241.29	2101.60	1682.55	984.12	717.54	848.31

* γ – Partial safety factor

DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	2241.29	2101.60	1682.55	984.12	717.54	848.31
b	mm	975.64	975.64	975.64	1205.73	838.62	838.62
Area of reinforcement provided A_{st}	mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement		Not Ok	Not Ok	Not Ok	Not Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}	m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A	mm	516.08	516.08	516.08	348.00	333.56	333.56
Critical Depth of N.A	mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	3724.00	3483.26	2768.36	1571.22	1121.05	1328.46
Moment Resisting capacity M_R	KN-m	7747.0991	7747.099	7747.0991	6920.69	4745.95	4745.95

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1	855.66	802.61	643.43	378.15	276.78	326.56
2	DL of Slab	1	804.55	754.14	602.90	350.83	254.73	301.82
Total Design Moment M_{ED}			1660.21	1556.74	1246.33	728.98	531.51	628.38

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	1660.21	1556.74	1246.33	728.98	531.51	628.38
b		mm	975.64	975.64	975.64	1205.73	838.62	838.62
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
A_c		m ²	0.753	0.753	0.753	0.753	0.910	0.910
u		m	4.8	4.8	4.8	4.8	4.8	4.8
$2A_c/u$			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			3.0	3.0	3.0	3.0	3.0	3.0
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(f_{cm})$			2.9	2.9	2.9	2.9	2.9	2.9
$\beta(t_0)$			0.7	0.7	0.7	0.7	0.7	0.7
t			7.0	7.0	7.0	7.0	7.0	7.0
t_0			3.0	3.0	3.0	3.0	3.0	3.0
$t-t_0$			4.0	4.0	4.0	4.0	4.0	4.0
$\beta(t,t_0)$			0.2	0.2	0.2	0.2	0.2	0.2
β_H			788.1	788.1	788.1	788.1	900.5	900.5
$E_{c,eff}$			19827.5	19827.5	19827.5	19827.5	20250.2	20250.2
Modular Ratio	m		10.09	10.09	10.09	10.09	9.88	9.88
Depth of N.A	xu	mm	522.98	522.98	522.98	450.87	444.94	444.94
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_c	mm ⁴	1.54E+09	1.54E+09	1.54E+09	9.84E+08	9.66E+08	9.66E+08
Moment of Inertia of transformed area	I_s	mm ⁴	1.21E+10	1.21E+10	1.21E+10	1.22E+10	8.78E+09	8.78E+09
Modulus of Section of Concrete	Z_c	mm ³	2.94E+06	2.94E+06	2.94E+06	2.18E+06	2.17E+06	2.17E+06
Modulus of Section of Steel	Z_s	mm ³	1.32E+07	1.32E+07	1.32E+07	1.21E+07	8.40E+06	8.40E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.36E+10	1.36E+10	1.36E+10	1.32E+10	9.75E+09	9.75E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	6.32	5.93	4.74	2.46	2.46	2.90
At C.G of Steel	fy'	N/mm ²	111.36	104.42	83.60	55.53	56.97	67.35
At bottom fibre of Steel	fy''	N/mm ²	121.76	114.17	91.41	59.06	58.72	69.42

STRESS IN GIRDER DUE TO GREEN WEIGHT OF SLAB**SLS CHECK AT EACH SECTION****Material Properties**

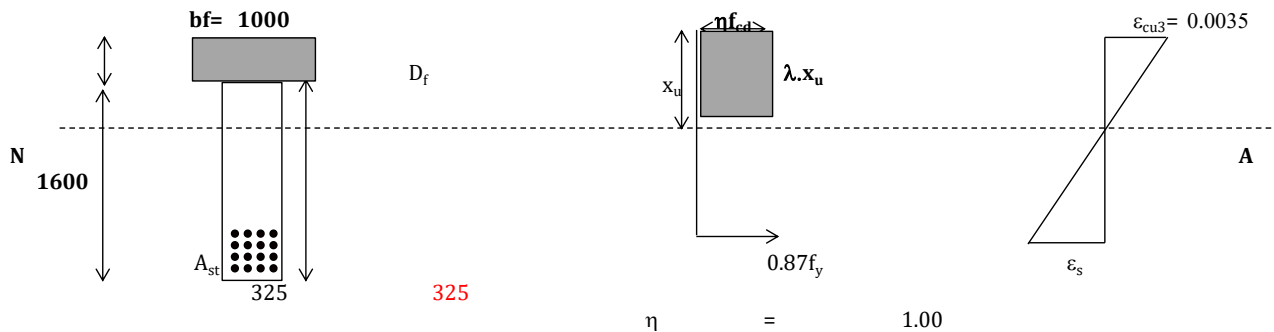
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35.00	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Green weight of slab	1.35	29.71	27.85	22.26	12.95	9.41	11.14
Total Design Moment M_{ED}			40.10	37.59	30.05	17.49	12.70	15.04



DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	40.10	37.59	30.05	17.49	12.70	15.04
b		mm	700.44	577.95	577.95	602.40	692.87	692.87
Area of reinforcement provided A_{st}		mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of girder, d_{eff}		m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A		mm	718.85	871.21	871.21	696.53	403.73	403.73
Critical Depth of N.A		mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	64.23	60.21	48.13	27.59	19.60	23.22
Moment Resisting capacity M_R		KN-m	7236.34	6852.57	6852.5702	6189.09	4647.7597	4647.7597

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1	29.71	27.85	22.26	12.95	9.41	11.14
Total Design Moment M_{ED}			29.71	27.85	22.26	12.95	9.41	11.14

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	29.71	27.85	22.26	12.95	9.41	11.14
b		mm	700.44	577.95	577.95	602.40	692.87	692.87
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	0.75	0.75	0.75	0.75	0.91	0.91
u		mm	4.8	4.8	4.8	4.8	4.8	4.8
2Ac/u			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-to)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_o			2.4	2.4	2.4	2.4	2.4	2.4
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(fcm)$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(to)$			0.6	0.6	0.6	0.6	0.6	0.6
t			14.0	14.0	14.0	14.0	14.0	14.0
t_o			7.0	7.0	7.0	7.0	7.0	7.0
t- t_o			7.0	7.0	7.0	7.0	7.0	7.0
$\beta(t,to)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			788.1	788.1	788.1	788.1	900.5	900.5
$E_{c,eff}$			20148.9	20148.9	20148.9	20148.9	20565.8	20565.8
Modular Ratio	m		9.93	9.93	9.93	9.93	9.72	9.72
Depth of N.A	xu	mm	589.55	632.40	632.40	582.68	467.71	467.71
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.24E+09	2.76E+09	2.76E+09	2.16E+09	1.14E+09	1.14E+09
Moment of Inertia of transformed area	I_S	mm ⁴	1.04E+10	9.36E+09	9.36E+09	9.24E+09	8.40E+09	8.40E+09
Modulus of Section of Concrete	Z_C	mm ³	3.79E+06	4.36E+06	4.36E+06	3.71E+06	2.44E+06	2.44E+06
Modulus of Section of Steel	Z_S	mm ³	1.23E+07	1.16E+07	1.16E+07	1.06E+07	8.22E+06	8.22E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.26E+10	1.21E+10	1.21E+10	1.14E+10	9.54E+09	9.54E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	0.14	0.15	0.12	0.07	0.05	0.06
At C.G of Steel	fy'	N/mm ²	1.99	1.85	1.48	0.99	1.01	1.19
At bottom fibre of Steel	fy''	N/mm ²	2.19	2.22	1.78	1.16	1.12	1.32

STRESS IN GIRDER DUE TO CONSTRUCTION LOAD**SLS CHECK AT EACH SECTION****Material Properties**

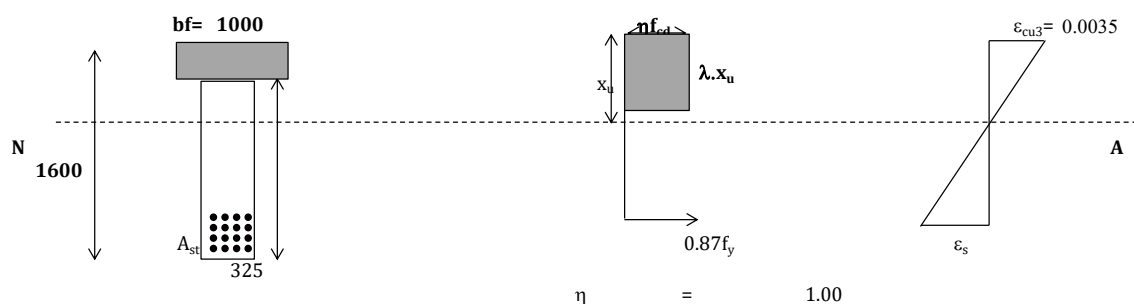
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{yd}	-	435	N/mm ²	
f_{ck}	-	35.00	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)				
			L/2	3L/8	L/4	L/8	Deff
1	Construction Load	1	486.11	455.65	364.27	211.97	192.64
Total Design Moment M_{ED}			486.11	455.65	364.27	211.97	192.64



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	486.11	455.65	364.27	211.97	192.64	182.41
b_{eff}	mm	700.44	700.44	700.44	747.99	926.13	926.13
Area of reinforcement provided A_{st}	mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement		Not Ok	Not Ok	Not Ok	Not Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}	m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A	mm	718.85	718.85	718.85	560.96	302.04	302.04
Critical Depth of N.A	mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	786.38	736.59	587.65	335.66	298.10	282.23
Moment Resisting capacity M_R	KN-m	6437.963	6263.548	5565.2921	3496.068	3180.6526	3066.6996

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Construction Load	1	486.11	455.65	364.27	211.97	192.64	182.41
Total Design Moment M_{ED}			486.11	455.65	364.27	211.97	192.64	182.41

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	486.11	455.65	364.27	211.97	192.64	182.41
b		mm	700.44	700.44	700.44	747.99	926.13	926.13
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
A_c		mm ²	0.8	0.8	0.8	0.8	0.9	0.9
u		mm	4.8	4.8	4.8	4.8	4.8	4.8
$2A_c/u$			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			2.4	2.4	2.4	2.4	2.4	2.4
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
t			14.0	14.0	14.0	14.0	14.0	14.0
t_0			7.0	7.0	7.0	7.0	7.0	7.0
$t-t_0$			7.0	7.0	7.0	7.0	7.0	7.0
$\beta(t,t_0)$			0.2	0.2	0.2	0.2	0.2	0.2
β_H			788.1	788.1	788.1	788.1	900.5	900.5
$E_{c,eff}$			20148.9	20148.9	20148.9	20148.9	20565.8	20565.8
Modular Ratio	m		9.93	9.93	9.93	9.93	9.72	9.72
Depth of N.A	xu	mm	589.55	589.55	589.55	536.77	415.34	415.34
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_c	mm ⁴	2.24E+09	2.24E+09	2.24E+09	1.69E+09	7.98E+08	7.98E+08
Moment of Inertia of transformed area	I_s	mm ⁴	1.04E+10	1.04E+10	1.04E+10	1.02E+10	9.29E+09	9.29E+09
Modulus of Section of Concrete	Z_c	mm ³	3.79E+06	3.79E+06	3.79E+06	3.14E+06	1.92E+06	1.92E+06
Modulus of Section of Steel	Z_s	mm ³	1.23E+07	1.23E+07	1.23E+07	1.11E+07	8.64E+06	8.64E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.26E+10	1.26E+10	1.26E+10	1.19E+10	1.01E+10	1.01E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	2.29	2.14	1.71	0.96	0.82	0.77
At C.G of Steel	fy'	N/mm ²	32.62	30.57	24.44	16.37	20.52	19.43
At bottom fibre of Steel	fy''	N/mm ²	35.90	33.65	26.90	17.51	21.14	20.01

STRESS IN GIRDER DUE TO SIDL AND SURFACING**SLS CHECK AT EACH SECTION****Material Properties**

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

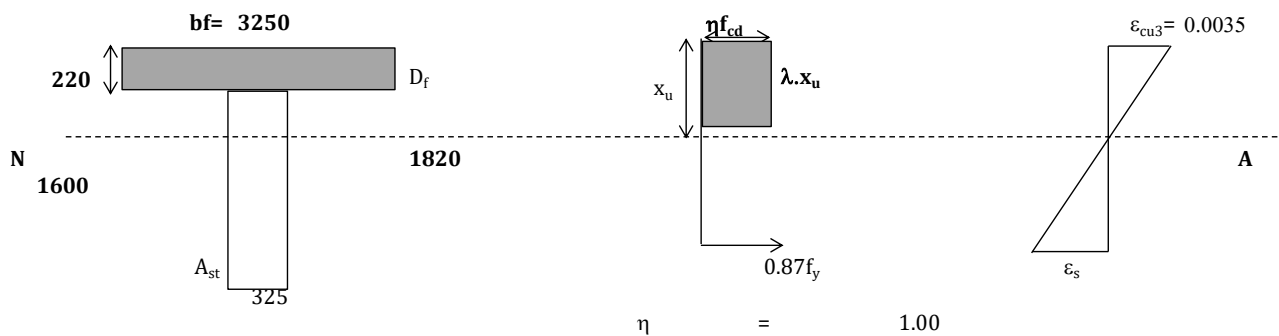
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	SIDL	1.35	393.66	367.14	284.07	137.42	103.36	67.68
2	Surfacing	1.75	310.63	291.14	232.60	132.64	108.47	82.83
Total Design Moment M_{ED}			1075.03	1005.13	790.53	417.64	329.36	236.32

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	1075.03	1005.13	790.53	417.64	329.36	236.32
b		mm	2610.13	2610.13	2610.13	2630.83	2620.51	2621.36
Area of reinforcement provided A_{st}		mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement			Not Ok	Not Ok	Not Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}		m	1.66	1.66	1.66	1.68	1.71	1.71
Depth of N.A		mm	192.91	192.91	192.91	159.49	106.75	106.71
Critical Depth of N.A		mm	1021.73	1021.73	1021.73	1034.89	1054.63	1054.63
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	1498.98	1401.07	1100.87	573.20	443.39	318.01
Moment Resisting capacity M_R		KN-m	9946.547	9946.5466	9946.5466	8470.883	5833.017	5833.0657

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	SIDL	1	393.66	367.14	284.07	137.42	103.36	67.68
2	Surfacing	1.2	310.63	291.14	232.60	132.64	108.47	82.83
Total Design Moment M_{ED}			766.41	716.51	563.18	296.59	233.52	167.07

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	766.41	716.51	563.18	296.59	233.52	167.07
b		mm	2610.13	2610.13	2610.13	2630.83	2620.51	2621.36
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	1.5	1.5	1.5	1.5	1.6	1.5
u		mm	10.3	10.3	10.3	10.3	10.3	10.3
2Ac/u			285.9	285.9	285.9	285.9	316.5	285.9
$\phi(t-t_0)$			0.9	0.9	0.9	0.9	0.9	0.9
ϕ_0			2.2	2.2	2.2	2.2	2.1	2.2
ϕ_{RH}			1.4	1.4	1.4	1.4	1.4	1.4
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
t			56.0	56.0	56.0	56.0	56.0	56.0
t_0			14.0	14.0	14.0	14.0	14.0	14.0
t-t ₀			42.0	42.0	42.0	42.0	42.0	42.0
$\beta(t, t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
βH			743.3	743.3	743.3	743.3	796.1	743.3
$E_{c,eff}$			16894.6	16894.6	16894.6	16894.6	17123.6	16894.6
Modular Ratio	m		11.84	11.84	11.84	11.84	11.68	11.84
Depth of N.A	xu	mm	405.36	405.36	405.36	373.28	310.64	312.48
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	6.10E+09	6.10E+09	6.10E+09	4.76E+09	2.78E+09	2.79E+09
Moment of Inertia of transformed area	I_S	mm ⁴	2.27E+10	2.27E+10	2.27E+10	2.05E+10	1.57E+10	1.57E+10
Modulus of Section of Concrete	Z_C	mm ³	1.50E+07	1.50E+07	1.50E+07	1.28E+07	8.95E+06	8.94E+06
Modulus of Section of Steel	Z_S	mm ³	1.81E+07	1.81E+07	1.81E+07	1.57E+07	1.13E+07	1.12E+07
Cracked Moment of Inertia	I_{CR}	mm ⁴	2.88E+10	2.88E+10	2.88E+10	2.53E+10	1.85E+10	1.85E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0.91	0.85	0.67	0.37	0.34	0.24
Top of Girder	fck''	N/mm ²	0.42	0.39	0.31	0.15	0.10	0.07
At C.G of Steel	fy'	N/mm ²	33.34	31.17	24.50	15.30	17.64	12.62
At bottom fibre of Steel	fy''	N/mm ²	35.62	35.24	27.70	16.96	19.02	13.61

STRESS IN GIRDER DUE TO CWLL**SLS CHECK AT EACH SECTION****Material Properties**

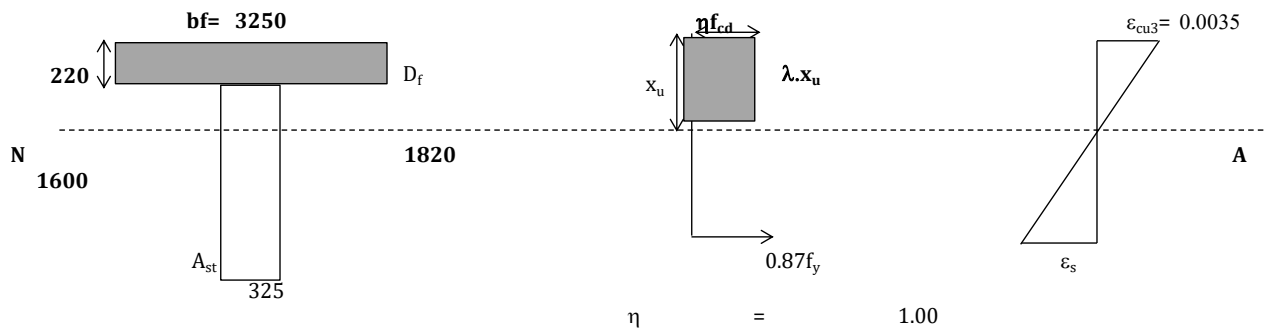
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Live Load	1.5	1788.05	1604.18	1168.29	590.16	476.72	229.96
2	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Impact factor			1.180	2109.90	1892.93	1378.59	696.39	562.52
Lane Reduction factor			0.900	1898.91	1703.64	1240.73	626.75	506.27
Total Design Moment M_{ED}			2848.37	2555.46	1861.09	940.13	759.41	366.33

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	2848.37	2555.46	1861.09	940.13	759.41	366.33
b		mm	2651.41	2651.41	2651.41	2646.91	2567.42	2570.13
Area of reinforcement provided A_{st}		mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement			Not Ok	Not Ok	Not Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}		m	1.66	1.66	1.66	1.68	1.71	1.71
Depth of N.A		mm	189.90	189.90	189.90	158.52	108.95	108.84
Critical Depth of N.A		mm	1021.73	1021.73	1021.73	1034.89	1054.63	1054.63
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	4003.25	3586.80	2604.00	1293.21	1024.24	493.24
Moment Resisting capacity M_R		KN-m	9954.112	9954.11184	9954.1118	8472.916	5829.9281	5830.089

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Live Load	1	1788.05	1604.18	1168.29	590.16	476.72	229.96
2	FPLL	1	0.00	0.00	0.00	0.00	0.00	0.00
Impact factor		1.180	2109.90	1892.93	1378.59	696.39	562.52	271.35
Lane Reduction factor		0.900	1898.91	1703.64	1240.73	626.75	506.27	244.22
Total Design Moment M_{ED}			1898.91	1703.64	1240.73	626.75	506.27	244.22

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	1898.91	1703.64	1240.73	626.75	506.27	244.22
b		mm	2651.41	2651.41	2651.41	2646.91	2567.42	2570.13
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	1.5	1.5	1.5	1.5	1.6	1.5
u		mm	10.3	10.3	10.3	10.3	10.3	10.3
$2Ac/u$			285.9	285.9	285.9	285.9	316.5	285.9
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_o			1.7	1.7	1.7	1.7	1.6	1.7
ϕ_{RH}			1.4	1.4	1.4	1.4	1.4	1.4
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
t			84.0	84.0	84.0	84.0	84.0	84.0
t_o			56.0	56.0	56.0	56.0	56.0	56.0
$t-t_o$			28.0	28.0	28.0	28.0	28.0	28.0
$\beta(t, t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
β_H			743.3	743.3	743.3	743.3	796.1	743.3
$E_{c,eff}$			19857.4	19857.4	19857.4	19857.4	20075.8	19857.4
Modular Ratio	m		10.07	10.07	10.07	10.07	9.96	10.07
Depth of N.A	x_u	mm	375.39	375.39	375.39	346.78	291.86	293.17
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	5.69E+09	5.69E+09	5.69E+09	4.49E+09	2.70E+09	2.71E+09
Moment of Inertia of transformed area	I_S	mm ⁴	2.38E+10	2.38E+10	2.38E+10	2.14E+10	1.62E+10	1.61E+10
Modulus of Section of Concrete	Z_C	mm ³	1.52E+07	1.52E+07	1.52E+07	1.29E+07	9.26E+06	9.24E+06
Modulus of Section of Steel	Z_S	mm ³	1.85E+07	1.85E+07	1.85E+07	1.61E+07	1.14E+07	1.14E+07
Cracked Moment of Inertia	I_{CR}	mm ⁴	2.95E+10	2.95E+10	2.95E+10	2.59E+10	1.89E+10	1.89E+10
Actual Stresses								
Top of Slab	f_{ck}'	N/mm ²	2.40	2.16	1.57	0.83	0.79	0.38
Top of Girder	f_{ck}''	N/mm ²	0.99	0.89	0.65	0.31	0.19	0.09
At C.G of Steel	f_y'	N/mm ²	82.60	74.11	53.97	32.26	38.03	18.35
At bottom fibre of Steel	f_y''	N/mm ²	88.10	83.55	60.85	35.70	40.98	19.78

CHECK FOR DEFLECTION

Refer 12.4.1 IRC 112-2011

Allowable deflection due to Vehicular load	=	$\frac{Span}{800}$	=	23.75 mm
Deflection due to LL(70R Wheeled)	=	3.91	mm	
Deflection due to LL(1 lane of class A)	=	3.274	mm	
Deflection due to LL (70R W + Class A)	=	7.184	mm	< 23.75 mm

SUMMARY OF STRESSES IN OUTER GIRDER**Allowable Stresses**

Concrete	16.8	N/mm ²
Steel	300	N/mm ²

As per Clause 5.3.2.5 of IRC 112 Amendment No. 1, Aug 2013

Description

Stress Location	
Top of Slab	f _{ck} '
Top of Girder	f _{ck} ''
At C.G of Steel	f _y '
At bottom fibre of Steel	f _y ''

LC-1 STAGE 1 DL + Green Weight of Slab + Construction Load AT 14 DAY

SLS - RARE		L/2		
LOAD	Load Factor	f _{ck} ''	f _y '	f _y ''
		N/mm ²		
DEAD LOAD	1	6.32	111.36	121.76
DL-Green Weight	1	0.10	1.64	1.80
Construction Load	1	2.29	32.62	35.90
Resultant Stress		8.71	145.62	159.46
CHECK		SAFE		

3L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
5.93	104.42	114.17
0.11	1.56	1.72
2.14	30.57	33.65
8.18	136.55	149.54
SAFE		

L/4		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
4.74	83.60	91.41
0.09	1.25	1.37
1.71	24.44	26.90
6.55	109.29	119.69
SAFE		

L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.46	55.53	59.06
0.06	0.85	0.91
0.96	16.37	17.51
3.48	72.75	77.48
SAFE		

Deff		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.46	56.97	58.72
0.04	0.88	0.91
0.82	20.52	21.14
3.32	78.38	80.76
SAFE		

Web Widening		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.90	67.35	69.42
0.05	1.04	1.08
0.77	19.43	20.01
3.73	87.83	90.51
SAFE		

LC-2 STAGE 2 DL - Construction Load + SIDL + LL AFTER 84 Days

SLS - RARE		L/2				3L/8				L/4				L/8				Deff				Web Widening			
LOAD	Load Factor	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''
		N/mm ²				N/mm ²				N/mm ²				N/mm ²				N/mm ²				N/mm ²			
DL+Construction	1	0	8.61	143.98	157.67	0	8.07	134.99	147.83	0	6.46	108.04	118.31	0	3.43	71.91	76.57	0	3.27	77.50	79.85	0	3.68	86.79	89.43
-ve Construction Load	1	0.00	-2.29	-25.83	-29.59	-2.02	-1.60	-24.21	-30.95	-1.61	-1.28	-19.36	-24.74	-0.94	-0.74	-13.35	-16.22	-0.90	-0.68	-17.50	-19.88	-0.85	-0.64	-16.57	-18.83
SIDL	1	0.91	0.42	33.34	35.62	0.85	0.39	31.17	35.24	0.67	0.31	24.50	27.70	0.37	0.15	15.30	16.96	0.34	0.10	17.64	19.02	0.24	0.07	12.62	13.61
Live Load	1	2.40	0.99	82.60	88.10	2.16	0.89	74.11	83.55	1.57	0.65	53.97	60.85	0.83	0.31	32.26	35.70	0.79	0.19	38.03	40.98	0.38	0.09	18.35	19.78
Differential Shrinkage	1	-0.08	0.30	0.54	0.54	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67
Resultant Stress		3.23	8.03	234.63	252.34	0.90	8.12	216.74	236.34	0.53	6.50	167.83	182.79	0.17	3.51	106.79	113.69	0.13	3.26	116.34	120.65	-0.33	3.57	101.87	104.67
CHECK		SAFE				SAFE				SAFE				SAFE				SAFE				SAFE			

LC-3 Effect of Temperature Rise & Diff.Shrinkage Without CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
Temperature Rise	1	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019
Differential Shrinkage	1	-0.08	0.54	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		8.90	145.24	8.44	136.27	7.07	108.77	5.26	70.73	4.45	75.28	5.57	79.88
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-4 Effect of Temperature Rise & Diff.Shrinkage With CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
CWLL	1	2.40	82.60	2.16	74.11	1.57	53.97	0.83	32.26	0.79	38.03	0.38	18.35
Temperature Rise	0.6	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		10.59	227.98	9.89	210.37	7.94	162.74	4.62	103.76	4.53	113.31	4.47	99.00
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-5 Effect of Temperature Fall & Diff.Shrinkage Without CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
Temperature fall	1	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		8.54	142.95	8.09	133.84	6.73	106.35	4.15	69.08	4.10	72.86	4.45	78.22
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-6 Effect of Temperature Fall & Diff.Shrinkage With CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
CWLL	1	2.40	82.60	2.16	74.11	1.57	53.97	0.83	32.26	0.79	38.03	0.38	18.35
Temperature fall	0.6	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		10.95	225.55	10.25	207.95	8.30	160.32	4.98	101.34	4.89	110.89	4.83	96.58
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-7 Effect of Temperature Rise & Diff.Shrinkage

SLS - QUASI PERMANENT		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
Temperature Rise	0.5	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019	1.76	-0.0019
Differential Shrinkage	1	-0.08	0.54	-0.10	0.67	-0.10	0.67	0.67	-0.10	-0.10	0.67	0.67	-0.10
Resultant Stress		8.03	145.24	7.56	136.27	6.19	108.77	4.39	70.73	3.57	75.28	4.69	79.88
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-8 Effect of Temperature Fall & Diff.Shrinkage

SLS - QUASI PERMANENT		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.23	144.70	6.78	135.59	5.41	108.10	2.83	70.83	2.79	74.61	3.14	79.98
Temperature fall	0.5	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43	1.41	-2.43
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		7.84	144.17	7.39	135.05	6.02	107.56	3.44	70.29	3.40	74.07	3.75	79.44
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

CRACK WIDTH CALUCLATION

Permissible Crack width in moderate conditions of exposure = **0.3 mm**

Crack Width W_k at Midspan

$$S_{r, \max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = \mathbf{0.13 \text{ mm}} \quad \mathbf{OK} \quad \text{Refer Cl. 12.3.4 IRC 112}$$

$S_{r, \max}$	-	Maximum Crack spacing
ϵ_{sm}	-	Mean Strain in Reinforcement
ϵ_{cm}	-	Mean Strain in Concrete
σ_{sc}	-	145.24 N/mm ² (from Quasi permanent combination)
K_t	-	0.5
$f_{ct, \text{eff}}$	-	2.8
$\rho_{p, \text{eff}}$	-	0.153
α_e	-	6.25
E_s	-	200000
A_s	-	14476 mm ²
c	-	50
ϕ	-	32
h	-	1820 mm
d	-	1656.67
x	-	947.60

$$h_{c, \text{eff}} = \text{Min} \begin{cases} 2.5(h-d) \\ (h-x)/3 \\ h/2 \end{cases} = \begin{matrix} 408.333 \\ 290.8 \\ 910 \end{matrix} = \mathbf{290.80 \text{ mm}}$$

$$\phi_{eq} = \frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2} = 32$$

$$A_{c, \text{eff}} = 94510 \text{ mm}^2$$

$$\epsilon_{sm} - \epsilon_{cm} = \frac{\sigma_{sc} - K_t \frac{f_{ct, \text{eff}}}{\rho_{p, \text{eff}}} (1 + \alpha_e \cdot \rho_{p, \text{eff}})}{E_s} \geq 0.6 \cdot \frac{\sigma_{sc}}{E_s}$$

$$= 0.00064 \geq 0.0004$$

For the case of Deformed bars associated with pure bending

$$S_{r, \max} = 3.4c + \frac{0.17 \cdot \phi}{\rho_{p, \text{eff}}} = 205.52$$

DESIGN OF INNER GIRDER

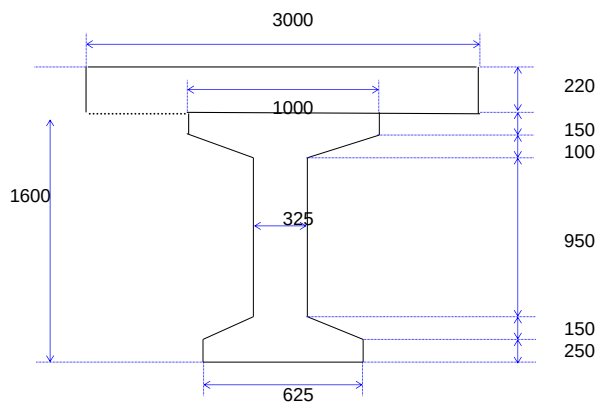
ULS DESIGN OF LONGITUDINAL MEMBERS FOR TORSION

ULTIMATE BENDING MOMENT AND SHEAR FORCE

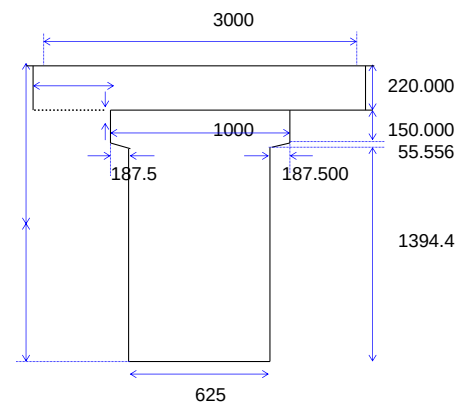
Bending Moment (kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	858.63	804.64	644.53	378.31	276.78	326.58
DL of Slab	1.35	742.66	696.13	556.52	323.85	235.14	278.60
SIDL	1.35	40.95	40.35	38.28	35.37	17.83	9.86
Surfacing	1.75	290.18	272.26	272.17	218.39	218.22	128.43
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	1906.41	1672.11	1159.04	589.37	456.93	313.25
Live Load(SV Loading)	1	2654.75	2488.58	1986.40	1053.65	817.13	555.35
	TOTAL M_u	5584.46	5065.13	4135.79	2431.50	1914.18	1610.42
Shear Force (KN)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.39	45.07	89.75	134.43	148.90	141.49
DL of Slab	1.35	0.00	39.19	78.38	117.56	129.42	123.75
SIDL	1.35	6.88	6.14	7.28	7.65	8.56	5.36
Surfacing	1.75	4.54	9.79	18.88	24.13	33.29	38.54
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	6.63	176.45	232.47	244.85	292.31	296.61
Live Load(SV Loading)	1	12.98	155.67	340.72	484.76	536.23	535.86
	TOTAL M_u	30.74	403.85	618.55	877.51	981.77	968.61

Torsional Moment(kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.00	0.00	0.00	0.00	0.00	0.00
DL of Slab	1.35	0.00	0.00	0.00	0.00	0.00	0.00
SIDL	1.35	0.90	2.47	3.84	3.84	6.80	7.65
Surfacing	1.75	0.02	0.02	0.03	0.03	0.02	0.02
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	7.13	13.00	14.62	12.14	12.88	9.56
Live Load(SV Loading)	1	3.77	12.87	24.44	39.23	41.68	45.72
	TOTAL M_u	11.95	22.85	29.68	44.46	50.89	56.09

Equivalent Section at Mid-Span



Equivalent Section at Support



Calculation of Section at mid span:

	width	depth	area	perimeter	tef,i	eff.depth	Ak	uk
Deck Slab	3.000	0.220	0.660	5.609	0.136	0.084	0.241	5.065
Top Flange	0.831	0.250	0.208	1.006	0.207	0.043	0.027	0.387
Web	0.325	0.950	0.309	1.900	0.163	0.788	0.128	1.575
Bottom Flange	0.550	0.400	0.220	1.575	0.156	0.244	0.096	0.951
					0.151		0.492	7.977

Total Torsional inertia of section at mid-span = 0.0228 m⁴

Ak for comosite section = 1.413 m²

perimeter u for composite section = 10.265 m

Thickness of web = 0.3250 m

Calculation of Section at Support:

	width	depth	area	perimeter	tef,i	eff.depth	Ak	uk
Deck Slab	3.000	0.220	0.660	5.534	0.136	0.084	0.241	4.990
Top Flange	0.906	0.206	0.186	0.692	0.269	0.000	0.000	0.000
Web	0.625	1.394	0.872	3.414	0.255	1.139	0.421	2.648
			0.2119				0.662	7.638

Total Torsional inertia of section at support = 0.094 m⁴
 Ak for comosite section = 1.727 m²
 perimeter u for composite section = 8.825 m
 Thickness of web = 0.625 m

Material Stress

fck = 35 N/mm² fcd = 15.63 N/mm²
 fyk = 500 N/mm² fyd = 434.78 N/mm²
 fctm = 2.77 N/mm² Es = 2.00E+05 N/mm²

ULS TORSIONAL MOMENT CAPACITY OF OF LONGITUDINAL SECTION

fck	=	40.00	N/mm ²	fcd	=	17.87	N/mm ²
fyd	=	434.78	N/mm ²				
		L/2	3L/8	L/4	L/8	Deff	Web Widening
Ultimate Torsion Moment , T_{Ed}	kNm	11.95	22.85	29.68	44.46	50.89	56.09
Total Area of section , A	m ²	1.413	1.413	1.413	1.413	1.727	1.570
effective cover	m	0.163	0.163	0.163	0.163	0.110	0.110
t _{ef,i}	m	0.151	0.151	0.151	0.151	0.212	0.212
Area of section , Ak	m ²	0.492	0.492	0.492	0.492	0.577	0.577
Perimeter of section , uk	m	7.977	7.977	7.977	7.977	7.808	7.808
Shear stress due to pure Torsion, τ _{t,i} = T _{Ed} / (2 . Ak. t _{ef,i})	N/mm ²	0.081	0.154	0.200	0.300	0.208	0.229
Length of section , Zi	m	0.325	0.325	0.325	0.325	0.625	0.625
Shear force due to torsion = V _{Ed,i} = τ _{t,i} . t _{ef} . Zi	kN	3.95	7.55	9.81	14.69	27.57	30.39
Mean compr. stress at c.g. sec. σ _{cp} = N _{Ed} / Ac ≤ 0.2 fcd	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000
Coefficient α _{cw} =1+σ _{cp} /fcd		1.000	1.000	1.000	1.000	1.000	1.000
strength reduction factor for section cracked in shear = v = 0.6 [1 - fck/310]		0.52	0.52	0.52	0.52	0.52	0.52
cot θ		2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22
Maximum Torsional Capacity T _{Rd,max} = 2 . v . α _{cw} . fcd . Ak . tef . sinθ . cosθ	kNm	477.27	477.27	477.27	477.27	787.08	787.08
Torsional Long. Reinf. Asl = T _{Ed} . cot θ / (2 . Ak . fyd)	mm ²	557.27	1065.62	1383.89	2073.42	1980.57	2182.78
Torsional reinf. In web on each face	mm ²	55.01	105.19	136.61	204.68	353.73	389.85
Torsional reinf. In bottom flange	mm ²	66.43	127.03	164.97	247.17	158.55	174.73
Torsional reinf. In top flange	mm ²	27.01	51.65	67.08	100.50	0.00	0.00
Total torsional steel in girder		148.46	283.88	368.67	552.36	512.28	564.58
Total torsional moment, T _{Rd,max}	kNm	477.27	477.27	477.27	477.27	787.08	787.08
Total shear force due to torsion, V _{Edi}	kN	3.95	7.55	9.81	14.69	27.57	30.39
Design shear force, Vu	kN	30.7	403.8	618.5	877.5	981.8	968.6
Total shear Force , V _{Ed} =V _u +V _{Edi}	kN	34.69	411.40	628.35	892.20	1009.35	999.00
Maximum shear capacity V _{Rd,max}	kN	1513.58	1513.58	1513.58	1513.58	2910.83	2910.83
T _{Ed} /T _{Rd,max} + V _{Ed} /V _{Rd,max} ≤ 1.0		0.048	0.320	0.477	0.683	0.411	0.414
Check		OK	OK	OK	OK	OK	OK

Characteristic yield stress of steel , f_{yk}	=	500 N/mm ²	
Design yield strength of shear reinforcement, $f_{ywd} = 0.8 \cdot f_{yk}$	=	400 N/mm ²	Refer clause 10.3.3.2 of IRC:112-2011
Strength reduction factor , v_1	=	0.6	Refer clause 10.3.3.2 of IRC:112-2011
Design Compressive strength of concrete , f_{cd}	=	15.63 N/mm ²	

Overall Depth , D	mm	1600.0	1600.0	1600.0	1600.0	1600.0	1600.0
Effective cover	mm	0.163	0.163	0.163	0.163	0.110	0.110
Effective depth , d	mm	1599.8	1599.8	1599.8	1599.8	1599.9	1599.9
Width of web , bw	mm	325.00	325.00	325.00	325.00	625.00	625.00
Coefficient , α_{cp} for $\sigma_{cp} = 0$		1.00	1.00	1.00	1.00	1.00	1.00
Lever arm , $Z = 0.9 d$	mm	1439.85	1439.85	1439.85	1439.85	1439.90	1439.90
$1 \geq \cot \theta \geq 2.5$		2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22
$V_{Rd,max}$	KN	1513.58	1513.58	1513.58	1513.58	2910.83	2910.83

DESIGN OF LONGITUDINAL GIRDER**ULS DESIGN****Material Properties**

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

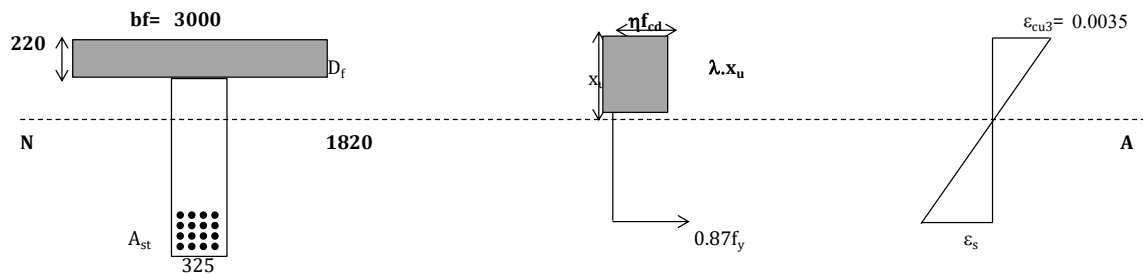
Design Constants

f_y	-	500 D	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{ctm}	-	2.77	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

LOADINGS**ULS Moments**

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1.35	858.63	804.64	644.53	378.31	276.78	326.58
2	DL of Slab	1.35	742.66	696.13	556.52	323.85	235.14	278.60
3	SIDL	1.35	40.95	40.35	38.28	35.37	17.83	9.86
4	Surfacing	1.75	290.18	272.26	272.17	218.39	218.22	128.43
5	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
6	Live Load (Class A + 70-R)	1.5	1906.41	1672.11	1159.04	589.37	456.93	313.25
7	Live Load (SV loading)	1	2654.75	2488.58	1986.40	1053.65	817.13	555.35
Total Design Moment M_{ED}			5584.46	5065.13	4135.79	2431.50	1914.18	1610.42

* γ – Partial safety factor

**At Midspan(L/2)**

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

Area of reinforcement provided A_{st} = **14476.5 mm²** Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = **163.33 mm**

Therefore, d_{eff} = **1.657 m**

Assuming N.A lies in the flange

$$b_f \cdot \eta f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

η	=	1.00	
λx_u	=	134.27 mm	N.A lies inside the flange
λ	=	0.8	
x_u	=	167.84 mm	

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$M_{ED} = M_{u,lim}$$

$$5.58E+09 = \eta \cdot f_{cd} \cdot \frac{x_{u,max}}{d} \cdot \left(1 - 0.4 \cdot \frac{x_{u,max}}{d}\right) b_f \cdot d^2$$

$$d_{reqd} = 506.2533 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$A_{st,Required} = 7925.01 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 775.89 \text{ mm}^2$$

$$b_t = \text{Mean width of tension Zone}$$

$$0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 148.46 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 8073.46 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 10009.694 \text{ KN-m} \quad \text{Safe}$$

At SECTION 2-2 (3L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

$$\text{Area of reinforcement provided } A_{st} = 14476.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 163.33 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.657 \text{ m}$$

Asssuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\eta = 1.00$$

$$\lambda x_u = 134.27 \text{ mm}$$

N.A lies inside the flange

$$\lambda = 0.8$$

$$x_u = 167.84 \text{ mm}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$M_{ED} = M_{u,lim}$$

$$5.07E+09 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 514.3945 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}} \right)$$

$$A_{st, \text{Required}} = 7172.58 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 775.89 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, \text{torsion}} = 283.88 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 7456.46 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 10009.694 \text{ KN-m} \quad \text{Safe}$$

At SECTION 3-3 (L/4)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	3	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	18		

$$\text{Area of reinforcement provided } A_{st} = 14476.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 163.33 \text{ mm}$$

$$\text{Therefore } d_{eff} = 1.657 \text{ m}$$

$$\text{Asssuming N.A lies in the flange} \quad b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 134.27 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 167.84 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u, \text{max}} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \varepsilon_{cu, 3}} \right)} = 1021.73 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$M_{ED} = M_{u, \text{lim}}$$

$$4.14 \text{E}+09 = \eta f_{cd} \cdot b_f \cdot x_{u, \text{max}} \cdot (d - 0.4 \cdot x_{u, \text{max}})$$

$$d_{reqd} = 495.00 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}} \right)$$

$$A_{st, \text{Required}} = 5834.26 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 775.89 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 699.94 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, \text{torsion}} = 368.67 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 6202.93 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 10009.694 \text{ KN-m} \quad \text{Safe}$$

At SECTION 4-4 (L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	5	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	15		

$$\text{Area of reinforcement provided } A_{st} = 12063.7 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 142.00 \text{ mm}$$

$$\text{Therefore } d_{eff} = 1.678 \text{ m}$$

Assuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda \cdot x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 111.89 \text{ mm} \quad \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 139.86 \text{ mm} \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} = 1034.89 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= M_{u,lim} \\ 2.43E+09 &= \eta f_{cd} \cdot b_f \cdot x_{u,max} \cdot (d - 0.4 \cdot x_{u,max}) \\ d_{reqd} &= 464.05 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$A_{st,Required} = 3362.39 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 785.88 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 708.96 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 552.36 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 3914.74 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 8512.0804 \text{ KN-m} \quad \text{Safe}$$

At Deff

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	0	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	10		

$$\text{Area of reinforcement provided } A_{st} = 8042.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 110.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.710 \text{ m}$$

Assuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\eta = 1.00$$

$$\lambda x_u = 74.59 \text{ mm}$$

$$\lambda = 0.8$$

$$x_u = 93.24 \text{ mm}$$

N.A lies inside the flange

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} = 1054.63 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$M_{ED} = M_{u,lim}$$

$$1.91\text{E}+09 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 460.55 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$A_{st,Required} = 2591.55 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 800.87 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 722.48 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 512.28 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 3103.83 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5851.9132 \text{ KN-m} \quad \text{Safe}$$

At Web Widening

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	5	32	78
2nd Layer	5	32	142
3rd Layer	0	32	206
4th Layer	0	32	270
5th Layer	0	32	334
6th Layer	0	32	398
7th Layer	0	32	462
8th Layer	0	25	523
Total no. of bars	10		

$$\text{Area of reinforcement provided } A_{st} = 8042.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 110.00 \text{ mm}$$

$$\text{Therefore, } d_{\text{eff}} = 1.710 \text{ m}$$

Assuming N.A lies in the flange

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 74.59 \text{ mm} & \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 93.24 \text{ mm} \end{aligned}$$

$$x_{u,\text{max}} = 1054.63 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{\text{ED}} &= \frac{d_{\text{eff}}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu3}}\right)} \\ 1.61\text{E}+09 &= \eta f_{cd} \cdot b_f \cdot x_{u,\text{max}} \cdot (d - 0.4 \cdot x_{u,\text{max}}) \\ d_{\text{reqd}} &= 454.41 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot \left(d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}}\right)$$

$$\begin{aligned} A_{\text{st,Required}} &= 2177.85 \text{ mm}^2 \\ A_{\text{s,min}} &= 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{\text{eff}} = 800.87 \text{ mm}^2 \\ &= 0.0013 \cdot b_t \cdot d_{\text{eff}} = 722.48 \text{ mm}^2 \end{aligned}$$

$$\text{due to torsion} \quad A_{\text{st,torsion}} = 564.58 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 2742.42 \text{ mm}^2$$

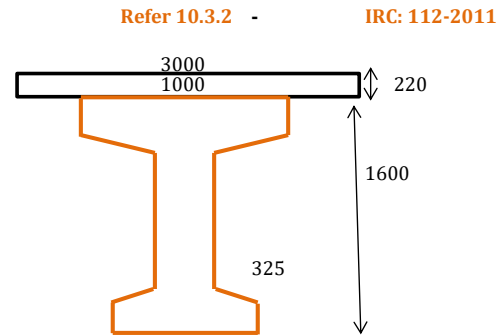
$$\text{Moment Resisting capacity } M_R = 5851.9132 \text{ KN-m} \quad \text{Safe}$$

Summary

SNO	Description	unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Design Bending Moment M_{ED}	KN-m	5584.46	5065.13	4135.79	2431.50	1914.18	1610.42
	Area of Steel required (flexure+torsion)	mm ²	8073.46	7456.46	6202.93	3914.74	3103.83	2742.42
2	Area of Steel Provided	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
	Check for steel provided		OK	OK	OK	OK	OK	OK
3	Depth of Neutral Axis x_u	mm	167.84	163.33	163.33	142.00	110.00	93.24
4	Critical Depth of N.A $x_{u,\text{max}}$	mm	1021.73	1021.73	1021.73	1034.89	1054.63	1054.63
5	Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
6	Total Depth required D_{reqd}	mm	646.25	654.39	635.00	604.05	600.55	594.41
7	Total Depth Provided	mm	1820.00	1820.00	1820.00	1820.00	1820.00	1820.00
8	Area of Steel Required A_{st}	mm ²	7925.01	7172.58	5834.26	3362.39	2591.55	2177.85
9	Moment of Resistance M_R	KN-m	10009.69	10009.69	10009.69	8512.08	5851.91	5851.91
Check for Moment Capacity			OK	OK	OK	OK	OK	OK
10	Additional tensile Force ΔF_d	KN	24.83	105.93	203.02	305.47	346.66	368.89
11	$M_{\text{ED}}/Z + \Delta F_d$	KN	3745.444	3397.13886	2773.8387	1630.784	1283.8196	1080.0953
12	M_{RD}/Z	KN	6713.41	6713.40968	6713.4097	5708.974	3924.8244	3924.8244
Check (As per 16.5.1.3)			OK	OK	OK	OK	OK	OK

DESIGN OF LONGITUDINAL GIRDER FOR SHEAR**Material Properties**

Grade of Concrete	-	M	35	
Grade of Reinforcement	-	Fe	500	
Elastic Modulus of Concrete	-		32000	N/mm ²
Elastic Modulus of Steel	-		200000	N/mm ²
f_y	=		500	N/mm ²
f_{yd}	=		435	N/mm ²
f_{ck}	=		35	N/mm ²
f_{ctm}	=		2.77	N/mm ²
f_{cd}	=		15.63	N/mm ²

* γ - Partial Safety factor

Shear corresponding to torsion and flexure = 1009.35 KN

Bending Moment at critical section = 1914.18 KN-m

$\tan \beta = 0.00$

Effective Depth d_{eff} = 1710.00 m

Net shear force $S = M \cdot \tan \beta / d'$ = 1009.35 KN

Hence, Shear stress = $\frac{1.706 \text{ N/mm}^2}{1706.42 \text{ KN/m}^2}$

Design Shear resistance of the member (without Shear R/F) $V_{Rd,c} = \left[0.12K(80 \cdot \rho_1 \cdot f_{ck})^{0.33} + 0.15 \cdot \sigma_{cp} \right] b_w \cdot d_{eff}$
= 297 KN

Minimum Shear resistance $U_{Rd,c} = (v_{min} + 0.15 \sigma_{cp}) b_w \cdot d_{eff}$
= 158 KN

Mean Compressive stress $\sigma_{cp} = 0.2 f_{cd} = 3.13 \text{ Mpa}$
(Limited to this value)
 $N_{Ed}/A_c < 0.2 f_{cd}$
0.00

Applied Longitudinal Force $N_{Ed} = 0.00$

Design value of concrete compressive strength $f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_m} = 15.633$

$v_{min} = 0.031 K^{3/2} \cdot f_{ck}^{1/2}$

$K = 1 + \sqrt{\frac{200}{d_{eff}}} = 1.342$

$\rho_1 = \frac{A_s}{b_w \cdot d_{eff}} < 0.02$

% of reinforcement = $\frac{0.01360}{1.360} = 0.01$

Check for Shear resistance = $V_{Rd,Max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}$

$\alpha_{cw} = 1.00$ for $\sigma_{cp} = 0$

$b_w = 325.00$

$z = 1.54$

$v_1 = 0.60$

$V_{Rd,Max} = 4691.64 \text{ KN}$

Design Shear resistance = 297.44 KN

Design Shear Force = 1009.35 KN

Provide Shear Reinforcement

SHEAR REINFORCEMENT

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot \theta \quad \longrightarrow \quad \frac{A_{sw}}{s} = 0.001640 \text{ mm}^2/\text{m}$$

$$V_{Rd,s} = 1009.35 \text{ KN}$$

$$Z = 1539$$

$$f_{ywd} = 0.8 \cdot f_y = 400 \text{ N/mm}^2$$

$$\cot \theta = 2.5$$

$$S = \text{Spacing of stirrups}$$

$$\text{For combined Shear and Torsion } A_{swt,Reqd} = 0.00164 \text{ mm}^2/\text{mm}$$

$$\text{Using } 2 \text{ L- } 12 @ A_{sw} = 1.51 \text{ mm}^2/\text{mm} \quad \text{OK}$$

$$\text{Spacing to be Provided } S = 150 \text{ mm}$$

Minimum Shear Reinforcement

$$\text{Shear Reinforcement ratio } \rho_w = \frac{A_{sw}}{(s \cdot b_w \cdot \sin \alpha)} = 0.0309 \quad \text{OK}$$

$$\text{Minimum Shear Reinforcement ratio } \rho_{w,min} = \frac{(0.072 \cdot \sqrt{f_{ck}})}{f_{yk}} = 0.00085 \quad \text{Refer 10.3.2 IRC: 112-2011}$$

INTERFACE SHEAR

Refer 10.3.4 IRC: 112-2011

Design Shear at Interface

$$\begin{aligned} \text{Transverse Shear } V_{Ed} &= 1009.35 \text{ KN} \\ \text{Inner lever arm of composite section } Z &= 1539 \text{ m} \\ \text{Interface width } b_i &= 325.00 \text{ mm} \\ \text{Interface area} &= 325000 \text{ mm}^2 \\ \text{Proportion of Longitudinal tensile force carried in top sl } \beta &= 1.00 \\ \text{Interface Shear Stress } \frac{A_{sf}}{S_f} &= \frac{\beta \cdot V_{Ed}}{Z \cdot b_i} = 2.018 \text{ N/mm}^2 \end{aligned}$$

Design Shear Resistance at Interface

$$\begin{aligned} \text{Interface shear capacity at section } V_{Rdi} &= \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot [\mu \sin \alpha + \cos \alpha] \geq 0.5 \cdot v \cdot f_{cd} \\ \text{Factor dependent on interface roughness } \mu &= 0.6 \\ \text{Stress resulting from forces normal to interface } \sigma_n &= 0.00 \text{ Mpa} \\ \text{Angle of reinforcement to interface } \alpha &= 90 \text{ degree} \\ \text{Strength reduction factor for concrete cracked in shear } v &= 0.5 \\ V_{Rdi} &= 4.03 \text{ N/mm}^2 \quad \text{OK} \end{aligned}$$

$$\begin{aligned} \text{Minimum reinforcement } A_{ci} &= 487.500 \text{ mm}^2 \\ \text{Provide } 10 @ 125 &= 628.3 \text{ mm}^2 \end{aligned}$$

SHEAR IN FLANGE PORTION

Refer 10.3.5 IRC: 112-2011

S.No	Type of Loading	γ^*	At support	At L/8	At L/4	At 3L/8	At L/2
1	Dead Load-Girder	1.35	2.90	758.87	1297.51	1620.69	1728.42
2	Dead Load-Slab	1.35	1.36	525.01	900.99	1126.57	1201.77
3	SIDL	1.35	40.95	40.35	38.28	35.37	17.83
4	Surfacing	1.75	0.81	134.02	243.45	303.98	324.14
5	Live Load	1.5	27.96	393.92	663.17	749.83	791.15
TOTAL			104.40	2613.14	4440.44	5413.25	5733.78

* γ - Partial Safety factor

$$\text{Longitudinal shear stress across junction of flange with web } V_{Edf} = \frac{\Delta F_d}{h_f \cdot \Delta x}$$

$$\begin{aligned} Z &= \text{Inner lever arm of composite section} = 1539 \text{ m} \\ h_f &= \text{Thickness of Flange} = 220 \text{ mm} \\ \Delta F_d &= \text{Change in Normal Force (unbalanced in plane SF at interface Section)} = 4336.05 \text{ KN} \end{aligned}$$

ϕ	-	Strength reduction factor	=	0.52
f_{ctd}	-		=	1.85

Description	Support - L/8		L/8 - L/4		L/4 - 3L/8		3L/8 - L/2	
ΔFd	1630.11		1187.33		632.10		208.27	
Δx	3.125		3.125		3.125		3.125	
$V_{edf} = \frac{\Delta Fd}{h_f \cdot \Delta x}$	2.37107		1.72703		0.91943		0.30294	
$\cot \theta_f$	2.00		2.00		2.00		2.00	
$\frac{A_{sw}}{s} \cdot \frac{V_{edf} \cdot h_f}{f_{yd} \cdot \cot \theta_f}$	599.58		436.72		232.50		76.60	
Provide	2L- 12 Dia		2L- 12 Dia		2L- 12 Dia		2L- 12 Dia	
A_{sf}	1809.557		1809.557		1507.964		1507.964	
S_f	125		125		150		150	
$V_{Ed} \leq \phi \cdot f_{cd} \cdot \sin \theta_f \cdot \cos \theta_f$	3.23	OK	3.23	OK	3.23	OK	3.23	OK
$0.4f_{ctd}$	0.74		0.74		0.74		0.74	

STRESS IN GIRDER DUE TO DEAD WEIGHT OF SLAB**SLS CHECK AT EACH STAGE**

IRC:112-2011 7.7.1

Material Properties

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

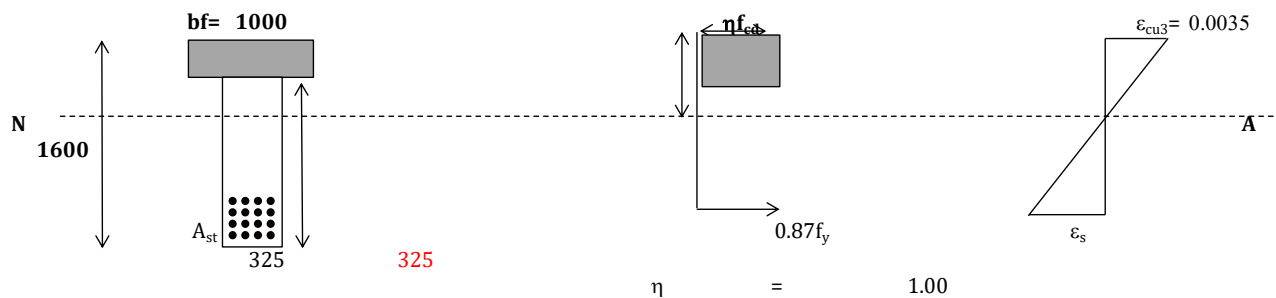
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	30.57	N/mm ²	at 14 day
f_{cm}	-	40.57	N/mm ²	
f_{ctm}	-	2.50	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1.35	858.63	804.64	644.53	378.31	276.78	326.58
2	DL of Slab	1.35	742.66	696.13	556.52	323.85	235.14	278.60
Total Design Moment M_{ED}			2161.74	2026.03	1621.42	947.92	691.09	816.99

* γ – Partial safety factor

DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	2161.74	2026.03	1621.42	947.92	691.09	816.99
b_{eff}	mm	698.38	698.38	698.38	763.63	973.57	973.57
Area of reinforcement provided A_{st}	mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}	m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A	mm	720.97	720.97	720.97	549.47	287.32	287.32
Critical Depth of N.A	mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	3643.10	3402.43	2695.21	1523.60	1077.38	1276.12
Moment Resisting capacity M_R	KN-m	7230.9977	7230.998	7230.9977	6497.776	4810.6525	4810.6525

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1	858.63	804.64	644.53	378.31	276.78	326.58
2	DL of Slab	1	742.66	696.13	556.52	323.85	235.14	278.60
Total Design Moment M_{ED}			1601.29	1500.76	1201.05	702.16	511.92	605.18

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	1601.29	1500.76	1201.05	702.16	511.92	605.18
b		mm	698.38	698.38	698.38	763.63	973.57	973.57
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		m ²	0.8	0.8	0.8	0.8	0.9	0.9
u		m	4.8	4.8	4.8	4.8	4.8	4.8
2Ac/u			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			3.0	3.0	3.0	3.0	3.0	3.0
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(f_{cm})$			2.9	2.9	2.9	2.9	2.9	2.9
$\beta(t_0)$			0.7	0.7	0.7	0.7	0.7	0.7
t			7.0	7.0	7.0	7.0	7.0	7.0
t_0			3.0	3.0	3.0	3.0	3.0	3.0
$t-t_0$			4.0	4.0	4.0	4.0	4.0	4.0
$\beta(t, t_0)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			788.1	788.1	788.1	788.1	900.5	900.5
$E_{c,eff}$			19827.5	19827.5	19827.5	19827.5	20250.2	20250.2
Modular Ratio	m		10.09	10.09	10.09	10.09	9.88	9.88
Depth of N.A	xu	mm	593.72	593.72	593.72	540.70	418.20	418.20
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.25E+09	2.25E+09	2.25E+09	1.70E+09	8.02E+08	8.02E+08
Moment of Inertia of transformed area	I_S	mm ⁴	1.03E+10	1.03E+10	1.03E+10	1.02E+10	9.24E+09	9.24E+09
Modulus of Section of Concrete	Z_C	mm ³	3.79E+06	3.79E+06	3.79E+06	3.14E+06	1.92E+06	1.92E+06
Modulus of Section of Steel	Z_S	mm ³	1.22E+07	1.22E+07	1.22E+07	1.11E+07	8.62E+06	8.62E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.25E+10	1.25E+10	1.25E+10	1.18E+10	1.00E+10	1.00E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	7.52	7.05	5.64	3.18	2.16	2.55
At C.G of Steel	fy'	N/mm ²	107.69	100.93	80.77	54.36	54.64	64.60
At bottom fibre of Steel	fy''	N/mm ²	118.59	111.15	88.95	58.15	56.27	66.53

STRESS IN GIRDER DUE TO GREEN WEIGHT OF SLAB**SLS CHECK AT EACH SECTION****Material Properties**

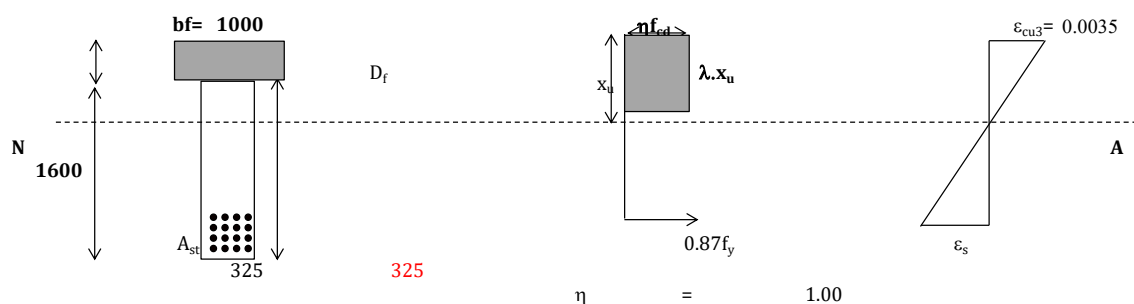
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.50	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)				
			L/2	3L/8	L/4	L/8	Deff
1	Green weight of slab	1.35	29.71	27.85	22.26	12.95	9.41
Total Design Moment M_{ED}			40.10	37.59	30.05	17.49	12.70
							15.04



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	40.10	37.59	30.05	17.49	12.70	15.04
b_{eff}	mm	700.44	1031.49	1031.49	1132.14	1407.80	1407.80
Area of reinforcement provided A_{st}	mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of girder, d_{eff}	m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A	mm	718.85	488.14	488.14	370.62	198.70	198.70
Critical Depth of N.A	mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	64.23	60.18	48.11	27.58	19.59	23.21
Moment Resisting capacity M_R	KN-m	7236.34	7817.482	7817.4825	6873.205	4934.6744	4934.6744

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	DL of Precast Girder	1	29.71	27.85	22.26	12.95	9.41	11.14
Total Design Moment M_{ED}			29.71	27.85	22.26	12.95	9.41	11.14

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	29.71	27.85	22.26	12.95	9.41	11.14
b		mm	700.44	1031.49	1031.49	1132.14	1407.80	1407.80
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	0.8	0.8	0.8	0.8	0.9	0.9
u		mm	4.8	4.8	4.8	4.8	4.8	4.8
2Ac/u			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			2.4	2.4	2.4	2.4	2.4	2.4
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
t			14.0	14.0	14.0	14.0	14.0	14.0
t_0			7.0	7.0	7.0	7.0	7.0	7.0
t-t ₀			7.0	7.0	7.0	7.0	7.0	7.0
$\beta(t, t_0)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			788.1	788.1	788.1	788.1	900.5	900.5
$E_{c,eff}$			20148.9	20148.9	20148.9	20148.9	20565.8	20565.8
Modular Ratio	m		9.93	9.93	9.93	9.93	9.72	9.72
Depth of N.A	xu	mm	589.55	508.52	508.52	455.57	347.83	347.83
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.24E+09	1.44E+09	1.44E+09	1.03E+09	4.69E+08	4.69E+08
Moment of Inertia of transformed area	I_S	mm ⁴	1.04E+10	1.25E+10	1.25E+10	1.21E+10	1.05E+10	1.05E+10
Modulus of Section of Concrete	Z_C	mm ³	3.79E+06	2.82E+06	2.82E+06	2.27E+06	1.35E+06	1.35E+06
Modulus of Section of Steel	Z_S	mm ³	1.23E+07	1.34E+07	1.34E+07	1.21E+07	9.19E+06	9.19E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.26E+10	1.39E+10	1.39E+10	1.32E+10	1.10E+10	1.10E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	0.14	0.10	0.08	0.05	0.03	0.04
At C.G of Steel	fy'	N/mm ²	1.99	1.86	1.49	0.99	0.98	1.16
At bottom fibre of Steel	fy''	N/mm ²	2.19	2.03	1.62	1.05	1.01	1.19

STRESS IN GIRDER DUE TO CONSTRUCTION LOAD**SLS CHECK AT EACH SECTION****Material Properties**

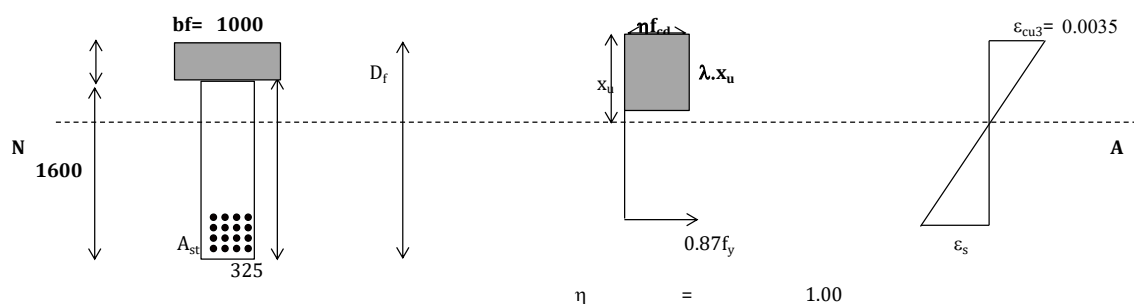
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)				
			L/2	3L/8	L/4	L/8	Deff
1	Construction Load	1	486.11	455.65	364.27	211.97	192.64
Total Design Moment M_{ED}			486.11	455.65	364.27	211.97	192.64



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	486.11	455.65	364.27	211.97	192.64	182.41
b	mm	700.52	700.52	700.52	748.08	928.16	928.16
Area of reinforcement provided A_{st}	mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}	m	1.44	1.44	1.44	1.46	1.49	1.49
Depth of N.A	mm	718.77	718.77	718.77	560.89	301.38	301.38
Critical Depth of N.A	mm	886.05	886.05	886.05	899.21	918.94	918.94
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	786.38	736.59	587.65	335.66	298.10	282.23
Moment Resisting capacity M_R	KN-m	7236.555	7236.555	7236.5546	6473.804	4790.9816	4790.9816

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Construction Load	1	486.11	455.65	364.27	211.97	192.64	182.41
Total Design Moment M_{ED}			486.11	455.65	364.27	211.97	192.64	182.41

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	486.11	455.65	364.27	211.97	192.64	182.41
b		mm	700.52	700.52	700.52	748.08	928.16	928.16
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	0.8	0.8	0.8	0.8	0.9	0.9
u		mm	4.8	4.8	4.8	4.8	4.8	4.8
2Ac/u			311.9	311.9	311.9	311.9	377.0	377.0
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.5	0.5
ϕ_0			2.4	2.4	2.4	2.4	2.4	2.4
ϕ_{RH}			1.4	1.4	1.4	1.4	1.3	1.3
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
t			14.0	14.0	14.0	14.0	14.0	14.0
t_0			7.0	7.0	7.0	7.0	7.0	7.0
$t-t_0$			7.0	7.0	7.0	7.0	7.0	7.0
$\beta(t, t_0)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			791.3	791.3	791.3	791.3	924.5	924.5
$E_{c,eff}$			20161.9	20161.9	20161.9	20161.9	20646.1	20646.1
Modular Ratio	m		9.92	9.92	9.92	9.92	9.69	9.69
Depth of N.A	xu	mm	589.39	589.39	589.39	536.62	414.29	414.29
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.24E+09	2.24E+09	2.24E+09	1.69E+09	7.95E+08	7.95E+08
Moment of Inertia of transformed area	I_S	mm ⁴	1.04E+10	1.04E+10	1.04E+10	1.02E+10	9.31E+09	9.31E+09
Modulus of Section of Concrete	Z_C	mm ³	3.79E+06	3.79E+06	3.79E+06	3.14E+06	1.92E+06	1.92E+06
Modulus of Section of Steel	Z_S	mm ³	1.23E+07	1.23E+07	1.23E+07	1.11E+07	8.65E+06	8.65E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.26E+10	1.26E+10	1.26E+10	1.19E+10	1.01E+10	1.01E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	2.29	2.14	1.71	0.96	0.82	0.77

At C.G of Steel	fy'	N/mm ²	32.61	30.57	24.44	16.37	20.51	19.42
At bottom fibre of Steel	fy''	N/mm ²	35.90	33.65	26.90	17.51	21.12	20.00

STRESS IN GIRDER DUE TO SIDL AND SURFACING**SLS CHECK AT EACH SECTION****Material Properties**

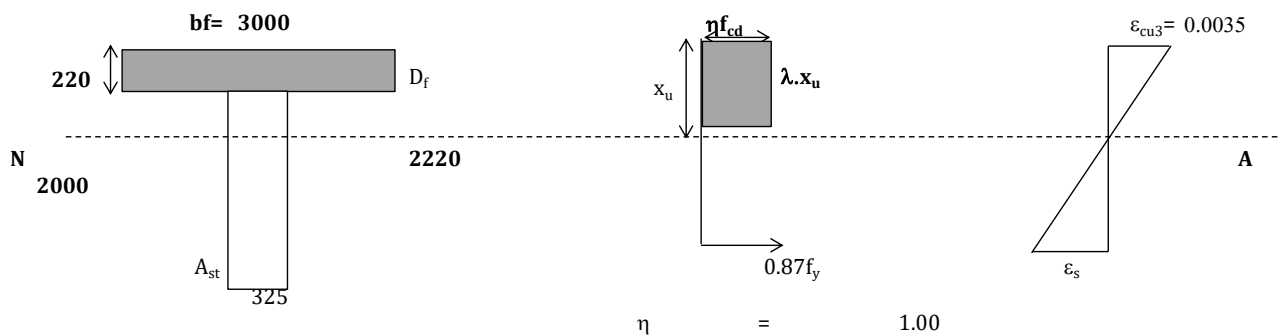
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	SIDL	1.35	40.95	40.35	38.28	35.37	17.83	9.86
2	Surfacing	1.75	290.18	272.26	272.17	218.39	218.22	128.43
Total Design Moment M_{ED}			563.10	530.94	527.97	429.94	405.96	238.08

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	563.10	530.94	527.97	429.94	405.96	238.08
b		mm	2495.66	2496.95	2496.95	2572.28	2694.08	2690.06
Area of reinforcement provided A_{st}		mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}		m	2.06	2.06	2.06	2.08	2.11	2.11
Depth of N.A		mm	201.75	201.65	201.65	163.12	103.83	103.99
Critical Depth of N.A		mm	1268.43	1268.43	1268.43	1281.59	1301.32	1301.32
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	630.48	594.41	591.09	476.22	442.78	259.55
Moment Resisting capacity M_R		KN-m	12443.16	12443.425	12443.425	10562.35	7236.4875	7236.2703

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	SIDL	1	40.95	40.35	38.28	35.37	17.83	9.86
2	Surfacing	1.2	324.14	303.98	243.45	134.02	88.35	71.72
Total Design Moment M_{ED}			429.92	405.12	330.43	196.20	123.85	95.93

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	429.92	405.12	330.43	196.20	123.85	95.93
b		mm	2495.66	2496.95	2496.95	2572.28	2694.08	2690.06
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	1.5	1.5	1.5	1.5	1.6	1.5
u		mm	10.3	10.3	10.3	10.3	10.3	10.3
2Ac/u			285.9	285.9	285.9	285.9	316.5	285.9
$\phi(t-t_0)$			0.9	0.9	0.9	0.9	0.9	0.9
ϕ_0			2.2	2.2	2.2	2.2	2.1	2.2
ϕ_{RH}			1.4	1.4	1.4	1.4	1.4	1.4
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
t			56.0	56.0	56.0	56.0	56.0	56.0
t_0			14.0	14.0	14.0	14.0	14.0	14.0
t-t ₀			42.0	42.0	42.0	42.0	42.0	42.0
$\beta(t, t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
βH			743.3	753.7	753.7	753.7	826.7	753.7
$E_{c,eff}$			16894.6	16926.1	16926.1	16926.1	17209.2	16926.1
Modular Ratio	m		11.84	11.82	11.82	11.82	11.62	11.82
Depth of N.A	xu	mm	467.22	466.73	466.73	425.22	344.66	347.50
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	8.62E+09	8.60E+09	8.60E+09	6.51E+09	3.52E+09	3.55E+09
Moment of Inertia of transformed area	I_S	mm ⁴	3.66E+10	3.66E+10	3.66E+10	3.30E+10	2.51E+10	2.50E+10
Modulus of Section of Concrete	Z_C	mm ³	1.84E+07	1.84E+07	1.84E+07	1.53E+07	1.02E+07	1.02E+07
Modulus of Section of Steel	Z_S	mm ³	2.30E+07	2.30E+07	2.30E+07	1.99E+07	1.42E+07	1.42E+07
Cracked Moment of Inertia	I_{CR}	mm ⁴	4.52E+10	4.52E+10	4.52E+10	3.95E+10	2.86E+10	2.85E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	0.38	0.35	0.29	0.18	0.13	0.10
Top of Girder	fck''	N/mm ²	0.20	0.19	0.15	0.09	0.05	0.04
At C.G of Steel	fy'	N/mm ²	15.12	14.25	11.62	8.22	7.65	5.93
At bottom fibre of Steel	fy''	N/mm ²	15.93	15.71	12.82	8.92	8.12	6.29

STRESS IN GIRDER DUE TO CWLL**SLS CHECK AT EACH SECTION****Material Properties**

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

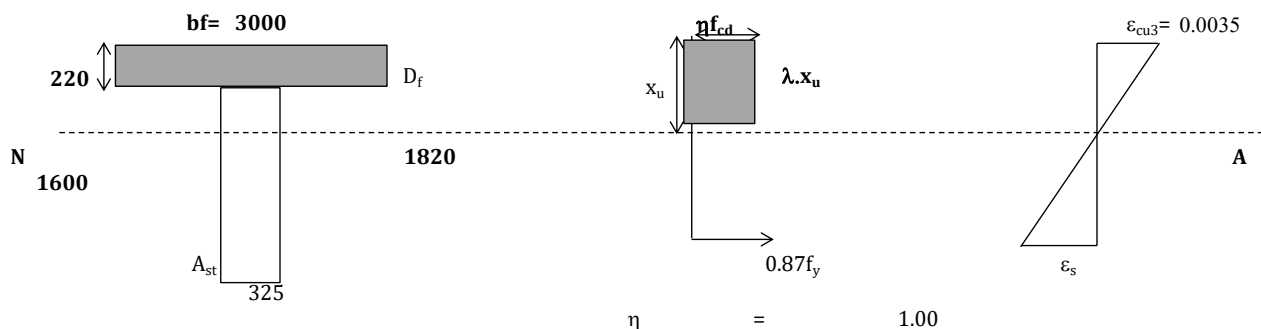
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	
f_{yd}	-	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	-	35	N/mm ²	
f_{cm}	-	45	N/mm ²	
f_{ctm}	-	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	-	15.63	N/mm ²	
Clear Cover to Reinforcement	=	50	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Live Load (class-A+70 R)	1.5	1906.41	1672.11	1159.04	589.37	456.93	313.25
2	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
3	Live Load (SV Loading)	1	2654.75	2488.58	1986.40	1053.65	817.13	555.35
Impact factor		1.180	2249.57	1973.09	1367.67	695.46	539.18	369.63
Lane Reduction factor		0.900	2024.61	1775.78	1230.90	625.91	485.26	332.67
Total Design Moment M_{ED}			3036.92	2663.67	1986.40	1053.65	817.13	555.35

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	3036.92	2663.67	1986.40	1053.65	817.13	555.35
b		mm	2651.41	2651.88	2651.88	2647.14	2566.38	2569.76
Area of reinforcement provided A_{st}		mm ²	14476	14476	14476	12064	8042	8042
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	163.33	163.33	163.33	142.00	110.00	110.00
Effective depth of Girder, d_{eff}		m	1.66	1.66	1.66	1.68	1.71	1.71
Depth of N.A		mm	189.90	189.87	189.87	158.51	109.00	108.85
Critical Depth of N.A		mm	1021.73	1021.73	1021.73	1034.89	1054.63	1054.63
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	4271.94	3740.52	2780.89	1450.08	1102.36	748.37
Moment Resisting capacity M_R		KN-m	9954.112	9954.19666	9954.1967	8472.946	5829.8664	5830.067

CHECK FOR STRESS

Stresses are calculated using Cracked section analysis

SLS MOMENTS

S.No	Type of Loading	γ	BM(KN-m)					
			L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Live Load(Class A +70-R)	1	1906.41	1672.11	1159.04	589.37	456.93	313.25
2	FPLL	1	0.00	0.00	0.00	0.00	0.00	0.00
3	Live Load(SV Loading)	1	2654.75	2488.58	1986.40	1053.65	817.13	555.35
Impact factor		1.180	2249.57	1973.09	1367.67	695.46	539.18	369.63
Lane Reduction factor		0.900	2024.61	1775.78	1230.90	625.91	485.26	332.67
Total Design Moment M_{ED}			2654.75	2488.58	1986.40	1053.65	817.13	555.35

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	2654.75	2488.58	1986.40	1053.65	817.13	555.35
b		mm	2651.41	2651.88	2651.88	2647.14	2566.38	2569.76
Area of steel Provided	A_{st}	mm ²	14476.5	14476.5	14476.5	12063.7	8042.5	8042.5
Ac		mm ²	1.5	1.5	1.5	1.5	1.6	1.5
u		mm	10.3	10.3	10.3	10.3	10.3	10.3
2Ac/u			285.9	285.9	285.9	285.9	316.5	285.9
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			1.7	1.7	1.7	1.7	1.6	1.7
ϕ_{RH}			1.4	1.4	1.4	1.4	1.4	1.4
$\beta(f_{cm})$			2.8	2.8	2.8	2.8	2.8	2.8
$\beta(t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
t			84.0	84.0	84.0	84.0	84.0	84.0
t_0			56.0	56.0	56.0	56.0	56.0	56.0
$t-t_0$			28.0	28.0	28.0	28.0	28.0	28.0
$\beta(t,t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
β_H			743.3	753.7	753.7	753.7	826.7	753.7
$E_{c,eff}$			19857.4	19887.8	19887.8	19887.8	20157.5	19887.8
Modular Ratio	m		10.07	10.06	10.06	10.06	9.92	10.06
Depth of N.A	x_u	mm	375.39	375.11	375.11	346.53	291.38	292.98
Goal seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	5.25E+09	5.25E+09	5.25E+09	4.14E+09	2.49E+09	2.50E+09
Moment of Inertia of transformed area	I_S	mm ⁴	2.38E+10	2.38E+10	2.38E+10	2.14E+10	1.62E+10	1.61E+10
Modulus of Section of Concrete	Z_C	mm ³	1.40E+07	1.40E+07	1.40E+07	1.19E+07	8.56E+06	8.54E+06
Modulus of Section of Steel	Z_S	mm ³	1.85E+07	1.86E+07	1.86E+07	1.61E+07	1.14E+07	1.14E+07
Cracked Moment of Inertia	I_{CR}	mm ⁴	2.90E+10	2.90E+10	2.90E+10	2.55E+10	1.87E+10	1.86E+10
Actual Stresses								
Top of Slab	fck'	N/mm ²	3.41	3.20	2.55	1.42	1.28	0.87
Top of Girder	fck''	N/mm ²	1.41	1.32	1.06	0.52	0.31	0.22
At C.G of Steel	fy'	N/mm ²	117.22	109.88	87.71	54.96	62.06	42.20
At bottom fibre of Steel	fy''	N/mm ²	125.03	123.89	98.89	60.82	66.87	45.47

CHECK FOR DEFLECTION

Refer 12.4.1 IRC 112-2011

Allowable deflection due to Vehicular load	=	$\frac{Span}{800}$	=	31.25 mm
Deflection due to LL(70R Wheeled)	=	5.995	mm	
Deflection due to LL(1 lane of class A)	=	2.8	mm	
Deflection due to LL(SV Loading)	=	8.74	mm	
Deflection due to LL (70R W + Class A/sv loading)	=	8.795	mm	< 31.25 mm

SUMMARY OF STRESSES IN INNER GIRDER**Allowable Stresses**

Concrete	16.8	N/mm ²
Steel	300	N/mm ²

As per Clause 5.3.2.5 of IRC 112 Amendment No. 1, Aug 2013

Description

Stress Location	
Top of Slab	f _{ck} '
Top of Girder	f _{ck} ''
At C.G of Steel	f _y '
At bottom fibre of Steel	f _y ''

LC-1 STAGE 1 DL + Green Weight of Slab + Construction Load AT 14 DAY

SLS - RARE		L/2		
LOAD	Load Factor	f _{ck} ''	f _y '	f _y ''
		N/mm ²		
DEAD LOAD	1	7.52	107.69	118.59
DL-Green Weight	1	0.14	1.99	2.19
Construction Load	1	2.29	32.61	35.90
Resultant Stress		9.95	142.30	156.69
CHECK		SAFE		

3L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
7.05	100.93	111.15
0.10	1.86	2.03
2.14	30.57	33.65
9.29	133.36	146.83
SAFE		

L/4		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
5.64	80.77	88.95
0.08	1.49	1.62
1.71	24.44	26.90
7.44	106.70	117.47
SAFE		

L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
3.18	54.36	58.15
0.05	0.99	1.05
0.96	16.37	17.51
4.18	71.72	76.71
SAFE		

Deff		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.16	54.64	56.27
0.03	0.98	1.01
0.82	20.51	21.12
3.01	76.14	78.41
SAFE		

Web Widening		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.55	64.60	66.53
0.04	1.16	1.19
0.77	19.42	20.00
3.36	85.18	87.72
SAFE		

LC-2 STAGE 2 DL - Construction Load + SIDL + LL AFTER 84 Days

SLS - RARE		L/2				3L/8				L/4				L/8				Deff				Web Widening			
LOAD	Load Factor	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''	f _{ck} '	f _{ck} ''	f _y '	f _y ''
		N/mm ²				N/mm ²				N/mm ²				N/mm ²				N/mm ²				N/mm ²			
DL+Construction	1	0	9.81	140.30	154.49	0	9.19	131.50	144.80	0	7.35	105.21	115.85	0	4.14	70.73	75.66	0	2.97	75.16	77.40	0	3.32	84.02	86.53
-ve'Construction Load	1	0.00	-2.29	-32.61	-35.90	0.00	-2.14	-30.57	-33.65	0.00	-1.71	-24.44	-26.90	0.00	-0.96	-16.37	-17.51	0.00	-0.82	-20.51	-21.12	0.00	-0.77	-19.42	-20.00
SIDL	1	0.38	0.20	15.12	15.93	0.35	0.19	14.25	15.71	0.29	0.15	11.62	12.82	0.18	0.09	8.22	8.92	0.13	0.05	7.65	8.12	0.10	0.04	5.93	6.29
Live Load	1	3.41	1.41	117.22	125.03	3.20	1.32	109.88	123.89	2.55	1.06	87.71	98.89	1.42	0.52	54.96	60.82	1.28	0.31	62.06	66.87	0.87	0.22	42.20	45.47
Differential Shrinkage	1	-0.12	0.26	0.43	0.43	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67	-0.10	0.37	0.67	0.67
Resultant Stress		3.67	9.39	240.46	259.98	3.46	8.93	225.74	251.42	2.74	7.22	180.78	201.33	1.50	4.15	118.21	128.57	1.32	2.89	125.03	131.95	0.87	3.17	113.39	118.97
CHECK		SAFE				SAFE				SAFE				SAFE				SAFE				SAFE			

LC-3 Effect of Temperature Rise & Diff.Shrinkage Without CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '	f _{ck} '	f _y '
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
Temperature Rise	1	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020
Differential Shrinkage	1	-0.12	0.43	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		9.40	123.24	8.93	115.85	7.45	93.07	5.65	62.48	3.81	62.96	4.95	70.42
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-4 Effect of Temperature Rise & Diff.Shrinkage With CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
CWLL	1	3.41	117.22	3.20	109.88	2.55	87.71	1.42	54.96	1.28	62.06	0.87	42.20
Temperature Rise	0.6	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		12.18	240.71	11.48	225.74	9.36	180.78	5.65	118.21	4.45	125.02	4.39	113.39
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-5 Effect of Temperature Fall & Diff.Shrinkage Without CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
Temperature fall	1	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		9.10	120.97	8.60	113.34	7.13	90.56	4.56	60.74	3.49	60.45	3.85	68.68
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-6 Effect of Temperature Fall & Diff.Shrinkage With CWLL

SLS - RARE		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
CWLL	1	3.41	117.22	3.20	109.88	2.55	87.71	1.42	54.96	1.28	62.06	0.87	42.20
Temperature fall	0.6	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		12.51	238.19	11.80	223.22	9.69	178.26	5.98	115.70	4.78	122.51	4.72	110.88
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-7 Effect of Temperature Rise & Diff.Shrinkage

SLS - QUASI PERMENANT		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
Temperature Rise	0.5	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020	1.62	-0.0020
Differential Shrinkage	1	-0.12	0.43	-0.10	0.67	-0.10	0.67	0.67	-0.10	-0.10	0.67	0.67	-0.10
Resultant Stress		8.59	123.24	8.12	115.85	6.64	93.07	4.84	62.48	3.00	62.96	4.14	70.42
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

LC-8 Effect of Temperature Fall & Diff.Shrinkage

SLS - QUASI PERMENANT		L/2		3L/8		L/4		L/8		Deff		Web Widening	
LOAD	Load Factor	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'	f_{ck}'	f_y'
		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²		N/mm ²	
DL+SIDL	1	7.90	122.81	7.40	115.18	5.93	92.40	3.36	62.58	2.29	62.29	2.65	70.52
Temperature fall	0.5	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51	1.30	-2.51
Differential Shrinkage	1	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67	-0.10	0.67
Resultant Stress		8.45	122.23	7.95	114.60	6.48	91.81	3.91	61.99	2.84	61.71	3.20	69.94
CHECK		SAFE		SAFE		SAFE		SAFE		SAFE		SAFE	

CRACK WIDTH CALUCLATION

Permissible Crack width in moderate conditions of exposur = **0.3 mm**

Crack Width W_k at Midspan

$$S_{r, \max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = \mathbf{0.11 \text{ mm}} \quad \mathbf{OK} \quad \text{Refer Cl. 12.3.4 IRC 112}$$

$S_{r, \max}$	-	Maximum Crack spacing
ϵ_{sm}	-	Mean Strain in Reinforcement
ϵ_{cm}	-	Mean Strain in Concrete
σ_{sc}	-	123.24 N/mm ² (from Quasi permanant combination)
K_t	-	0.5
$f_{ct, \text{eff}}$	-	2.77127
$\rho_{p, \text{eff}}$	-	0.128
α_e	-	6.25
E_s	-	200000
A_s	-	12064 mm ²
c	-	50
ϕ	-	32
h	-	1820 mm
d	-	1656.67
x	-	947.60

$$h_{c, \text{eff}} = \text{Min} \begin{cases} 2.5(h-d) \\ (h-x)/3 \\ h/2 \end{cases} = \begin{matrix} 408.333 \\ 290.8 \\ 910 \end{matrix} = \mathbf{290.80 \text{ mm}}$$

$$\phi_{eq} = \frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2} = 32$$

$$A_{c, \text{eff}} = 94510 \text{ mm}^2$$

$$\epsilon_{sm} - \epsilon_{cm} = \frac{\sigma_{sc} - K_t \frac{f_{ct, \text{eff}}}{\rho_{p, \text{eff}}} (1 + \alpha_e \cdot \rho_{p, \text{eff}})}{E_s} \geq 0.6 \cdot \frac{\sigma_{sc}}{E_s}$$

$$= 0.00052 \geq 0.0004$$

For the case of Deformed bars associated with pure bending

$$S_{r, \max} = 3.4c + \frac{0.17 \cdot \phi}{\rho_{p, \text{eff}}} = 212.62$$

DESIGN OF END DIAPHRAGM

DESIGN OF LIFTING HOOK

Vertical Load on hook = **548.921** kN

Diameter of the cable 12.7

Cable provided = **5-12.7mm HTS strands 7 Ply class II**

Area of Bar = 493.5 mm² IRC: 112 -2011 Table 18.4

Ultimate capacity of Bar = 1861.196 N/mm²

Allowable direct tension stress = 0.87 fy 1619.24 N/mm²

Actual Direct tension = **1112.30** N/mm² **OK**

CHECK FOR BONDING

Bonding stress = **3.00** N/mm² (IRC: 112-2011 Table 15.3)

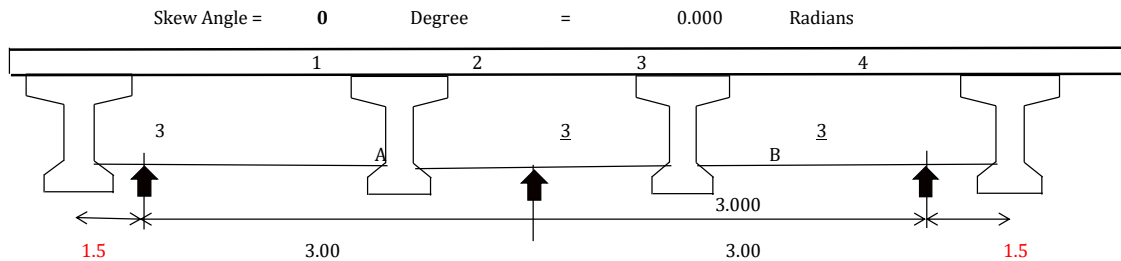
Ld $\frac{\phi \sigma_s}{4\tau_{bd}}$ = 1177.19 mm (IRC: 112-2011 Cl 15.2.3.3)

DESIGN OF END DIAPHRAGM**Material Specification**

f_{ck}	=	35	N/mm ²	
f_y	=	500	N/mm ²	
f_{yd}	=	435	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{ck}	=	35	N/mm ²	
f_{ctm}	=	2.8	N/mm ²	As per Table 6.5, IRC: 112:2011
f_{cd}	=	15.63	N/mm ²	
Clear cover to reinforcement	=	50	mm	
Elastic Modulus of Steel	E_s	=	200000	N/mm ²
Elastic Modulus of Concrete	E_{cm}	=	32000	N/mm ²

(Jacking condition)

All The Girder are supported on bearing and the reaction transferred to the bearing directly. Hence End Diaphragm is critical in Jacking up condition. The reactions below the each girder due to SIDL & DL has been calculated with the help of STAAD.

**Dead Load Load Over Each Bearing**

Bearing No.	DL (kN)	SIDL	Surfacing
1	381.70	126.017	67.792
2	367.97	41.017	60.608
3	367.97	41.017	60.608
4	381.70	126.017	67.792
5	0.00	0	0
6	0.00	0	0

No of Jacks	N	=	3	
	D	=	1570	mm
	B	=	0.4	
c/c distance between jacks	L1	=	3.00	m

Results from Staad Analysis

LOAD	λ FOR SLS	λ FOR ULS	Hogging Moment (kNm)	Sagging Moment (kNm)	Max Shear Force (kN)
Dead Load	1	1.35	300	793	492
SIDL	1	1.35	89	6.3	127
Surfacing	1	1.75	48	92.2	69
ULS			609.15	1240.41	956.40
SLS			437	891.5	

Vertical load on jack 1	=	696.74	KN
Vertical load on jack 2	=	696.74	KN
Vertical load on jack 3	=	0.00	KN

Maximum Jack Reaction	=	696.7365	KN	=	69.7	MT
				say	80.1	MT

$$L/D = 1.911 > 2.5 \text{ Design as Deep Beam}$$

Calculation of Hogging Reinforcement

$$\text{Design Bending Moment } M_z = 609.15 \text{ kNm}$$

$$b_f \cdot \eta \cdot f_{cd} \cdot \lambda_x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda_{x_u} &= 239026.26 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 298782.83 \text{ mm} \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 929.74 \text{ mm}$$

$$\begin{aligned} x_u &= 239026.260 \text{ mm} \\ x_{u,\max} &= 929.74 \text{ mm} \end{aligned} \quad \text{over reinforced}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= M_{u,\lim} \\ 6.09\text{E}+08 &= \eta \cdot f_{cd} \cdot \frac{x_{u,\max}}{d} \cdot (1 - 0.4 \cdot \frac{x_{u,\max}}{d}) \cdot b_f \cdot d^2 \\ d_{\text{reqd}} &= 14480.05 \text{ mm} \quad \text{OK} \end{aligned}$$

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot (d - \frac{0.87 \cdot A_{st} \cdot f_y}{2 \cdot b_f \cdot \eta \cdot f_{cd}})$$

$$A_{st,\text{Required}} = -18.08 \text{ mm}^2$$

Minimum Longitudinal Reinforcement

$$A_{s,\text{Min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b \cdot d$$

- Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011

Whichever is higher

$$\text{OR} = 0.0013 \cdot b \cdot d$$

-Refer Clause 16.9 of IRC:112-2011'

$$b = 0.40 \text{ mm}$$

$$d = 1507.50 \text{ mm}$$

$$A_{st\text{ min}} = 192.40 \text{ mm}^2/\text{m}$$

Governing Reinf. Ast

$$= 192.40 \text{ mm}^2/\text{m}$$

25	T @	5	Nos	$A_{st} =$	2454.37	mm^2	
25	T @	2	Nos	$A_{st} =$	981.75	mm^2	
0	T @	0	Nos	$A_{st} =$	0.00	mm^2	
				$A_{st} =$	3436.12	mm^2	Reinf. OK

$$\text{Hence } A_{st} \text{ provided} = 3436 \text{ mm}^2$$

Calculation of Span Reinforcement

$$\begin{aligned} \text{Design Bending Moment } M_z &= 1240.41 \text{ kNm} \\ \eta &= 1.00 \\ \lambda x_u &= 204879.65 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 256099.56 \text{ mm} \end{aligned}$$

$$x_{u,\max} = \frac{d_{\text{eff}}}{(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}})} = 929.74 \text{ mm}$$

$$x_u = 204879.651 \text{ mm}$$

$$x_{u,\max} = 929.74 \text{ mm} \quad \text{over reinforced}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= M_{u,\lim} \\ 1.24\text{E}+09 &= \\ d_{\text{reqd}} &= 20662.83 \text{ mm} \quad \text{OK} \end{aligned}$$

$$A_{st,\text{Required}} = 25.51 \text{ mm}^2$$

Minimum Longitudinal Reinforcement

$$A_{s,\text{Min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b \cdot d$$

- Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011

Whichever is higher

$$\text{OR} = 0.133 \cdot f_{ck} \cdot b \cdot d^2$$

-Refer Clause 16.9 of IRC:112-2011'

$$b = 0.40 \text{ mm}$$

$$d = 1507.50 \text{ mm}$$

$$A_{st\text{ min}} = 0.78 \text{ mm}^2/\text{m}$$

Governing Reinf. Ast

$$= 25.51 \text{ mm}^2/\text{m}$$

25	T @	3	Nos	$A_{st} =$	1472.62	mm^2	
25	T @	3	Nos	$A_{st} =$	1472.62	mm^2	
0	T @	0	Nos	$A_{st} =$	0.00	mm^2	
				$A_{st} =$	2945.24	mm^2	Reinf. OK

Hence Ast provided = 2945 mm²

DESIGN OF DIAPHRAGM BEAM FOR SHEAR

Shear Force	=	956.40 KN
Bending Moment at critical section	=	176.46 KN-m
$\tan \beta$	=	0.00
Effective Depth d_{eff}	=	1507.500 m
Net shear force $S = M \cdot \tan \beta / d'$	=	956.40 KN
Hence, Shear stress	=	1586.070 N/mm ²
Design Shear resistance of the member (without Shear R/F)	$V_{Rd,c}$	$= \left[0.12K(80\rho_1 \cdot f_{ck})^{0.33} + 0.15\sigma_{cp} \right] b_w d_{eff}$
		2 KN
Minimum Shear resistance	$v_{Rd,c}$	$= (v_{min} + 0.15\sigma_{cp}) b_w d_{eff}$
		0 KN
Mean Compressive stress	σ_{cp}	$= 0.2f_{cd} \quad 3.13 \text{ Mpa}$
		$\frac{N_{Ed}}{A_c} < 0.2f_{cd}$
Applied Longitudinal Force	N_{Ed}	0.00
Design value of concrete compressive strength	f_{cd}	$= \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_m} = 15.63$
	v_{min}	$= 0.031K^{3/2} \cdot f_{ck}^{1/2}$
	K	$= 1 + \sqrt{\frac{200}{d_{eff}}} = 1.36$
	ρ_1	$= \frac{A_s}{b_w \cdot d_{eff}} < 0.02$
% of reinforcement		$= \frac{5.69837}{569.837} \%$
Check for Shear resistance		$V_{Rd,Max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}$
α_{cw}	=	1.00 for $\sigma_{cp} = 0$
b_w	=	0.40 mm
z	=	1356.75 mm
v_1	=	0.60
$V_{Rd,Max}$	=	5.09 KN
Design Shear resistance	=	2.41 KN
Design Shear Force	=	956.40 KN
		Provide Shear Reinforcement

SHEAR REINFORCEMENT

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot \theta$$

$V_{Rd,s}$	=	956.40 KN
z	=	1356.75 mm
f_{ywd}	=	$0.87 \cdot f_{yk} = 435 \text{ N/mm}^2$
$\cot \theta$	=	2.5

$$\begin{aligned} \text{Using } 2 \text{ L-} & \quad S & \quad - & \quad \text{Spacing of stirrups} \\ & \quad 12 @ & \quad A_{sw} & \quad = & \quad 226.19 \text{ mm}^2 \\ \text{Spacing required } & \quad Sv & \quad = & \quad 348.96 \\ \text{Spacing to be Provided } & \quad S & \quad = & \quad 150 \text{ mm} \quad \sim \quad 150 \text{ mm} \end{aligned}$$

Minimum Shear Reinforcement

$$\text{Shear Reinforcement ratio } \rho_w = \frac{A_{sw}}{(s.b_w \cdot \sin \alpha)} = 3.7699 \quad \text{OK}$$

$$\text{Minimum Shear Reinforcement ratio } \rho_{w,min} = \frac{(0.072 \cdot \sqrt{f_{ck}})}{f_{yk}} = 0.00085$$

STRESS CHECK FOR SUPPORTS**SLS Moments at Supports**

S.No	Type of Loading	γ	BM(KN-m)
1	DL+SIDL	1	891.50
Total			891.50

KN-m

* γ – Partial safety factor-Rare combination**CHECK FOR STRESS**

Stresses are calculated using Cracked section analysis

Limiting Stresses

Sno	Combination	Allowable Stresses		Actual Stress	Check
1	Concrete	$0.48f_{ck}$	16.8	0.04	Safe
2	Steel		300	-0.73	Safe

$$\begin{aligned} \text{Area of steel Provided } & \quad A_{st} & \quad = & \quad 2945.2 \text{ mm}^2 \\ \text{Depth of Neutral axis } & \quad x_u & \quad = & \quad 256099.6 \text{ mm} \end{aligned}$$

Sectional Properties

$$\begin{aligned} \text{Moment of Inertia of Concrete section } & \quad I_c & \quad = & \quad 1.21\text{E}+14 \text{ mm}^4 \\ \text{Modular Ratio } & \quad m & \quad = & \quad 18.54 \\ \text{Moment of Inertia of transformed area } & \quad I_s & \quad = & \quad 1.91\text{E}+14 \text{ mm}^4 \\ \text{Modulus of Section of Concrete } & \quad Z_c & \quad = & \quad 4.72\text{E}+08 \text{ mm}^3 \\ \text{Modulus of Section of Steel } & \quad Z_s & \quad = & \quad -7.50\text{E}+08 \text{ mm}^3 \\ \text{Cracked Moment of Inertia } & \quad I_{CR} & \quad = & \quad 3.12\text{E}+14 \end{aligned}$$

Calculation of Modular Ratio (m)

$$\begin{aligned} \text{Area of Cross section of composite section } & \quad A_c & \quad = & \quad 628 \text{ mm}^2 \\ \text{Perimeter of Section } & \quad u & \quad = & \quad 3141 \text{ mm} \end{aligned}$$

$$= 0.400 \text{ mm}$$

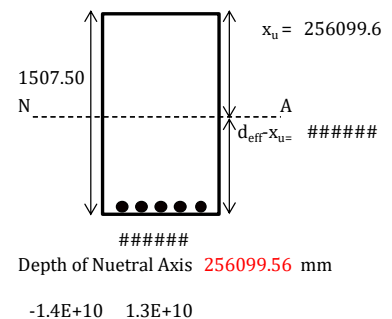
$$\text{For dry atmospheric conditions } \phi = 1.97$$

$$E_{c,eff} = \frac{Ec}{(1 + \phi)} = 10787.16$$

$$m = \frac{E_s}{E_{c,eff}} = 18.5$$

Actual Stresses

$$\begin{aligned} f_{ck}' & \quad = & \quad 0.04 \text{ N/mm}^2 \\ f_{y'} & \quad = & \quad -0.73 \text{ N/mm}^2 \end{aligned}$$



CRACK WIDTH CALUCLATION

Permissible Crack width in moderate conditions of exposure = 0.3 mm

Crack Width W_k = $S_{r, \max} \cdot (\epsilon_{sm} - \epsilon_{cm})$ = 0.00 mm **OK**

$S_{r, \max}$ - Maximum Crack spacing
 ϵ_{sm} - Mean Strain in Reinforcement
 ϵ_{cm} - Mean Strain in Concrete

σ_{sc} - -0.73
 K_t - 0.5
 $f_{ct, eff}$ - 2.8
 $\rho_{p, eff}$ - -0.101
 α_e - 6.25
 E_s - 200000
 A_s - 3436 mm²
 c - 50
 ϕ - 25.00
 h - 1570 mm
 d - 1507.50
 x - 256099.56

$h_{c, eff}$ - Min $\begin{cases} 2.5(h-d) \\ (h-x/3) \\ h/2 \end{cases}$ = $\begin{matrix} 156.25 \\ -84843.19 \\ 785 \end{matrix}$ = -84843.19 mm

ϕ_{eq} = $\frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2}$ = 25.00

$A_{c, eff}$ = -33937 mm²

$\epsilon_{sm} - \epsilon_{cm}$ = $\frac{\sigma_{sc} - K_t \frac{f_{ct, eff}}{\rho_{p, eff}} (1 + \alpha_e \cdot \rho_{p, eff})}{E_s}$ $\geq$ $0.6 \cdot \frac{\sigma_{sc}}{E_s}$
 = 0.00002

For the case of Deformed bars associated with pure bending

$S_{r, \max}$ = $3.4c + \frac{0.17 \cdot \phi}{\rho_{p, eff}}$ = 128.02

ANNEXURE-I

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 28-jan-2015

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 0.5 0 0; 3 1.5 0 0; 4 2 0 0; 5 2.5 0 0; 6 3 0 0; 7 5 0 0; 8 7.5 0 0;
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MEMBER INCIDENCES

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ANNEXURE-I

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DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 2.17184e+007
POISSON 0.17
DENSITY 23.6158
ALPHA 5e-006
DAMP 0.05
ISOTROPIC MATERIAL1
E 3e+007
POISSON 0.2
DENSITY 0
END DEFINE MATERIAL
MEMBER PROPERTY INDIAN
*****OUTTER GIRDER
51 52 61 62 131 132 141 142 PRIS AX 1.6246 IX 0.05899 IY 0.663 IZ 0.53535
49 50 63 64 129 130 143 144 PRIS AX 1.78175 IX 0.0947 IY 0.67285 IZ 0.5762
53 TO 60 133 TO 140 PRIS AX 1.4675 IX 0.02329 IY 0.65316 IZ 0.49451
*****INNER GIRDER
195 196 205 206 227 228 237 -
238 PRIS AX 1.56963 IX 0.05855 IY 0.52916 IZ 0.52189
193 194 207 208 225 226 239 240 PRIS AX 1.72675 IX 0.09425 IY 0.5385 IZ 0.56079
197 TO 204 229 TO 236 PRIS AX 1.4125 IX 0.02284 IY 0.51983 IZ 0.483
*****CROSS GIRDER
293 307 310 324 327 341 344 358 361 375 429 443 446 460 463 -
477 PRIS AX 1.019 IX 0.0747 IY 0.0484 IZ 0.2262
*****DUMMY
1 TO 48 65 TO 128 145 TO 192 209 TO 224 241 257 258 274 275 291 292 308 309 -
325 326 342 343 359 360 376 377 393 394 410 411 427 428 444 445 461 462 -
478 PRIS AX 0.0001 IX 1e-005 IY 1e-005 IZ 1e-005
*****TRANSVERSE(DUMMY)
243 255 260 272 277 289 294 306 311 323 328 340 345 357 362 374 379 391 396 -
408 413 425 430 442 447 459 464 476 PRIS YD 0.22 ZD 1.5
244 254 261 271 278 288 295 305 312 322 329 339 346 356 363 373 380 390 397 -
407 414 424 431 441 448 458 465 475 PRIS YD 0.22 ZD 0.5
245 253 262 270 279 287 296 304 313 321 330 338 347 355 364 372 381 389 398 -
406 415 423 432 440 449 457 466 474 PRIS YD 0.22 ZD 0.5
246 252 263 269 280 286 297 303 314 320 331 337 348 354 365 371 382 388 399 -
405 416 422 433 439 450 456 467 473 PRIS YD 0.22 ZD 1.5
247 TO 251 264 TO 268 281 TO 285 298 TO 302 315 TO 319 332 TO 336 349 TO 353 -
366 TO 370 383 TO 387 400 TO 404 417 TO 421 434 TO 438 451 TO 455 -
468 TO 472 PRIS YD 0.22 ZD 2.5
MEMBER PROPERTY INDIAN
242 256 259 273 276 290 378 392 395 409 412 426 PRIS YD 0.22 ZD 0.85
CONSTANTS
MATERIAL CONCRETE MEMB 49 TO 64 129 TO 144 193 TO 208 225 TO 240 293 307 310 -
324 327 341 344 358 361 375 429 443 446 460 463 477
MATERIAL MATERIAL1 MEMB 1 TO 48 65 TO 128 145 TO 192 209 TO 224 241 TO 292 -
294 TO 306 308 309 311 TO 323 325 326 328 TO 340 342 343 345 TO 357 359 360 -
362 TO 374 376 TO 428 430 TO 442 444 445 447 TO 459 461 462 464 TO 476 478
SUPPORTS
1001 1018 1035 1052 FIXED BUT FX FZ MX MY MZ
1015 1032 1049 1066 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 40 60 60 85 85 85 85
DIST 3.96 1.52 2.13 1.37 3.05 1.37 WID 1.93
TYPE 2 LOAD 13.5 13.5 57 57 34 34 34
DIST 1.1 3.2 1.2 4.3 3 3 3 WID 1.8
*LOAD 1 LOADTYPE None TITLE SIDL
*MEMBER LOAD
**CB
*415 417 419 421 423 425 427 429 431 433 435 437 439 441 443 445 448 450 452 -
*454 456 458 460 462 464 466 468 470 472 474 476 478 UNI GY -8
**Railling
*481 483 485 487 489 491 493 495 497 499 501 503 505 507 509 511 514 516 518 -
*520 522 524 526 528 530 532 534 536 538 540 542 544 UNI GY -6
*LOAD 2 LOADTYPE None TITLE WEARING COAT
*MEMBER LOAD
***Surfacing
*33 TO 48 65 TO 80 97 TO 112 129 TO 144 UNI GY -5.2
***Surfacing Footpath
*547 549 551 553 555 557 559 561 563 565 567 569 571 573 575 577 580 582 584 -
*586 588 590 592 594 596 598 600 602 604 606 608 610 UNI GY -3
*LOAD 3 LOADTYPE None TITLE FPLL
*MEMBER LOAD
*547 549 551 553 555 557 559 561 563 565 567 569 571 573 575 577 580 582 584 -
*586 588 590 592 594 596 598 600 602 604 606 608 610 UNI GY -6
*Class A + 70R (Most Eccentric Class A )
LOAD GENERATION 397

```

ANNEXURE-I

TYPE 2 -18.8 0 4.7 XINC 0.1
TYPE 1 -13.4 0 8.61 XINC 0.1
*Class A + 70R (Most Eccentric Class 70R)
LOAD GENERATION 397
TYPE 1 -13.4 0 6.060 XINC 0.1
TYPE 2 -18.8 0 9.740 XINC 0.1
*Class A + 70R (Concentric)
LOAD GENERATION 397
TYPE 2 -18.8 0 6.455 XINC 0.1
TYPE 1 -13.4 0 10.265 XINC 0.1
*3 Lane Class A
LOAD GENERATION 397
TYPE 2 -18.8 0 4.7 XINC 0.1
TYPE 2 -18.8 0 8.2 XINC 0.1
TYPE 2 -18.8 0 11.7 XINC 0.1
PERFORM ANALYSIS
FINISH

ANNEXURE-II

PAGE NO. 1

```

*****
*                               *
*   STAAD.Pro                   *
*   Version 2007 Build 05      *
*   Proprietary Program of    *
*   Bentley Systems, Inc.     *
*   Date= NOV 16, 2015        *
*   Time= 12:27:38           *
*                               *
*   USER ID:                  *
*****

```

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1. STAAD SPACE
INPUT FILE: Diaphragm 1.STD
2. START JOB INFORMATION
3. ENGINEER DATE 24-SEP-15
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER KN
7. JOINT COORDINATES
8. 1 0 0 0; 2 2.60001 0 0; 3 5.20001 0 0; 4 7.80002 0 0; 5 10.4 0 0; 6 13 0 0
9. 7 1.3 0 0; 8 6.50001 0 0; 9 11.7 0 0
10. MEMBER INCIDENCES
11. 1 1 7; 2 2 3; 3 3 8; 4 4 5; 5 5 9; 6 7 2; 7 8 4; 8 9 6
12. DEFINE MATERIAL START
13. ISOTROPIC CONCRETE
14. E 2.17184E+007
15. POISSON 0.17
16. DENSITY 23.6158
17. ALPHA 5E-006
18. DAMP 0.05
19. END DEFINE MATERIAL
20. CONSTANTS
21. MATERIAL CONCRETE ALL
22. MEMBER PROPERTY INDIAN
23. 1 TO 8 PRIS AX 0.6155 IX 0.01742 IY 0.01048 IZ 0.14
24. SUPPORTS
25. 7 TO 9 PINNED
26. LOAD 1 LOADTYPE NONE TITLE SIDL
27. JOINT LOAD
28. 1 FY -147.21
29. 2 FY 13.69
30. 3 FY 1.641
31. 4 FY 1.641
32. 5 FY 13.69
33. 6 FY -147.21
34. LOAD 2 LOADTYPE NONE TITLE SURFACING
35. JOINT LOAD
36. 1 FY -29.38
37. 2 FY -47.24
38. 3 FY -49.62
39. 4 FY -49.62
40. 5 FY -47.24
STAAD SPACE

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-- PAGE NO. 2

```

41. 6 FY -29.38
42. LOAD 3 LOADTYPE NONE TITLE DEAD LOAD
43. JOINT LOAD

```


44. 1 FY -336.32
 45. 2 FY -325.98
 46. 3 FY -325.98
 47. 4 FY -325.98
 48. 5 FY -325.98
 49. 6 FY -336.32
 50. SELFWEIGHT Y -1 LIST 1 TO 8
 51. PERFORM ANALYSIS

52. PRINT MEMBER FORCE
 STAAD SPACE

-- PAGE NO. 3

MEMBER END FORCES STRUCTURE TYPE = SPACE

 ALL UNITS ARE -- KN METE (LOCAL)

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z

1	1	1	0.00	-147.21	0.00	0.00	0.00	0.00
	7		0.00	147.21	0.00	0.00	0.00	-191.37
2	1		0.00	-29.38	0.00	0.00	0.00	0.00
	7		0.00	29.38	0.00	0.00	0.00	-38.19
3	1		0.00	-336.32	0.00	0.00	0.00	0.00
	7		0.00	355.22	0.00	0.00	0.00	-449.50
2	1	2	0.00	60.09	0.00	0.00	0.00	131.05
	3		0.00	-60.09	0.00	0.00	0.00	25.18
	2	2	0.00	-2.06	0.00	0.00	0.00	-20.53
	3		0.00	2.06	0.00	0.00	0.00	15.17
	3	2	0.00	47.43	0.00	0.00	0.00	-48.22
	3		0.00	-9.64	0.00	0.00	0.00	122.40
3	1	3	0.00	61.73	0.00	0.00	0.00	-25.18
	8		0.00	-61.73	0.00	0.00	0.00	105.43
	2	3	0.00	-51.68	0.00	0.00	0.00	-15.17
	8		0.00	51.68	0.00	0.00	0.00	-52.02
	3	3	0.00	-316.34	0.00	0.00	0.00	-122.40
	8		0.00	335.24	0.00	0.00	0.00	-301.13
4	1	4	0.00	-60.09	0.00	0.00	0.00	-25.18
	5		0.00	60.09	0.00	0.00	0.00	-131.05
	2	4	0.00	2.06	0.00	0.00	0.00	-15.17
	5		0.00	-2.06	0.00	0.00	0.00	20.53
	3	4	0.00	-9.64	0.00	0.00	0.00	-122.40
	5		0.00	47.43	0.00	0.00	0.00	48.22
5	1	5	0.00	-46.40	0.00	0.00	0.00	131.05
	9		0.00	46.40	0.00	0.00	0.00	-191.37
	2	5	0.00	-45.18	0.00	0.00	0.00	-20.53
	9		0.00	45.18	0.00	0.00	0.00	-38.19
	3	5	0.00	-373.41	0.00	0.00	0.00	-48.22
	9		0.00	392.31	0.00	0.00	0.00	-449.50
6	1	7	0.00	46.40	0.00	0.00	0.00	191.37
	2		0.00	-46.40	0.00	0.00	0.00	-131.05
	2	7	0.00	45.18	0.00	0.00	0.00	38.19
	2		0.00	-45.18	0.00	0.00	0.00	20.53
	3	7	0.00	392.30	0.00	0.00	0.00	449.50
	2		0.00	-373.41	0.00	0.00	0.00	48.22
7	1	8	0.00	-61.73	0.00	0.00	0.00	-105.43
	4		0.00	61.73	0.00	0.00	0.00	25.18

STAAD SPACE

-- PAGE NO. 4

MEMBER END FORCES STRUCTURE TYPE = SPACE

 ALL UNITS ARE -- KN METE (LOCAL)

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z

2	8		0.00	51.68	0.00	0.00	0.00	52.02
	4		0.00	-51.68	0.00	0.00	0.00	15.17
3	8		0.00	335.24	0.00	0.00	0.00	301.13
	4		0.00	-316.34	0.00	0.00	0.00	122.40
8	1	9	0.00	147.21	0.00	0.00	0.00	191.37
	6		0.00	-147.21	0.00	0.00	0.00	0.00
	2	9	0.00	29.38	0.00	0.00	0.00	38.19

	6	0.00	-29.38	0.00	0.00	0.00	0.00
3	9	0.00	355.22	0.00	0.00	0.00	449.50
	6	0.00	-336.32	0.00	0.00	0.00	0.00

***** END OF LATEST ANALYSIS RESULT *****

53. FINISH

FORCE CALCULATION FOR DESIGN OF SPHERICAL BEARING

LOAD CALCULATION FOR SPHERICAL BEARING**Design Loads (Refer Superstructure Design)**

c/c Bearing (span)	=	19.000	m		
Skew Angle	=	0	deg	0.000	rad
Total No. of Pin Beaing	B2	=	0		
Total No. of Metallic. Guided bearings	B3	=	0		
Total No. of Free Bearings	B1	=	2		
Total No. of fix Bearings	B2	=	2		
Total No. of Trans. Guided bearings	B3	=	2		
Total No. of Long. Guided bearings	B4	=	2		
Total No. of bearings at Each Side	n	=	4		
Total No. of bearings for one Span		=	8		
C/C distance of bearings (Trans)		=	3.00	m	
Total Width of Superstructure		=	12.5	m	
Maximum Free Length For Expansion in Transverse Direction		=	6.25	m	

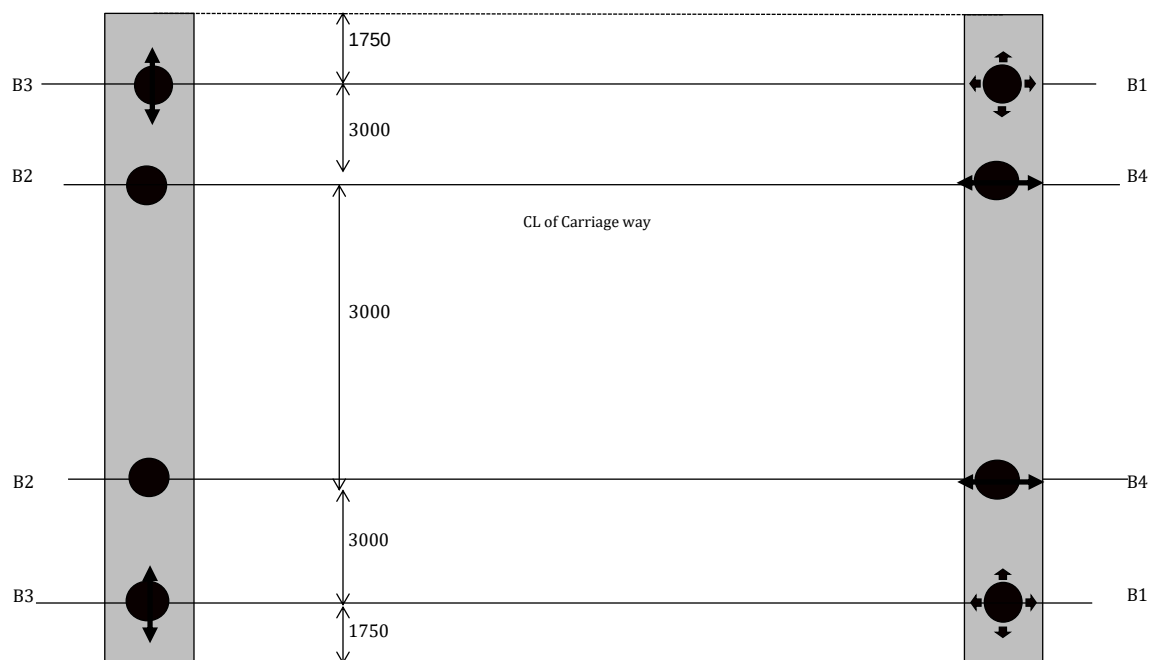
Seismic Coefficient Calculation : **Seismic Zone-IV**

Z	=	0.24
I	=	1.2
R	=	1
Sa/g	=	2.5
Ah	=	0.36
Av	=	0.24

SEISMIC COEFFICIENTS		
Horizontal	A _h	0.36
Vertical	A _v	0.24

IMPACT FACTOR for LIVE LOAD	
Class A	1.180
Class 70R W	1.180
Class SV Loading	1.00

Lane Reduction Factor = 0.9



Vertical Load (Refer Staad Files for Superstructure)**1 Lane Class 70R + 1 Lane Class A**

Load Case	Vertical Load (kN) on Bearing					
	1	2	3	4	5	6
Dead Load	381.70	367.97	367.97	381.70	0.00	0.00
SIDL	126.02	-41.02	-41.02	126.02	0.00	0.00
SURFACING	67.79	60.61	60.61	67.79	0.00	0.00
FPLL	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(CWLL Max)	423.59	572.10	637.94	253.30	0.00	0.00
Live Load(CWLL Min)	-3.55	-12.36	-3.05	-30.87	0.00	0.00
SV Loading (Max)	0.63	493.46	904.35	101.37	0.00	0.00
SV Loading (Min)	-43.42	-11.64	-2.87	-29.07	0.00	0.00

Dead Load Reaction on one End	=	1499 kN
SIDL Reaction on one End	=	427 kN
Max Live Load(CWLL) reaction on one	=	1886.92 kN
Min Live Load(CWLL) reaction	=	-49.83 kN
Max Live Load(SV Load) reaction on o	=	1499.80 kN
Min Live Load(SV Load) reaction	=	-87.00 kN

Calculation of Translation of Superstructure due to Temp., Shrinkage and Creep*(At Free End)*Coefficient of Linear Expansion α = 1.20E-05 /°C (Refer Cl:215.4 of IRC:6 - 2000)

1	Due to temp. (Rise and fall of 25 degree considered)	=	1.20E-05	x	47.5	x	19000	=	+ -	10.830 mm
2	Due to shrinkage	=	shrinkage strain		3.75E-05	x	19000	=		0.712 mm
3	Due to creep	=	creep strain		-4.0E-04 10	x	19000	=		-0.760 mm
Total movement in contraction	=	-10.830	+		0.712	+	-0.760	=		-10.878 mm
Design movement in contraction	=	-10.878	x		1.3			=		-14.141 -14.200 (say)
Total movement in elongation	=	10.830	+		0.712	+	-0.760	=		10.782 mm
Design movement in elongation	=	10.782	x		1.3			=		14.017 14.100 (say)
Movement in transverse direction	=	1.20E-05	x		50	x	6250.00	=	+ -	3.750 mm
Total movement	=	3.750	x		1.3	=	+ -			4.88 mm 4.90 (say)

Calculation of RotationGrade of concrete = **M35**

For simply supported span it may be computed as

$$\alpha_d = \frac{400 M_{max} L \times 10^{-3}}{(E I)}$$

$$E_s = 2E+11$$

$$\text{Where } E_c = 3.20E+10 \text{ Pa}$$

$$E = E_c / 2 = 1.60E+10 \text{ Pa} \quad \text{For permanent load}$$

S. No.	Load case	Mmax. (Nm)	E (Pa)	I (m ⁴)	L (m)	α_d
1	DL	859977.87	3.20E+10	0.216	19.000	9.45E-04
2	SIDL	331129.00	3.20E+10	0.483	19.000	1.63E-04
3	F.P.L.L	0	3.20E+10	0.483	19.000	0.00E+00
4	CWLL	2654.75	3.20E+10	0.483	19.000	1.31E-06
Total						1.11E-03

$$\text{Design Rotation} = 1.11E-03 \times 1.3 = 0.0014 \text{ radian} \quad \mathbf{0.0020} \text{ (say)}$$

Calculation of Horizontal Force**Normal Case**

Bearing Force

1-70R Wheel + 1-Class A

Load on spa: First train	=	900.00 kN	1-70R Wheel
Second train	=	498.6 kN	1-Class A
Third Train	=	0 kN	

$$\text{Bearing Force (Fh)} = 204.93 \text{ kN} \quad (20\% \text{ of First Train and } 5\% \text{ of Second and Third})$$

Horizontal Force due to Secondary effect (Temp, Shrinkage etc)

$$\text{Friction Coeff. } \mu = 0.05$$

$\mu (Rg+Rq)$	=	190.65 kN	91.96 kN	96.31 kN
		(LL Max)	(LL Min)	(No LL)
Fh- $\mu(Rg+Rq)$		14.28 kN	112.97 kN	108.62 kN
Fh/2+ $\mu(Rg+Rq)$		293.12 kN	194.42 kN	198.7721 kN
Max=		293.12 kN	194.42 kN	198.77 kN

Seismic (Long) Case

Longitudinal Seismic Force

Seismic Force (Fs)	=	1386.82 kN
Braking Force	=	40.986 kN

$\mu (R_g+R_q)$	=	115.18 kN	95.81 kN	96.31 kN
		(LL Max)	(LL Min)	No LL
$F_h - \mu(R_g+R_q)$		1312.63 kN	1332.00 kN	
$F_h/2 + \mu(R_g+R_q)$		829.08 kN	809.71 kN	
$F_h + F_s$		1427.81 kN		
Max=		1427.81 kN	1427.81 kN	1386.82 kN

Transverse Seismic Force

F_t	=	$0.3 \times A_h \times (DL+SIDL+LL_{max}/5)$	
	=	242.56 kN	Max LL
F_t	=	$0.3 \times A_h \times (DL+SIDL+LL_{min}/5)$	
	=	207.11 kN	Min LL
F_t	=	$0.3 \times A_h \times (DL+SIDL)$	
	=	208.02	No LL

Vertical Seismic Force

$$\begin{aligned}
 F_v &= 0.3 \times a_v \times (DL + SIDL + LL_{max}/5) \\
 &= 165.85 \text{ kN} \quad \text{at fixed end} \\
 F_v &= 0.3 \times a_v \times (DL + SIDL + LL_{min}/5) \\
 &= 137.96 \text{ kN} \quad \text{at free end}
 \end{aligned}$$

Seismic (Trans) Case

Longitudinal Seismic Force

$$\begin{aligned}
 \text{Seismic Force (Fs)} &= 416.05 \text{ kN} \\
 \text{Braking Force} &= 40.99 \text{ kN}
 \end{aligned}$$

$\mu (R_g + R_q) =$	115.18 kN	95.81 kN
	(LL Max)	(LL Min)

$$F_h - \mu(R_g + R_q) \quad 341.86 \text{ kN} \quad 361.22 \text{ kN}$$

$$F_h/2 + \mu(R_g + R_q) \quad 343.69 \text{ kN} \quad 324.33 \text{ kN}$$

$$F_h + F_s \quad 457.033 \text{ kN}$$

Max=	457.03	457.03

Transverse Seismic Force

$$\begin{aligned}
 F_t &= A_h \times (DL + SIDL + LL_{max}/5) \\
 &= 808.55 \text{ kN} \quad \text{with LL} \\
 &= 690.37 \text{ kN} \quad \text{Min LL} \\
 &= 459.88 \text{ kN} \quad \text{without LL}
 \end{aligned}$$

Summary of Forces on Bearing

			Vertical Force (kN)	Horizontal Force (kN)		Rotation (rad)	Movement (mm)	
				Long	Trans		Long	Trans
FREE BEARING (B1)	Normal Case	Max Rn	1213	121	0	0.002	-14.2/14.1	4.90
		Min Rn	357	36	0	0.002		
		No LL	576	58	0	0.002		
	Seismic Long Case	Max Rn	754	0	0	0.002		
		Min Rn	470	0	0	0.002		
		No LL	617	0	0	0.002		
	Seismic Trans Case	Max Rn	754	0	0	0.002		
		Min Rn	470	0	0	0.002		
		No LL	617	0	0	0.002		
FIX BEARING (B2)	Normal Case	Max Rn	1213	121	0	0.002	-	-
		Min Rn	357	49	0	0.002		
		No LL	576	58	0	0.002		
	Seismic Long Case	Max Rn	754	0	121	0.002		
		Min Rn	470	0	104	0.002		
		No LL	617	0	104	0.002		
	Seismic Trans Case	Max Rn	754	0	404	0.002		
		Min Rn	470	0	345	0.002		
		No LL	617	0	230	0.002		
TRANS GUIDED BEARING (B3)	Normal Case	Max Rn	1213	121	0	0.002	-	4.90
		Min Rn	357	49	0	0.002		
		No LL	576	58	0	0.002		
	Seismic Long Case	Max Rn	754	357	0	0.002		
		Min Rn	470	357	0	0.002		
		No LL	617	347	0	0.002		
	Seismic Trans Case	Max Rn	754	114	0	0.002		
		Min Rn	470	114	0	0.002		
		No LL	617	114	0	0.002		
LONG GUIDED BEARING (B4)	Normal Case	Max Rn	1213	121	0	0.002	-14.2/14.1	-
		Min Rn	357	36	0	0.002		
		No LL	576	58	0	0.002		
	Seismic Long Case	Max Rn	754	75	121	0.002		
		Min Rn	470	47	104	0.002		
		No LL	617	62	104	0.002		
	Seismic Trans Case	Max Rn	754	75	404	0.002		
		Min Rn	470	47	345	0.002		
		No LL	617	62	230	0.002		

DETAILED DESIGN OF CAST IN SITU DECK SLAB OVER GIRDER

DESIGN OF RCC DECK SLAB

Deck is designed as a continuous slab of one metre width supported over longitudinal girders.

Dimesnions of Deck Slab

Slab Thcikness	=	220 mm
Total Width of Slab	=	12.5 m

CALCULATION OF DEAD LOAD & SUPERIMPOSED DEAD LOAD

1. Deck slab

Self weight of slab	=	5.500 kN/m ²

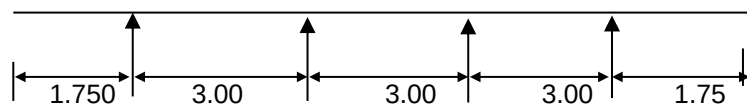
2. Wearing coat 65 thick + additional load

Load =2 kN/m ²	with start and end offset of 0.5m

3. Crash barrier

load = 8.0 kN

IDEALIZATION OF DECK SLAB



Load Factors (As per IRC:6-2014)

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

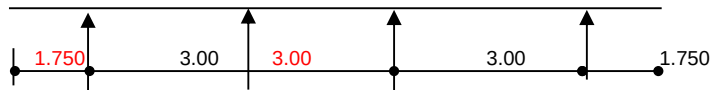
Loads	Basic Combination
Dead Load+SIDL except wearing course	1.350
Wearing Course only	1.750
CWLL and Associate load and FPLL(Service)	1.500

Table 3.3 Partial Safety Factor For Verification of Serviceability Limit State

-Refer Table 3.3 of IRC:6-2014

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL excluding surfacing	1.000	1.00	1.00
Surfacing	1.200	1.20	1.20
Shrinkage Creep Effect	1.000	1.00	1.00
CWLL and Associate load and FPLL	1.000	0.75	0.00

CALCULATION OF EFFECTIVE WIDTH AND LOAD INTENSITY FOR CRITICAL LOAD POSITIONS



As per IRC: 21, clause 305.16.2

$$b_{ef} = \alpha \cdot a \cdot (1 - a/l_o) + b_1 \quad (\text{for solid slab spanning in one direction})$$

$$= 1.2 a + b_1 \quad (\text{for solid slab cantilever})$$

b_{ef} = effective width of slab

l_o = effective span = 3.00 m

a = distance of c.g. of conc. load from nearer support

b_1 = breadth of concentration area of the load i.e. the dimension of tyre over road surface in the direction at right angle to span plus twice the thickness of wearing coat

α = a constant having the values depending upon ratio b/l_o

$$= 2.6$$

Thickness of wearing coat = 65 mm

Thickness of deck slab = 220 mm

Effective depth of deck slab = 164 mm

Contact width perpendicular to span = 250 mm

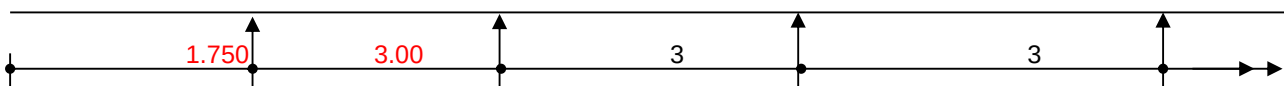
Contact width parallel to span = 500 mm

Distance between two loads perpendicular to span = 1.2 m

Concentrated load = 57 kN

$$b_1 = 0.25 + 0.065 \times 2 = 0.38 \text{ m}$$

Loading 1 One lane CI A (Cantilever Moment)

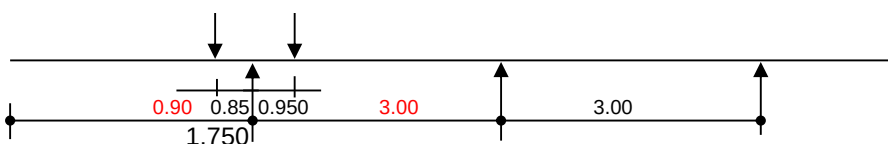


Length of dispersion along the span =

$$0.5 + 2 \times (0.065 + 0.164) = 0.958 \text{ m}$$

Load Nos.	Load (kN)	a (m)	b _{ef} (m)	b _{ef1} (m)	b _{eff2} (m)	Load (kN/m ²)
1	57	0.850	1.400	1.300	0.958	45.8
2	57	0.950	2.068	1.634	0.958	36.4

$b_{ef1} = (b_{ef} + 1.2)/2$, if $b_{ef} > 1.2$



Max. load on a wheel = $\frac{200}{4} = 50 \text{ kN}$

Max. tyre pressure = 5.273 kg/cm^2

Contact width parallel to span = 410 mm

Contact area = $\frac{5000}{5.273} = 948 \text{ cm}^2$

$= 94823 \text{ mm}^2$

Contact width perpendicular to span = $\frac{94823}{410} = 231 \text{ mm}$

Distance between two loads perpendicular to span = 1.22 m

b1 = $0.231 + 0.065 \times 2 = 0.361 \text{ m}$

1 M type (for maximum sagging moment)

Load Nos.	Load (kN)	a (m)	bef (m)	bef1 (m)	beff2(m)	Load (kN/m ²)
1	50	0.310	1.084	1.084	0.832	55.5
2	50	1.105	2.176	1.698	0.793	37.2
3	50	1.105	2.176	1.698	0.793	37.2
4	50	0.310	1.084	1.084	0.832	55.5

Diagram of a continuous beam with 5 spans. The total length is 5.06. The spans are 1.750, 3.00, 0.795, 0.790, and 1.750. The beam is supported by 6 piers. The first span is 1.750, the second is 3.00, the third is 0.795, the fourth is 0.790, and the fifth is 1.750. The total length is 5.06.

2 M type (Inner Support Hogging Moment)

Load Nos.	Load (kN)	a (m)	bef (m)	bef1 (m)	beff2(m)	Load (kN/m ²)
1	50	1.190	2.228	1.724	0.832	34.9
2	50	0.395	1.253	1.237	0.793	51.0
3	50	0.395	1.253	1.237	0.793	51.0
4	50	1.190	2.228	1.724	0.832	34.9

Diagram of a beam with a uniformly distributed load of 3.56 kN/m. The beam is supported by a pin support at the left end and a roller support at the right end. The beam is divided into segments of 1.750m, 3.00m, 0.795m, 0.790m, 0.795m, 3.00m, and 1.750m. The total length is 10.00m.

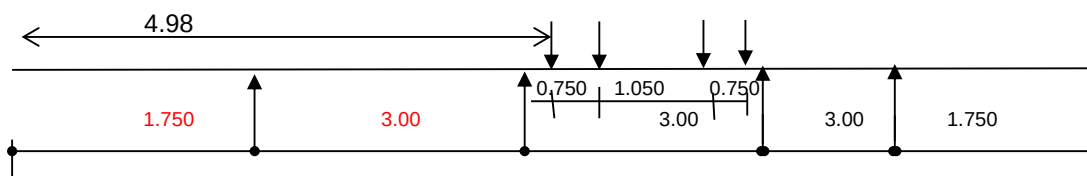
IRC Class SV Loading

Contact width parallel to span = 450 mm
 Contact width perpendicular to span = 274 mm

Length of dispersion across the span , b1 = 0.404 m
 Length of dispersion along the span = 0.908 m

Maximum weight of single axle = 180.00 KN
 nos. of wheel considered under each axle = 4.00
 Weight shared by each wheel = 45.00 KN
 Distance betn two axle perpendicular to span = 1.5 m

Load Nos.	Load (kN)	a (m)	bef (m)	bef1 (m)	beff2(m)	Load (kN/m ²)
1	45	0.225	0.945	0.945	0.829	57.4
2	45	0.975	2.115	1.058	0.829	51.3
3	45	0.975	2.115	1.058	0.829	51.3
4	45	0.225	0.945	0.945	0.829	57.4



Material Specification

Concrete Grade	=	M 35	
Characteristic Compressive	=	35.00	Mpa at 28 days
Design Compressive strength of Concrete, fcd	=	15.63	Mpa at 28 days (0.67/1.5 * fck) - Refer Fig 6.5 of IRC : 112-2011
Tensile strength of concrete , fctm	=	2.77	MPa
Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
Modulus of Elasticity of Concrete (Ec)	=	2.96E+04	N/mm ² (5000 x sqrt (fck)
Ecm	=	3.23E+04	N/mm ²
Steel Grade	=	Fe 500	D (HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa (1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	2.00E+05	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Severe Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48 fck	(Refer Clause 12.2.1 of IRC:112-2011)
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²

Staad Results:

Load Cases	Moment (kNm/m)			Shear Forces at deff from Support Face (kN/m)	
	Cantilever Portion	Intermediate Support	InterMeediate Span	Cantilever Portion	Intermediate Support
DL+SIDL excluding Wearing Coat	7.807	1.217	3.970	7.148	5.148
Wearing Coat	0.563	0.238	0.762	1.372	1.872
Class A	15.738	1.705	0.000	35.073	0.000
Class 70R Wheel	0.000	2.563	27.800	0.000	38.491
SV Loading	0.000	9.640	30.400	0.000	46.690

Impact Factor for Cantilever Span = 1.5 For Span < 3 m (Fig 5 of IRC: 6-2014)

Impact Factor for Continuous Span = 1.25 For Span < 9 m (Clause 208.3 of IRC: 6-2014)

Note: No impact factor shall be applied on SV loading

Factored Forces For Design of Deck Slab

Load Cases	Load Factor	Moment (kNm/m)			Shear Forces at deff from Support Face (kN/m)	
		Cantilever Portion	Intermediate Support	InterMeediate Span	Cantilever Portion	Intermediate Support
DL+SIDL excluding Wearing Coat	1.350	10.539	1.643	5.360	9.650	6.950
Wearing Coat	1.750	0.985	0.417	1.334	2.401	3.276
Class A	1.500	35.411	3.836	0.000	78.914	0.000
Class 70R Wheel	1.500	0.000	4.806	52.125	0.000	72.171
SV Loading	1.000	0.000	9.640	30.400	0.000	46.690
Total =		46.935	11.699	58.818	90.965	82.396

Support Reinforcement Calculation:

Thickness of slab = 220 mm at Interior span

Thickness of slab = 220 mm (at cantilever)

Clear Cover to outer steel = 50 mm

Maximum Diameter of Reinforcement = 12 mm

Effective Depth Provided (d_{eff}) = 164 mm

Design bending moment = 46.94 kNm/m

M_{lim} = 0.167 x f_{ck} x b x d² = 46.94 kNm/m (Equation derived based on IRC:112-2011)

Effective Depth of Cap Required (d_{req}) = $\text{SQRT}\left(\frac{46.94 \times 1000000}{0.167 \times 35.00 \times 1000}\right)$

Effective Depth of Cap Required (d_{req}) = 89.610 mm

Total Depth Required (D_{req}) = 145.61 mm

Total Depth Provided (D_{prov}) = 220.00 mm

OK

R = M_{RD} / (b d²) = 1.75

Design Moment of Resistance **M_{RD}** = 0.87f_yA_{st} * (d - (λ_{x_u}/2))

46.94 = 0.87f_yA_{st} * (d - (λ_{x_u}/2))

$$\begin{aligned}\eta &= 1 \\ f_{cd} &= 15.61 \text{ N/mm}^2 \\ f_y &= 500 \text{ N/mm}^2 \\ \lambda &= 0.8\end{aligned}$$

$$M_{RD} = 0.87 f_y A_{st} * (d - (0.87 * f_y * A_{st} / (b * \eta * f_{cd} * 2)))$$

$$\text{Area of Steel Required, } A_{st_{red}} = 699.4766 \text{ mm}^2/\text{m}$$

$$A_{st_{req}} = 699.4766 \text{ mm}^2/\text{m}$$

Minimum Longitudinal Reinforcement :

$$\text{As. Min} = 0.26 \times \frac{f_{ctm}}{f_y} \times b \cdot d \quad \text{- Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

$$\text{Whichever is higher} \quad \text{OR} \quad = 0.0015 \times b \cdot d \quad \text{-Refer Clause 16.9 of IRC:112-2011'}$$

$$b = 1000.00 \text{ mm}$$

$$d = 164.00 \text{ mm}$$

$$A_{st \text{ min}} = 246.00 \text{ mm}^2/\text{m}$$

$$\text{Governing Reinf. Ast} = 699.48 \text{ mm}^2/\text{m}$$

$$\text{Provide } 12 \text{ mm dia @ } 100 \text{ mm c/c} + 0 \text{ mm dia @ } 100 \text{ mm c/c}$$

Area provided=	1130.97 mm ² /m	>	699.48 mm ² /m	OK
Percentage of Steel (pt%)	=	0.69	%	

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

$$S_{max} = 2h = 328.00$$

$$\text{OR} = 250.00 \text{ mm} \quad \text{whichever is max}$$

Provided Spacing is less than Smax, Hence OK

Moment of Resistance of Section corresponding to Provided Ast

$$\begin{aligned}M_{RD} &= 0.87 f_y A_{st} * (d - (0.87 * f_y * A_{st} / (b * \eta * f_{cd} * 2))) \\ &= 72.93 \text{ kNm} > 46.94 \text{ kNm} \\ &\quad \text{SAFE}\end{aligned}$$

Distribution reinforcement:

$$\text{Effective Depth in Long. Direction} = 153 \text{ mm}$$

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$$\frac{20.00}{100.00} \times 1130.97 = 226.1947 \text{ mm}^2/\text{m}$$

$$\text{Provide } 10 \text{ dia bar } 200 \text{ mm c/c at Trans direction in top face. (Providing = } 392.699 \text{ mm}^2$$

Span Reinforcement Calculation:

$$\text{Clear Cover to outer steel} = 50 \text{ mm}$$

$$\text{Maximum Diameter of Reinforcement} = 12 \text{ mm}$$

$$\text{Effective Depth Provided (deff)} = 164 \text{ mm}$$

$$\text{Design bending moment} = 58.82 \text{ kNm/m}$$

$$\text{Mult} = 0.167 \times f_{ck} \times b \times d^2 = 58.82 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \text{SQRT} \left(\frac{58.82 \times 1000000}{0.167 \times 35.00 \times 1000} \right)$$

$$\text{Effective Depth of Cap Required (dreq)} = 100.3143 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 156.31 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 220.00 \text{ mm} \quad \text{OK}$$

$$R = M_u / (b \cdot d^2) = 2.19$$

Ast Required:

$$M_{RD} = 0.87 f_{yk} A_{st} \left(d - \frac{0.87 f_{yk} A_{st}}{b \cdot n \cdot f_{cd}} \right)$$

$$\text{Area of Steel Required, } A_{st_{req}} = 892.087 \text{ mm}^2/\text{m}$$

$$A_{st_{req}} = 892.087 \text{ mm}^2/\text{m}$$

Minimum Longitudinal Reinforcement :

$$A_{s, \text{Min}} = 0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \cdot d \quad \text{- Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

Whichever is higher OR $0.0015 \times b \cdot d$ -Refer Clause 16.9 of IRC:112-2011'

$$b = 1000.00 \text{ mm}$$

$$d = 164.00 \text{ mm}$$

$$A_{st \text{ min}} = 246.00 \text{ mm}^2/\text{m}$$

$$\text{Governing Reinf. Ast} = 892.09 \text{ mm}^2/\text{m}$$

Provide 12 mm dia @ 100 mm c/c + 0 mm dia @ 100 mm c/

Area provided=	1130.97 mm ² /m	>	892.09 mm ² /m	OK
Percentage of Steel (pt%)	=	0.69	%	

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

$$S_{max} = 2h = 328.00$$

OR $= 250.00 \text{ mm}$ whichever is max

Provided Spacing is less than Smax, Hence OK

Moment of Resistance of Section corresponding to Provided Ast

$$M_{RD} = 0.87 f_{yk} A_{st} \left(d - \frac{0.87 f_{yk} A_{st}}{b \cdot n \cdot f_{cd}} \right)$$

$$= 72.93 \text{ kNm} > 58.82 \text{ kNm}$$

SAFE

Distribution reinforcement:

Effective Depth in Long. Direction = 155 mm

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$$\frac{20.00}{100.00} \times 1130.97 = 226.1946711 \text{ mm}^2/\text{m}$$

Provide 10 dia bar 200 direction in top face. (Providing = 392.699 mm²)

Check For Shear**Design Shear Strength of Concrete, (τ_c) without Shear Reinforcement:**

As per Clause 10.3.2 of IRC:112-2011,

Design shear resistance of the member without shear reinforcement is given by:

$$V_{Rd,c} = \left[0.12 K (80 \rho_1 f_{ck})^{0.33} + 0.15 \sigma_{cp} \right] b_w d \quad \text{eq.1}$$

$$\text{Subjected to minimum of } V_{Rd,c} = (V_{min} + 0.15 \sigma_{cp}) b_w d \quad \text{eq.2}$$

where,

$$K = 1 + \sqrt{200/d} \leq 2.0, \text{ where } d \text{ is depth in mm}$$

$$K = 2.00$$

$$v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}, \quad f_{ck} = 35.00 \text{ N/mm}^2$$

Hence $v_{min} = 0.5187 \text{ N/mm}^2$

σ_{cp} = Concrete compressive stress in concrete at centroidal axis in the direction of axial load or prestressing

$$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd} \quad \text{where, } f_{cd} = 0.67 f_{ck}/1.5$$

$$\sigma_{cp} = 0.00 \text{ N/mm}^2$$

Hence,

$$\tau_c = V_{Rd,c}/(b_w \cdot d) = V_{min} + 0.15 \sigma_{cp} = 0.5187 \text{ N/mm}^2 \quad \text{From eq.1}$$

$$\rho_1 = \text{Steel Ratio} = A_s/(b_w \cdot d) \leq 0.02$$

$$\rho_1 = 0.0069$$

$$\tau_c = V_{Rd,c}/(b_w \cdot d) = 0.6439 \text{ N/mm}^2 \quad \text{From eq.2}$$

Max of eq.1 & eq.2

$$\tau_c = V_{Rd,c}/(b_w \cdot d) = 0.6439 \text{ N/mm}^2$$

Corresponds to steel ratio = 0.69% & M35
Grade of Concrete

$$\text{Shear stress (} v_{Ed} \text{)} = V_{Ed}/(b_w \cdot z) \quad , \text{ where } z = 0.9 \text{ deff}$$

$$v_{Ed} = \frac{90965.050}{1000.00 \times 147.60} = 0.62 \text{ N/mm}^2 < 0.644 \text{ MPa}$$

AsTv is lesser than Tc Hence No Shear Reinforcement is need to be provided.

Stress Check of Deck Slab

Design Forces in Rare Combination		Moments (kNm/m)		
Load Cases	Load Factors	Cantilever Portion	Intermediate Support	InterMeediate Span
DL+SIDL excluding Wearing Coat	1	7.807	1.217	3.97
Wearing Coat	1.2	0.563	0.238	0.762
Class A	1	23.607	2.5575	0
Class 70R Wheel	1	0	3.20375	34.75
SV Loading	1	0	9.64	30.4
			11.10	39.48

Maximum Support moment (Mxx)	=	39.482	kNm/m
Maximum compressive stress in concrete under rare combination	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²

Section Properties of Uncracked Section:

width of section (Dx)	=	1000.00	mm
Depth of Section (Dy)	=	220.00	mm
Gross Cross sectional area (Ag)	=	220000	mm ²
Gross Moment of Inertia I _{gx}	=	8.87E+08	mm ⁴
Gross Moment of Inertia I _{gy}	=	1.83E+10	mm ⁴

Calculation of Modular Ratio:

Perimeter of section (u)	=	2440	mm
2 Ac/u	=	180.328	mm
For Dry atmosperic condition			
Creep coeff.	=	1.48	
E _{eff}	=	$\frac{E_{cm}}{(1 + \phi)}$	1.30E+04

Modular Ratio (m)	=	E _c / E _{eff}	=	15.35

Details of Reinforcements:

at Support

12 mm dia @	100 mm c/c
0 mm dia @	100 mm c/c

Total Ast provided	=	1130.97	mm ² /m
pt = Ast/(b*D)	=	0.0069	
effective cover	=	56	mm
C.g. Of Tension steel from N.A. (y)	=	54.00	mm
Area of concrete, Ac = (Ag - Ast)	=	218869.03	mm ²
Transformed Area of Section AstT	=	236230.3306	mm ²
Transformed M.O.I (I _{xx})	=	$I_{gx} + (m-1).Ast . y^2$	8.87E+08 + 47327643.95
	=	9.35E+08	mm ⁴
Section modulus (Z _{xx})	=	$I_{xx} / (Dy/2)$	
	=	8496917.975	mm ³
Bending stress (M / Z _{xx})	=	4.647	N/mm ²
Tensile strength of concrete , f _{ctm}	=	2.77	N/mm ²

Cracked

Sectional Properties of Cracked Section:

Stresses under Service load are usually within Linear Elastic Range hence such analysis involved use of Modulus ratio.

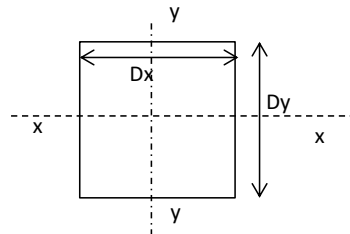
Effective Depth deff (dy)	=	164.00	mm
Depth of neutral axis (yb)	=	k * dy	
where , k	=	$\sqrt{2 pt * m + (pt*m)^2} - pt * m$	
	=	0.3663	
Depth of Neutral axis from Extreme compression face(yb)	=	60.0720	mm
Depth of Neutral axis from Extreme Tension face(yt)	=	159.9280	mm
Depth of Neutral axis from c.g. Of tesion steel (ys)	=	103.9280	mm

Cracked Moment of Inertia (I _{xxcr})	=	$Dx * (k * dy)^3 / 3 + m Ast * (dy - k * dy)^2$	=	2.5978E+08 mm ⁴	
Maximum compressive stress in concrete=	$Mxx / I_{xxcr} * y_b$	=	9.13 N/mm ²	<	16.8
Maximum Tensile stress in concrete=	$Mxx / I_{xxcr} * y_t$	=	24.31 N/mm ²	>	2.77
Maximum Tensile stress in steel=	$m * Mxx / I_{xxcr} * y_s$	=	242.47 N/mm ²	<	300
					SAFE

Crack Width Calculation of Deck Slab

Design Forces in Quasi-Permanent Combination		Moments (kNm/m)		
Load Cases	Load Factors	Cantilever Portion	Intermediate Support	InterMeediate Span
DL+SIDL excluding Wearing Coat	1	7.807	1.217	3.97
Wearing Coat	1.20	0.563	0.238	0.762
Class A	0	0	0	0
Class 70R Wheel	0	0	0	0
SV Loading	0	0	0	0
		1.46	4.73	

Maximum Support moment (M_{xx}) = 4.732 kNm/m

Section Properties of Uncracked Section:

width of section (D_x) = 1000.00 mm
 Depth of Section (D_y) = 220.00 mm
 Gross Cross sectional area (A_g) = 220000 mm²
 Gross Moment of Inertia I_{gxx} = 8.87E+08 mm⁴
 Gross Moment of Inertia I_{gyy} = 1.83E+10 mm⁴

Calculation of Modular Ratio:

Perimeter of section (u) = 2440 mm
 2 A_c/u = 180.328 mm
 For Dry atmosperic condition
 Creep coeff. = 1.48
 $E_{eff} = \frac{E_{cm}}{(1 + \phi)}$ = 1.30E+04

Modular Ratio (m) = E_c / E_{eff} = 15.35

Details of Reinforcements:

at Support	12 mm dia @	100 mm c/c
	0 mm dia @	100 mm c/c

Total Ast provided = 1130.97 mm²/m
 p_t = A_{st}/(b*D) = 0.0069
 effective cover = 56 mm
 C.g. Of Tension steel from N.A. (y) = 54.00 mm
 Area of concrete, A_c = (A_g - A_{st}) = 218869.03 mm²
 Transformed Area of Section A_{st}T = A_c + m A_{st} = 236230.3306 mm²
 Transformed M.O.I (I_{xx}) = I_{gxx} + (m-1).A_{st} . y² = 8.87E+08 + 47327643.95 = 9.35E+08 mm⁴
 Section modulus (Z_{xx}) = I_{xx} / (D_y/2) = 8496917.975 mm³
 Bending stress (M / Z_{xx}) = 0.557 N/mm²
 Tensile strength of concrete , f_{ctm} = 2.77 N/mm²

Uncracked

No need to check for Crack width as the section is cracked

ANNEXURE-I : STAAD ANALYSIS FOR DECK SLAB DESIGN

STAAD PLANE ANALYSIS FOR DESIGN OF SLAB (DL+SIDL+CWLL)

PAGE NO. 1

```

*****
*                               *
*      STAAD.Pro                *
*      Version  2007   Build 05   *
*      Proprietary Program of      *
*      Bentley Systems, Inc.       *
*      Date=   SEP 23, 2015        *
*      Time=   18: 5:27            *
*                               *
*      USER ID:                  *
*****

```

1. STAAD SPACE

INPUT FILE: Deck Slab.STD

2. START JOB INFORMATION

3. ENGINEER DATE 11-NOV-10

4. END JOB INFORMATION

5. INPUT WIDTH 79

6. UNIT METER KN

7. JOINT COORDINATES

8. 1 -0.250001 0 0; 2 1.5 0 0; 3 4.5 0 0; 4 7.5 0 0; 5 10.5 0 0; 6 12.25 0 0

9. MEMBER INCIDENCES

10. 1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6

11. DEFINE MATERIAL START

12. ISOTROPIC CONCRETE

13. E 2.17185E+007

14. POISSON 0.17

15. DENSITY 25

16. ALPHA 1E-005

17. DAMP 0.05

18. END DEFINE MATERIAL

19. MEMBER PROPERTY INDIAN

20. 1 TO 5 PRIS YD 0.22 ZD 1

21. CONSTANTS

22. MATERIAL CONCRETE ALL

23. SUPPORTS

24. 2 TO 5 PINNED

25. *****

26. LOAD 1 SELF WEIGHT OF SLAB

27. SELFWEIGHT Y -1

28. MEMBER LOAD

29. 1 UNI GY -8 0.05 0.5

30. 5 UNI GY -8 1.25 1.7

31. *****

32. LOAD 2 WEARING COAT

33. MEMBER LOAD

34. 2 TO 4 UNI GY -2

35. 1 UNI GY -2 0.5 1.75

36. 5 UNI GY -2 0 1.25

37. *****

38. LOAD 3 1 LANE CLASS-A
 39. MEMBER LOAD
 40. 1 UNI GY -45.8 0.421 1.379
 STAAD SPACE

-- PAGE NO. 2

41. 2 UNI GY -36.4 0.471 1.429
 42. *****
 43. LOAD 4 1-LANE 70-WHEEL (MAX SAGGING MOMENT)
 44. MEMBER LOAD
 45. 2 UNI GY -53.1 2.876 3
 46. 3 UNI GY -54.4 0 0.744
 47. 3 UNI GY -33.9 0.671 1.539
 48. 3 UNI GY -33.9 1.461 2.329
 49. 3 UNI GY -53.1 2.256 3
 50. 4 UNI GY -53.1 0 0.124
 51. *****
 52. LOAD 5 70R WHEEL (MAX HOGGING MOMENT)
 53. MEMBER LOAD
 54. 2 UNI GY -33.4 1.376 2.244
 55. 2 UNI GY -46.6 2.171 3
 56. 3 UNI GY -46.6 0 0.829
 57. 3 UNI GY -33.4 0.756 1.624
 58. *****
 59. LOAD 6 IRC SV LOADING
 60. MEMBER LOAD
 61. 2 UNI GY -52.4 2.776 3
 62. 3 UNI GY -52.4 0 0.684
 63. 3 UNI GY -46.9 0.526 1.434
 64. 3 UNI GY -46.9 1.576 2.484
 65. 3 UNI GY -52.4 2.326 3
 66. 4 UNI GY -52.4 0 0.234
 67. *****
 68. PERFORM ANALYSIS

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 6/ 5/ 4

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH= 1/ 1/ 9 DOF
 TOTAL PRIMARY LOAD CASES = 6, TOTAL DEGREES OF FREEDOM = 24
 SIZE OF STIFFNESS MATRIX = 1 DOUBLE KILO-WORDS
 REQRD/AVAIL. DISK SPACE = 12.0/ 127799.0 MB

***WARNING - INSTABILITY AT JOINT 6 DIRECTION = MX
 PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
 K-MATRIX DIAG= 1.4352283E+05 L-MATRIX DIAG= -2.9103830E-11 EQN NO 22
 ***NOTE - VERY WEAK SPRING ADDED FOR STABILITY
 STAAD SPACE -- PAGE NO. 3

***NOTE** STAAD DETECTS INSTABILITIES AS EXCESSIVE LOSS OF SIGNIFICANT DIGITS
DURING DECOMPOSITION. WHEN A DECOMPOSED DIAGONAL IS LESS THAN THE

BUILT-IN REDUCTION FACTOR TIMES THE ORIGINAL STIFFNESS MATRIX DIAGONAL, STAAD PRINTS A SINGULARITY NOTICE. THE BUILT-IN REDUCTION FACTOR IS 1.000E-09

THE ABOVE CONDITIONS COULD ALSO BE CAUSED BY VERY STIFF OR VERY WEAK ELEMENTS AS WELL AS TRUE SINGULARITIES.

69. PRINT MEMBER FORCES ALL
STAAD SPACE

-- PAGE NO. 4

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KN METE (LOCAL)

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z

1	1	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	13.23	0.00	0.00	0.00	-13.73
	2	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	2.50	0.00	0.00	0.00	-1.56
	3	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	43.88	0.00	0.00	0.00	-37.29
	4	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	0.00	0.00	0.00	0.00	0.00
	5	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	0.00	0.00	0.00	0.00	0.00
	6	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	0.00	0.00	0.00	0.00	0.00

2	1	2	0.00	12.09	0.00	0.00	0.00	13.73
		3	0.00	4.41	0.00	0.00	0.00	-2.22
	2	2	0.00	3.02	0.00	0.00	0.00	1.56
		3	0.00	2.98	0.00	0.00	0.00	-1.49
	3	2	0.00	36.98	0.00	0.00	0.00	37.29
		3	0.00	-2.11	0.00	0.00	0.00	2.15
	4	2	0.00	-6.39	0.00	0.00	0.00	0.00
		3	0.00	12.97	0.00	0.00	0.00	-19.57
	5	2	0.00	7.86	0.00	0.00	0.00	0.00
		3	0.00	59.76	0.00	0.00	0.00	-26.93
	6	2	0.00	-7.17	0.00	0.00	0.00	0.00
		3	0.00	18.91	0.00	0.00	0.00	-22.83

3	1	3	0.00	8.25	0.00	0.00	0.00	2.22
		4	0.00	8.25	0.00	0.00	0.00	-2.22
	2	3	0.00	3.00	0.00	0.00	0.00	1.49
		4	0.00	3.00	0.00	0.00	0.00	-1.49
	3	3	0.00	-0.89	0.00	0.00	0.00	-2.15
		4	0.00	0.89	0.00	0.00	0.00	-0.53
	4	3	0.00	69.80	0.00	0.00	0.00	19.57
		4	0.00	69.03	0.00	0.00	0.00	-19.50
	5	3	0.00	58.36	0.00	0.00	0.00	26.93
		4	0.00	9.27	0.00	0.00	0.00	-4.22

6	3	0.00	78.15	0.00	0.00	0.00	22.83
	4	0.00	78.18	0.00	0.00	0.00	-22.89
4	1	4	0.00	4.41	0.00	0.00	2.22
	5	0.00	12.09	0.00	0.00	0.00	-13.73
2	4	0.00	2.98	0.00	0.00	0.00	1.49
	5	0.00	3.02	0.00	0.00	0.00	-1.56
3	4	0.00	0.18	0.00	0.00	0.00	0.53
	5	0.00	-0.18	0.00	0.00	0.00	0.00

STAAD SPACE -- PAGE NO. 5

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KN METE (LOCAL)

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z

4	4	0.00	12.95	0.00	0.00	0.00	19.50
	5	0.00	-6.36	0.00	0.00	0.00	0.00
5	4	0.00	1.41	0.00	0.00	0.00	4.22
	5	0.00	-1.41	0.00	0.00	0.00	0.00
6	4	0.00	19.41	0.00	0.00	0.00	22.89
	5	0.00	-7.15	0.00	0.00	0.00	0.00
5	1	5	0.00	13.23	0.00	0.00	13.73
	6	0.00	0.00	0.00	0.00	0.00	0.00
2	5	0.00	2.50	0.00	0.00	0.00	1.56
	6	0.00	0.00	0.00	0.00	0.00	0.00
3	5	0.00	0.00	0.00	0.00	0.00	0.00
	6	0.00	0.00	0.00	0.00	0.00	0.00
4	5	0.00	0.00	0.00	0.00	0.00	0.00
	6	0.00	0.00	0.00	0.00	0.00	0.00
5	5	0.00	0.00	0.00	0.00	0.00	0.00
	6	0.00	0.00	0.00	0.00	0.00	0.00
6	5	0.00	0.00	0.00	0.00	0.00	0.00
	6	0.00	0.00	0.00	0.00	0.00	0.00

***** END OF LATEST ANALYSIS RESULT *****

70. FINISH

Design of Projecting Deck Slab Beyond End Cross Girder
(This will be designed as cantilever slab)

Data Given

Thickness of Deck slab at edge = 0.370 m
 Length of corbel = 0.500 m
 Thickness of W.C. = 0.065 m

Calculation of dead Load & SIDL Moment at the Face of Cantilever

Sl. no.	Item	Load Factors	Length (m)	Depth (m)	Unit Wt. (kN/m ³)	Wt. per m (kN/m)	Dis. of cg. from the face (m)	Moment @ the cant. face (kN-m/m)
1	Cantilever Slab seg.(1)	1.350	0.500	0.370	25.000	4.625	0.250	1.561
2	Wt. of the Wearing Course (Wt. of W.C. 2.00 kN/m ²)	1.750	0.500	x	2.000	1.000	0.250	0.438
	Total					5.625		1.998

Moment in Cantilever Slab due to CWLL

Class-70R Bogie Axle placed symmetrically(M Type)

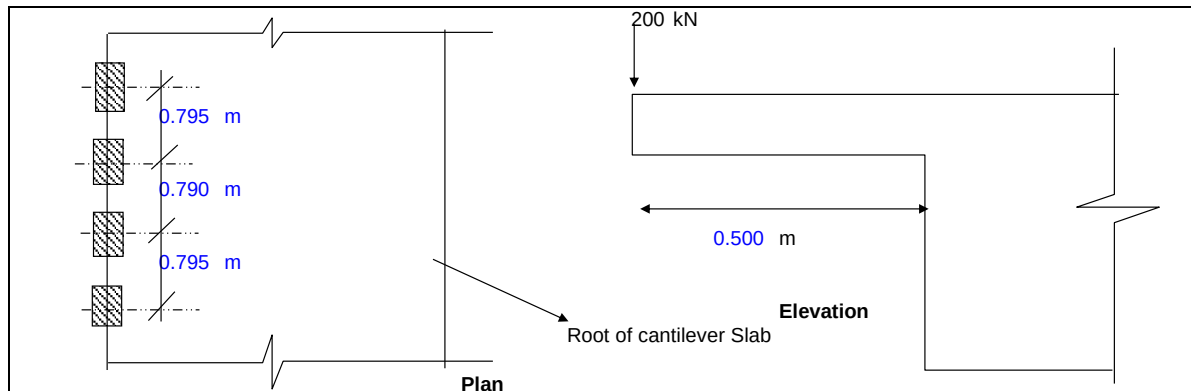
Considering 200.00 kN Axle passing through the section
 Dis. of Nearest Axle for Cl.70R Bogie Axle = 1.220 m

Width of tyre of 70R Bogie Axle (M-Type) = 0.410 - 0.050 = 0.360 m

Calculation of Contact Length along Longitudinal direction of span For 70R Bogie Axle

Maximum Tyre Pressure = 5.273 Kg/m²

Contact length of tyre for 50.000 kN Load = $\frac{50.000}{360} \times \frac{10}{5.273} = 0.263$ mm
 (for 70R "M-Type")



Effective width of the Slab	be	=	1.2 a	+	b1

b1	=	0.360	+	2	x	0.065	=	0.490 m
						Th. of W.C.		

a	=	0.500 m

be	=	1.200	x	0.500	+	0.490	=	1.09 m

Hence, effective width	=	$\frac{1.09}{2}$	+	0.795	+	0.790	+	0.795	+	$\frac{1.09}{2}$
	=	3.47 m								

Impact Factor for Cl. 70R Wheeled Loading (for cantilever slab portion having length < 9m)	=	1.250

Considering no dispersion of Load along the Span

Intensity of the Load Live Load	=	$\frac{200.000}{3.470}$	=	57.637 kN/m

Live load Factor	=	1.500 -Refer Table 3.2 of IRC:6-2014

B.M. at the root of Cantilever due to Live Load	=	1.250	x	57.637	x	0.500	x	1.500
	=	54.035 kN-m/m						

S.F. at the root of Cantilever due to Live Load	=	1.250	x	57.637	x	1.500
	=	108.069 kN per m				

Total Ultimate B.M. at the root of Canl	=	1.998	+	54.035	=	56.033 kN-m/m

Material Properties:

Grade of Concrete = M 35

fck	=	35 N/mm ²	
fctm	=	2.77 N/mm ²	
Ec	=	29580.40 N/mm ²	(5000 x sqrt (fck)
Grade of Steel	=	Fe 500 D	(HYSD Bars)
fy or fyk	=	500.00 N/mm ²	
fyd	=	434.783 N/mm ²	(fy/1.15)
Es	=	200000.0 N/mm ²	
Clear cover to reinforcement	=	40 mm	
Maximum Dia of Bar along long.	=	12 mm	
Maximum Dia of Bar along cant.	=	12 mm	
Unit width of slab	=	1000 mm	

Calculation of Flexure Reinforcement

Check For Depth of Pier Cap : (Eq. For Mulim. Derived based on IRC:112-2011)
 Mulim = $0.167 \times fck \times b \times d^2$ = 56.033 kNm/m

$$\text{Effective Depth of Cap Required (drec)} = \frac{\text{SQRT} \left(\frac{56.033}{0.167} \times \frac{1000000}{35 \times 1000} \right)}{1000}$$

$$\text{Effective Depth of Cap Required (drec)} = 97.911 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 155.911 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 370 \text{ mm}$$

$$\text{Deff provided} = 312 \text{ mm}$$

OK

$$R = \frac{Mu}{(b \times d^2)} = 0.576$$

Minimum Longitudinal Reinforcement in Pier Cap :

$$\text{As. Min} = 0.26 \times \frac{fctm}{fyk} \times b \times d \text{ - Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

$$\text{OR} = 0.0015 \times b \times d \text{ -Refer Clause 16.9 of IRC:112-2011'}$$

Whichever is higher

$$b = 1000 \text{ mm}$$

$$d = 312 \text{ mm}$$

$$\text{Ast min} = 468 \text{ mm}^2$$

Area of Steel Required:

$$\text{Design Moment of Resistance } M_{RD} = 0.87 f_y A_{st} \times (d - (\lambda x_u / 2))$$

$$\eta = 1$$

$$f_{cd} = 15.61 \text{ N/mm}^2$$

$$f_y = 500.00 \text{ N/mm}^2$$

$$\lambda = 0.8$$

$$M_{RD} = 0.87 f_y A_{st} \times (d - (0.87 f_y A_{st} / (b \times n \times f_{cd} \times 2)))$$

$$\text{Area of Steel Required, } A_{st_{red}} = 347.76 \text{ mm}^2/\text{m}$$

$$A_{st_{req}} = 347.758 \text{ mm}^2/\text{m}$$

$$\text{Ast required} = \max(A_{st_{min}}, A_{st_{req}}) : 468.0000 \text{ mm}^2/\text{m}$$

Provide	12 dia @ spacing	200 +	10 dia @ spac	200	=	958.19 mm ² /m

DESIGN OF ABUTMENT(A1) WITH
OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
13+225

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 50 m
Design speed of vehicle = 0 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 18.750m	18.750m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side) =	0.600m	0.600m
Slab projection Beyond C/L of Bearing (Span Side) =	0.600m	0.600m
Span -c/c of E.J.	= 20.000m	20.00m
Type of Superstructure	= Precast RCC Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 11.000m	
Projection beyond crash barrier	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 0.000m	
Railing/kerb on footpath edge	= 0.250m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 4
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 370 kN/m² (as per geotechnical report)
SBC of soil-Normal Case = 400 kN/m²
SBC of soil-Seismic Case = 500 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1.5 Normal Case
= 1.25 Seismic Case

Permissible FOS against Overturning = 2 Normal Case
= 1.5 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 2.120m	
Height of Trapering portion	= 0.772m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 1.850m	1.850m
Width of Abutmentcap at bottom of Tapering Portion	= 1.000m	1.000m
Projection of Abutment Cap (Span Side)	= 0.400m	0.400m
Projection of Abutment Cap Back Side	= 0.450m	0.450m
Abutmentcap thickness (Uniform portion)	= 0.500m	

Abutment cap thickness (Tapering Portion)	=	0.500m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	1.000m	1.000m
Thickness of Abutment shaft at Bottom	=	1.000m	1.000m

Solid Return Wall

Length of Return wall	=	4.600m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.750m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.750m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	8.400m
abutment pedestal width	=	1.000m
abutment pedestal Height	=	1.000m
Width of Toe Slab	=	2.800m
Width of Heel Slab	=	4.600m
Thickness of Toe slab at tip	=	0.400m
Thickness of Toe slab near shaft	=	1.000m
Thickness of heel slab at tip	=	0.400m
Thickness of heel slab near shaft	=	1.000m
Width of backfill on heel slab	=	4.600m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	9.154m
Height of back fill at back fill edge of heel slab	=	8.554m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1405.947m	Cross Slope (Bi-directional)	=	6.500%
Deck level at Outer Edge =	1406.321m	Height of Superstructure	=	1.820m
Deck level at center line =	1405.947m	Min. Height of Footpath Side Pedestal (1)	=	0.300m
Soffit Level at center of bridge =	1404.062m	Height of Pedestal (2)	=	0.495m
Abutment cap top level =	1403.242m	Height of Pedestal (3)	=	0.690m
Abutment cap bottom lvl (uniform portion ends)	1402.742m	Height of Pedestal (4)	=	0.885m
Abutment cap bottom lvl (corbel portion ends)	1402.242m	Height of Pedestal (5)	=	0.000m
Abutment shaft top level =	1402.242m	Height of Pedestal (6)	=	0.000m
Ground level/LBL =	1399.583m	Distance of nearest girder to c.l. of deck	=	1.500m
Abutment shaft bottom level =	1397.580m	Height (Avg.) of Dirt Wall	=	2.892m
Foundation level =	1396.580m	Abutment shaft Above G.L	=	2.659m
HFL	= 1399.583m	Abutment Shaft below G.L	=	2.003m
		Height of abutment shaft	=	4.662m
		MSL	=	1399.583m
		Wedge over girder flange	=	0.0650m

Material Specification

Concrete Grade	=	M 35
Characteristic compressive strength of concrete, f _{ck}	=	35.00 Mpa at 28 days
Design Compressive strength of Concrete, f _{cd}	=	15.63 Mpa at 28 (0.67/1.5 * f _{ck})
Tensile strength of concrete , f _{ctm}	=	2.77 MPa

Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E _{cm}	=	32308.250	N/mm ²
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f _y or f _{yk}	=	500	Mpa
Design Yield Strength of Reinforcement, f _{yd}	=	434.78	Mpa (1/1.15 * f _y)
Modulus of Elasticity of Steel (E _s)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48	f _{ck}
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm
<u>Seismic Data:</u>			
Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction
<u>Properties of backfill material :</u>			
c	=	0	
φ	=	35	
θ	=	90	
β	=	0	
δ	=	22.5	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.000m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

		eT
DL Reaction on Bearing No.-1	=	450.94 4.50
DL Reaction on Bearing No.-2	=	432.46 1.50
DL Reaction on Bearing No.-3	=	432.46 -1.50
DL Reaction on Bearing No.-4	=	450.94 -4.50
DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		1766.80 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	135.917 4.50
SIDL Reaction on Bearing No.-2	=	-55.917 1.50
SIDL Reaction on Bearing No.-3	=	-55.917 -1.50
SIDL Reaction on Bearing No.-4	=	135.917 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		160.00 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	60.954 4.50
SIDL Reaction on Bearing No.-2	=	66.495 1.50
SIDL Reaction on Bearing No.-3	=	66.495 -1.50
SIDL Reaction on Bearing No.-4	=	60.954 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		254.90 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 2-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	981.82	KN	Corr. Transv	=	932.730 kNm	ecc.	0.95
min	=	368.18	KN	Corr. Transv	=	349.770 kNm	ecc.	0.95

SV Loading

max	=	2003.04	KN	Corr. Transv	=	600.912 kNm	ecc.	0.30
min	=	1416.96	KN	Corr. Transv	=	425.088 kNm	ecc.	0.30

**MAXIMUM TRASVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	758.73	KN	Corr. Transv	=	2393.795 kNm	ecc.	3.16
min	=	241.27	KN	Corr. Transv	=	761.205 kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.300 x	0.800 x	0.800 =	0.1920 m ³	4.500
Pedestal 2	=	0.495 x	0.800 x	0.800 =	0.3168 m ³	1.500
Pedestal 3	=	0.690 x	0.800 x	0.800 =	0.4416 m ³	-1.500
Pedestal 4	=	0.885 x	0.800 x	0.800 =	0.5664 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					1.5168 m ³	-1.234

VOLUME CALCULATION

C.G. Of Footing	=	4.200 m
C.G. Of shaft from toe tip	=	3.300 m
Distance between c.g. of shaft and footing	=	0.900 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	2.120	7.950	-0.800	0.100	-4.100	0.000
-Trapering portion	1	12.50	0.300	0.772	2.895	-0.800	0.100	-4.100	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.100	-0.200	-4.400	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-1.050	-0.150	-4.350	0.000
Cap (uniform portion)	1	12.50	1.850	0.500	11.563	-0.025	0.875	-3.325	0.000
Cap (Corbel Portion)	1	12.50	1.850	0.500	8.771	-0.025	0.875	-3.325	0.000
		12.50	1.000						
Shaft above HFL	1	12.50	1.000	2.659	33.237	0.000	0.900	-3.300	0.000
Shaft below HFL	1	12.50	1.000	1.603	20.038	0.000	0.900	-3.300	0.000
Solid Return Wall	1	4.60	0.625	9.041	25.992	-2.800	-1.900	-6.100	5.938
Cantilever Return wall(Rectangular portion)	1	0.00	0.500	0.000	0.000	-0.500	0.400	-3.800	5.938
Cantilever Return wall(Traingular portion)	1	0.00	0.625	0.000	0.000	-0.500	0.400	-3.800	5.938
Footing									
Heel Slab	1	12.50	4.600	0.700	40.250		-1.571	-5.771	0.000
Toe Slab	1	12.50	2.800	0.700	24.500		2.600	-1.600	0.000
Portion between Heel and Toe	1	12.50	1.000	1.000	12.500		0.900	-3.300	0.000
Back filling above HFL over Heel Slab	1	12.50	4.600	6.364	365.930		-1.900	-6.100	0.000
Back filling below HFL over Heel Slab	1	12.50	4.600	2.303	132.423		-2.000	-6.200	0.000
Backfill above Heel slab	1	12.50	4.600	8.854	509.098		-1.926	-6.126	0.000
Front Filling over Toe Slab	1	12.50	2.800	2.303	80.605		2.739	-1.461	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	2.800	2.303	80.605		1.839	-2.361	0.000
Side filling between heel and toe	1	0.00	1.000	2.303	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-1.100	-0.200	-4.400	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.000	6.364	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.000	2.303	0.000		0.000	0.000	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barr	2	8 kN/m	1.750	28.00kN		-0.800	0.100	-4.100

SECTIONAL PROPERTIES

Width of Footing (B)	=	8.400 m
Length of Footing (L)	=	12.500 m

A	=	8.400	x	12.500	=	105.000 m ²
ZL	=	12.500	x	11.760	=	147.000 m ³
ZT	=	IT1	+	IT2		
		distance of extreme point from centre				
IT1	=	8.400	x	162.760	=	1367.19 m ⁴
IT2 (moment of inertia of triangle)	=	8.400	x	0.000	+	0.500 x 8.400 x 0.000 x 39.063
from centre of footing	=	0.000	m ⁴			
Moment of inertia of two triangle	=	0.000	m ⁴			
Total moment of inertia	=	1367.19	m ⁴			
Distance of extreme point from centre of footing	=	6.250	+	0.000	=	6.250 m
Total Section modulus (ZT)	=	218.750	m ³			

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.000	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Serviceability Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.00	1.00	1.00
wearing course	1.20	1.20	1.20
Back Filling Weight	1.00	1.00	1.00
Shrinkage Creep Effect	1.00	1.00	1.00
Earth Pressure due to back filling	1.00	1.00	1.00
CWLL and Associate load and FPLL	1.00	0.75	0.00
Live Load Surcharge	0.80	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

Horizontal Seismic Force For Zone

4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.) or horizontal vibrations.

$$= 2.0 \frac{D^{1/2}}{1000F}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 6.962 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 8.765 m

Abutment Cap Top Level - Foundation Level

= 6.662 m

Dimensions of Abutment Shaft

Length = 12.50 m

Width = 1.00 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 0.781 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 162.760 m^4

Ecm = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 239.896 KN
 D = 2181.70 KN
 T = 0.1907 sec

Transverse Direction

Force = 36207.241 KN
 D = 2451.697 KN
 T = 0.0165 sec

For Hard or Rocky Strata, with $N > 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.247

For Medium Strata, with $N > 10 \& N < 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.247

For Soft Soil Strata, with $N < 10$

$\frac{S_a}{g}$ = 2.5

$\frac{S_a}{g}$ = 1.247

Medium Soil Strata

Sa/g = 2.5

Sa/g = 1.247

Seismic Coeff. In Longitudinal Direction = 0.12

Seismic Coeff. In Transverse Direction = 0.180

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.180	In Transverse direction
Av	=	0.120	In Vertical direction

For Design of Foundation

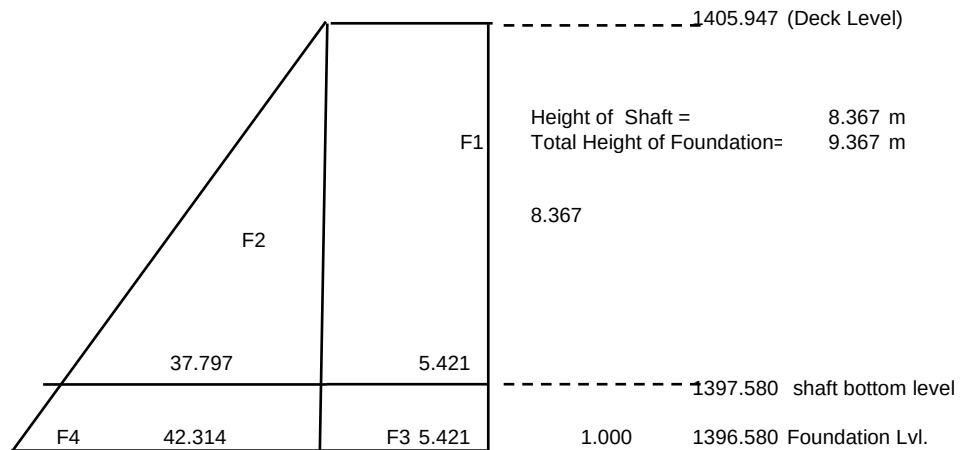
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.162	In Longitudinal direction
Ah	=	0.180	In Transverse direction
Av	=	0.120	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = 1.2 m
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1397.580 m (at Shaft Base)			
@ RL					1396.580 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	8.367	x	12.500	=	566.949 kN
M1	=	566.95	x	4.18	=	2371.830 kN.m		at Shaft Bottom
F3	=	5.421	x	9.367	x	12.500	=	634.709 kN
M3	=	634.709	x	4.683	=	2972.658 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	8.367	= 37.797 kN/m ²
F2	=	0.5	x	37.80	x	8.367	x 12.50
	=	1976.525 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	1976.52	x	3.51	=	6945.785 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	9.367	= 42.314 kN/m ²
F4	=	0.5	x	42.31	x	9.367	x 12.50
	=	2477.215 kN					
M4	=	2477.22	x	3.93	=	9745.712 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m ³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	8.367	=	40.162 kN/m ²	
F	=	0.5	x	40.162	x	8.367	x 12.500
	=	2100.201 kN					

$$M = 2100.20 \times 2.789 = 5857.460 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 9.367 = 44.96 \text{ kN/m}^2$$

$$F = 0.5 \times 44.962 \times 9.37 \times 12.50 = 2632.221 \text{ kN}$$

$$M = 2632.22 \times 3.122 = 8218.670 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50$$

$$= 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

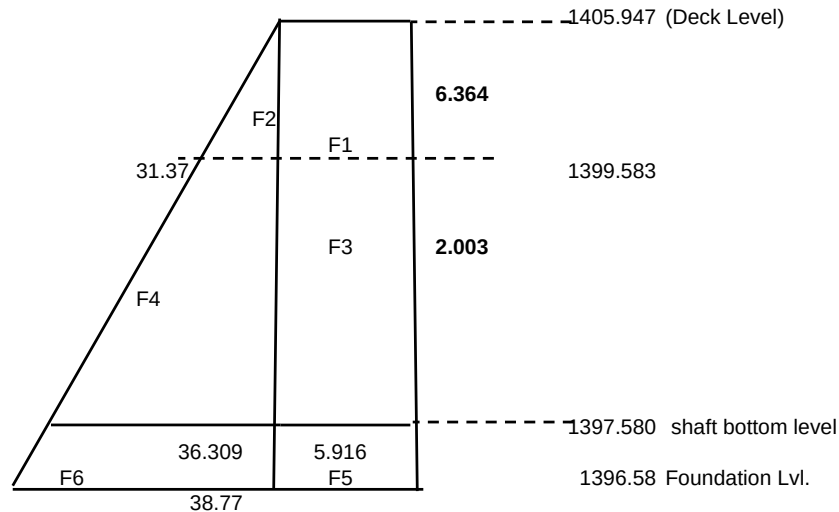
$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	6945.785	9745.712	1976.525	2477.215
Due to Minimum Fluid Pressure	5857.460	8218.670	2100.201	2632.221
Governing of Two	6945.785	9745.712	2100.201	2632.221
Due to Live Load Surcharge	2371.830	2972.658	566.949	634.709
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	11.3 degree	0.196 radians
	Ka'	=	0.246 active component	
	Kp'	=	5.249 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	
Assuming				

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1397.6 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m ²
F1	=	5.916	x	8.367	x	12.500	=	618.690 kN
M1	=	618.69	x	4.18	=	2588.290 kN.m		at Shaft Bottom
F3	=	5.916	x	9.367	x	12.500	=	692.634 kN
M3	=	692.63	x	4.68	=	3243.952 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	6.364	=	31.372 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	2.003	=	4.937 kN/m ²
F2	=	0.5	x	31.37	x	6.364	x	12.50
	=	1247.822 kN						
F4	=	$\left(\frac{31.37 + 36.31}{2} \right)$	x	2.00	x	12.50		
	=	847.282 kN						
Total Force	=	2095.103 kN						
M2	=	1247.82	x	4.68	=	5834.664 kN.m		
M4	=	847.28	x	0.98	=	827.920 kN.m		
Total Moment	=	6662.58 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 6.364 = 31.372 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 3.003 = 7.402 \text{ kN/m}^2$$

$$F2 = 0.5 \times 31.37 \times 6.364 \times 12.50 = 1247.822 \text{ kN}$$

$$F6 = \left(\frac{31.37 + 38.77}{2} \right) \times 3.00 \times 12.50 = 1316.549 \text{ kN}$$

$$\text{Total Force} = 2564.371 \text{ kN}$$

$$M2 = 1247.82 \times 5.68 = 7082.486 \text{ kN.m}$$

$$M6 = 1316.55 \times 1.45 = 1907.268 \text{ kN.m}$$

$$\text{Total Moment} = 8989.75 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

$$= 5.249 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2$$

$$\begin{aligned} \text{Force due to passive @ Foundation, F} &= 0.5 \times 0.000 \times 12.50 \\ &= 0.000 \text{ kN} \end{aligned}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	6662.584	8989.754	2095.103	2564.371
Due to Minimum Fluid Pressure	5857.460	8218.670	2100.201	2632.221
Governing of Two	6662.584	8989.754	2100.201	2632.221
Due to Live Load Surcharge	2588.290	3243.952	618.690	692.634
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2} (1 \pm \alpha v)$$

αh	=	0.120	
αv	=	0.120	
ϕ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

αh	=	HORIZONTAL SEISMIC COEFFICIENT
αv	=	VERTICAL SEISMIC COEFFICIENT
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.107 \\ 0.135 \end{matrix}$$

$$C_a = \begin{matrix} 1 \\ 0.321 \end{matrix} \begin{matrix} 2 \\ 0.270 \end{matrix}$$

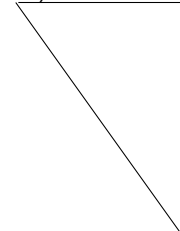
Ca	=	0.321	
Ka	=	0.226	
Dynamic Increment	=	0.321	-0.226 0.096

3 Earth Pressure : DRY CASE (Seismic case)

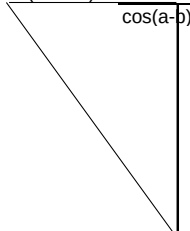
Equivalent Live Load Surcharge height	=	1.2 m
Assuming	γ_{dry}	= 20 kN/m ³
	γ_{water}	= 10.00 kN/m ³

Active Earth Pressure

$$3 (Ca - Ka) = 0.287$$

**Surcharge Earth Pressure**

$$2 (Ca - Ka) \frac{\cos a}{\cos(a-b)} = 0.19 \quad \text{--- 1405.947 (Deck Level)}$$

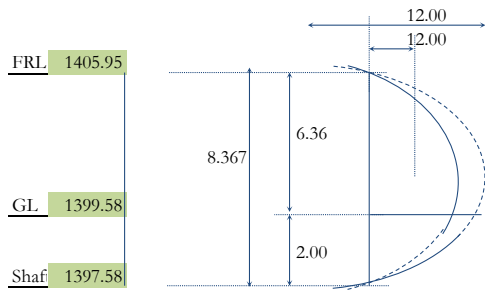


--- 1397.580 (Shaft bottom level)

--- 1396.580 (Foundation Bottom level)

Earth Pressure Diagram for Dynamic Increment**Due to Dynamic Live Load Surcharge**

	=	0.191	x	20.00	x	1.2	=	4.589 kN/m ²
at Shaft Bottom Level								
E1	=	0.50	x	4.589	x	8.367	x	12.500
	=	239.992	kN					
M1	=	239.992	x	5.606			=	1345.370 kN.m
at Foundation Bottom Level								
E2	=	0.50	x	4.589	x	8.367	x	12.500
	=	239.992	kN					
M2	=	239.992	x	6.606			=	1585.362 kN.m

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	12.00 t/m ²
h	=	8.367 m
y	=	2.1805 m
L	=	12.500 m

Parabola below Water Level

p_mid_height	=	12.00 t/m ²
h	=	8.367 m
y	=	-2.1805 m
L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	715.8	4.7
Parabola below Water Level	120.9	1.3
Total Dynamic Earth Pressure	836.7	4.2

$$E3 = 836.673 \text{ kN}$$

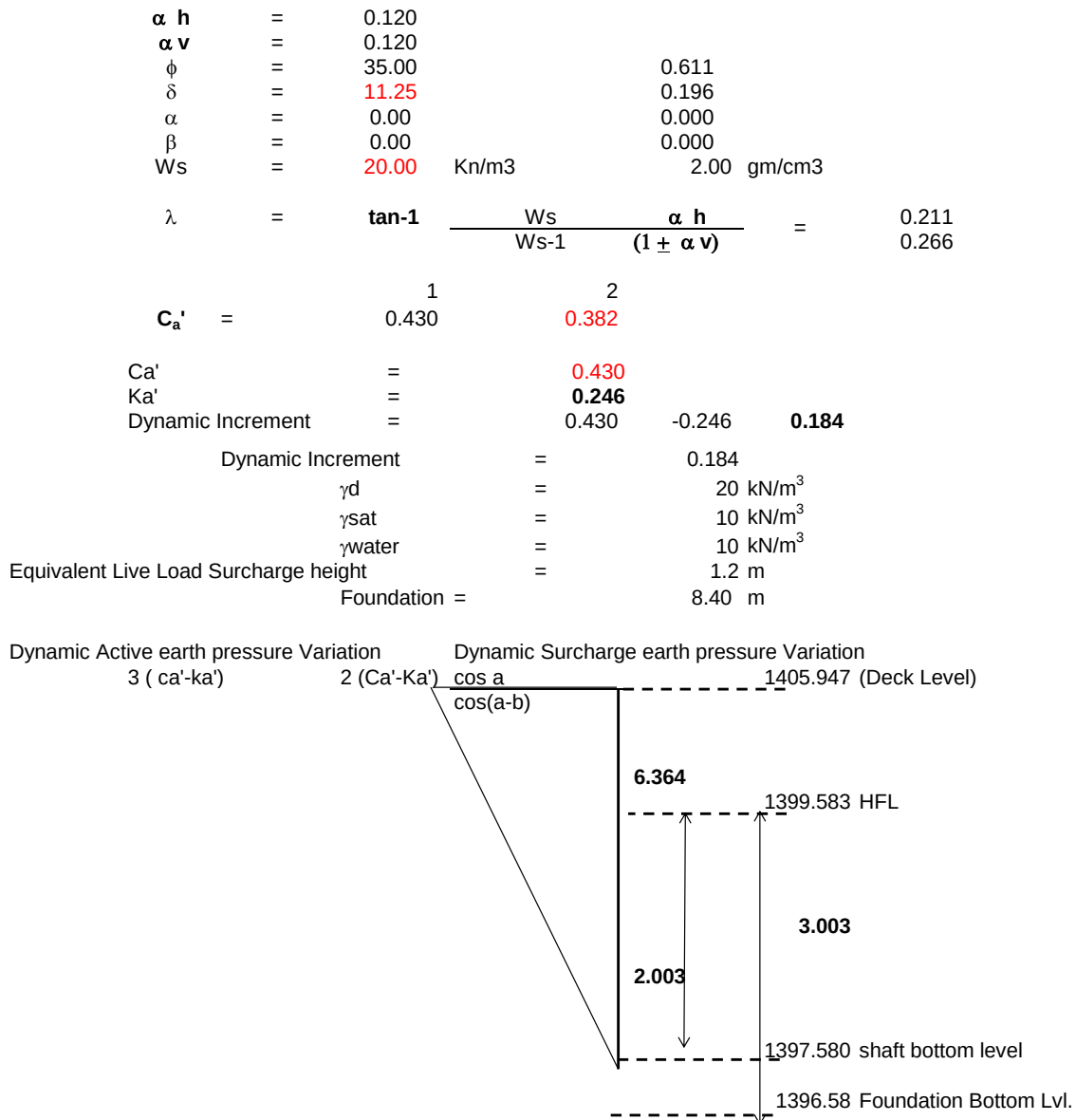
$$E4 = 836.673 \text{ kN}$$

$$M3 = 836.67 \times 4.18 = 3500.221 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 836.67 \times 5.18 = 4336.893 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	6945.785	9745.712	1976.525	2477.215
Due to active Earth Pressure (dynamic Increment)	3500.221	4336.893	836.673	836.673
Total Earth Pressure	10446.005	14082.605	2813.197	3313.888
Due to Minimum Fluid Pressure	5857.460	8218.670	2100.201	2632.221
Governing of Two	6945.785	14082.605	2813.197	3313.888
Due to Live Load Surcharge (Static)	2371.830	2972.658	566.949	634.709
Due to Live Load Surcharge(Dynamic)	1345.370	1585.362	239.992	239.992
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case**Earth Pressure Diagram**

h'	=	2.003
h	=	8.367

Dynamic active earth pressure

3 (ca'-ka')	=	0.287 at deck level
3 (ca'-ka')	=	0.551 at deck level
3 (ca'-ka')h'/h	=	0.132 at HFL level

Dynamic Surcharge earth pressure

2 (Ca'-Ka') $\frac{\cos a}{\cos(a-b)}$	=	0.367 at deck level	Pressure	=	8.82 kN/m ²
	=	0.088 at HFL level	Pressure	=	2.11 kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	4.589 kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	2.11 kN/m ²

at Shaft Bottom Level

F1	=	3.350	x	6.364	x	12.500	=	266.513 kN
M1	=	266.51	x	5.19	=	1381.869 kN.m		
F2	=	0.50	x	2.111	x	2.003	x	12.500
	=	26.430 kN						

$$M2 = 26.43 \times 1.34 = 35.469 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 292.942 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 1417.338 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

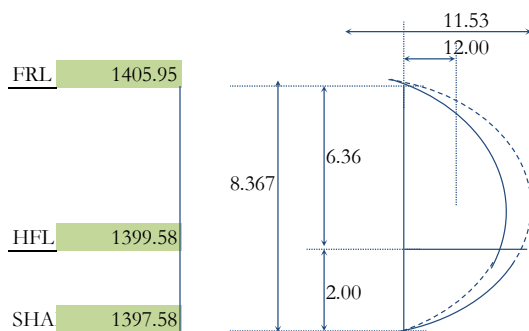
$$F3 = 3.350 \times 6.364 \times 12.500 = 266.513 \text{ kN}$$

$$M3 = 266.51 \times 6.19 = 1648.382 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 2.111 \times 2.003 \times 12.500 \\ &= 26.430 \text{ kN} \end{aligned}$$

$$M4 = 26.43 \times 2.34 = 61.898 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 292.942 \text{ kN} \\ \text{Total Moment at Founding level} &= 1710.280 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 12.00 \text{ t/m}^2 \\ h &= 8.367 \text{ m} \\ y &= 2.1805 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 11.53 \text{ t/m}^2 \\ h &= 8.367 \text{ m} \\ y &= -2.1805 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	715.8	4.7
Parabola below Water Level	116.2	1.3
Total Dynamic Earth Pressure	831.9	4.2

Total Force (F1 + F2)	=	831.936 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	831.936 kN	at Foundation Bottom Level

$$M1 = 831.94 \times 4.20 = 3494.046 \text{ kN.m}$$

$$\text{Total Mome} = 3494.046 \text{ kN.m} \quad \text{at Shaft Bottom}$$

$$M1 = 831.94 \times 5.20 = 4325.982 \text{ kN.m}$$

$$\text{Total Mome} = 4325.982 \text{ kN.m} \quad \text{at Foundation Bottom Level}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Bottom	At Shaft Bottom Lvl	At Foundation Bottom Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure(Static)	6662.584	8989.754	2095.103	2564.371
Due to active Earth Pressure (Dynamic Increment)	3494.046	4325.982	831.936	831.936
Total Earth Pressure	10156.630	13315.735	2927.039	3396.307
Due to Minimum Fluid Pressure	5857.460	8218.670	2100.201	2632.221
Governing of Two	10156.630	13315.735	2927.039	3396.307
Due to Live Load Surcharge(Static)	2588.290	3243.952	618.690	692.634
Due to Live Load Surcharge (Dynamic Increment)	1417.338	1710.280	292.942	292.942
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - SPHERICAL BEARING

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1766.80	1.35	1.35	2385.18	2385.18	
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00	
WC	=	254.90	1.75	1.75	446.07	446.07	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	981.82	1.5	0.20	1472.73	196.36	(1-70RW + 2-CLASS A)
CWLLmax-Reaction case	=	2003.04	1	0.20	2003.04	400.61	SV Loading
CWLLmin	=	368.18	1.5	0.20	552.27	73.64	(1-70RW + 2-CLASS A)
CWLLmin	=	1416.96	1	0.20	1416.96	283.39	SV Loading
CWLLmax-Transv. Moment Case		758.73	1.5	0.20	1138.10	151.75	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 1108 = 255.4 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3047.25	0	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.1	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.1	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.1	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	785.41	152.362	545.068	633.048	785.411	
DL+SIDL+LL-Max Reaction case		3120.89	836.49	156.044	574.290	680.447	836.491	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	836.49	162.181	580.426	674.310	836.491	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3199.00	836.49	159.950	578.195	676.541	836.491	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	235.623	152.362	270.174	83.261	270.174	
DL+SIDL+LL-Max Reaction case		3120.89	286.703	156.044	299.396	130.659	299.396	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	286.703	162.181	305.532	124.523	305.532	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3199.00	286.703	159.950	303.301	126.753	303.301

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.35	1.35	2385.18	2385.18
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	254.90	1.75	1.75	446.07	446.07
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	981.82	1.5	0.75	1472.73	736.37
CWLLmax-Transv. Moment Case	=	758.73	1.5	0.75	1138.10	569.05
CWLLmin	=	368.18	1.5	0.75	552.27	276.13

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3047.25	0.000	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.100	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0.000	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.100	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0.000	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.100	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	1060.30	152.362	682.515	907.942	1060.305	
DL+SIDL+LL-Max Reaction case		3323.38	1111.38	166.169	721.862	945.216	1111.385	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	1111.38	189.181	744.873	922.204	1111.385	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	1111.38	180.815	736.507	930.570	1111.385	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	318.091	152.362	311.408	165.729	318.091	
DL+SIDL+LL-Max Reaction case		3323.38	369.171	166.169	350.755	203.002	369.171	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	369.171	189.181	373.766	179.991	373.766	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	369.171	180.815	365.401	188.357	369.171	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1	1.00	1766.80	1766.80
SIDL except wc	=	160.00	1	1.00	160.00	160.00
WC	=	254.90	1	1.00	254.90	254.90
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1	0.20	981.82	196.36
CWLLmax- Transv. Moment Case		758.73	1	0.20	758.73	151.75
CWLLmin	=	368.18	1	0.20	368.18	73.64

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	0.000	109.085	109.085	-109.085	109.085	
DL+SIDL+LL-Max Reaction case	2549.88	255.400	127.494	255.194	127.906	255.194	(1-70RW + 2-CLASS A)
	3598.66	0.000	179.933	179.933	-179.933	179.933	SV Loading
DL+SIDL+LL-Min Reaction case	3163.52	255.400	158.176	285.876	97.224	285.876	(1-70RW + 2-CLASS A)
	4184.74	0.000	209.237	209.237	-209.237	209.237	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2940.43	255.400	147.021	274.721	108.379	274.721	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	706.870	109.085	462.520	597.785	706.870	
DL+SIDL+LL-Max Reaction case	2255.33	757.950	112.767	491.742	645.183	757.950	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	757.950	118.903	497.878	639.047	757.950	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	757.950	116.672	495.647	641.278	757.950	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	212.061	109.085	215.115	102.976	215.115	
DL+SIDL+LL-Max Reaction case	2255.33	263.141	112.767	244.337	150.374	263.141	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	263.141	118.903	250.474	144.238	263.141	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	263.141	116.672	248.243	146.469	263.141	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.1	1.10	1943.48	1943.48
SIDL except wc	=	160.00	1.1	1.10	176.00	176.00
WC	=	254.90	1.35	1.35	344.11	344.11
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1.5	0.00	1472.73	0.00
CWLLmax- Transv. Moment Case		758.73	1.5	0.00	1138.10	0.00
CWLLmin	=	368.18	1.5	0.00	552.27	0.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2463.59	0.000	123.180	123.180	-123.180	123.180	
DL+SIDL+LL-Max Reaction case	3015.86	383.100	150.793	342.343	232.307	342.343	(1-70RW + 2-CLASS A)
	3880.55	0.000	194.028	194.028	-194.028	194.028	SV Loading
DL+SIDL+LL-Min Reaction case	3936.32	383.100	196.816	388.366	186.284	388.366	(1-70RW + 2-CLASS A)
	4466.63	0.000	223.332	223.332	-223.332	223.332	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3601.69	383.100	180.084	371.634	203.016	371.634	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	1060.30	123.180	653.332	937.125	1060.305	
DL+SIDL+LL-Max Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	1060.30	123.180	653.332	937.125	1060.305	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	318.091	123.180	282.225	194.912	318.091	
DL+SIDL+LL-Max Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	318.091	123.180	282.225	194.912	318.091	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1766.80	1	1	1	1766.80	1766.80	1766.80
SIDL except wc	=	160.00	1	1	1	160.00	160.00	160.00
WC	=	254.90	1.20	1.20	1.20	305.88	305.88	305.88
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1	0.75	0	981.82	736.37	0.00
CWLLmax- Transv. Moment Case	=	758.73	1	0.75	0	758.73	569.05	0.00
CWLLmin	=	368.18	1	0.75	0	368.18	276.13	0.00

Braking Force = 255.4 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2600.86	255.400	130.043	257.743	125.357	257.743	(1-70RW + 2- CLASS A)
	3649.64	0.000	182.482	182.482	-182.482	182.482	SV Loading
DL+SIDL+LL-Min Reaction case	3214.50	255.400	160.725	288.425	94.675	288.425	(1-70RW + 2- CLASS A)
	4235.72	0.000	211.786	211.786	-211.786	211.786	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2991.41	255.400	149.570	277.270	105.830	277.270	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2508.81	191.550	125.441	221.216	66.109	221.216	Dry Case
DL+SIDL+LL-Min Reaction case	2969.04	191.550	148.452	244.227	43.098	244.227	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2801.72	191.550	140.086	235.861	51.464	235.861	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	Dry Case
DL+SIDL+LL-Min Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2232.68	0.000	111.634	111.634	-111.634	111.634	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	0.00	kmph	0.00	kmph
Live Load	W	=	981.82	kN	981.82	kN
Radius of Curvature	R	=	50.00	m	50.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

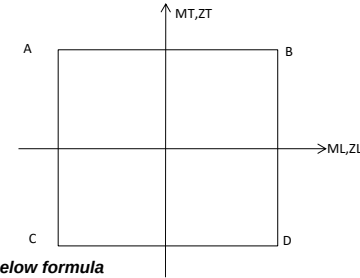
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1396.580 m

Properties of Footing Base:

A	=	105.000	m ²
ZL	=	147.000	m ³
ZT	=	218.750	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1766.799	0.900	1590.120	0.000	0.000
SIDL except Wearing Course	1			160.000	0.900	144.000	0.000	0.000
Wearing Course	1			254.898	0.900	229.408	0.000	0.000
Bearing Pedestal	1	25	1.517	37.920	0.900	34.128	-1.234	-46.800
				2219.617		1997.655		-46.800
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	7.950	198.750	0.100	19.875	0.000	0.000
Dirt Wall-Tapered portion	1	25	2.895	72.363	0.100	7.236	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	-0.200	-5.625	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	-0.150	-2.109	0.000	0.000
Cap - (uniform portion)	1	25	11.563	289.063	0.875	252.930	0.000	0.000
Cap - (corbel portion)	1	25	8.771	219.278	0.875	191.869	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.400	0.000	5.938	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.400	0.000	5.938	0.000
RCC Railing or Crash Barrier	1			28.000	0.100	2.800	0.000	0.000
Approach Slab	1	25	6.563	164.063	-0.200	-32.813	0.000	0.000
				1013.704		434.163		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	25.992	649.804	-1.900	-1234.627	5.938	3858.211
Abutment Shaft	1	25	53.275	1331.875	0.900	1198.687	0.000	0.000
Back filling over heel slab	1	20	509.098	10181.956	-1.926	-19610.217	0.000	0.000
Front Filling over toe slab	1	20	80.605	1612.100	2.739	4415.880	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	40.250	1006.250	-1.571	-1581.250	0.000	0.000
Toe slab	1	25	24.500	612.500	2.600	1592.500	0.000	0.000
portion between heel & toe	1	25	12.500	312.500	0.900	281.250	0.000	0.000
Vertical Components of active earth pressure	1			1090.302	-4.200	-4579.266	0.000	0.000
				16797.287		-19517.043		3858.211
Total				20030.608		-17085.225		3811.411

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.900	1590.120	-3.300	-5247.394
0.900	144.000	-3.300	-475.200
1.000	254.898	-3.300	-841.162
0.900	34.128	-3.300	-112.622
	2023.145		-6676.379
0.900	178.875	-4.100	-733.388
0.900	65.127	-4.100	-267.021
0.900	25.313	-4.400	-111.375
0.900	12.656	-4.350	-55.055
0.900	260.156	-3.325	-865.020
0.900	197.351	-3.325	-656.191
0.900	0.000	-3.800	0.000
0.900	0.000	-3.800	0.000
0.900	25.200	-4.100	-103.320
0.900	147.656	-4.400	-649.688
	912.334		-3441.056
0.900	584.824	-6.100	-3567.423
0.900	1198.687	-3.300	-3955.669
0.900	9163.761	-6.126	-56136.990
0.900	1450.890	-1.461	-2119.446
0.900	0.000	0.000	0.000
0.900	905.625	-5.771	-5226.750
0.900	551.250	-1.600	-882.000
0.900	281.250	-3.300	-928.125
0.900	981.271	-8.400	-8242.679
	15117.558		-81059.082
	18053.037		-91176.517

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1404.062	816.173
due to Earth pressure	1	2632.221		9745.712

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1404.062	921.629
due to Earth pressure	1.5	3948.331		14618.568
		4071.511		15540.197

Summary of Forces For SBC

P	20030.608	KN
ML	-6523.340	kNm
MT	3811.411	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		20030.608		-17085.225		3811.411
CWLL-Max. Reaction case	1	981.821	0.900	883.639	0.950	932.730
Vertical Components of LL Surcharge	1	262.905	-4.200	-1104.201	0.000	0.000
Total		21275.334		-17305.787		4744.140

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	255.194	1404.062	1909.360
due to Earth pressure	1	2632.221		9745.712
due to Live load surcharge	1	634.709		2972.658

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		342.343	1404.062	2561.410
due to Earth pressure	1.5	3948.331		14618.568
due to Live load surcharge	1.2	761.650		3567.190
		5052.324		20747.168

Summary of Forces For SBC

P	21275.334	KN
ML	-2678.057	kNm
MT	4744.140	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	816.17	0.00	0.00
2632.22	9745.71	0.00	0.00
2741.306	10561.885	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	921.63	0.00	0.00
3948.33	14618.57	0.00	0.00
4071.511	15540.197	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	18053.03687		-91176.52
0.000		-3.300	0.00
0.900		-8.400	-1987.56
	18053.037		-93164.08

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
255.19	1909.36	0.00	0.00
2632.22	9745.71	0.00	0.00
634.709	2972.658	0.000	0.000
3522.12	14627.73	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
342.34	2561.41	0.00	0.00
3948.33	14618.57	0.00	0.00
761.65	3567.19	0.00	0.00
5052.324	20747.168	0.000	0.000

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				2219.617		1997.655		-46.800
Substructure & Foundation -Portion 1				1013.704		434.163		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	25.992	649.804	-1.900	-1234.627	5.938	3858.211
Shaft above HFL	1	25	33.237	830.937	0.900	747.844	0.000	0.000
Shaft below HFL	1	15	20.038	300.563	0.900	270.506	0.000	0.000
Back filling above HFL over heel slab	1	20	365.930	7318.600	-1.900	-13905.340	0.000	0.000
Back filling below HFL over heel slab	1	10	132.423	1324.225	-2.000	-2648.278	0.000	0.000
Front Filling over toe slab	1	10	80.605	806.050	1.839	1482.495	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	40.250	603.750	-1.571	-948.750	0.000	0.000
Toe slab	1	15	24.500	367.500	2.600	955.500	0.000	0.000
Portion between Heel & Toe	1	15	12.500	187.500	0.900	168.750	0.000	0.000
Vertical Components of active earth pressure	1			1090.302	-4.200	-4579.266	0.000	0.000
				13479.230		-19691.166		3858.211
Total				16712.552		-17259.348		3811.411

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	2023.145		-6676.379
	912.334		-3441.056
0.900	584.824	-6.100	-3567.423
0.900	747.844	-3.300	-2467.884
0.900	270.506	-3.300	-892.671
0.900	6586.740	-6.100	-40179.114
0.900	1191.803	-6.200	-7389.020
0.900	725.445	-2.361	-1712.624
0.900	0.000	0.000	0.000
0.900	543.375	-5.771	-3136.050
0.900	330.750	-1.600	-529.200
0.900	168.750	-3.300	-556.875
0.900	981.271	-8.400	-8242.679
	12131.307		-68673.541
	15066.786		-78790.975

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1404.062	816.173
due to Earth pressure	1	2632.221		8989.754

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	816.17	0.00	0.00
2632.22	8989.75	0.00	0.00
2741.31	9805.93	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1404.062	921.629
due to Earth pressure	1.5	3948.331		13484.630
		4071.511		14406.260

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	921.63	0.00	0.00
3948.33	13484.63	0.00	0.00
4071.51	14406.26	0.00	0.00

Summary of Forces For SBC

P	16712.552	kN
ML	-7453.422	kNm
MT	3811.411	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		16712.552		-17259.348		3811.411
CWLL-Min. Reaction case	1	368.179	0.900	331.361	0.950	349.770
Vertical Components of LL Surcharge	1	286.898	-4.200	-1204.974	0.000	0.000
Total		17367.629		-18132.960		4161.181

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	15066.786		-78790.975
0.000	0.000	-3.300	0
0.900	258.209	-8.400	-2168.952
	15324.995		-80959.927

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	285.876	1404.062	2138.923
due to Earth pressure	1	2632.221		8989.754
due to Live load surcharge	1	692.634		3243.952

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
285.88	2138.92	0.00	0.00
2632.22	8989.75	0.00	0.00
692.634	3243.952	0.000	0.000
3610.73	14372.63	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		388.366	1404.062	2905.755
due to Earth pressure	1.5	3948.331		13484.630
due to live load surcharge	1.2	831.161		3892.743
		5167.858		20283.128

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
388.37	2905.76	0.00	0.00
3948.33	13484.63	0.00	0.00
831.161	3892.743	0.000	0.000
5167.86	20283.13	0.00	0.00

Summary of Forces For SBC

P	17367.629	kN
ML	-3760.331	kNm
MT	4161.181	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1

ah= 0.162 In Longitudinal direction
ah= 0.180 In Transverse direction
av= 0.120 In Vertical direction

Weight of shaft below Ground level

= 625.938 KN

Weight of back fill below Ground level

= 2303.450 KN

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1766.799		95.168	63.445	0.900	1590.120	57.101	1405.275		0.000	0.000	827.516
SIDL except Wearing Course	1			160.000		8.618	5.746	0.900	144.000	5.171	1406.396		0.000	0.000	84.597
Wearing Course	1			254.898		13.730	9.153	0.900	229.408	8.238	1405.947		0.000	0.000	128.608
Bearing Pedestal	1	25	1.517	37.920		2.043	1.362	0.900	34.128	1.226			-1.234	-46.800	
				2219.617		119.559	79.706		1997.655	71.735				-46.800	1040.721
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	7.950	198.750	32.198	10.706	7.137	0.100	19.875	0.714	1404.887	267.465	0.000	0.000	88.931
Dirt Wall-Tapered portion	1	25	2.895	72.363	11.723	3.898	2.599	0.100	7.236	0.260	1403.441	80.431	0.000	0.000	26.743
Bracket - Uniform portion	1	25	1.125	28.125				-0.200	-5.625						
Bracket - Tapered portion	1	25	0.563	14.063				-0.150	-2.109						
Cap - (uniform portion)	1	25	11.563	289.063	46.828	15.570	10.380	0.875	252.930	9.083	1402.992	300.262	0.000	0.000	99.836
Cap - (corbel portion)	1	25	8.771	219.278	35.523	11.811	7.874	0.875	191.869	6.890	1402.492	210.013	0.000	0.000	69.829
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.400	0.000	0.000	1406.321	0.000	5.938	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.400	0.000	0.000	1406.321	0.000	5.938	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.100	2.800				0.000	0.000	
Approach Slab	1	25	6.563	164.063				-0.200	-32.813				0.000	0.000	

				1013.704	126.272	41.985	27.990		434.163	16.946		858.170		0.000	285.339
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	25.992	649.804	105.268	35.001	23.334	-1.900	-1234.627	-44.335	1401.520	520.032	5.938	3858.211	172.909
Abutment Shaft	1	25	53.275	1331.875	114.362	38.025	25.350	0.900	1198.687	22.815	1400.913	495.473	0.000	0.000	164.743
Back filling over heel slab	1	20	509.098	10181.956	0.000	0.000	0.000	-1.926	-19610.217	0.000	1401.520	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	80.605	1612.100				2.739	4415.880				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	40.250	1006.250				-1.571	-1581.250				0.000	0.000	
Toe slab	1	25	24.500	612.500				2.600	1592.500				0.000	0.000	
portion between heel & toe	1	25	12.500	312.500				0.900	281.250				0.000	0.000	
Vertical component of active earth pressure	1			1026.096				-4.200	-4309.604				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			346.561				-4.200	-1455.557				0.000	0.000	
				17079.643	219.630	73.026	48.684		-20702.938	-21.520		1015.504		3858.211	337.652
Total =				20312.964	345.902	234.570	156.380		-18271.120	67.161		1873.675		3811.411	1663.712
								-156.380		-67.161					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Pxel2	MLs due to Fv
Superstructure								
Dead Load	0.9			1590.120	57.101	-3.300	-5247.394	-188.432
SIDL except Wearing Course	0.9			144.000	5.171	-3.300	-475.200	-17.064
Wearing Course	1.00			254.898	9.153	-3.300	-841.162	-30.206
Bearing Pedestal	0.9	25	1.5168	34.128		-3.300	-112.622	
				2023.145	71.425		-6676.379	-235.702
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	7.950	178.875	6.423	-4.100	-733.388	-26.336
Dirt Wall-Tapered portion	0.9	25	2.895	65.127	2.339	-4.100	-267.021	-9.589
Bracket - Uniform portion	0.9	25	1.125	25.313				
Bracket - Tapered portion	0.9	25	0.563	12.656				
Cap - (uniform portion)	0.9	25	11.563	260.156	9.342	-3.325	-865.020	-31.063
Cap - (corbel portion)	0.9	25	8.771	197.351	7.087	-3.325	-656.191	-23.564
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-3.800	0.000	0.000
Cantilever Return Wall-Traingle portion	0.9	25	0.000	0.000	0.000	-3.800	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-4.100	-103.320	
Approach Slab	0.9	25	6.563	147.656		-4.400	-649.688	
				912.334	25.191		-3274.626	-90.550
Substructure & Foundation -Portion 2								
Abutment Shaft	0.9	25	53.275	1198.687	20.567	-3.300	-3955.669	-67.872
Solid Return wall	0.9	25	25.992	584.824	21.001	-6.100	-3567.423	-128.105
Back filling over heel slab	0.9	20	509.098	9163.761	0.000	-6.126	-56137.0	0.000
Front Filling over toe	0.9	20	80.605	1450.890		-1.461	-2119.446	
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000	
Heel slab	0.9	25	40.250	905.625		-5.771	-5226.750	
Toe slab	0.9	25	24.500	551.250		-1.600	-882.000	
portion between heel & toe	0.9	25	12.500	281.250		-3.300	-928.125	
Vertical component of active earth pressure	0.9			981.271		-8.400	-8242.679	
Vertical component of dynamic increment of earth pressure	0.9			311.905		-8.400	-2620.003	
				15429.463	41.568		-83679.09	-195.977
Total =				18364.942	138.184		-93630.09	-522.230
						-138.184		522.230

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	32.198	1404.887	267.465
1.0	11.723	1403.441	80.431
1.0	46.828	1402.992	300.262
1.0	35.523	1402.492	210.013
1.0	0.000	1406.321	0.000
1.0	0.000	1406.321	0.000
	126.272		858.170
1.0	103.095	1400.913	446.659
1.0	105.268	1401.520	520.032
1.0	0.000	1401.520	0.000
	208.363		966.691
	334.635		1824.861

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	119.559	1404.062	5288.800	1040.721
due to Substructure	1	345.902	115.011		1873.675	622.991
due to Earth pressure	1	3313.888			14082.605	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1404.062	7933.20
due to Substructure	1.5	501.952		2737.29
due to Active Earth pressure	1	2477.215		9745.71
due to dynamic Earth pressure	1.5	1255.009		6505.34
		5294.481		26921.543

Summary of Forces For SBC

	Downward	Upward	
P	20469.344	20156.584	kN
ML	3041.121	2906.799	kNm
MT	5475.122	5475.122	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	5288.80	0.00	0.00	0.00	0.00	119.56	1040.72
345.90	1873.67	0.00	0.00	0.00	0.00	115.01	622.99
3313.89	14082.61						
4366.66	21245.08	0.00	0.00	0.00	0.00	234.57	1663.71

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	7933.20	0.00	0.00
501.95	2737.29	0.00	0.00
2477.215	9745.712	0.000	0.000
1255.009	6505.340	0.000	0.000
5294.48	26921.54	0.00	0.00

Summary of Restoring Forces

Vertical Load	18226.758	kN
Moment	-94152.319	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				2219.617			79.706		1997.655	71.735				-46.800	1040.721
Forces from Substructure				18093.347	345.902	115.011	76.674		-20268.775	-4.574		1873.675		3858.211	622.991
CWLL-Max. Reaction case	0.20			196.36		10.577	7.051	0.900	176.728	6.346	1407.147		0.950	186.546	111.768
Vertical component of LL Surcharge	0.20			52.58				-4.200	-220.840				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			19.88				-4.200	-83.503				0.000	0.000	
Total =				20581.791	345.902	245.147	163.431		-18398.735	73.507		1873.675		3997.957	1775.479
							-163.431			-73.507					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Forces from Superstructure				2023.145	71.425	0.000	-6676.38	-235.702
Forces from Substructure				16341.797	66.759	0.000	-86953.71	-286.527
CWLL-Max. Reaction case	0.00			0.00	0.000	-3.300	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-8.400	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-8.400	0.00	
Total =				18364.942	138.184		-93630.09	-522.230
					-138.184			522.230

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	757.950	130.136	1404.062	5670.981	1152.488
due to Substructure	1	345.902	115.011		1873.675	622.991
due to Earth pressure	1	3313.888			14082.605	
due to Live load surcharge	0.20	174.940			911.604	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1404.062	7933.200
due to Substructure	1.5	501.952		2737.292
due to Active Earth pressure	1	2477.215		9745.712
due to dynamic Earth pressure	1.5	1255.009		6505.33998
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		5294.481		26921.543

Summary of Forces For SBC

	Downward	Upward	
P	20745.222	20418.359	kN
ML	4213.637	4066.622	kNm
MT	5773.436	5773.436	kNm

Summary of Restoring Forces

Vertical Load	18226.758	kN
Moment	-94152.319	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
757.95	5670.98	0.00	0.00	0.00	0.00	130.14	1152.49
345.90	1873.67	0.00	0.00	0.00	0.00	115.01	622.99
3313.89	14082.61						
174.94	911.60						
4592.68	22538.86	0.00	0.00	0.00	0.00	245.15	1775.48

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	7933.20	0.00	0.00
501.95	2737.29	0.00	0.00
2477.215	9745.712		
1255.009	6505.340		
0.000	0.000		
0.000	0.000		
5294.48	26921.54	0.00	0.00

Case 7 : DL+SIDL-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2219.617		119.559	79.706		1997.655	71.735				-46.800	1040.721
Substructure & Foundation -Portion 1				1013.704	126.272	41.985	27.990		434.163	16.946		858.170		0.000	285.339
Substructure & Foundation -Portion 2															
Solid Return wall	1		25.99	649.80	105.268	35.001	23.334	-1.90	-1234.627	-44.335	1401.52	520.032	5.94	3858.211	172.909
Shaft above HFL	1	25	33.237	830.937	134.612	44.758	29.839	0.900	747.844	26.855	1400.913	583.206	0.000	0.000	193.914
Shaft below HFL	1	15	20.038	300.563	-20.250	-6.733	-4.489	0.900	270.506	-4.040	1399.583	-60.811	0.000	0.000	-20.219
Back filling above HFL over heel slab	1	20	365.930	7318.600	0.000	0.000	0.000	-1.900	-13905.340	0.000	1402.765	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	132.423	1324.225	0.000	0.000	0.000	-2.000	-2648.278	0.000	1398.332	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	80.605	806.050				1.839	1482.495				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	40.250	603.750				-1.571	-948.750				0.000	0.000	
Toe slab	1	15	24.500	367.500				2.600	955.500				0.000	0.000	
portion between heel & toe	1	15	12.500	187.500				0.900	168.750				0.000	0.000	
Vertical component of active earth pressure	1			1062.197				-4.200	-4461.228				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			344.599				-4.200	-1447.317				0.000	0.000	
				13795.725	219.630	73.026	48.684		-21020.445	-21.520		1042.427		3858.211	346.604
Total =				17029.047	345.902	234.570	156.380		-18588.627	67.161		1900.597		3811.411	1672.663

-156.380

-67.161

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLS due to Fv
Superstructure				2023.145	71.425		-6676.379	-235.702
Substructure & Foundation -Portion 1				912.334	25.191		-3274.626	-90.550
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	25.992156	584.824	21.001	-6.10	-3567.423	-128.105
Shaft above HFL	0.9	25	33.237	747.844	26.855	-3.300	-2467.884	-88.621
Shaft below HFL	0.9	15	20.038	270.506	-6.288	-3.300	-892.671	20.749
Back filling above HFL over heel slab	0.9	20	365.930	6586.740	0.000	-6.100	-40179.114	0.000
Back filling below HFL over heel slab	0.9	10	132.423	1191.803	0.000	-6.200	-7389.020	0.000
Front Filling ovetoe	0.9	10	80.605	725.445		-2.361	-1712.624	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	40.250	543.375		-5.771	-3136.050	
Toe slab	0.9	15	24.500	330.750		-1.600	-529.200	
portion between heel & toe	0.9	15	12.500	168.750		-3.300	-556.875	
Vertical component of active earth pressure	0.9			955.977		-8.400	-8030.211	
Vertical component of dynamic increment of earth pressure	0.9			310.139		-8.400	-2605.170	
				12416.153	41.568		-71066.24	-195.977
Total =				15351.632	138.184		-81017.25	-522.230

-138.184

522.230

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLS due to FL
	126.272		858.170
1.0	105.268	1401.520	520.032
1.0	134.612	1400.913	583.206
1.0	-31.517	1399.583	-94.645
1.0	0.000	1402.765	0.000
1.0	0.000	1398.332	0.000
	208.363		1008.592
	334.635		1866.763

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	119.559	1404.062	5288.800	1040.721
due to Substructure	1	345.902	115.011		1900.597	631.943
due to Earth pressure	1	3396.307			13315.735	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1404.062	7933.200
due to Substructure	1.5	501.952		2800.144
due to Active Earth pressure	1	2564.371		8989.754
due to dynamic Earth pressure	1.5	1247.904		6488.973
		5374.532		26212.071

Summary of Forces For SBC

	Downward	Upward	
P	17185.426	16872.667	kN
ML	1983.667	1849.345	kNm
MT	5484.074	5484.074	kNm

Forces along Long. Axis

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	5288.80	0.00	0.00	0.00	0.00	119.56	1040.72
345.90	1900.60	0.00	0.00	0.00	0.00	115.01	631.94
3396.31	13315.74						
4449.08	20505.13	0.00	0.00	0.00	0.00	234.57	1672.66

Forces along Long. Axis

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	7933.20	0.00	0.00
501.95	2800.14	0.00	0.00
2564.371	8989.754		
1247.904	6488.973		
5374.53	26212.07	0.00	0.00

Summary of Restoring Forces

Vertical Load	15213.448	kN
Moment	-81539.477	kNm

MT	5595.941	5595.941	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x α_h x P (kN)	FT = α_h x P (kN)	Fv = 0.3 x α_v x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2219.617		398.529	79.706		1997.655	71.735				-46.800	3469.068
Substructure & Foundation -Portion 1				1013.704	37.881	139.950	27.990		434.163	16.946		257.451		0.000	951.130
Substructure & Foundation -Portion 2				17079.643	65.889	243.421	48.684		-20702.938	-21.520		304.651		3858.211	1125.507
Total =				20312.964	103.771	781.899	156.380		-18271.120	67.161		562.102		3811.411	5545.706
										-156.380		-67.161			

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x α_v x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Superstructure				2023.145	71.425		-6676.38	-235.702
Substructure & Foundation -Portion 1				912.334	25.191		-3274.63	-90.550
Substructure & Foundation -Portion 2				15429.463	41.568		-83679.09	-195.977
Total =				18364.942	138.184		-93630.09	-522.230
					-138.184		522.230	

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x α_h x P (kN)	C.g. of Force (m)	MLs due to FL
	126.272		858.170
	208.363		966.691
	334.635		1824.861

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.115	398.529	1404.062	1609.493	3469.068
due to Substructure	1	103.771	383.371		562.102	2076.637
due to Earth pressure	1	3313.888			14082.605	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	1609.49	0.00	0.00	0.00	0.00	398.53	3469.07
103.77	562.10	0.00	0.00	0.00	0.00	383.37	2076.64
3313.89	14082.61						
3632.77	16254.20	0.00	0.00	0.00	0.00	781.90	5545.71

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1404.062	2379.960
due to Substructure	1.5	501.952		2737.29178
due to Active Earth pressure	1	2477.215		9745.71172
due to dynamic Earth pressure	1.5	376.503		1951.60199
		3673.762		16814.565

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2379.96	0.00	0.00
501.95	2737.29	0.00	0.00
2477.215	9745.712		
376.503	1951.602		
3673.76	16814.57	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	20469.344	20156.584	kN
ML	-1949.758	-2084.081	kNm
MT	9357.116	9357.116	kNm

Summary of Restoring Forces

Vertical Load	18226.758	kN
Moment	-94152.319	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Total =	20581.791	163.431	-18398.73	73.507	3997.957
		-163.431		-73.507	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	433.785	1404.062	1968.821	3841.628
due to Substructure	1	103.771	383.371		562.102	2076.637
due to Earth pressure	1	3313.888			14082.605	
due to Live load surcharge	0.2	174.940			911.604	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1404.062	2379.960
due to Substructure	1.5	150.59		821.188
due to Active Earth pressure	1	2477.22		9745.712
due to dynamic Earth pressure	1.5	376.50		1951.602
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		3322.40		14898.461

Summary of Forces For SBC

	Downward	Upward	
P	20745.222	20418.359	kN
ML	-800.095	-947.110	kNm
MT	9916.222	9916.222	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Superstructure	2219.617	79.706	1997.655	71.735	-46.800
Substructure & Foundation -Portion 1	1013.704	27.990	434.163	16.946	0.000
Substructure & Foundation -Portion 2	13795.725	48.684	-21020.445	-21.520	3858.211
Total =	17029.047	156.380	-18588.627	67.161	3811.411
		-156.380		-67.161	

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L2}$	MLs due to F_v
Total =	18364.942	138.184	-93630.09	-522.230
		-138.184		522.230

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.14	1968.82	0.00	0.00	0.00	0.00	433.79	3841.63
103.77	562.10	0.00	0.00	0.00	0.00	383.37	2076.64
3313.89	14082.61						
174.94	911.60						
3855.74	17525.13	0.00	0.00	0.00	0.00	817.16	5918.26

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2379.96	0.00	0.00
150.59	821.19	0.00	0.00
2477.215	9745.712		
376.503	1951.602		
0.000	0.000		
0.000	0.000		
3322.40	14898.46	0.00	0.00

Summary of Restoring Forces

Vertical Load	18226.758	kN
Moment	-94152.319	kNm

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	Long. Ecc. (e_{L2}) @ Toe (m)	ML = $P \times e_{L2}$	MLs due to F_v
Superstructure	2023.145	71.425		-6676.379	-235.702
Substructure & Foundation	912.334	25.191		-3274.626	-90.550
Substructure & Foundation	12416.153	41.568		-71066.242	-195.977
Total =	15351.632	138.184		-81017.247	-522.230
		-138.184			522.230

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.12	398.53	1404.06	1609.49	3469.07
due to Substructure	1	103.77	383.37		570.18	2106.48
due to Earth pressure	1	3396.31			13315.74	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1404.062	2379.96
due to Substructure	1.5	150.586		840.04
due to Active Earth pressure	1	2564.371		8989.75
due to dynamic Earth pressure	1.5	374.371		1946.69
		3407.419		14156.45

Summary of Forces For SBC

	Downward	Upward	
P	17185.426	16872.667	kN
ML	-3026.058	-3160.381	kNm
MT	9386.955	9386.955	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	ML = $P \times eL1$	MLs due to F_v	MT = $P \times eT$
Total =	17184.330	159.024	-18865.28	69.541	3881.365
		-159.024		-69.541	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	411.74976	1404.062	1968.821	3608.7767
due to Substructure	1	103.771	383.37086		570.179	2106.47596
due to Earth pressure	1	3396.307			13315.735	
due to Live load surcharge	0.2	197.115			990.847	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1404.062	2379.96
due to Substructure	1.5	150.59		840.04
due to Active Earth pressure	1	2564.37		8989.75
due to dynamic Earth pressure	1.5	374.37		1946.69
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		3407.42		14156.45

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	1609.49	0.00	0.00	0.00	0.00	398.53	3469.07
103.77	570.18	0.00	0.00	0.00	0.00	383.37	2106.48
3396.31	13315.74						
3715.19	15495.41	0.00	0.00	0.00	0.00	781.90	5575.54

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2379.96	0.00	0.00
150.59	840.04	0.00	0.00
2564.371	8989.754		
374.371	1946.692		
3407.42	14156.45	0.00	0.00

Summary of Restoring Forces

Vertical Load	15213.448	kN
Moment	-81539.477	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = $P \times eL2$	MLs due to F_v
Total =	15351.632	138.184		-81020.55	-522.230
		-138.184			522.230

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.14	1968.82	0.00	0.00	0.00	0.00	411.75	3608.78
103.77	570.18	0.00	0.00	0.00	0.00	383.37	2106.48
3396.31	13315.74						
197.12	990.85						
3960.33	16845.58	0.00	0.00	0.00	0.00	795.12	5715.25

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2379.96	0.00	0.00
150.59	840.04	0.00	0.00
2564.371	8989.754		
374.371	1946.692		
0.000	0.000		
0.000	0.000		
3407.42	14156.45	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	17343.354	17025.306	kN
ML	-1950.153	-2089.235	kNm
MT	9596.617	9596.617	kNm

Summary of Restoring Forces

Vertical Load	15213.448	kN
Moment	-81542.777	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1407.521 -	1396.580)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1407.521 -	1396.580)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	20030.608	-6523.340	3811.411	252.568	163.815	217.721	128.968	252.568	128.968	4071.511	12637.126	3.10	15540.197	91176.52	5.87
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	21275.334	-2678.057	4744.140	242.528	206.092	199.153	162.717	242.528	162.717	5052.324	12637.126	2.50	20747.168	93164.08	4.49
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	16712.552	-7453.422	3811.411	227.294	125.887	192.447	91.040	227.294	91.040	4071.511	10546.750	2.59	14406.260	78790.97	5.47
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	17367.629	-3760.331	4161.181	210.009	158.848	171.964	120.803	210.009	120.803	5167.858	10727.496	2.08	20283.128	80959.93	3.99
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	20469.344	3041.121	5475.122	199.287	240.663	149.229	190.605	240.663	149.229	5294.481	12758.731	2.41	26921.543	94152.32	3.50
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	20745.222	4213.637	5773.436	195.302	252.631	142.516	199.845	252.631	142.516	5294.481	12758.731	2.41	26921.543	94152.32	3.50
								SAFE	SAFE			SAFE			SAFE

Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	17185.426	1983.667	5484.074	175.246	202.235	125.106	152.095	202.235	125.106	5374.532	10649.413	1.98	26212.071	81539.48	3.11
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	17343.354	3082.425	5595.941	169.787	211.725	118.624	160.562	211.725	118.624	5374.532	10649.413	1.98	26212.071	81542.78	3.11
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	20469.344	-1949.758	9357.116	250.985	224.458	165.434	138.907	250.985	138.907	3673.762	12758.731	3.47	16814.565	94152.32	5.60
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	20745.222	-800.095	9916.222	248.348	237.462	157.685	146.799	248.348	146.799	3322.395	12758.731	3.84	14898.461	94152.32	6.32
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	17185.426	-3026.058	9386.955	227.168	185.997	141.344	100.174	227.168	100.174	3407.419	10649.413	3.13	14156.449	81539.48	5.76
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	17343.354	-1950.153	9596.617	222.311	195.779	134.571	108.038	222.311	108.038	3407.419	10649.413	3.13	14156.449	81542.78	5.76
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

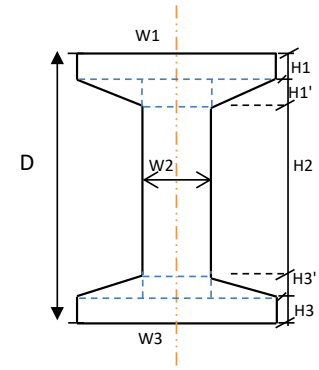
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	325
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	675	100	33750
1' (Rect.)	325	100	32500
2 (Rect.)	325	950	308750
3' (Tri.)	300	150	22500
3' (Rect.)	325	150	48750
3 (Rect.)	625	250	156250
			752500



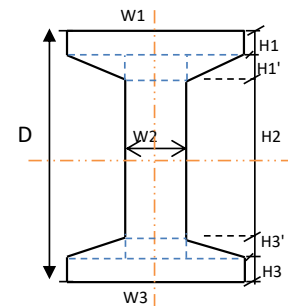
Total Area of Section A = 0.753 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	78
H2	=	1122
H3'	=	0
H3	=	250
W1	=	1000
W2	=	475
W3	=	475

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	525	78	20417
1' (Rect.)	475	78	36944
2 (Rect.)	475	1122	533056
3' (Tri.)	0	0	0
3' (Rect.)	475	0	0
3 (Rect.)	475	250	118750
			859167



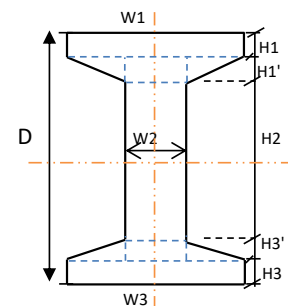
Total Area of Section A = 0.859 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

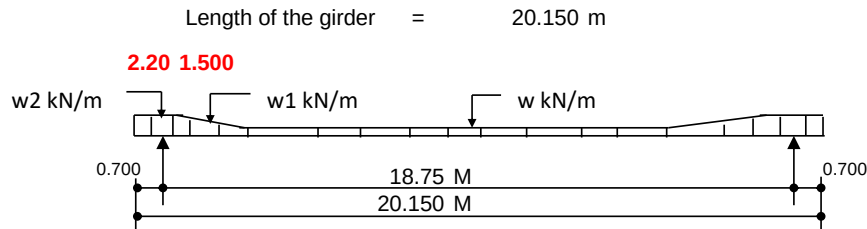
D	=	1600
H1	=	150
H1'	=	56
H2	=	1394
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	56	10417
1' (Rect.)	625	56	34722
2 (Rect.)	625	1394	871528
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066667



Total Area of Section A = 1.067 m²

1.0 Self weight of Inner Girder & Outer Girder



UDL , w	=	18.81 kN/m
Avg. , w ₁	=	21.48 kN/m
UDL , w ₂	=	26.67 kN/m
Supp. Reaction , R	=	210.82 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	
UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder
Supp. Reaction , R	=	180.091 kN	For Outer Girder
	=	166.238 kN	For Inner Girder
weight of deck slab has been Increase by 5 %			
Supp. Reaction , R	=	189.095 kN	For Outer Girder
	=	174.549 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

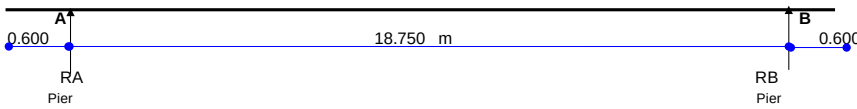
4.0 Weight due to wearing coat

Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	
UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder
Supp. Reaction , R	=	60.954 kN	For Outer Girder
	=	66.495 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)

Maximum Live Load Reaction Case**Due to Class A 1-Lane , 2-Lanes, 3-Lanes & 4-Lanes**Considering **114.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**

Nos. of Lane for design purpose = **3 LANE**

Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

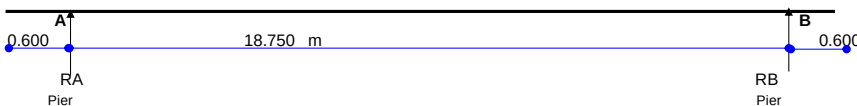
1st Train	Total Load	500.0kN				
					363.637	136.363
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	363.637	136.363

Due to Class 70R WheeledConsidering **170.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

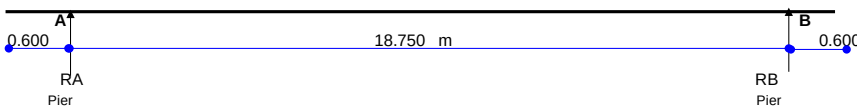
1st Train	Total Load	1000.0kN				
					758.731	241.269
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	758.731	241.269

Due to Class A 1-Lane/2-Lane (along with Class 70R)Considering **68.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

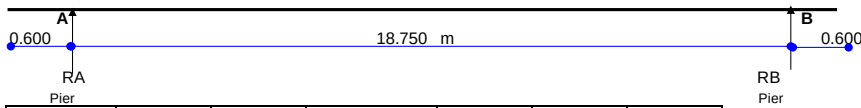
1st Train	Total Load	554.0kN				
					284.864	269.136
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
27.000	1.100	18.800	18.200	491.400		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	284.864	269.136

Due to Class SV LoadingConsidering **180.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = 18.75 m
 Projection beyond c.L. of bearing at Abut. end = 0.600 m
 Projection beyond c.L. of bearing at far end = 0.600 m
 Total length including Overhangs = 19.95 m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	3420.0kN				
180.000		0.000	-0.600	-108.000	2003.040	1416.960
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		
180.000	1.500	9.000	8.400	1512.000		
180.000	1.500	10.500	9.900	1782.000		
180.000	1.500	12.000	11.400	2052.000		
180.000	1.500	13.500	12.900	2322.000		
180.000	1.500	15.000	14.400	2592.000		
180.000	1.500	16.500	15.900	2862.000		
180.000	1.500	18.000	17.400	3132.000		
180.000	1.500	19.500	18.900	3402.000		
0.000	1.500	0.000	-0.600	0.000		
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	2003.040	1416.960

Deck Width = 12.500 m

transverse eccentricity of superstructure about c.g of abutment and foundation = 0 m

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	2003.040 KN	Corr. MT	= 600.912 kNm
Minimum Reaction (RB)	=	1416.960 KN	Corr. MT	= 425.088 kNm

1-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-0.9	0	0	=	4.45 m
Maximum Reaction (RA)	=	363.637 KN		Corr. MT	=	1618.186 kNm			
Minimum Reaction (RB)	=	136.363 KN		Corr. MT	=	606.814 kNm			

2-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-0.85	0	=	2.70 m
Maximum Reaction (RA)	=	727.275	KN	Corr. MT	=	1963.642	kNm		
Minimum Reaction (RB)	=	272.725	KN	Corr. MT	=	736.358	kNm		

3-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	0.95 m
Maximum Reaction (RA)	=	1090.912 KN	Corr. MT	=	1036.366 kNm				
Minimum Reaction (RB)	=	409.088 KN	Corr. MT	=	388.634 kNm				

4-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-0.8 m
Maximum Reaction (RA)	=	1454.549 KN		Corr. MT	=	-1163.639 kNm				
Minimum Reaction (RB)	=	545.451 KN		Corr. MT	=	-436.361 kNm				

1- 70RW

Transverse Ecc.	=	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m
Maximum Reaction (RA)	=	758.731 KN		Corr. MT	=	2394			
Minimum Reaction (RB)	=	241.269 KN		Corr. MT	=	761			

1-CLASS A + 1-70RW

CLASS A - 70RW		SEAL									
Transverse Ecc.	for Class A	=	6.25	-0.5	-0.4	-0.9	=	4.45 m			
	for 70RW	=	6.25	-0.5	-0.4	-1.8	-0.25	-1.8	-1.395	=	0.105
Maximum Reaction (RA)	=	1043.595 KN	Corr. MT	=	284.86 x	4.45 +	758.73 x	0.105	=	1347	
Minimum Reaction (RB)	=	510.405 KN	Corr. MT	=	269.136	4.45 +	241.269	0.105	=	1223	

2-CLASS A + 1-70RW

<u>CORNER AT 70RW</u>												
Transverse Ecc.	=	for 1st Class A for 2nd Class A for 70RW	= = =	4.45 m 4.45 2.7	-0.90 -0.85	-0.85	0 -1.8	= -0.25	2.7 m -1.2	-1.395	=	-2.795
Maximum Reaction (RA)	=	1328.459 KN		Corr. MT	=	284.86 x	4.45 + +	284.86 x 758.73 x	2.700 -2.795		=	-84
Minimum Reaction (RB)	=	779.541 KN		Corr. MT	=	269.136	4.45 + +	269.14 x 241.27 x	2.700 -2.795		=	1250

1- 70RW + 1-CLASS A

Transverse Ecc.	=	6.25	-0.5	-1.2	-1.395	=	3.155	m
	=	6.25	-0.5	-7.25	-0.25	-0.9 =	-2.650	m
Maximum Reaction (RA)	=	1043.595 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650
Minimum Reaction (RB)	=	510.405 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650

1- 70RW + 2-CLASS A

70RW + 2 CLASS A		4 LANE									
Transverse Ecc.		=	3.155 m								
for 70RW		=	-2.650 m								
for 1st Class		=	-2.650	-0.9	-0.85	=	-4.400 m				
for 2nd Class		=									
Maximum Reaction (RA)	=	1328.459 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650	=	386	
					+		284.86 x	-4.400			
Minimum Reaction (RB)	=	779.541 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650	=	-1136	
					+		269.14 x	-4.400			

2- 70RW

[illegible]

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	363.637	1618.186
2-CLASS A	727.275	1963.642
3-CLASS A	1090.912	1036.366
4-CLASS A	0.000	0.000
1- 70RW	758.731	2393.795
1-CLASS A + 1-70RW	1043.595	1347.312
1- 70RW + 1-CLASS A	1043.595	1638.906
2-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 2-CLASS A	0.000	0.000
2- 70RW	0.000	0.000

Reaction RB, , (kN)	Corr. MT (kNm)
136.363	606.814
272.725	736.358
409.088	388.634
0.000	0.000
241.269	761.205
510.405	1222.988
510.405	47.994
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 1090.912 KN
 Reaction RB = 409.088 KN

& Corresponding moment = 1036.366 kNm
 & Corresponding moment = 388.634 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 758.731 KN
 Reaction RB = 241.269 KN

& Max moment = 2393.795 kNm
 & Max moment = 761.205 kNm

Stability Check Summary

Description	P _{max} (kN/m ²)	P _{min} (kN/m ²)	Sliding	Overturnin g	Shear (Heel slab)	Shear (Toe slab)
Normal Dry Case						
Case 1 : DL+SIDL-Normal Dry Case	252.57	128.97	3.10	5.87	0.224	0.736
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	242.53	162.72	2.50	4.49		
Permissible	400	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Normal HFLCase						
Case 3 : DL+SIDL-Normal HFL Case	227.29	91.04	2.59	5.47		
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	210.01	120.80	2.08	3.99		
Permissible	400	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic Dry Case						
Case 5 : DL+SIDL-Long. Seismic Dry Case	240.66	149.23	2.41	3.50		
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	252.63	142.52	2.41	3.50		
Permissible	500	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic HFL Case						
Case 7 : DL+SIDL-Long. Seismic HFL Case	202.235	125.106	1.98	3.11		
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	211.725	118.624	1.98	3.11		
Permissible	500	0	1.25	1.50		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic Dry Case						
Case 9 : DL+SIDL-Trans. Seismic Dry Case	250.985	138.907	3.47	5.60		
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	248.348	146.799	3.84	6.32		
Permissible	500	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic HFL Case						
Case 11 : DL+SIDL-Trans. Seismic HFL Case	227.168	100.174	3.13	5.76		
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	222.311	108.038	3.13	5.76		
Permissible	500	0	1.25	1.50	4.192	4.192
Remarks	O.K	O.K	O.K	O.K	O.K	O.K

Reinforcement summary

Type of reinforcement	Area of steel required	Area of steel provided						
Heel Slab								
Main steel at bottom face	1319	20	mm bar @	200	mm c/c (i.e.)	2576	mm ²	(O.K)
		16	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	515	12	mm bar @	200	mm c/c (i.e.)	565	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 20 Legged bar @	200	mm c/c (i.e.)			
Toe Slab								
Main steel at bottom face	3687	25	mm bar @	200	mm c/c (i.e.)	4025	mm ²	(O.K)
		20	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	805	16	mm bar @	150	mm c/c (i.e.)	1340	mm ²	(O.K)
Shear Reinforcement	Provide Shear Reinf.	10	mm 22 Legged bar @	150	mm c/c (i.e.)			

Abutment Shaft								
Vertical steel at earth face	5184	32	mm bar @	200	mm c/c (i.e.)	6476	mm ²	(O.K)
		25	mm bar @	200	mm c/c (i.e.)			
Vertical steel at non-earth face	1200	20	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
Distribution steel	2012	16	mm 2 Legged bar @	150	mm c/c (i.e.)	2681	mm ²	(O.K)
Abutment Cap								
Top Longitudinal Reinf.	1049	12	mm dia bar	12	Nos.	1357	mm ²	(O.K)
Bottom Longitudinal Reinf.	1049	12	mm dia bar	12	Nos.	1357	mm ²	(O.K)
Side Face Longitudinal Reinf.		12	mm dia bar	200	mm c/c (i.e.)			
Transverse Reinf.	2.08E+06	12	mm 2 Legged bar @	150	mm c/c (i.e.)	2.62E+06	mm ³ /m	(O.K)
Corbel Reinforcement								
Primary tensile reinf.	2100	25	mm bar @	200	mm c/c (i.e.)	2454	mm ²	(O.K)
Horizontal Stirrups	200.95	12	mm 2 Legged bar @	in 2 Vertical Layers		452	mm ² /m	(O.K)
Vertical Stirrups	525.00	10	mm 2 Legged bar @	200	mm c/c (i.e.)	785	mm ³ /m	(O.K)

DESIGN OF COUNTER FORT
TYPE ABUTMENT WITH OPEN
FOUNDATION

FOR MINOR BRIDGE AT CH:-
14+400

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 50 m
Design speed of vehicle = 50 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 18.750m	18.750m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side)	= 0.600m	0.600m
Slab projection Beyond C/L of Bearing (Span Side)	= 0.600m	0.600m
Span -c/c of E.J.	= 20.000m	20.000m
Type of Superstructure	= Precast PSC Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 11.000m	
Projection beyond RCC Railing	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 0.000m	
Railing/kerb on footpath edge	= 0.250m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	
Increase in length due to curvature effect	= 0 in skew	

Bearings

No of Bearings = 4
Type of Bearing = Spherical Bearing
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 330 kN/m² As per Geo tech Report
SBC of soil-Normal Case = 407 kN/m²
SBC of soil-Seismic Case = 509 kN/m²

Coeff. of friction between concrete and soil = 0.5 for cohesive/cohesionless soil

Permissible FOS against Sliding = 1.5 Normal Case
= 1.25 Seismic Case

Permissible FOS against Overturning = 2 Normal Case
= 1.5 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 2.120m	
Height of Trapering portion	= 0.790m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 1.850m	1.850m
Width of Abutmentcap at bottom of Tapering Portion	= 1.850m	1.850m
Projection of Abutment Cap (Span Side)	= 0.500m	0.500m
Projection of Abutment Cap Back Side	= 0.550m	0.550m
Abutment cap thickness (Uniform portion)	= 0.500m	

Design Calculation	RODIC		INPUT
Abutment cap thickness (Tapering Portion)	=	0.000m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m
<u>Abutment- Counter Fort Wall Type</u>			
Thickness of Abutment	=	0.800m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	0.800m	0.800m
Thickness of Abutment shaft at HFL	=	0.800m	0.800m
Thickness of Abutment shaft at Bottom	=	0.800m	0.800m
Thickness of counterfort	=	0.600m	0.600m
<u>Solid Return Wall & Counter Fort</u>			
Length of Return wall	=	9.400m	
Thickness of Return wall at Top	=	0.500m	
Thickness of Return wall at Bottom	=	1.200m	
Length of counterfort @ bottom	=	9.400m	
Length of counterfort @ top	=	0.550m	
Thickness of counterfort	=	0.600m	
No. of Counter Fort	=	2	
<u>Cantilever Return Wall</u>			
Height of Return Wall-Free edge	=	0.600m	
Height of wall at abutment	=	0.000m	
Length of Return wall	=	0.000m	
Thickness of Return wall at Top	=	0.500m	
Thickness of Return wall at Bottom	=	1.200m	
<u>Foundation</u>			
<u>Along Traffic Direction:</u>			
Total Width of Footing	=	15.200m	
abutment pedestal width	=	0.800m	
abutment pedestal Height	=	2.000m	
Width of Toe Slab	=	5.000m	
Width of Heel Slab	=	9.400m	
Thickness of Toe slab at tip	=	0.750m	
Thickness of Toe slab near shaft	=	2.000m	
Thickness of heel slab at tip	=	0.750m	
Thickness of heel slab near shaft	=	0.750m	
Width of backfill on heel slab	=	9.400m	
Thickness of heel slab at back fill edge	=	0.750m	
Height of back fill at bottom edge of heel slab	=	16.523	
Height of back fill at back fill edge of heel slab	=	16.523	
<u>Across Traffic Direction:</u>			
Width of foundation -Uniform portion	=	12.500m (skew dimension)	
Width of foundation -Tapering portion	=	12.500m (skew dimension)	
<u>Levels</u>			
Deck Level at Median Edge=	1473.357m	Cross Slope (Bi-directional)	= 7.000%
Deck level at Outer Edge =	1473.760m	Height of Superstructure	= 2.020m
Deck level at center line =	1473.357m	Min. Height of Footpath Side Pedestal (1)	= 0.300m
Soffit Level at center of bridge =	1471.272m	Height of Pedestal (2)	= 0.510m
Abutment cap top level =	1470.447m	Height of Pedestal (3)	= 0.720m
Abutment cap bottom lvl (uniform portion ends)	1469.947m	Height of Pedestal (4)	= 0.930m
Abutment cap bottom lvl (corbel portion ends)	1469.947m	Height of Pedestal (5)	= 0.000m
Abutment shaft top level =	1469.947m		
Ground level/LBL =	1462.735m	Distance of nearest girder to c.l. of deck	= 1.500m
Abutment shaft bottom level =	1458.285m	Height (Avg.) of Dirt Wall	= 2.910m
Foundation level =	1456.285m	Abutment shaft Above G.L	= 7.212m
HFL =	1456.285m	Abutment Shaft below G.L	= 4.450m
		Height of abutment shaft	= 11.662m
		MSL	= 1456.285m
		Wedge over girder flange	= 0.0700m
<u>Material Specification</u>			

Concrete Grade

	=	M 35	
Characteristic compressive strength of concrete, fck	=	35 MPa at 28 days	
Design Compressive strength of Concrete, fcd	=	15.63 MPa at 28 days	(0.67/1.5 * fck)
Tensile strength of concrete, fctm	=	2.77 MPa	
Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.002	
Ultimate Strain, ϵ_{cu2}	=	0.0035	
Ecm	=	32308 MPa	

Steel Grade

	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500 MPa	
Design Yield Strength of Reinforcement, fyd	=	435 MPa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000 MPa	

Dry weight of Concrete	=	25 kN/m ³	
Dry unit weight of soil	=	20 kN/m ³	
Permissible Crack Width	=	0.3 mm	- For moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48 fck	
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²

Creep Coefficient

For Abutment Shaft	=	1.2	for 365 days
For Footing	=	1.2	for 365 days

Clear Cover to Reinforcement

Earth Face	=	75	mm
Non-Earth Face	=	40	mm

Seismic Data:

Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction

Properties of backfill material :

c	=	0	
ϕ	=	35	
θ	=	90	
β	=	0	
δ	=	22.50	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.000m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

DL Reaction on Bearing No.-1	=	443.73	eT 4.500
DL Reaction on Bearing No.-2	=	425.40	1.500
DL Reaction on Bearing No.-3	=	425.40	-1.500
DL Reaction on Bearing No.-4	=	443.73	-4.500
DL Reaction on Bearing No.-5	=	0.00	0.000
		1738.25	0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	135.917	4.50
SIDL Reaction on Bearing No.-2	=	-55.917	1.50
SIDL Reaction on Bearing No.-3	=	-55.917	-1.50
SIDL Reaction on Bearing No.-4	=	135.917	-4.50
SIDL Reaction on Bearing No.-5	=	0.000	0.00
		160.00	0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	60.349	4.50
SIDL Reaction on Bearing No.-2	=	60.349	1.50
SIDL Reaction on Bearing No.-3	=	60.349	-1.50
SIDL Reaction on Bearing No.-4	=	60.349	-4.50
SIDL Reaction on Bearing No.-5	=	0.000	0.00
		241.40	0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 1-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	955.66	kN	Corr. Transv	=	1428.527 kNm	ecc.	1.49
min	=	442.94	kN	Corr. Transv	=	1141.773 kNm	ecc.	2.58

SV Loading

max	=	1618.56	kN	Corr. Transv	=	485.568 kNm	ecc.	0.30
min	=	1261.44	kN	Corr. Transv	=	378.432 kNm	ecc.	0.30

**MAXIMUM TRANSVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	774.00	kN	Corr. Transv	=	2393.795 kNm	ecc.	3.09
min	=	225.00	kN	Corr. Transv	=	761.205 kNm	ecc.	3.38

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.300 x	0.800 x	0.800 =	0.1920 m ³	4.500
Pedestal 2	=	0.510 x	0.800 x	0.800 =	0.3264 m ³	1.500
Pedestal 3	=	0.720 x	0.800 x	0.800 =	0.4608 m ³	-1.500
Pedestal 4	=	0.930 x	0.800 x	0.800 =	0.5952 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					1.5744 m ³	-1.280

VOLUME CALCULATION

C.G. Of Footing	=	7.600 m
C.G. Of shaft from toe tip	=	5.400 m
Distance between c.g. of shaft and footing	=	2.200 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	2.120	7.950	-0.800	1.400	-6.200	0.000
-Trapering portion	1	12.50	0.300	0.790	2.963	-0.800	1.400	-6.200	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.100	1.100	-6.500	0.000
(Corbel) 0.5	1	12.50	0.300	0.300	0.563	-1.050	1.150	-6.450	0.000
Cap (uniform portion)	1	12.50	1.850	0.500	11.563	-0.025	2.175	-5.425	0.000
Cap (Corbel Portion)	1	12.50	1.850	0.000	0.000	-0.025	2.175	-5.425	0.000
		12.50	1.850						
Shaft above HFL	1	12.50	0.800	11.662	116.620	0.000	2.200	-5.400	0.000
Shaft below HFL	1	12.50	0.800	0.000	0.000	0.000	2.200	-5.400	0.000
Solid Return Wall	2	9.40	0.850	16.724	267.258	-5.100	-2.900	-10.500	0.000
Cantilever Return wall(Rectangular portion)	0	0.00	0.500	0.600	0.000	-0.400	1.800	-5.800	0.000
Cantilever Return wall(Traingular portion)	0	0.00	0.850	-0.600	0.000	-0.400	1.800	-5.800	0.000
Counter fort wall	2	9.40	0.600	11.662	65.774	-3.533	-2.900	-10.500	0.000
Front Counter fort wall	0	5.00	0.600	2.500	0.000	2.067	4.997	-2.603	0.000
Footing									
Heel Slab	1	12.50	9.400	0.750	88.125		-2.900	-10.500	0.000
Toe Slab	1	12.50	5.000	1.375	85.938		4.721	-2.879	0.000
Portion between Heel and Toe	1	12.50	0.800	2.000	20.000		2.200	-5.400	0.000
Back filling above HFL over Heel Slab	1	12.50	9.400	16.322	1584.804		-2.900	-10.500	0.000
Back filling below HFL over Heel Slab	1	12.50	9.400	0.000	0.000		-2.900	-10.500	0.000
Backfill above Heel slab	1	12.50	9.400	16.523	1608.451		-2.900	-10.500	0.000
Front Filling over Toe Slab	1	12.50	5.000	5.075	317.187		4.997	-2.603	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	5.000	-1.375	0.000		3.279	-4.321	0.000
Side filling between heel and toe	1	0.00	0.800	5.075	0.000		0.000	0.000	0.000
Aproach Slab	1	12.500	1.750	0.300	6.563	-1.100	1.100	-6.500	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.000	16.322	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.000	0.000	0.000		0.000	0.000	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barrie	2	0 kN/m	1.750	0.00kN		-0.800	1.400	-6.200

SECTIONAL PROPERTIES

Width of Footing (B)	=	15.2 m							
Length of Footing (L)	=	12.500 m							
A	=	15.200	x	12.500	=	190.000	m ²		
ZL	=	12.500	x	38.507	=	481.333	m ³		
ZT	=	IT1	+	IT2					
		distance of extreme point from centre							
IT1	=	15.200	x	162.760	=	2473.96	m ⁴		
IT2 (moment of inertia of triangle)	=	15.200	x	0.000	+	0.500 x	15.200 x	0.000 x	39.063
from centre of footing	=	0.000	m ⁴						
Moment of inertia of two triangle	=	0.000	m ⁴						
Total moment of inertia	=	2473.96	m ⁴						
Distance of extreme point from centre of footing	=	6.250	+	0.000	=	6.250	m		
Total Section modulus (ZT)	=	395.833	m ³						

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	1.000	1.000	1.000
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table B.2 of IRC:6-2017

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Serviceability Limit State

-Refer Table B.3 of IRC:6-2017

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.000	1.00	1.00
wearing course	1.200	1.20	1.20
Back Filling Weight	1.000	1.00	1.00
Shrinkage Creep Effect	1.000	1.00	1.00
Earth Pressure due to back filling	1.000	1.000	1.000
CWLL and Associate load and FPLL	1.000	0.750	0.000
Live Load Surcharge	0.800	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

-Refer Table B.4 of IRC:6-2017

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

F(Horizontal Seismic Force For Zone

4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.)
or horizontal vibrations.

$$= 2.0 \frac{D}{1000F}^{1/2}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 14.462 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 16.402 m

Abutment Cap Top Level - Foundation Level

= 14.162 m

Dimensions of Abutment Shaft

Length	=	12.50	m
Width	=	0.80	m
Width of counter for @ top	=	0.55	m
Width of counter for @ Bottom	=	9.40	m
Height of Counter fort	=	11.66	m
Thk. Of counter fort	=	0.60	m
No. counter fort	=	2.00	m
Width of Solid Return Wall	=	9.40	m
Thk. of Return wall	=	0.85	m

Moment of Inertia , $I_{longitudinal}$ = 3421.087 m⁴

Moment of Inertia , $I_{transverse}$ = 415.484 m⁴

Ecm = 3.231E+07 kN/m²

Longitudinal Direction

Force	=	56573.093	KN
D	=	2139.64	KN
T	=	0.0123	sec

Transverse Direction

Force	=	5729.623	KN
D	=	2419.362	KN
T	=	0.0411	sec

For Hard or Rocky Strata, with N>30

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 2.500

For Medium Strata, with N>10&N<30

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 2.500

Design Calculation

RODIC

Seismic Coeff.

For Soft Soil Strata, with $N < 10$

$$\frac{S_a}{g} = 2.5000$$

$$\frac{S_a}{g} = 2.500$$

Medium Soil Strata

$$S_a/g = 2.50000$$

$$S_a/g = 2.500$$

$$\text{Seismic Coeff. In Longitudinal Direction} = 0.120$$

$$\text{Seismic Coeff. In Transverse Direction} = 0.360$$

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.360	In Transverse direction
Av	=	0.240	In Vertical direction

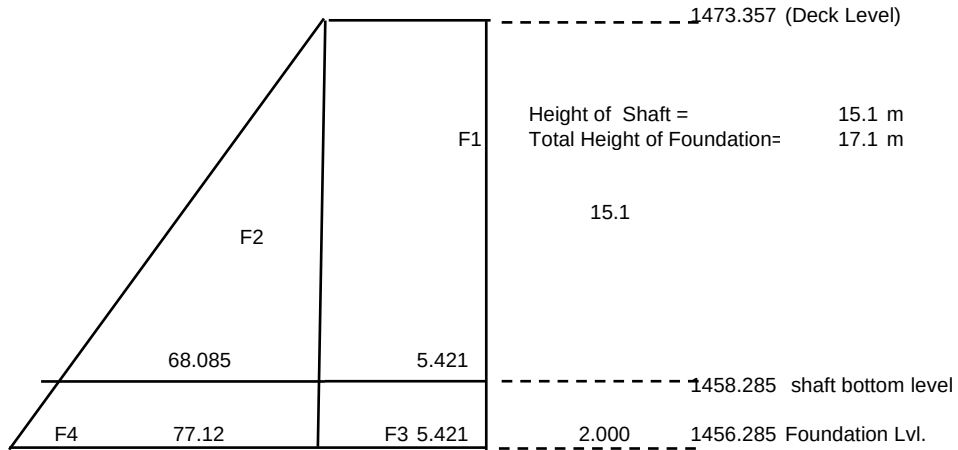
For Design of Foundation

Ah	=	0.162	In Longitudinal direction
Ah	=	0.360	In Transverse direction
Av	=	0.240	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = 1.2 m
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1458.285 m (at Shaft Base)			
@ RL					1456.285 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	15.072	x	8.900	=	727.151 kN
M1	=	727.15	x	7.54	=	5479.813 kN.m		at Shaft Bottom
F3	=	5.421	x	17.072	x	8.900	=	823.642 kN
M3	=	823.642	x	8.536	=	7030.606 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	15.072	= 68.085 kN/m ²
F2	=	0.5	x	68.09	x	15.072	x 8.90
	=	4566.511 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	4566.51	x	6.33	=	28907.108 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	17.072	= 77.120 kN/m ²
F4	=	0.5	x	77.12	x	17.072	x 8.90
	=	5858.838 kN					
M4	=	5858.84	x	7.17	=	42009.276 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m ³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	15.072	=	72.346 kN/m ²	
F	=	0.5	x	72.346	x	15.072	x 8.900
	=	4852.248 kN					

$$M = 4852.25 \times 5.024 = 24377.696 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 17.072 = 81.95 \text{ kN/m}^2$$

$$F = 0.5 \times 81.946 \times 17.07 \times 8.900 = 6225.440 \text{ kN}$$

$$M = 6225.44 \times 5.691 = 35426.904 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 8.90 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	28907.108	42009.276	4566.511	5858.838
Due to Minimum Fluid Pressure	24377.696	35426.904	4852.248	6225.440
Governing of Two	28907.108	42009.276	4852.248	6225.440
Due to Live Load Surcharge	5479.813	7030.606	727.151	823.642
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2017****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

α h	=	0.120	
α v	=	0.240	
ϕ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

α h	=	HORIZONTAL SEISMIC COEFFICIENT
α v	=	VERTICAL SEISMIC COEFFICIENT
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \frac{0.096}{0.157}$$

$$C_a = \frac{1}{0.348} \frac{2}{0.245}$$

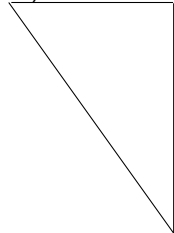
Ca	=	0.348	
Ka	=	0.226	
Dynamic Increment	=	0.348	-0.226 0.122

3 Earth Pressure : DRY CASE (Seismic case)

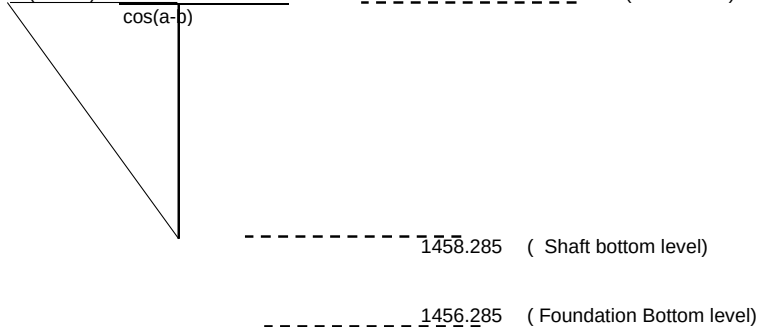
Equivalent Live Load Surcharge height	=	1.2 m
Assuming γ_{dry}	=	20 kN/m ³
γ_{water}	=	10.00 kN/m ³

Active Earth Pressure

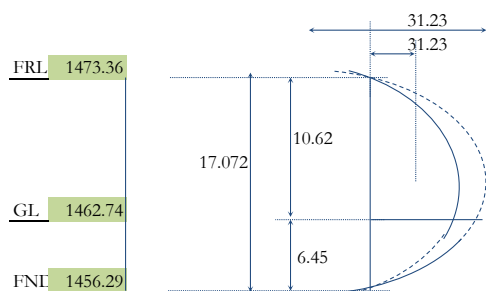
$$3 (Ca - Ka) = 0.366$$

**Surcharge Earth Pressure**

$$\frac{2 (Ca - Ka) \cos a}{\cos(a - b)} = 0.24$$

**Earth Pressure Diagram for Dynamic Increment****Due to Dynamic Live Load Surcharge**

	=	0.244	x	20.00	x	1.2	=	5.853 kN/m ²
at Shaft Bottom Level								
E1	=	0.50	x	5.853	x	15.072	x	8.900
	=	392.587	kN					
M1	=	392.587	x	10.098			=	3964.4 kN.m
at Foundation Bottom Level								
E2	=	0.50	x	5.853	x	17.072	x	8.900
	=	444.682	kN					
M2	=	444.682	x	13.438			=	5975.7 kN.m

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p _{mid_height}	=	31.23 t/m ²
h	=	17.072 m
y	=	2.086 m
L	=	8.900 m

Parabola below Water Level

p _{mid_height}	=	31.23 t/m ²
h	=	17.072 m
y	=	-2.086 m
L	=	8.900 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	2149.8	10.6
Parabola below Water Level	1013.4	4.1
Total Dynamic Earth Pressure	3163.2	8.5

$$E3 = 3163.169 \text{ kN}$$

$$E4 = 3163.169 \text{ kN}$$

$$M3 = 3163.17 \times 8.54 = 27000.8 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 3163.17 \times 10.54 = 33327.1 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	28907.1	42009.276	4566.511	5858.838
Due to active Earth Pressure (dynamic Increment)	27000.8	33327.147	3163.169	3163.169
Total Earth Pressure	55907.9	75336.423	7729.680	9022.007
Due to Minimum Fluid Pressure	24377.7	35426.904	4852.248	6225.440
Governing of Two	28907.1	75336.423	7729.680	9022.007
Due to Live Load Surcharge (Static)	5479.8	7030.606	727.151	823.642
Due to Live Load Surcharge(Dynamic)	3964.436	5975.739	392.587	444.682
Due to Passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing

Spherical Brg

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1738.25	1.35	1.35	2346.63	2346.63	
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00	
WC	=	241.40	1.75	1.75	422.44	422.44	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	955.66	1.5	0.20	1433.49	191.13	(1-70RW + 1-CLASS A)
CWLLmax-Reaction case	=	1618.56	1	0.20	1618.56	323.71	SV Loading
CWLLmin	=	442.94	1.5	0.20	664.41	88.59	(1-70RW + 1-CLASS A)
CWLLmin	=	1261.44	1	0.20	1261.44	252.29	SV Loading
CWLLmax-Transv. Moment Case		774.00	1.5	0.20	1161.00	154.80	(1-70RW)

$$\text{Braking Force} = 0.2 \times 0 + 0.05 \times 0 = 0 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μ R	Fh/2 + μ R	Fh- μ R	Governing Long. Force (kN)	
DL+SIDL	2985.08	0	149.254	149.254	-149.254	149.254	
DL+SIDL+LL-Max Reaction case	3649.48	0	182.474	182.474	-182.474	182.474	(1-70RW + 1-CLASS A)
	4910.92	0	245.546	245.546	-245.546	245.546	SV Loading
DL+SIDL+LL-Min Reaction case	4418.57	0	220.928	220.928	-220.928	220.928	(1-70RW + 1-CLASS A)
	4603.64	0	230.182	230.182	-230.182	230.182	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4146.08	0	207.304	207.304	-207.304	207.304	

Longitudinal Seismic Case: Seismic effect = 0.00

Longitudinal Seismic Case:		Seismic Effect		=		0.00		Governing Long. Force (kN)	
	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Fh/2 + μ R	Fh- μ R			
DL+SIDL	2139.64	2985.08	0.00	149.254	149.254	-149.254	149.254	Dry Case	
DL+SIDL+LL-Max Reaction case		3073.66	0.00	153.683	153.683	-153.683	153.683		
DL+SIDL+LL-Min Reaction case		3176.21	0.00	158.810	158.810	-158.810	158.810	HFL Case	
DL+SIDL+LL-Max Transv. Moment case		3139.88	0.00	156.994	156.994	-156.994	156.994		

Transverse Seismic Case: Seismic effect = 0.00

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Fh/2 + μ R	Fh- μ R	Governing Long. Force (kN)	
DL+SIDL	2139.64	2985.08	0.000	149.254	149.254	-149.254	149.254	
DL+SIDL+LL-Max Reaction case		3073.66	0.00	153.683	153.683	-153.683	153.683	Dry Case
DL+SIDL+LL-Min Reaction case		3176.21	0.00	158.810	158.810	-158.810	158.810	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3139.88	0.00	156.994	156.994	-156.994	156.994

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1738.25	1.35	1.35	2346.63	2346.63
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.40	1.75	1.75	422.44	422.44
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	955.66	1.5	0.75	1433.49	716.75
CWLLmax-Transv. Moment Case	=	774.00	1.5	0.75	1161.00	580.50
CWLLmin	=	442.94	1.5	0.75	664.41	332.20

Braking Force = 0 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2985.08	0.000	149.254	149.254	-149.254	149.254	
DL+SIDL+LL-Max Reaction case	3649.48	0.000	182.474	182.474	-182.474	182.474	(1-70RW + 1-CLASS A)
	4910.92	0.000	245.546	245.546	-245.546	245.546	SV Loading
DL+SIDL+LL-Min Reaction case	4418.57	0.000	220.928	220.928	-220.928	220.928	(1-70RW + 1-CLASS A)
	4603.64	0.000	230.182	230.182	-230.182	230.182	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4146.08	0.000	207.304	207.304	-207.304	207.304	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 0.00

	Unfactored Vertical Force	Vertical Force	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.64	2985.08	0.00	149.254	149.254	-149.254	149.254	
DL+SIDL+LL-Max Reaction case		3317.28	0.00	165.864	165.864	-165.864	165.864	Dry Case
DL+SIDL+LL-Min Reaction case		3701.82	0.00	185.091	185.091	-185.091	185.091	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3565.58	0.00	178.279	178.279	-178.279	178.279	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.64	2985.08	0.000	149.254	149.254	-149.254	149.254	
DL+SIDL+LL-Max Reaction case		3317.28	0.000	165.864	165.864	-165.864	165.864	Dry Case
DL+SIDL+LL-Min Reaction case		3701.82	0.000	185.091	185.091	-185.091	185.091	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3565.58	0.000	178.279	178.279	-178.279	178.279	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1738.25	1	1.00	1738.25	1738.25
SIDL except wc	=	160.00	1	1.00	160.00	160.00
WC	=	241.40	1	1.00	241.40	241.40
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	955.66	1	0.20	955.66	191.13
CWLLmax- Transv. Moment Case		774.00	1	0.20	774.00	154.80
CWLLmin	=	442.94	1	0.20	442.94	88.59

Braking Force = 0 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.64	0.000	106.982	106.982	-106.982	106.982	
DL+SIDL+LL-Max Reaction case	2582.58	0.000	129.129	129.129	-129.129	129.129	(1-70RW + 1-CLASS A)
	3401.08	0.000	170.054	170.054	-170.054	170.054	SV Loading
DL+SIDL+LL-Min Reaction case	3095.30	0.000	154.765	154.765	-154.765	154.765	(1-70RW + 1-CLASS A)
	3758.20	0.000	187.910	187.910	-187.910	187.910	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2913.64	0.000	145.682	145.682	-145.682	145.682	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.64	0.000	106.982	106.982	-106.982	106.982	
DL+SIDL+LL-Max Reaction case	2228.23	0.000	111.412	111.412	-111.412	111.412	Dry Case
DL+SIDL+LL-Min Reaction case	2330.77	0.000	116.539	116.539	-116.539	116.539	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2294.44	0.000	114.722	114.722	-114.722	114.722	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.64	0.000	106.982	106.982	-106.982	106.982	
DL+SIDL+LL-Max Reaction case	2228.23	0.000	111.412	111.412	-111.412	111.412	Dry Case
DL+SIDL+LL-Min Reaction case	2330.77	0.000	116.539	116.539	-116.539	116.539	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2294.44	0.000	114.722	114.722	-114.722	114.722	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1738.25	1.1	1.10	1912.07	1912.07
SIDL except wc	=	160.00	1.1	1.10	176.00	176.00
WC	=	241.40	1.35	1.35	325.88	325.88
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax- Reaction case	=	955.66	1.5	0.00	1433.49	0.00
CWLLmax- Transv. Moment Case	=	774.00	1.5	0.00	1161.00	0.00
CWLLmin	=	442.94	1.5	0.00	664.41	0.00

Braking Force = 0 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh- μR	Governing Long. Force (kN)	
DL+SIDL	2413.96	0.000	120.698	120.698	-120.698	120.698	
DL+SIDL+LL-Max Reaction case	3078.36	0.000	153.918	153.918	-153.918	153.918	(1-70RW + 1-CLASS A)
	3675.40	0.000	183.770	183.770	-183.770	183.770	SV Loading
DL+SIDL+LL-Min Reaction case	3847.45	0.000	192.372	192.372	-192.372	192.372	(1-70RW + 1-CLASS A)
	4032.52	0.000	201.626	201.626	-201.626	201.626	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3574.96	0.000	178.748	178.748	-178.748	178.748	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 0.00

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh- μR	Governing Long. Force (kN)	
DL+SIDL	2139.64	2413.96	0.00	120.698	120.698	-120.698	120.698	
DL+SIDL+LL-Max Reaction case		2413.96	0.00	120.698	120.698	-120.698	120.698	Dry Case
DL+SIDL+LL-Min Reaction case		2413.96	0.00	120.698	120.698	-120.698	120.698	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2413.96	0.00	120.698	120.698	-120.698	120.698	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh- μR	Governing Long. Force (kN)	
DL+SIDL	2139.64	2413.96	0.00	120.698	120.698	-120.698	120.698	
DL+SIDL+LL-Max Reaction case		2413.96	0.000	120.698	120.698	-120.698	120.698	Dry Case
DL+SIDL+LL-Min Reaction case		2413.96	0.000	120.698	120.698	-120.698	120.698	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2413.96	0.000	120.698	120.698	-120.698	120.698	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1738.25	1	1	1	1738.25	1738.25	1738.25
SIDL except wc	=	160.00	1	1	1	160.00	160.00	160.00
WC	=	241.40	1.20	1.20	1.20	289.67	289.67	289.67
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax- Reaction case	=	955.66	1	0.75	0	955.66	716.75	0.00
CWLLmax- Transv. Moment Case	=	774.00	1	0.75	0	774.00	580.50	0.00
CWLLmin	=	442.94	1	0.75	0	442.94	332.20	0.00

Braking Force = 0 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2187.92	0.000	109.396	109.396	-109.396	109.396	
DL+SIDL+LL-Max Reaction case	2630.86	0.000	131.543	131.543	-131.543	131.543	(1-70RW + 1- CLASS A)
	3449.36	0.000	172.468	172.468	-172.468	172.468	SV Loading
DL+SIDL+LL-Min Reaction case	3143.58	0.000	157.179	157.179	-157.179	157.179	(1-70RW + 1- CLASS A)
	3806.48	0.000	190.324	190.324	-190.324	190.324	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2961.92	0.000	148.096	148.096	-148.096	148.096	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2187.92	0.000	109.396	109.396	-109.396	109.396	
DL+SIDL+LL-Max Reaction case	2520.13	0.000	126.006	126.006	-126.006	126.006	Dry Case
DL+SIDL+LL-Min Reaction case	2904.67	0.000	145.233	145.233	-145.233	145.233	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2768.42	0.000	138.421	138.421	-138.421	138.421	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2187.92	0.000	109.396	109.396	-109.396	109.396	
DL+SIDL+LL-Max Reaction case	2187.92	0.000	109.396	109.396	-109.396	109.396	Dry Case
DL+SIDL+LL-Min Reaction case	2187.92	0.000	109.396	109.396	-109.396	109.396	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2187.92	0.000	109.396	109.396	-109.396	109.396	

Centrifugal Force Calculation

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	50.00	kmph	50.00	kmph
Live Load	W	=	955.66	kN	955.66	kN
Radius of Curvature	R	=	50.00	m	50.00	m
CENTRIFUGAL FORCE	C	=	376.24	kN	376.24	kN

Possible Load Combination

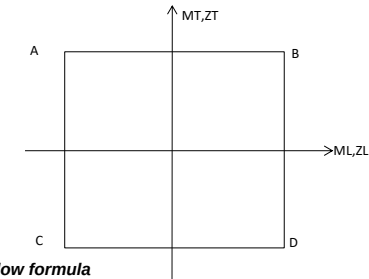
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1456.29 m

Properties of Footing Base:

A	=	190.00	m ²
ZL	=	481.33	m ³
ZT	=	395.83	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1.00			1738.25	2.20	3824.14	0.00	0.00
SIDL except Wearing Course	1.00			160.00	2.20	352.00	0.00	0.00
Wearing Course	1.00			241.40	2.20	531.07	0.00	0.00
Bearing Pedestal	1.00	25.00	1.57	39.36	2.20	86.59	-1.28	-50.40
				2179.00		4793.81		-50.40
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25.00	7.95	198.75	1.40	278.25	0.00	0.00
Dirt Wall-Tapered portion	1.00	25.00	2.96	74.06	1.40	103.69	0.00	0.00
Bracket - Uniform portion	1.00	25.00	1.13	28.13	1.10	30.94	0.00	0.00
Bracket - Tapered portion	1.00	25.00	0.56	14.06	1.15	16.17	0.00	0.00
Cap - (uniform portion)	1.00	25.00	11.56	289.06	2.18	628.71	0.00	0.00
Cap - (corbel portion)	1.00	25.00	0.00	0.00	2.18	0.00	0.00	0.00
Cantilever Return Wall-Rectangle portion	1.00	25.00	0.00	0.00	1.80	0.00	0.00	0.00
Cantilever Return Wall-Triangle portion	1.00	25.00	0.00	0.00	1.80	0.00	0.00	0.00
RCC Railing or Crash Barrier	1.00			0.00	1.40	0.00	0.00	0.00
Approach Slab	1.00	25.00	6.56	164.06	1.10	180.47	0.00	0.00
				768.13		1238.23		0.00
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25.00	267.26	6681.44	-2.90	-19376.17	0.00	0.00
Counterfort wall	1.00	5.00	65.77	328.87	-2.90	-953.72		
Front Counterfort wall	1.00	5.00	0.00	0.00	5.00	0.00		
Abutment Shaft	1.00	25.00	116.62	2915.50	2.20	6414.10	0.00	0.00
Back filling over heel slab	1.00	20.00	1608.45	32169.01	-2.90	-93290.14	0.00	0.00
Front Filling over toe slab	1.00	20.00	317.19	6343.75	5.00	31702.08	0.00	0.00
Side filling between heel and toe	1.00	20.00	0.00	0.00	0.00	0.00	0.00	0.00
Heel slab	1.00	25.00	88.13	2203.13	-2.90	-6389.06	0.00	0.00
Toe slab	1.00	25.00	85.94	2148.44	4.72	10143.23	0.00	0.00
portion between heel & toe	1.00	25.00	20.00	500.00	2.20	1100.00	0.00	0.00
Vertical Components of active earth pressure	1.00			2578.66	-7.60	-19597.83	0.00	0.00
				55868.79		-90247.51		0.00
Total				58815.92		-84215.47		-50.40

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.90	1564.42	-5.40	-8447.88
0.90	144.00	-5.40	-777.60
1.00	241.40	-5.40	-1303.53
0.90	35.42	-5.40	-191.29
	1985.24		-10720.30
0.90	178.88	-6.20	-1109.03
0.90	66.66	-6.20	-413.27
0.90	25.31	-6.50	-164.53
0.90	12.66	-6.45	-81.63
0.90	260.16	-5.43	-1411.35
0.90	0.00	-5.43	0.00
0.90	0.00	-5.80	0.00
0.90	0.00	-5.80	0.00
0.90	0.00	-6.20	0.00
0.90	147.66	-6.50	-959.77
	691.31		-4139.57
0.90	6013.29	-10.50	-63139.59
0.90	295.98	-10.50	-3107.81
0.90	0.00	-2.60	0.00
0.90	2623.95	-5.40	-14169.33
0.90	28952.11	-10.50	-303997.18
0.90	5709.37	-2.60	-14859.37
0.90	0.00	0.00	0.00
0.90	1982.81	-10.50	-20819.53
0.90	1933.59	-2.88	-5566.41
0.90	450.00	-5.40	-2430.00
1.00	2578.66	-15.20	-39195.66
	50539.78		-467284.87
	53216.33		-482144.75

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1.00	106.98	1471.27	1603.34
due to Earth pressure	1.00	6225.44		42009.28

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		120.70	1471.27	1808.90
due to Earth pressure	1.50	9338.16		63013.91

Summary of Forces For SBC

P	58815.92	kN
ML	-40602.86	kNm
MT	-50.40	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		58815.92		-84215.47		-50.40
CWLL-Max. Reaction case	1.00	955.66	2.20	2102.45	1.49	1428.53
Vertical Components of LL Surcharge	1.00	341.16	-7.60	-2592.84	0.00	0.00
Total		60112.75		-84705.86		1378.13

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1.00	129.13	1471.27	1935.26
due to Earth pressure	1.00	6225.44		42009.28
due to Live load surcharge	1.00	823.64		7030.61

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		153.92	1471.27	2306.77
due to Earth pressure	1.50	9338.16		63013.91
due to Live load surcharge	1.20	988.37		8436.73
		10480.45		73757.41

Summary of Forces For SBC

P	60112.75	kN
ML	-33730.72	kNm
MT	1378.13	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
106.98	1603.34	0.00	0.00
6225.44	42009.28	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.70	1808.90	0.00	0.00
9338.16	63013.91	0.00	0.00

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.00	53216.33		-482144.75
1.00		-5.40	0.00
		-15.20	-5185.69
	53216.33		-487330.44

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
129.13	1935.26	0.00	0.00
6225.44	42009.28	0.00	0.00
823.64	7030.61	0.00	0.00
7178.21	50975.14	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
153.92	2306.77	0.00	0.00
9338.16	63013.91	0.00	0.00
988.37	8436.73	0.00	0.00
10480.45	73757.41	0.00	0.00

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1.00 $\alpha_h = 0.12$ In Longitudinal direction
 $\alpha_h = 0.36$ In Transverse direction
 $\alpha_v = 0.24$ In Vertical direction

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha_h \times P$ (kN)	FT = $0.3 \times \alpha_h \times P$ (kN)	Fv = $0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = $P \times eL1$	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = $P \times eT$	MTs due to FT
Superstructure															
Dead Load	1.00			1738.25		187.73	125.15	2.20	3824.14	275.34	1472.62		0.00	0.00	3066.33
SIDL except Wearing Course	1.00			160.00		11.52		2.20	352.00	25.34	1473.81		0.00	0.00	302.76
Wearing Course	1.00			241.40		26.07	17.38	2.20	531.07	38.24	1473.36		0.00	0.00	445.08
Bearing Pedestal	1.00	25.00	1.57	39.36		4.25	2.83	2.20	86.59	6.23			-1.28	-50.40	
				2179.00		235.33	156.89		4793.81	345.15				-50.40	3814.17
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.00	25.00	7.95	198.75	23.85	21.47	14.31	1.40	278.25	20.03	1472.30	381.89	0.00	0.00	343.70
Dirt Wall-Tapered portion	1.00	25.00	2.96	74.06	8.89	8.00	5.33	1.40	103.69	7.47	1470.84	129.38	0.00	0.00	116.44
Bracket - Uniform portion	1.00	25.00	1.13	28.13				1.10	30.94						
Bracket - Tapered portion	1.00	25.00	0.56	14.06				1.15	16.17						
Cap - (uniform portion)	1.00	25.00	11.56	289.06	34.69	31.22	20.81	2.18	628.71	45.27	1470.20	482.57	0.00	0.00	434.32
Cap - (corbel portion)	1.00	25.00	0.00	0.00	0.00	0.00	0.00	2.18	0.00	0.00	1469.95	0.00	0.00	0.00	0.00
Cantilever Return Wall-Rectangle portion	1.00	25.00	0.00	0.00	0.00	0.00	0.00	1.80	0.00	0.00	1473.46	0.00	0.00	0.00	0.00
Cantilever Return Wall-Triangle portion	1.00	25.00	0.00	0.00	0.00	0.00	0.00	1.80	0.00	0.00	1473.36	0.00	0.00	0.00	0.00
RCC Railing or Crash Barrier	1.00			0.00				1.40	0.00				0.00	0.00	0.00
Approach Slab	1.00	25.00	6.56	164.06				1.10	180.47				0.00	0.00	
				768.13	67.43	60.68	40.46		1238.23	72.77		993.83		0.00	894.45
Substructure & Foundation -Portion 2															
Solid Return wall	1.00	25.00	267.26	6681.44	801.77	721.60	481.06	-2.90	-19376.17	-1395.08	1465.10	7063.92	0.00	0.00	6357.52
	0.00	1.00	5.00	65.77	328.87	39.46	23.68	-2.90	-953.72	-68.67	1462.34	239.01	0.00	0.00	215.11
	0.00	1.00	5.00	0.00	0.00	0.00	0.00	5.00	0.00	0.00	1459.12	0.00	0.00	0.00	0.00
Abutment Shaft	1.00	25.00	116.62	2915.50	349.86	314.87	209.92	2.20	6414.10	461.82	1466.34	3518.19	0.00	0.00	3166.37
Back filling over heel slab	1.00	20.00	1608.45	32169.01	0.00	0.00	0.00	-2.90	-93290.14	0.00	1465.10	0.00	0.00	0.00	0.00
Front Filling over toe	1.00	20.00	317.19	6343.75				5.00	31702.08				0.00	0.00	0.00
Side filling between heel and toe	1.00	20.00	0.00	0.00				0.00	0.00				0.00	0.00	0.00
Heel slab	1.00	25.00	88.13	2203.13	264.38	237.94	158.63	-2.90	-6389.06		1457.66	363.52	0.00	0.00	327.16
Toe slab	1.00	25.00	85.94	2148.44	257.81	232.03	154.69	4.72	10143.23		1457.66	354.49	0.00	0.00	319.04
portion between heel & toe	1.00	25.00	20.00	500.00	60.00	54.00	36.00	2.20	1100.00		1457.29	60.00	0.00	0.00	54.00
Vertical component of active earth pressure	1.00			2426.81				-7.60	-18443.76				0.00	0.00	
Vertical component of dynamic increment of earth pressure	1.00			1310.23				-7.60	-9957.73				0.00	0.00	
				57027.17	1773.28	1595.96	1063.97		-99051.16	-1001.94		11599.13		0.00	10439.22
Total =				59974.30	1840.71	1891.97	1261.31		-93019.13	-584.02		12592.97		-50.40	15147.84

-1261.31

584.02

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = $P \times eL2$	MLs due to Fv
Superstructure								
Dead Load	0.90			1564.42	112.64	-5.40	-8447.88	-608.25
SIDL except Wearing Course	0.90			144.00	10.37	-5.40	-777.60	-55.99
Wearing Course	1.00			241.40	17.38	-5.40	-1303.53	-93.85
Bearing Pedestal	0.90	25.00	1.57	35.42		-5.40	-191.29	
				1985.24	140.39		-10720.30	-758.09

For Overturning or Sliding Effect

Load Factor	FL = $\alpha_h \times P$ (kN)	C.g. of Force (m)	MLs due to FL

Design Calculation

RODIC

Stability of Foundation

Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	0.90	25.00	7.95	178.88	12.88	-6.20	-1109.03	-79.85	1.00	23.85	1472.30	381.89
Dirt Wall-Tapered portion	0.90	25.00	2.96	66.66	4.80	-6.20	-413.27	-29.76	1.00	8.89	1470.84	129.38
Bracket - Uniform portion	0.90	25.00	1.13	25.31								
Bracket - Tapered portion	0.90	25.00	0.56	12.66								
Cap - (uniform portion)	0.90	25.00	11.56	260.16	18.73	-5.43	-1411.35	-101.62	1.00	34.69	1470.20	482.57
Cap - (corbel portion)	0.90	25.00	0.00	0.00	0.00	-5.43	0.00	0.00	1.00	0.00	1469.95	0.00
Cantilever Return Wall-Rectangle portion	0.90	25.00	0.00	0.00	0.00	-5.80	0.00	0.00	1.00	0.00	1473.46	0.00
Cantilever Return Wall-Triangle portion	0.90	25.00	0.00	0.00	0.00	-5.80	0.00	0.00	1.00	0.00	1473.36	0.00
RCC Railing or Crash Barrier	0.90			0.00		-6.20	0.00					
Approach Slab	0.90	25.00	6.56	147.66		-6.50	-959.77					
				691.31	36.41		-3893.41	-211.22		67.43		993.83
Substructure & Foundation -Portion 2												
Abutment Shaft	0.90	25.00	116.62	2623.95	188.92	-5.40	-14169.33	-1020.19	1.00	349.86	1466.34	3518.19
	0.00	0.90	5.00	65.77	295.98	-10.50	-3107.81	-223.76	1.00	39.46	1462.34	239.01
	0.00	0.90	5.00	0.00	0.00	-2.60	0.00	0.00	1.00	0.00	1459.12	0.00
Solid Return wall	0.90	25.00	267.26	6013.29	432.96	-10.50	-63139.59	-4546.05	1.00	801.77	1465.10	7063.92
Back filling over heel slab	0.90	20.00	1608.45	28952.11	0.00	-10.50	-303997.18	0.00	1.00	0.00	1465.10	0.00
Front Filling over toe	0.90	20.00	317.19	5709.37		-2.60	-14859.37					
Side filling between heel and toe	0.90	20.00	0.00	0.00		0.00	0.00					
Heel slab	0.90	25.00	88.13	1982.81		-10.50	-20819.53		1.00	264.38	1457.66	363.52
Toe slab	0.90	25.00	85.94	1933.59		-2.88	-5566.41		1.00	257.81	1457.66	354.49
portion between heel & toe	0.90	25.00	20.00	450.00		-5.40	-2430.00		1.00	60.00	1457.29	60.00
Vertical component of active earth pressure	1.00			2578.66		-15.20	-39195.66					
Vertical component of dynamic increment of earth pressure	1.00			1310.23		-15.20	-19915.46					
				51850.01	643.19		-487200.33	-5790.00		1773.28		11599.13
Total =				54526.56	819.99		-501814.04	-6759.32		1840.71		12592.97

-819.99

6759.32

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1.00	106.98	235.33	1471.27	1603.34	3814.17
due to Substructure	1.00	1840.71	1656.64		12592.97	11333.67
due to Earth pressure	1.00	9022.01			75336.42	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		120.70	1471.27	1808.90
due to Substructure	1.50	2761.06		18889.45
due to Active Earth pressure	1.00	5858.84		42009.28
due to dynamic Earth pressure	1.00	3163.17		33327.15
		11903.77		96034.77

Summary of Forces For SBC

	Downward	Upward	
P	61235.61	58712.98	kN
ML	-4070.42	-2902.39	kNm
MT	15097.44	15097.44	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
106.98	1603.34	0.00	0.00	0.00	0.00	235.33	3814.17
1840.71	12592.97	0.00	0.00	0.00	0.00	1656.64	11333.67
9022.01	75336.42						
10969.70	89532.73	0.00	0.00	0.00	0.00	1891.97	15147.84

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.70	1808.90	0.00	0.00
2761.06	18889.45	0.00	0.00
5858.84	42009.28	0.00	0.00
3163.17	33327.15	0.00	0.00
11903.77	96034.77	0.00	0.00

Summary of Restoring Forces

Vertical Load	53706.57	kN
Moment	-508573.36	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha h \times P$ (kN)	FT = $0.3 \times \alpha h \times P$ (kN)	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				2179.00		235.33	156.89		4793.81	345.15				-50.40	3814.17
Forces from Substructure				57795.30	1840.71	1656.64	1104.43		-97812.94	-929.17		12592.97		0.00	11333.67
CWLL-Max. Reaction case	0.20			191.13		20.64	13.76	2.20	420.49	30.28	1474.56		1.49	285.71	377.18
Vertical component of LL Surcharge	0.20			68.23				-7.60	-518.57				0.00	0.00	
Vertical component of dynamic increment LL Surcharge	0.20			36.84				-7.60	-279.97				0.00	0.00	
Total =				60270.50	1840.71	1912.61	1275.08		-93397.18	-553.74		12592.97		235.31	15525.02
									-1275.08		553.74				

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				1985.24	140.39	0.00	-10720.30	-758.09
Forces from Substructure				52541.32	679.60	0.00	-491093.74	-6001.23
CWLL-Max. Reaction case	0.00			0.00	0.00	-5.40	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-15.20	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.00		-15.20	0.00	
Total =				54526.56	819.99		-501814.04	-6759.32
				-819.99		6759.32		

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1.00	111.41	255.97	1471.27	1669.72	4191.35
due to Substructure	1.00	1840.71	1656.64		12592.97	11333.67
due to Earth pressure	1.00	9022.01			75336.42	
due to Live load surcharge	0.20	253.66			2601.27	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
111.41	1669.72	0.00	0.00	0.00	0.00	255.97	4191.35
1840.71	12592.97	0.00	0.00	0.00	0.00	1656.64	11333.67
9022.01	75336.42						
253.66	2601.27						
11227.79	92200.38	0.00	0.00	0.00	0.00	1912.61	15525.02

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		120.70	1471.27	1808.90
due to Substructure	1.50	2761.06		18889.45
due to Active Earth pressure	1.00	5858.84		42009.28
due to dynamic Earth pressure	1.00	3163.17		33327.15
due to Live load surcharge	0.00	0.00		0.00
due to dynamic increment of live load surcharge	0.00	0.00		0.00
		11903.77	96034.77	

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.70	1808.90	0.00	0.00
2761.06	18889.45	0.00	0.00
5858.84	42009.28		
3163.17	33327.15		
0.00	0.00		
0.00	0.00		
11903.77	96034.77	0.00	0.00

Summary of Forces For SBC			
	Downward	Upward	
P	61545.58	58995.43	kN
ML	-1750.54	-643.06	kNm
MT	15760.32	15760.32	kNm

Summary of Restoring Forces

Vertical Load	53706.57	kN
Moment	-508573.36	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x ah x P (kN)	FT = ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2179.00		784.44	156.89		4793.81	345.15				-50.40	12713.90
Substructure & Foundation -Portion 1				768.13	20.23	202.28	40.46		1238.23	72.77		298.15		0.00	2981.50
Substructure & Foundation -Portion 2				57027.17	531.99	5319.85	1063.97		-99051.16	-1001.94		3479.74		0.00	34797.39
Total =				59974.30	552.21	6306.57	1261.31		-93019.13	-584.02		3777.89		-50.40	50492.80
-1261.31 584.02															

Summary of Forces For SBC			
	Downward	Upward	
P	61235.61	58712.98	kN
ML	-12885.49	-11717.46	kNm
MT	50442.40	50442.40	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times a_v \times P$ (kN)	$ML = P \times eL1$	MLs due to F_v	$MT = P \times eT$
Total =	60270.50	1275.08	-93397.18	-553.74	235.31

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1.00	111.41	853.25	1471.27	1669.72	13971.15
due to Substructure	1.00	552.21	5522.13		3777.89	37778.90
due to Earth pressure	1.00	9022.01			75336.42	
due to Live load surcharge	0.20	253.66			2601.27	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		120.70	1471.27	1808.90
due to Substructure	1.50	828.32		5666.83
due to Active Earth pressure	1.00	5858.84		42009.28
due to dynamic Earth pressure	1.00	948.95		9998.14
due to Live load surcharge	0.00	0.00		0.00
due to dynamic increment of live load surcharge	0.00	0.00		0.00
		7756.81		59483.15

Summary of Forces For SBC			
	Downward	Upward	
P	61545.58	58995.43	kN
ML	-10565.62	-9458.14	kNm
MT	51985.36	51985.36	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.) = 1.00 x 376.24 = 376.24 kN
 Transverse Moment due to C.F. = 376.24 x (1474.96 - 1456.29) = 6700.14 kNm

Summary of Restoring Forces

Vertical Load	53706.57	kN
Moment	-508573.36	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times a_v \times P$ (kN)	$ML = P \times eL2$	MLs due to F_v
Total =	54526.56	819.99	-501814.04	-6759.32

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
111.41	1669.72	0.00	0.00	0.00	0.00	853.25	13971.15
552.21	3777.89	0.00	0.00	0.00	0.00	5522.13	37778.90
9022.01	75336.42						
253.66	2601.27						
9939.30	83385.31	0.00	0.00	0.00	0.00	6375.38	51750.05

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.70	1808.90	0.00	0.00
828.32	5666.83	0.00	0.00
5858.84	42009.28		
948.95	9998.14		
0.00	0.00		
0.00	0.00		
7756.81	59483.15	0.00	0.00

Summary of Restoring Forces

Vertical Load	53706.57	kN
Moment	-508573.36	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
376.24	7026.18	0.00	0.00

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	376.24	=	75.25 KN	Seismic	75.25	1405.24	0.00	0.00
Transverse Moment due to C.F.	=	75.25	x (	1474.96 -	=	1405.24 kNm					

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	58815.92	-40602.86	-50.40	393.79	225.08	394.04	225.33	394.04	225.08	9458.86	26608.17	2.81	64822.81	482144.75	7.44
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	60112.75	-26704.55	1378.13	375.34	264.38	368.38	257.42	375.34	257.42	10480.45	26608.17	2.54	73757.41	487330.44	6.61
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	61235.61	-2902.39	15097.44	366.46	354.40	290.18	278.12	366.46	278.12	11903.77	26853.29	2.26	96034.77	508573.36	5.30
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	61545.58	762.17	15760.32	362.16	365.32	282.53	285.69	365.32	282.53	11903.77	26853.29	2.26	96034.77	508573.36	5.30
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	61235.61	-11717.46	50442.40	474.07	425.38	219.20	170.52	474.07	170.52	9689.55	26853.29	2.77	72705.77	508573.4	6.99
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	61545.58	-9458.14	53390.59	478.46	439.16	208.69	169.39	478.46	169.39	7756.81	26853.29	3.46	59483.15	508573.4	8.55
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

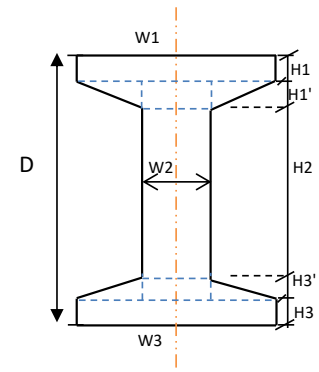
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	325
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	675	100	33750
1' (Rect.)	325	100	32500
2 (Rect.)	325	950	308750
3' (Tri.)	300	150	22500
3' (Rect.)	325	150	48750
3 (Rect.)	625	250	156250
			752500



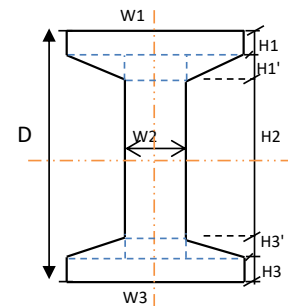
Total Area of Section A = 0.753 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	78
H2	=	1122
H3'	=	0
H3	=	250
W1	=	1000
W2	=	475
W3	=	475

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	525	78	20417
1' (Rect.)	475	78	36944
2 (Rect.)	475	1122	533056
3' (Tri.)	0	0	0
3' (Rect.)	475	0	0
3 (Rect.)	475	250	118750
			859167



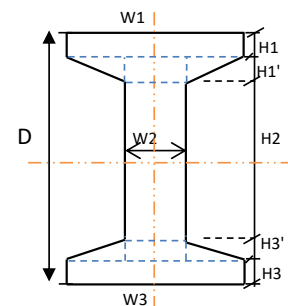
Total Area of Section A = 0.859 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	56
H2	=	1394
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

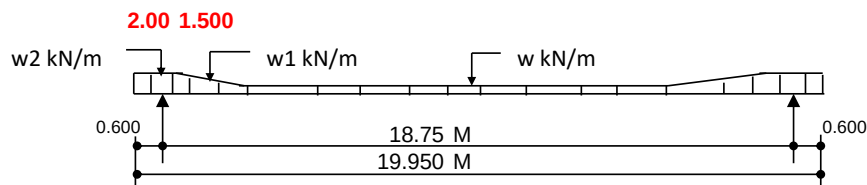
Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	56	10417
1' (Rect.)	625	56	34722
2 (Rect.)	625	1394	871528
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066667



Total Area of Section A = 1.067 m²

1.0 Self weight of Inner Girder & Outer Girder

Length of the girder = 19.950 m



UDL , w	=	18.81 kN/m
Avg. , w ₁	=	21.48 kN/m
UDL , w ₂	=	26.67 kN/m
Supp. Reaction , R	=	205.48 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	

UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder

Supp. Reaction , R	=	178.303 kN	For Outer Girder
	=	164.588 kN	For Inner Girder

weight of deck slab has been Increase by 5 %

Supp. Reaction , R	=	187.218 kN	For Outer Girder
	=	172.817 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

4.0 Weight due to wearing coat

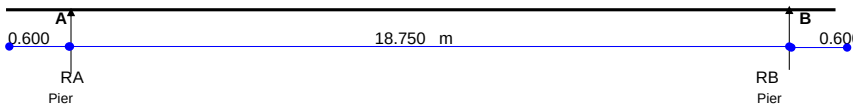
Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	

UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder

Supp. Reaction , R	=	60.349 kN	For Outer Girder
	=	65.835 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)**Maximum Live Load Reaction Case**Nos. of Lane for design purpose = **3 LANE****Due to Class A 1-Lane , 2-Lanes & 3-Lanes**Considering **27.00** kN(i.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c.l. of bearing at Abut. end = **0.600 m**
 Projection beyond c.l. of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



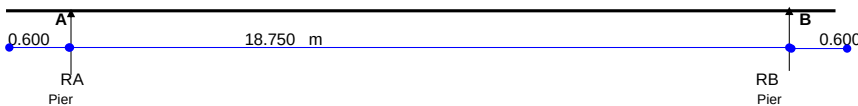
Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	554.0kN				
27.000		0.000	-0.600	-16.200	303.115	250.885
27.000	1.100	1.100	0.500	13.500		
114.000	3.200	4.300	3.700	421.800		
114.000	1.200	5.500	4.900	558.600		
68.000	4.300	9.800	9.200	625.600		
68.000	3.000	12.800	12.200	829.600		
68.000	3.000	15.800	15.200	1033.600		
68.000	3.000	18.800	18.200	1237.600		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	303.115	250.885

Due to Class 70R WheeledConsidering **170.00** kN(i.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c.l. of bearing at Abut. end = **0.600 m**
 Projection beyond c.l. of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



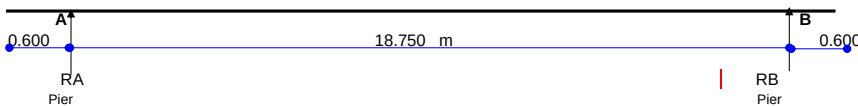
Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	1000.0kN				
170.000		0.000	-0.600	-102.000	758.731	241.269
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	758.731	241.269

Due to Class A 1-Lane (along with Class 70R)Considering **68.00** kN(i.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c.l. of bearing at Abut. end = **0.600 m**
 Projection beyond c.l. of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



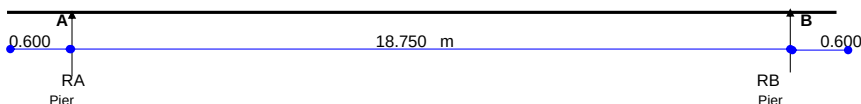
Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	554.0kN				
68.000		0.000	-0.600	-40.800	284.864	269.136
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
27.000	1.100	18.800	18.200	491.400		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	284.864	269.136

Due to Class SV LoadingConsidering **180.00** kN(i.e. **0.000m** from extreme Left End)

Length of the Span = **18.75** m
 Projection beyond cl. of bearing at Abut. end = **0.600** m
 Projection beyond cl. of bearing at far end = **0.600** m
 Total length including Overhangs = **19.95** m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
1st Train	Total Load	2880.0kN				
180.000		0.000	-0.600	-108.000	1618.560	1261.440
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		
180.000	1.500	9.000	8.400	1512.000		
180.000	1.500	10.500	9.900	1782.000		
180.000	1.500	12.000	11.400	2052.000		
180.000	1.500	13.500	12.900	2322.000		
180.000	1.500	15.000	14.400	2592.000		
180.000	1.500	16.500	15.900	2862.000		
180.000	1.500	18.000	17.400	3132.000		
180.000	1.500	19.500	18.900	3402.000		
0.000	1.500	0.000	-0.600	0.000		
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
0.000	0.000	3.000	2.400	0.000		
0.000	0.000	3.000	2.400	0.000		
0.000	0.000	3.000	2.400	0.000		
0.000	0.000	3.000	2.400	0.000		
0.000	0.000	3.000	2.400	0.000		
0.000	0.000	3.000	2.400	0.000		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	1618.560	1261.440

Deck Width = **12.500** mtransverse eccentricity of superstructure about c.g of abutment and foundation = **0** m**SV LOAD**

transverse ecc. = **0.3** m
 Maximum Reaction (RA) = **1618.560** KN Corr. MT = **485.568** kNm
 Minimum Reaction (RB) = **1261.440** KN Corr. MT = **378.432** kNm

1-CLASS A

transverse ecc. = **6.25** -0.5 -0.4 -0.9 **0** **0** = **4.45** m
 Maximum Reaction (RA) = **303.115** KN Corr. MT = **1348.860** kNm
 Minimum Reaction (RB) = **250.885** KN Corr. MT = **1116.440** kNm

2-CLASS A

transverse ecc. = **6.25** -0.5 -0.4 -1.8 **-0.85** **0** = **2.70** m
 Maximum Reaction (RA) = **606.229** KN Corr. MT = **1636.819** kNm
 Minimum Reaction (RB) = **501.771** KN Corr. MT = **1354.781** kNm

3-CLASS A

transverse ecc. = **6.25** -0.5 -0.4 -1.8 **-1.7** **-0.9** = **0.95** m
 Maximum Reaction (RA) = **909.344** KN Corr. MT = **863.877** kNm
 Minimum Reaction (RB) = **752.656** KN Corr. MT = **715.023** kNm

1- 70RWTransverse Ecc. for 70RW = **6.25** -0.5 -1.2 **-1.395** = **3.155** m

Maximum Reaction (RA) = **758.731** KN Corr. MT = **2394**
 Minimum Reaction (RB) = **241.269** KN Corr. MT = **761**

1-CLASS A + 1-70RW

Transverse Ecc. for Class A = **3 LANE** = **6.25** -0.5 -0.4 -0.9 = **4.45** m
 for 70RW = **6.25** -0.5 -0.4 -1.8 -0.25 **-1.8** -1.395 = **0.105**

Maximum Reaction (RA) = **1061.845** KN Corr. MT = **303.11 x 4.45 + 758.73 x 0.105** = **1429**
 Minimum Reaction (RB) = **492.155** KN Corr. MT = **250.885 x 4.45 + 241.269 x 0.105** = **1142**

1- 70RW + 1-CLASS A

Transverse Ecc. for 70RW = **3 LANE** = **6.25** -0.5 -1.2 **-1.395** = **3.155** m
 for Class A = **6.25** -0.5 -7.25 **-0.25** -0.9 = **-2.650** m

Maximum Reaction (RA) = **1061.845** KN Corr. MT = **758.73 x 3.155 + 303.11 x -2.650** = **1591**
 Minimum Reaction (RB) = **544.384** KN Corr. MT = **241.269 x 3.155 + 303.115 x -2.650** = **-42**

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	303.115	1348.860
2-CLASS A	606.229	1636.819
3-CLASS A	909.344	863.877
1- 70RW	758.731	2393.795
1-CLASS A + 1-70RW	1061.845	1428.527
1- 70RW + 1-CLASS A	1061.845	1590.541

Reaction RB, , (kN)	Corr. MT (kNm)
250.885	1116.440
501.771	1354.781
752.656	715.023
241.269	761.205
492.155	1141.773
544.384	-42.049

Maximum Reaction & Corresponding Transverse Moment

Reaction RA	=	1061.845	KN
Reaction RB	=	492.155	KN

& Corresponding moment	=	1428.527	kNm
& Corresponding moment	=	1141.773	kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA	=	758.731	KN
Reaction RB	=	241.269	KN

& Max moment	=	2393.795	kNm
& Max moment	=	761.205	kNm

DESIGN OF ABUTMENT(A1&A2
WITH OPEN FOUNDATION
FOR MAJOR BRIDGE AT CH-
16+463

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 0 m
Design speed of vehicle = 0 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 23.750m	23.750m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side) =	0.600m	0.600m
Slab projection Beyond C/L of Bearing (Span Side) =	0.600m	0.600m
Span -c/c of E.J.	= 25.000m	25.00m
Type of Superstructure	= PSC precast Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 7.500m	
Projection beyond crash barrier	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 1.500m	
Railing/kerb on footpath edge	= 0.500m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 4
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 350 kN/m² (as per geotechnical report)
SBC of soil-Normal Case = 422 kN/m²
SBC of soil-Seismic Case = 527 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1 Normal Case
= 1 Seismic Case
Permissible FOS against Overturning = 1 Normal Case
= 1 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 0.900m	
Height of Trapering portion	= 1.413m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 1.950m	1.950m
Width of Abutmentcap at bottom of Tapering Portion	= 1.000m	1.000m
Projection of Abutment Cap (Span Side)	= 0.500m	0.500m
Projection of Abutment Cap Back Side	= 0.450m	0.450m
Abutmentcap thickness (Uniform portion)	= 0.500m	

Abutment cap thickness (Tapering Portion)	=	0.500m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	1.138m	1.138m
Thickness of Abutment shaft at Bottom	=	1.200m	1.200m

Solid Return Wall

Length of Return wall	=	5.500m
Thickness of Return wall at Top	=	0.600m
Thickness of Return wall at Bottom	=	0.600m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.600m
Thickness of Return wall at Bottom	=	0.600m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	10.500m
abutment pedestal width	=	1.200m
abutment pedestal Height	=	0.000m
Width of Toe Slab	=	3.800m
Width of Heel Slab	=	5.500m
Thickness of Toe slab at tip	=	0.500m
Thickness of Toe slab near shaft	=	1.500m
Thickness of heel slab at tip	=	0.500m
Thickness of heel slab near shaft	=	1.500m
Width of backfill on heel slab	=	5.500m
Thickness of heel slab at back fill edge	=	1.500m
Height of back fill at bottom edge of heel slab	=	11.108m
Height of back fill at back fill edge of heel slab	=	10.108m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1487.055m	Cross Slope (Bi-directional)	=	2.500%	
Deck level at Outer Edge =	1486.911m	Height of Superstructure	=	1.820m	
Deck level at center line =	1487.055m	Min. Height of Footpath Side Pedestal (1)	=	0.300m	
Soffit Level at center of bridge =	1485.170m	Height of Pedestal (2)	=	0.375m	
Abutment cap top level =	1484.670m	Height of Pedestal (3)	=	0.450m	
Abutment cap bottom lvl (uniform portion ends)	1484.170m	Height of Pedestal (4)	=	0.000m	
Abutment cap bottom lvl (corbel portion ends)	1483.670m	Height of Pedestal (5)	=	0.000m	
Abutment shaft top level =	1483.670m	Height of Pedestal (6)	=	0.000m	
Ground level/LBL =	1478.968m	Distance of nearest girder to c.l. of deck	=	1.500m	
Abutment shaft bottom level =	1476.875m	Height (Avg.) of Dirt Wall	=	2.313m	
Foundation level =	1475.375m	Abutment shaft Above G.L	=	4.702m	
HFL	=	1478.968m	Abutment Shaft below G.L	=	2.093m
			Height of abutment shaft	=	6.795m
			MSL	=	1478.968m
			Wedge over girder flange	=	0.0250m

Material Specification

Concrete Grade	=	M 35
Characteristic compressive strength of concrete, f _{ck}	=	35.00 Mpa at 28 days
Design Compressive strength of Concrete, f _{cd}	=	15.63 Mpa at 28 (0.67/1.5 * f _{ck})
Tensile strength of concrete , f _{ctm}	=	2.77 MPa

Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E _{cm}	=	32308.250	N/mm ²
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f _y or f _{yk}	=	500	Mpa
Design Yield Strength of Reinforcement, f _{yd}	=	434.78	Mpa (1/1.15 * f _y)
Modulus of Elasticity of Steel (E _s)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48	f _{ck}
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm
<u>Seismic Data:</u>			
Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction
<u>Properties of backfill material :</u>			
c	=	0	
φ	=	35	
θ	=	90	
β	=	0	
δ	=	22.5	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.000m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

		eT
DL Reaction on Bearing No.-1	=	550.85 4.50
DL Reaction on Bearing No.-2	=	528.77 1.50
DL Reaction on Bearing No.-3	=	528.77 -1.50
DL Reaction on Bearing No.-4	=	550.85 -4.50
DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		2159.25 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	90.000 4.50
SIDL Reaction on Bearing No.-2	=	10.000 1.50
SIDL Reaction on Bearing No.-3	=	10.000 -1.50
SIDL Reaction on Bearing No.-4	=	90.000 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		200.00 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	76.079 4.50
SIDL Reaction on Bearing No.-2	=	82.995 1.50
SIDL Reaction on Bearing No.-3	=	82.995 -1.50
SIDL Reaction on Bearing No.-4	=	76.079 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		318.15 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 2-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	1059.33	KN	Corr. Transv	=	1006.366 kNm	ecc.	0.95
min	=	290.67	KN	Corr. Transv	=	276.134 kNm	ecc.	0.95

SV Loading

max	=	1926.19	KN	Corr. Transv	=	577.857 kNm	ecc.	0.30
min	=	1493.81	KN	Corr. Transv	=	448.143 kNm	ecc.	0.30

**MAXIMUM TRASVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	809.52	KN	Corr. Transv	=	2554.049 kNm	ecc.	3.16
min	=	190.48	KN	Corr. Transv	=	600.951 kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.076
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.076
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.300 x	0.800 x	0.800 =	0.1920 m ³	4.500
Pedestal 2	=	0.375 x	0.800 x	0.800 =	0.2400 m ³	1.500
Pedestal 3	=	0.450 x	0.800 x	0.800 =	0.2880 m ³	-1.500
Pedestal 4	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					0.7200 m ³	1.100

VOLUME CALCULATION

C.G. Of Footing	=	5.250 m
C.G. Of shaft from toe tip	=	4.400 m
Distance between c.g. of shaft and footing	=	0.850 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	0.900	3.375	-0.800	0.050	-5.200	0.000
-Trapering portion	1	12.50	0.300	1.413	5.299	-0.800	0.050	-5.200	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.100	-0.250	-5.500	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-1.050	-0.200	-5.450	0.000
Cap (uniform portion)	1	12.50	1.950	0.500	12.188	0.025	0.875	-4.375	0.000
Cap (Corbel Portion)	1	12.50	1.950	0.500	9.055	0.025	0.875	-4.375	0.000
		12.50	1.000						
Shaft above HFL	1	12.50	1.069	4.702	62.842	0.065	0.915	-4.335	0.000
Shaft below HFL	1	12.50	1.169	1.593	23.282	0.015	0.865	-4.385	0.000
Solid Return Wall	1	5.50	0.600	10.680	35.244	-3.350	-2.500	-7.750	5.950
Cantilever Return wall(Rectangular portion)	1	0.00	0.600	0.000	0.000	-0.600	0.250	-5.000	5.950
Cantilever Return wall(Traingular portion)	1	0.00	0.600	0.000	0.000	-0.600	0.250	-5.000	5.950
Footing									
Heel Slab	1	12.50	5.500	1.000	68.750		-2.042	-7.292	0.000
Toe Slab	1	12.50	3.800	1.000	47.500		3.033	-2.217	0.000
Portion between Heel and Toe	1	12.50	1.200	1.500	22.500		0.850	-4.400	0.000
Back filling above HFL over Heel Slab	1	12.50	5.500	8.087	555.981		-2.500	-7.750	0.000
Back filling below HFL over Heel Slab	1	12.50	5.500	2.593	178.269		-2.677	-7.927	0.000
Backfill above Heel slab	1	12.50	5.500	10.608	729.309		-2.543	-7.793	0.000
Front Filling over Toe Slab	1	12.50	3.800	2.593	123.168		3.228	-2.022	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	3.800	2.593	123.168		2.378	-2.872	0.000
Side filling between heel and toe	1	0.00	1.200	2.593	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-1.100	-0.250	-5.500	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.138	8.087	13.990		0.322	-4.928	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.062	2.593	1.997		0.271	-4.979	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barr	2	8 kN/m	1.750	28.00kN		-0.800	0.050	-5.200

SECTIONAL PROPERTIES

Width of Footing (B)	=	10.500 m
Length of Footing (L)	=	12.500 m

A	=	10.500	x	12.500	=	131.250 m ²
ZL	=	12.500	x	18.375	=	229.688 m ³
ZT	=	IT1	+	IT2		
		distance of extreme point from centre				
IT1	=	10.500	x	162.760	=	1708.98 m ⁴
IT2 (moment of inertia of triangle)	=	10.500	x	0.000	+	0.500 x 10.500 x 0.000 x 39.063
from centre of footing	=	0.000	m ⁴			
Moment of inertia of two triangle	=	0.000	m ⁴			
Total moment of inertia	=	1708.98	m ⁴			
Distance of extreme point from centre of footing	=	6.250	+	0.000	=	6.250 m
Total Section modulus (ZT)	=	273.438	m ³			

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.000	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.500
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Servicibility Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.00	1.00	1.00
wearing course	1.20	1.20	1.20
Back Filling Weight	1.00	1.00	1.00
Shrinkage Creep Effect	1.00	1.00	1.00
Earth Pressure due to back filling	1.00	1.00	1.00
CWLL and Associate load and FPLL	1.00	0.75	0.00
Live Load Surcharge	0.80	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.500
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

Horizontal Seismic Force For Zone 4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.) or horizontal vibrations.

$$= 2.0 \frac{D^{1/2}}{1000F}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 9.595 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 11.185 m

Abutment Cap Top Level - Foundation Level

= 9.295 m

Dimensions of Abutment Shaft

Length = 12.50 m

Width = 1.10 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 1.040 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 179.036 m^4

Ecm = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 119.708 KN
 D = 2677.40 KN
 T = 0.2991 sec

Transverse Direction

Force = 16558.332 KN
 D = 2947.398 KN
 T = 0.0267 sec

For Hard or Rocky Strata, with $N > 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.400

For Medium Strata, with $N > 10 \& N < 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.400

For Soft Soil Strata, with $N < 10$

$\frac{S_a}{g}$ = 2.5

$\frac{S_a}{g}$ = 1.400

Medium Soil Strata

Sa/g = 2.5

Sa/g = 1.400

Seismic Coeff. In Longitudinal Direction = 0.12

Seismic Coeff. In Transverse Direction = 0.202

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.202	In Transverse direction
Av	=	0.134	In Vertical direction

For Design of Foundation

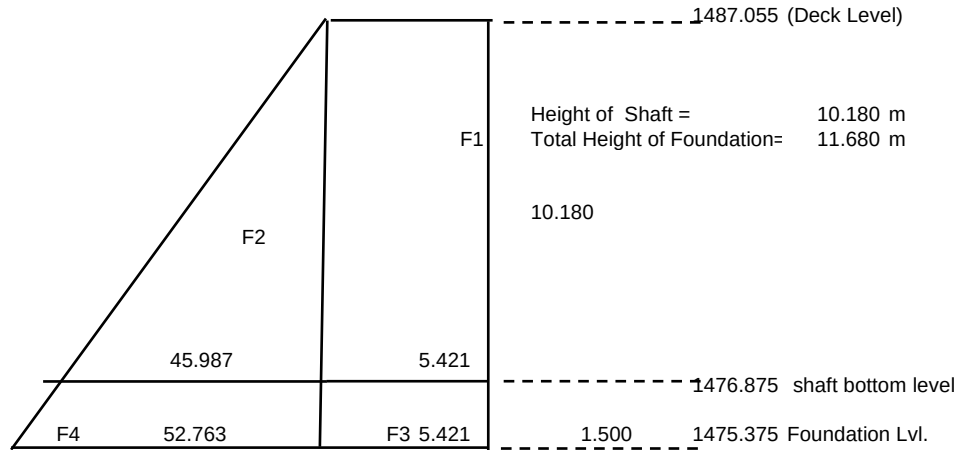
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.162	In Longitudinal direction
Ah	=	0.202	In Transverse direction
Av	=	0.134	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = **1.2 m**
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1476.875 m (at Shaft Base)			
@ RL					1475.375 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	10.180	x	12.500	=	689.798 kN
M1	=	689.80	x	5.09	=	3511.070 kN.m		at Shaft Bottom
F3	=	5.421	x	11.680	x	12.500	=	791.438 kN
M3	=	791.438	x	5.840	=	4621.997 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	10.180	= 45.987 kN/m ²
F2	=	0.5	x	45.99	x	10.180	x 12.50
	=	2925.892 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	2925.89	x	4.28	=	12509.943 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	11.680	= 52.763 kN/m ²
F4	=	0.5	x	52.76	x	11.680	x 12.50
	=	3851.664 kN					
M4	=	3851.66	x	4.91	=	18894.723 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	10.180	=	48.864 kN/m ²	
F	=	0.5	x	48.864	x	10.180	x 12.500
	=	3108.972 kN					

$$M = 3108.97 \times 3.393 = 10549.778 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 11.680 = 56.06 \text{ kN/m}^2$$

$$F = 0.5 \times 56.064 \times 11.68 \times 12.50 = 4092.672 \text{ kN}$$

$$M = 4092.67 \times 3.893 = 15934.136 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50$$

$$= 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

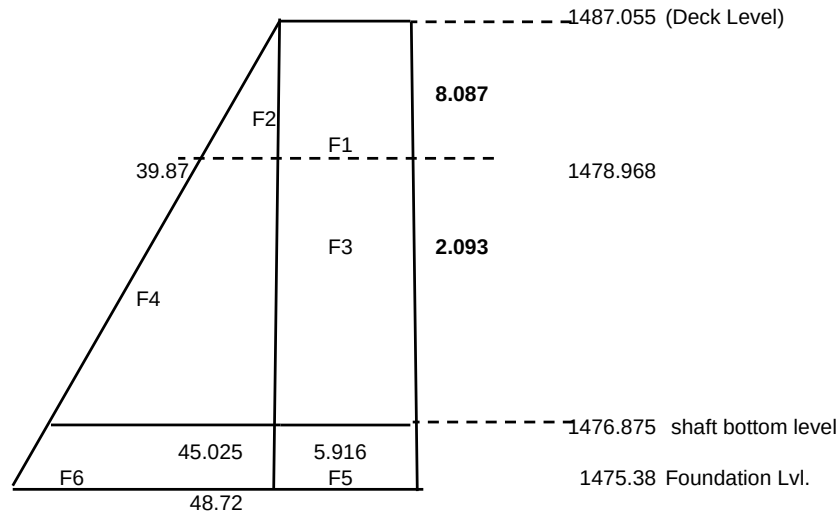
$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	12509.943	18894.723	2925.892	3851.664
Due to Minimum Fluid Pressure	10549.778	15934.136	3108.972	4092.672
Governing of Two	12509.943	18894.723	3108.972	4092.672
Due to Live Load Surcharge	3511.070	4621.997	689.798	791.438
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	11.3 degree	0.196 radians
	Ka'	=	0.246 active component	
	Kp'	=	5.249 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	
Assuming				

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1476.9 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m ²
F1	=	5.916	x	10.180	x	12.500	=	752.751 kN
M1	=	752.75	x	5.09	=	3831.501 kN.m		at Shaft Bottom
F3	=	5.916	x	11.680	x	12.500	=	863.667 kN
M3	=	863.67	x	5.84	=	5043.814 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	8.087	=	39.866 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	2.093	=	5.159 kN/m ²
F2	=	0.5	x	39.87	x	8.087	x	12.50
	=	2014.963 kN						
F4	=	$\frac{(39.87 + 45.02)}{2}$	x	2.09	x	12.50		
	=	1110.471 kN						
Total Force	=	3125.434 kN						
M2	=	2014.96	x	5.49	=	11061.219 kN.m		
M4	=	1110.47	x	1.03	=	1138.567 kN.m		
Total Moment	=	12199.79 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 8.087 = 39.866 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 3.593 = 8.856 \text{ kN/m}^2$$

$$F2 = 0.5 \times 39.87 \times 8.087 \times 12.50 = 2014.963 \text{ kN}$$

$$F6 = \left(\frac{39.87 + 48.72}{2} \right) \times 3.59 \times 12.50 = 1989.342 \text{ kN}$$

$$\text{Total Force} = 4004.305 \text{ kN}$$

$$M2 = 2014.96 \times 6.99 = 14083.663 \text{ kN.m}$$

$$M6 = 1989.34 \times 1.74 = 3454.762 \text{ kN.m}$$

$$\text{Total Moment} = 17538.42 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

$$= 5.249 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	12199.786	17538.425	3125.434	4004.305
Due to Minimum Fluid Pressure	10549.778	15934.136	3108.972	4092.672
Governing of Two	12199.786	17538.425	3125.434	4092.672
Due to Live Load Surcharge	3831.501	5043.814	752.751	863.667
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

αh	=	0.120	
αv	=	0.134	
ϕ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

αh	=	HORIZONTAL SEISMIC COEFFICIENT
αv	=	VERTICAL SEISMIC COEFFICIENT
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.105 \\ 0.138 \end{matrix}$$

$$C_a = \begin{matrix} 1 \\ 0.325 \end{matrix} \begin{matrix} 2 \\ 0.267 \end{matrix}$$

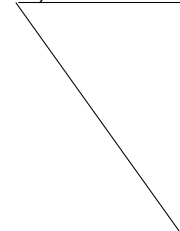
Ca	=	0.325	
Ka	=	0.226	
Dynamic Increment	=	0.325	-0.226 0.099

3 Earth Pressure : DRY CASE (Seismic case)

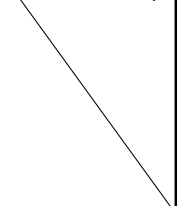
Equivalent Live Load Surcharge height	=	1.2 m
Assuming	γ_{dry}	= 20 kN/m ³
	γ_{water}	= 10.00 kN/m ³

Active Earth Pressure

$$3 (Ca - Ka) = 0.296$$

**Surcharge Earth Pressure**

$$2 (Ca - Ka) \frac{\cos a}{\cos(a-b)} = 0.20 \quad \text{--- 1487.055 (Deck Level)}$$

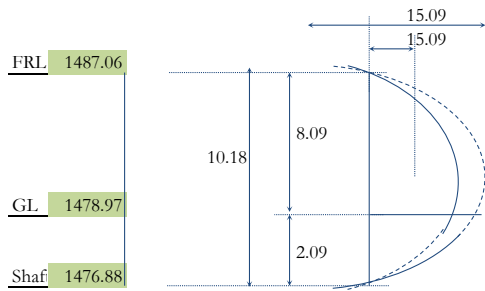


--- 1476.875 (Shaft bottom level)

--- 1475.375 (Foundation Bottom level)

Earth Pressure Diagram for Dynamic Increment**Due to Dynamic Live Load Surcharge**

	=	0.198	x	20.00	x	1.2	=	4.744 kN/m ²
at Shaft Bottom Level								
E1	=	0.50	x	4.744	x	10.180	x	12.500
	=	301.807	kN					
M1	=	301.807	x	6.821			=	2058.502 kN.m
at Foundation Bottom Level								
E2	=	0.50	x	4.744	x	10.180	x	12.500
	=	301.807	kN					
M2	=	301.807	x	8.321			=	2511.212 kN.m

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	15.09 t/m ²
h	=	10.18 m
y	=	2.997 m
L	=	12.500 m

Parabola below Water Level

p_mid_height	=	15.09 t/m ²
h	=	10.18 m
y	=	-2.997 m
L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	1140.1	5.5
Parabola below Water Level	140.1	1.4
Total Dynamic Earth Pressure	1280.2	5.1

$$E3 = 1280.163 \text{ kN}$$

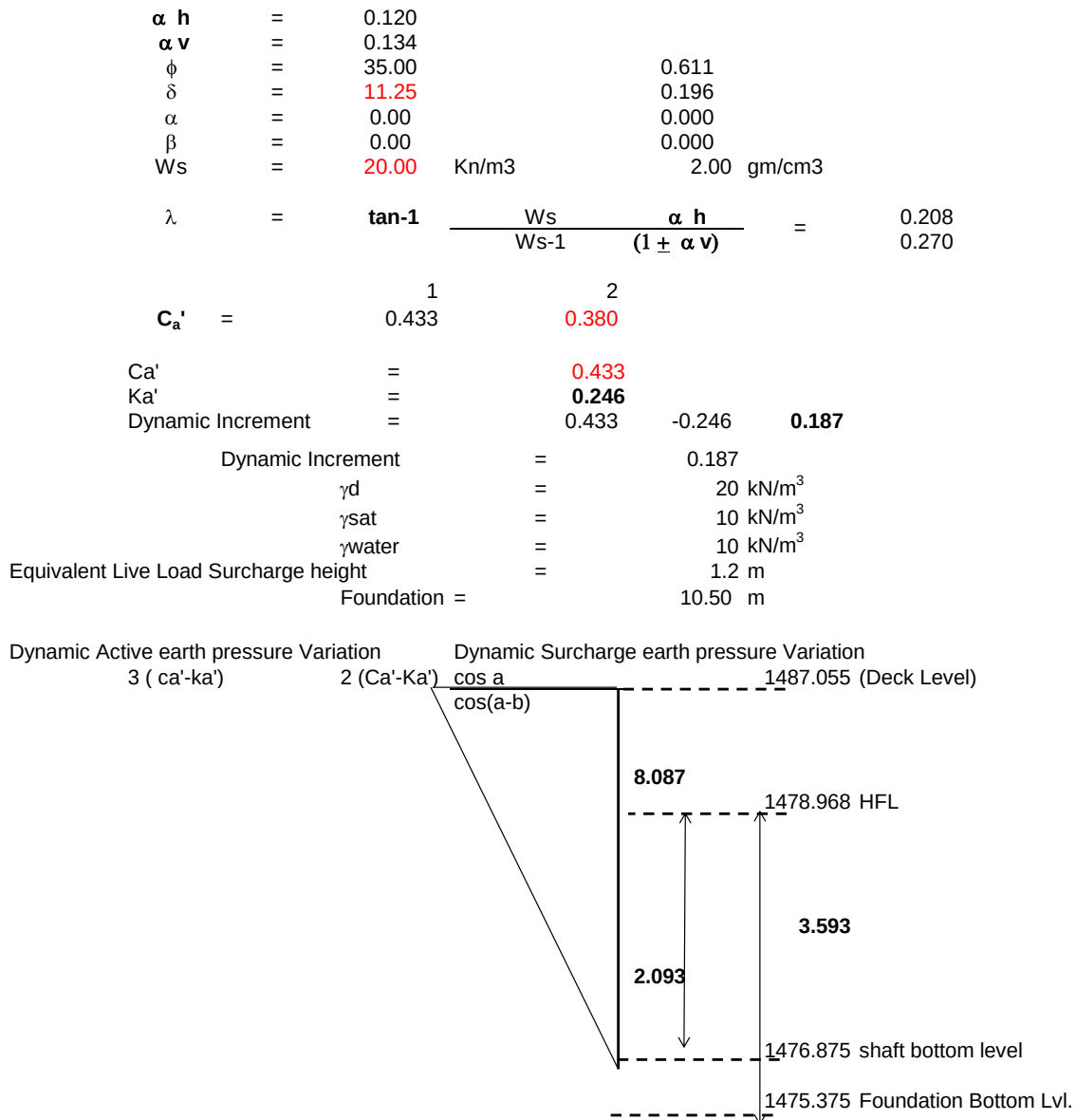
$$E4 = 1280.163 \text{ kN}$$

$$M3 = 1280.16 \times 5.09 = 6516.031 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 1280.16 \times 6.59 = 8436.276 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	12509.943	18894.723	2925.892	3851.664
Due to active Earth Pressure (dynamic Increment)	6516.031	8436.276	1280.163	1280.163
Total Earth Pressure	19025.974	27330.999	4206.055	5131.827
Due to Minimum Fluid Pressure	10549.778	15934.136	3108.972	4092.672
Governing of Two	12509.943	27330.999	4206.055	5131.827
Due to Live Load Surcharge (Static)	3511.070	4621.997	689.798	791.438
Due to Live Load Surcharge(Dynamic)	2058.502	2511.212	301.807	301.807
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case**Earth Pressure Diagram**

h'	=	2.093
h	=	10.180

Dynamic active earth pressure

$3 (ca - ka)$	=	0.296 at deck level
$3 (ca' - ka')$	=	0.561 at deck level
$3 (ca' - ka')h'/h$	=	0.115 at HFL level

Dynamic Surcharge earth pressure

$2 (Ca' - Ka') \frac{\cos a}{\cos(a-b)}$	=	0.374 at deck level	Pressure =	8.97 kN/m ²
	=	0.077 at HFL level	Pressure =	1.84 kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	4.744 kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	1.84 kN/m ²

at Shaft Bottom Level

F1	=	3.294	x	8.087	x	12.500	=	332.965 kN
M1	=	332.97	x	6.14	=	2043.242 kN.m		
F2	=	0.50	x	1.844	x	2.093	x	12.500
	=	24.124 kN						

$$M2 = 24.12 \times 1.40 = 33.829 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 357.089 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 2077.070 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

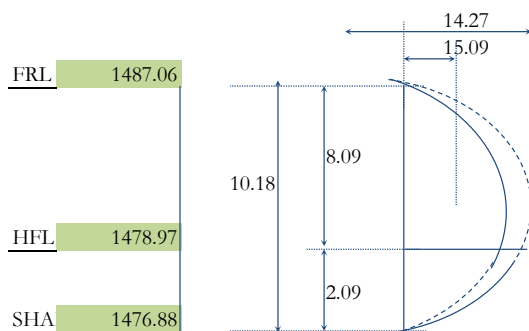
$$F3 = 3.294 \times 8.087 \times 12.500 = 332.965 \text{ kN}$$

$$M3 = 332.97 \times 7.64 = 2542.689 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 1.844 \times 2.093 \times 12.500 \\ &= 24.124 \text{ kN} \end{aligned}$$

$$M4 = 24.12 \times 2.90 = 70.014 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 357.089 \text{ kN} \\ \text{Total Moment at Founding level} &= 2612.704 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 15.09 \text{ t/m}^2 \\ h &= 10.18 \text{ m} \\ y &= 2.997 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 14.27 \text{ t/m}^2 \\ h &= 10.18 \text{ m} \\ y &= -2.997 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	1140.1	5.5
Parabola below Water Level	132.4	1.4
Total Dynamic Earth Pressure	1272.5	5.1

Total Force (F1 + F2)	=	1272.522 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	1272.522 kN	at Foundation Bottom Level

$$M1 = 1272.52 \times 5.11 = 6505.581 \text{ kN.m}$$

$$\text{Total Mome} = 6505.581 \text{ kN.m} \quad \text{at Shaft Bottom}$$

$$M1 = 1272.52 \times 6.61 = 8414.364 \text{ kN.m}$$

$$\text{Total Mome} = 8414.364 \text{ kN.m} \quad \text{at Foundation Bottom Level}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Bottom	At Shaft Bottom Lvl	At Foundation Bottom Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure(Static)	12199.786	17538.425	3125.434	4004.305
Due to active Earth Pressure (Dynamic Increment)	6505.581	8414.364	1272.522	1272.522
Total Earth Pressure	18705.367	25952.789	4397.956	5276.828
Due to Minimum Fluid Pressure	10549.778	15934.136	3108.972	4092.672
Governing of Two	18705.367	25952.789	4397.956	5276.828
Due to Live Load Surcharge(Static)	3831.501	5043.814	752.751	863.667
Due to Live Load Surcharge (Dynamic Increment)	2077.070	2612.704	357.089	357.089
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - SPHERICAL BEARING

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	2159.25	1.35	1.35	2914.99	2914.99	
SIDL except wc	=	200.00	1.35	1.35	270.00	270.00	
WC	=	318.15	1.75	1.75	556.76	556.76	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	1059.33	1.5	0.20	1589.00	211.87	(1-70RW + 2-CLASS A)
CWLLmax-Reaction case	=	1926.19	1	0.20	1926.19	385.24	SV Loading
CWLLmin	=	290.67	1.5	0.20	436.00	58.13	(1-70RW + 2-CLASS A)
CWLLmin	=	1493.81	1	0.20	1493.81	298.76	SV Loading
CWLLmax-Transv. Moment Case		809.52	1.5	0.20	1214.29	161.90	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 1108 = 255.4 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3741.75	0	187.087	187.087	-187.087	187.087	
DL+SIDL+LL-Max Reaction case	4177.75	383.1	208.887	400.437	174.213	400.437	(1-70RW + 2-CLASS A)
	5671.56	0	283.578	283.578	-283.578	283.578	SV Loading
DL+SIDL+LL-Min Reaction case	5330.74	383.1	266.537	458.087	116.563	458.087	(1-70RW + 2-CLASS A)
	5667.94	0	283.397	283.397	-283.397	283.397	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4956.03	383.1	247.802	439.352	135.298	439.352	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2677.40	3741.75	963.86	187.087	669.019	776.776	963.863	
DL+SIDL+LL-Max Reaction case		3799.88	1014.94	189.994	697.466	824.949	1014.943	Dry Case
DL+SIDL+LL-Min Reaction case		3953.61	1014.94	197.681	705.152	817.263	1014.943	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3903.65	1014.94	195.183	702.654	819.761	1014.943	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2677.40	3741.75	289.159	187.087	331.667	102.072	331.667	
DL+SIDL+LL-Max Reaction case		3799.88	340.239	189.994	360.113	150.245	360.113	Dry Case
DL+SIDL+LL-Min Reaction case		3953.61	340.239	197.681	367.800	142.558	367.800	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3903.65	340.239	195.183	365.302	145.056	365.302

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	2159.25	1.35	1.35	2914.99	2914.99
SIDL except wc	=	200.00	1.35	1.35	270.00	270.00
WC	=	318.15	1.75	1.75	556.76	556.76
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	1059.33	1.5	0.75	1589.00	794.50
CWLLmax-Transv. Moment Case	=	809.52	1.5	0.75	1214.29	607.14
CWLLmin	=	290.67	1.5	0.75	436.00	218.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3741.75	0.000	187.087	187.087	-187.087	187.087	
DL+SIDL+LL-Max Reaction case	4177.75	383.100	208.887	400.437	174.213	400.437	(1-70RW + 2-CLASS A)
	5671.56	0.000	283.578	283.578	-283.578	283.578	SV Loading
DL+SIDL+LL-Min Reaction case	5330.74	383.100	266.537	458.087	116.563	458.087	(1-70RW + 2-CLASS A)
	5667.94	0.000	283.397	283.397	-283.397	283.397	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4956.03	383.100	247.802	439.352	135.298	439.352	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2677.40	3741.75	1301.22	187.087	837.695	1114.128	1301.215	
DL+SIDL+LL-Max Reaction case		3959.75	1352.30	197.987	874.135	1154.308	1352.295	Dry Case
DL+SIDL+LL-Min Reaction case		4536.25	1352.30	226.812	902.960	1125.483	1352.295	HFL Case
DL+SIDL+LL-Max Transv. Moment case		4348.89	1352.30	217.444	893.592	1134.851	1352.295	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2677.40	3741.75	390.365	187.087	382.270	203.277	390.365	
DL+SIDL+LL-Max Reaction case		3959.75	441.445	197.987	418.710	243.457	441.445	Dry Case
DL+SIDL+LL-Min Reaction case		4536.25	441.445	226.812	447.535	214.632	447.535	HFL Case
DL+SIDL+LL-Max Transv. Moment case		4348.89	441.445	217.444	438.167	224.000	441.445	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	2159.25	1	1.00	2159.25	2159.25
SIDL except wc	=	200.00	1	1.00	200.00	200.00
WC	=	318.15	1	1.00	318.15	318.15
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	1059.33	1	0.20	1059.33	211.87
CWLLmax- Transv. Moment Case		809.52	1	0.20	809.52	161.90
CWLLmin	=	290.67	1	0.20	290.67	58.13

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2677.40	0.000	133.870	133.870	-133.870	133.870	
DL+SIDL+LL-Max Reaction case	2968.07	255.400	148.403	276.103	106.997	276.103	(1-70RW + 2-CLASS A)
	4171.21	0.000	208.560	208.560	-208.560	208.560	SV Loading
DL+SIDL+LL-Min Reaction case	3736.73	255.400	186.837	314.537	68.563	314.537	(1-70RW + 2-CLASS A)
	4603.59	0.000	230.179	230.179	-230.179	230.179	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3486.92	255.400	174.346	302.046	81.054	302.046	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2677.40	867.477	133.870	567.608	733.607	867.477	
DL+SIDL+LL-Max Reaction case	2735.53	918.557	136.777	596.055	781.780	918.557	Dry Case
DL+SIDL+LL-Min Reaction case	2889.26	918.557	144.463	603.742	774.094	918.557	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2839.30	918.557	141.965	601.244	776.592	918.557	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2677.40	260.243	133.870	263.991	126.373	263.991	
DL+SIDL+LL-Max Reaction case	2735.53	311.323	136.777	292.438	174.547	311.323	Dry Case
DL+SIDL+LL-Min Reaction case	2889.26	311.323	144.463	300.125	166.860	311.323	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2839.30	311.323	141.965	297.627	169.358	311.323	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	2159.25	1.1	1.10	2375.18	2375.18
SIDL except wc	=	200.00	1.1	1.10	220.00	220.00
WC	=	318.15	1.35	1.35	429.50	429.50
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	1059.33	1.5	0.00	1589.00	0.00
CWLLmax-Transv. Moment Case		809.52	1.5	0.00	1214.29	0.00
CWLLmin	=	290.67	1.5	0.00	436.00	0.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	3024.67	0.000	151.234	151.234	-151.234	151.234	
DL+SIDL+LL-Max Reaction case	3460.68	383.100	173.034	364.584	210.066	364.584	(1-70RW + 2-CLASS A)
	4518.49	0.000	225.924	225.924	-225.924	225.924	SV Loading
DL+SIDL+LL-Min Reaction case	4613.67	383.100	230.684	422.234	152.416	422.234	(1-70RW + 2-CLASS A)
	4950.86	0.000	247.543	247.543	-247.543	247.543	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4238.96	383.100	211.948	403.498	171.152	403.498	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2677.40	3024.67	1301.22	151.234	801.841	1149.982	1301.215	
DL+SIDL+LL-Max Reaction case		3024.67	1301.22	151.234	801.841	1149.982	1301.215	Dry Case
DL+SIDL+LL-Min Reaction case		3024.67	1301.22	151.234	801.841	1149.982	1301.215	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3024.67	1301.22	151.234	801.841	1149.982	1301.215	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2677.40	3024.67	390.365	151.234	346.416	239.131	390.365	
DL+SIDL+LL-Max Reaction case		3024.67	390.365	151.234	346.416	239.131	390.365	Dry Case
DL+SIDL+LL-Min Reaction case		3024.67	390.365	151.234	346.416	239.131	390.365	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3024.67	390.365	151.234	346.416	239.131	390.365	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	2159.25	1	1	1	2159.25	2159.25	2159.25
SIDL except wc	=	200.00	1	1	1	200.00	200.00	200.00
WC	=	318.15	1.20	1.20	1.20	381.78	381.78	381.78
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	1059.33	1	0.75	0	1059.33	794.50	0.00
CWLLmax-Transv. Moment Case	=	809.52	1	0.75	0	809.52	607.14	0.00
CWLLmin	=	290.67	1	0.75	0	290.67	218.00	0.00

Braking Force = 255.4 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2741.03	0.000	137.051	137.051	-137.051	137.051	
DL+SIDL+LL-Max Reaction case	3031.70	255.400	151.585	279.285	103.815	279.285	(1-70RW + 2-CLASS A)
	4234.84	0.000	211.742	211.742	-211.742	211.742	SV Loading
DL+SIDL+LL-Min Reaction case	3800.36	255.400	190.018	317.718	65.382	317.718	(1-70RW + 2-CLASS A)
	4667.22	0.000	233.361	233.361	-233.361	233.361	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3550.55	255.400	177.528	305.228	77.872	305.228	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2741.03	0.000	137.051	137.051	-137.051	137.051	
DL+SIDL+LL-Max Reaction case	2959.03	191.550	147.951	243.726	43.599	243.726	Dry Case
DL+SIDL+LL-Min Reaction case	3535.53	191.550	176.776	272.551	14.774	272.551	HFL Case
DL+SIDL+LL-Max Transv. Moment case	3348.17	191.550	167.409	263.184	24.141	263.184	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2741.03	0.000	137.051	137.051	-137.051	137.051	
DL+SIDL+LL-Max Reaction case	2741.03	0.000	137.051	137.051	-137.051	137.051	Dry Case
DL+SIDL+LL-Min Reaction case	2741.03	0.000	137.051	137.051	-137.051	137.051	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2741.03	0.000	137.051	137.051	-137.051	137.051	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	0.00	kmph	0.00	kmph
Live Load	W	=	1059.33	kN	1059.33	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

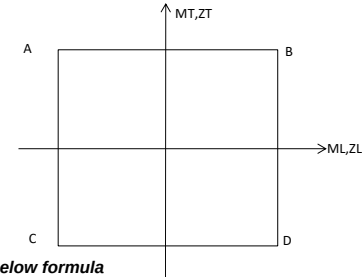
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1475.375 m

Properties of Footing Base:

A	=	131.250	m ²
ZL	=	229.688	m ³
ZT	=	273.438	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			2159.251	0.850	1835.363	0.000	0.000
SIDL except Wearing Course	1			200.000	0.850	170.000	0.000	0.000
Wearing Course	1			318.148	0.850	270.425	0.000	0.000
Bearing Pedestal	1	25	0.720	18.000	0.850	15.300	1.100	19.800
				2695.398		2291.088		19.800
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	3.375	84.375	0.050	4.219	0.000	0.000
Dirt Wall-Tapered portion	1	25	5.299	132.480	0.050	6.624	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	-0.250	-7.031	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	-0.200	-2.813	0.000	0.000
Cap - (uniform portion)	1	25	12.188	304.688	0.875	266.602	0.000	0.000
Cap - (corbel portion)	1	25	9.055	226.376	0.875	198.079	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.250	0.000	5.950	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.250	0.000	5.950	0.000
RCC Railing or Crash Barrier	1			28.000	0.050	1.400	0.000	0.000
Approach Slab	1	25	6.563	164.063	-0.250	-41.016	0.000	0.000
				982.169		426.064		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	35.244	881.100	-2.500	-2202.750	5.950	5242.545
Abutment Shaft	1	25	86.124	2153.094	0.865	1862.999	0.000	0.000
Back filling over heel slab	1	20	729.309	14586.172	-2.543	-37095.638	0.000	0.000
Front Filling over toe slab	1	20	123.168	2463.350	3.228	7951.389	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	68.750	1718.750	-2.042	-3509.115	0.000	0.000
Toe slab	1	25	47.500	1187.500	3.033	3602.083	0.000	0.000
portion between heel & toe	1	25	22.500	562.500	0.850	478.125	0.000	0.000
Vertical Components of active earth pressure	1			1695.240	-5.250	-8900.011	0.000	0.000
				25567.443		-37718.270		5242.545
Total				29245.010		-35001.117		5262.345

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.900	1943.326	-4.400	-8550.632
0.900	180.000	-4.400	-792.000
1.000	318.148	-4.400	-1399.849
0.900	16.200	-4.400	-71.280
	2457.673		-10813.761
0.900	75.938	-5.200	-394.875
0.900	119.232	-5.200	-620.009
0.900	25.313	-5.500	-139.219
0.900	12.656	-5.450	-68.977
0.900	274.219	-4.375	-1199.707
0.900	203.739	-4.375	-891.356
0.900	0.000	-5.000	0.000
0.900	0.000	-5.000	0.000
0.900	25.200	-5.200	-131.040
0.900	147.656	-5.500	-812.109
	883.952		-4257.292
0.900	792.990	-7.750	-6145.673
0.900	1937.785	-4.385	-8496.671
0.900	13127.555	-7.793	-102305.736
0.900	2217.015	-2.022	-4483.079
0.900	0.000	0.000	0.000
0.900	1546.875	-7.292	-11279.297
0.900	1068.750	-2.217	-2369.063
0.900	506.250	-4.400	-2227.500
0.900	1525.716	-10.500	-16020.020
	23010.699		-154745.277
	26352.324		-169816.330

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	133.870	1485.170	1311.256
due to Earth pressure	1	4092.672		18894.723

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		151.234	1485.170	1481.334
due to Earth pressure	1.5	6139.008		28342.084
		6290.242		29823.419

Summary of Forces For SBC

P	29245.010	kN
ML	-14795.139	kNm
MT	5262.345	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		29245.010		-35001.117		5262.345
CWLL-Max. Reaction case	1	1059.332	0.850	900.432	0.950	1006.366
Vertical Components of LL Surcharge	1	327.824	-5.250	-1721.077	0.000	0.000
Total		30632.167		-35821.762		6268.711

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	276.103	1485.170	2704.432
due to Earth pressure	1	4092.672		18894.723
due to Live load surcharge	1	791.438		4621.997

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		364.584	1485.170	3571.099
due to Earth pressure	1.5	6139.008		28342.084
due to Live load surcharge	1.2	949.725		5546.396
		7453.317		37459.579

Summary of Forces For SBC

P	30632.167	kN
ML	-9600.611	kNm
MT	6268.711	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
133.87	1311.26	0.00	0.00
4092.67	18894.72	0.00	0.00
4226.542	20205.979	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
151.23	1481.33	0.00	0.00
6139.01	28342.08	0.00	0.00
6290.242	29823.419	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	26352.32394		-169816.33
0.000		-4.400	0.00
0.900		-10.500	-3097.94
	26352.324		-172914.27

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
276.10	2704.43	0.00	0.00
4092.67	18894.72	0.00	0.00
791.438	4621.997	0.000	0.000
5160.21	26221.15	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
364.58	3571.10	0.00	0.00
6139.01	28342.08	0.00	0.00
949.73	5546.40	0.00	0.00
7453.317	37459.579	0.000	0.000

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				2695.398		2291.088		19.800
Substructure & Foundation -Portion 1				982.169		426.064		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	35.244	881.100	-2.500	-2202.750	5.950	5242.545
Shaft above HFL	1	25	62.842	1571.053	0.915	1436.971	0.000	0.000
Shaft below HFL	1	15	23.282	349.225	0.865	302.172	0.000	0.000
Back filling above HFL over heel slab	1	20	555.981	11119.625	-2.500	-27799.063	0.000	0.000
Back filling below HFL over heel slab	1	10	178.269	1782.688	-2.677	-4771.823	0.000	0.000
Front Filling over toe slab	1	10	123.168	1231.675	2.378	2928.771	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	68.750	1031.250	-2.042	-2105.469	0.000	0.000
Toe slab	1	15	47.500	712.500	3.033	2161.250	0.000	0.000
Portion between Heel & Toe	1	15	22.500	337.500	0.850	286.875	0.000	0.000
Vertical Components of active earth pressure	1			1695.240	-5.250	-8900.011	0.000	0.000
				21011.625		-38567.718		5242.545
Total				24689.192		-35850.565		5262.345

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	2457.673		-10813.761
	883.952		-4257.292
0.900	792.990	-7.750	-6145.673
0.900	1413.947	-4.335	-6129.951
0.900	314.302	-4.385	-1378.132
0.900	10007.663	-7.750	-77559.384
0.900	1604.419	-7.927	-12717.839
0.900	1108.508	-2.872	-3183.771
0.900	0.000	0.000	0.000
0.900	928.125	-7.292	-6767.578
0.900	641.250	-2.217	-1421.438
0.900	303.750	-4.400	-1336.500
0.900	1525.716	-10.500	-16020.020
	18910.462		-133990.872
	22252.087		-149061.925

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	133.870	1485.170	1311.256
due to Earth pressure	1	4092.672		17538.425

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
133.87	1311.26	0.00	0.00
4092.67	17538.42	0.00	0.00
4226.54	18849.68	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		151.234	1485.170	1481.334
due to Earth pressure	1.5	6139.008		26307.637
		6290.242		27788.972

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
151.23	1481.33	0.00	0.00
6139.01	26307.64	0.00	0.00
6290.24	27788.97	0.00	0.00

Summary of Forces For SBC

P	24689.192	kN
ML	-17000.885	kNm
MT	5262.345	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		24689.192		-35850.565		5262.345
CWLL-Min. Reaction case	1	290.668	0.850	247.068	0.950	276.134
Vertical Components of LL Surcharge	1	357.743	-5.250	-1878.148	0.000	0.000
Total		25337.602		-37481.646		5538.479

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.000	22252.087	-4.400	-149061.925
0.900	0.000	-10.500	0
	321.968		-3380.667
	22574.056		-152442.592

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	314.537	1485.170	3080.885
due to Earth pressure	1	4092.672		17538.425
due to Live load surcharge	1	863.667		5043.814

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
314.54	3080.89	0.00	0.00
4092.67	17538.42	0.00	0.00
863.667	5043.814	0.000	0.000
5270.88	25663.12	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		422.234	1485.170	4135.779
due to Earth pressure	1.5	6139.008		26307.637
due to live load surcharge	1.2	1036.400		6052.577
		7597.642		36495.993

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
422.23	4135.78	0.00	0.00
6139.01	26307.64	0.00	0.00
1036.400	6052.577	0.000	0.000
7597.64	36495.99	0.00	0.00

Summary of Forces For SBC

P	25337.602	kN
ML	-11818.522	kNm
MT	5538.479	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1

ah= 0.162 In Longitudinal direction
ah= 0.202 In Transverse direction
av= 0.134 In Vertical direction

Weight of shaft below Ground level

= 715.874 KN

Weight of back fill below Ground level

= 2877.875 KN

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			2159.251		130.615	87.077	0.850	1835.363	74.015	1486.383		0.000	0.000	1437.852
SIDL except Wearing Course	1			200.000		12.098	8.065	0.850	170.000	6.856	1487.504		0.000	0.000	146.738
Wearing Course	1			318.148		19.245	12.830	0.850	270.425	10.906	1487.055		0.000	0.000	224.782
Bearing Pedestal	1	25	0.720	18.000		1.089	0.726	0.850	15.300	0.617			1.100	19.800	
				2695.398		163.047	108.698		2291.088	92.393				19.800	1809.372
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	3.375	84.375	13.669	5.104	3.403	0.050	4.219	0.170	1486.605	153.500	0.000	0.000	57.317
Dirt Wall-Tapered portion	1	25	5.299	132.480	21.462	8.014	5.343	0.050	6.624	0.267	1485.448	216.194	0.000	0.000	80.727
Bracket - Uniform portion	1	25	1.125	28.125				-0.250	-7.031						
Bracket - Tapered portion	1	25	0.563	14.063				-0.200	-2.813						
Cap - (uniform portion)	1	25	12.188	304.688	49.359	18.431	12.287	0.875	266.602	10.751	1484.420	446.456	0.000	0.000	166.707
Cap - (corbel portion)	1	25	9.055	226.376	36.673	13.694	9.129	0.875	198.079	7.988	1483.920	313.370	0.000	0.000	117.013
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.250	0.000	0.000	1487.055	0.000	5.950	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.250	0.000	0.000	1487.055	0.000	5.950	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.050	1.400				0.000	0.000	
Approach Slab	1	25	6.563	164.063				-0.250	-41.016				0.000	0.000	

				982.169	121.163	45.242	30.162		426.064	19.177		1129.520		0.000	421.763
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	35.244	881.100	142.738	53.298	35.532	-2.500	-2202.750	-88.831	1481.751	910.090	5.950	5242.545	339.828
Abutment Shaft	1	25	86.124	2153.094	232.830	86.939	57.959	0.865	1862.999	50.150	1481.319	1383.940	0.000	0.000	516.763
Back filling over heel slab	1	20	729.309	14586.172	0.000	0.000	0.000	-2.543	-37095.638	0.000	1481.751	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	123.168	2463.350				3.228	7951.389				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	68.750	1718.750				-2.042	-3509.115				0.000	0.000	
Toe slab	1	25	47.500	1187.500				3.033	3602.083				0.000	0.000	
portion between heel & toe	1	25	22.500	562.500				0.850	478.125				0.000	0.000	
Vertical component of active earth pressure	1			1595.411				-5.250	-8375.910				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			530.261				-5.250	-2783.870				0.000	0.000	
				25997.875	375.568	140.237	93.491		-39978.039	-38.681		2294.029		5242.545	856.591
Total =				29675.442	496.731	348.526	232.351		-37260.886	72.889		3423.550		5262.345	3087.726
								-232.351		-72.889					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Superstructure								
Dead Load	0.9			1943.326	78.369	-4.400	-8550.632	-344.823
SIDL except Wearing Course	0.9			180.000	7.259	-4.400	-792.000	-31.939
Wearing Course	1.00			318.148	12.830	-4.400	-1399.849	-56.452
Bearing Pedestal	0.9	25	0.72	16.200		-4.400	-71.280	
				2457.673	98.458		-10813.761	-433.214
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	3.375	75.938	3.062	-5.200	-394.875	-15.924
Dirt Wall-Tapered portion	0.9	25	5.299	119.232	4.808	-5.200	-620.009	-25.003
Bracket - Uniform portion	0.9	25	1.125	25.313				
Bracket - Tapered portion	0.9	25	0.563	12.656				
Cap - (uniform portion)	0.9	25	12.188	274.219	11.058	-4.375	-1199.707	-48.381
Cap - (corbel portion)	0.9	25	9.055	203.739	8.216	-4.375	-891.356	-35.946
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-5.000	0.000	0.000
Cantilever Return Wall-Triangle portion	0.9	25	0.000	0.000	0.000	-5.000	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-5.200	-131.040	
Approach Slab	0.9	25	6.563	147.656		-5.500	-812.109	
				883.952	27.145		-4049.096	-125.254
Substructure & Foundation -Portion 2								
Abutment Shaft	0.9	25	86.124	1937.785	49.276	-4.385	-8496.671	-216.063
Solid Return wall	0.9	25	35.244	792.990	31.979	-7.750	-6145.673	-247.838
Back filling over heel slab	0.9	20	729.309	13127.555	0.000	-7.793	-102305.7	0.000
Front Filling over toe	0.9	20	123.168	2217.015		-2.022	-4483.079	
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000	
Heel slab	0.9	25	68.750	1546.875		-7.292	-11279.297	
Toe slab	0.9	25	47.500	1068.750		-2.217	-2369.063	
portion between heel & toe	0.9	25	22.500	506.250		-4.400	-2227.500	
Vertical component of active earth pressure	0.9			1525.716		-10.500	-16020.020	
Vertical component of dynamic increment of earth pressure	0.9			477.235		-10.500	-5010.966	
				23487.934	81.255		-159756.24	-463.901
Total =				26829.559	206.859		-174619.10	-1022.370
					-206.859	1022.370		

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	13.669	1486.605	153.500
1.0	21.462	1485.448	216.194
1.0	49.359	1484.420	446.456
1.0	36.673	1483.920	313.370
1.0	0.000	1487.055	0.000
1.0	0.000	1487.055	0.000
	121.163		1129.520
1.0	219.944	1481.319	1307.347
1.0	142.738	1481.751	910.090
1.0	0.000	1481.751	0.000
	362.682		2217.437
	483.845		3346.957

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	867.477	163.047	1485.170	8496.937	1809.372
due to Substructure	1	496.731	185.479		3423.550	1278.354
due to Earth pressure	1	5131.827			27330.999	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1301.215	1485.170	12745.41
due to Substructure	1.5	725.768		5020.44
due to Active Earth pressure	1	3851.664		18894.72
due to dynamic Earth pressure	1.5	1920.245		12654.41
		7798.892		49314.977

Summary of Forces For SBC

	Downward	Upward	
P	29907.793	29443.092	kN
ML	2063.488	1917.710	kNm
MT	8350.071	8350.071	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
867.48	8496.94	0.00	0.00	0.00	0.00	163.05	1809.37
496.73	3423.55	0.00	0.00	0.00	0.00	185.48	1278.35
5131.83	27331.00						
6496.03	39251.49	0.00	0.00	0.00	0.00	348.53	3087.73

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1301.22	12745.41	0.00	0.00
725.77	5020.44	0.00	0.00
3851.664	18894.723	0.000	0.000
1920.245	12654.413	0.000	0.000
7798.89	49314.98	0.00	0.00

Summary of Restoring Forces

Vertical Load	26622.700	kN
Moment	-175641.471	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				2695.398		163.047	108.698		2291.088	92.393				19.800	1809.372
Forces from Substructure				26980.044	496.731	185.479	123.653		-39551.975	-19.504		3423.550		5242.545	1278.354
CWLL-Max. Reaction case	0.20			211.87		12.816	8.544	0.850	180.086	7.262	1488.255		0.950	201.273	165.070
Vertical component of LL Surcharge	0.20			65.56				-5.250	-344.215				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			25.00				-5.250	-131.263				0.000	0.000	
Total =				29977.876	496.731	361.342	240.895		-37556.278	80.151		3423.550		5463.618	3252.796
							-240.895			-80.151					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Forces from Superstructure				2457.673	98.458	0.000	-10813.76	-433.214
Forces from Substructure				24371.886	108.401	0.000	-163805.34	-589.156
CWLL-Max. Reaction case	0.00			0.00	0.000	-4.400	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-10.500	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-10.500	0.00	
Total =				26829.559	206.859		-174619.10	-1022.370
					-206.859			1022.370

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	918.557	175.863	1485.170	8997.266	1974.441
due to Substructure	1	496.731	185.479		3423.550	1278.354
due to Earth pressure	1	5131.827			27330.999	
due to Live load surcharge	0.20	218.649			1426.642	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1301.215	1485.170	12745.405
due to Substructure	1.5	725.768		5020.436
due to Active Earth pressure	1	3851.664		18894.723
due to dynamic Earth pressure	1.5	1920.245		12654.4134
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		7798.892		49314.977

Summary of Forces For SBC

	Downward	Upward	
P	30218.771	29736.981	kN
ML	3702.329	3542.026	kNm
MT	8716.414	8716.414	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2695.398		163.047	108.698		2291.088	92.393				19.800	1809.372
Substructure & Foundation -Portion 1				982.169	121.163	45.242	30.162		426.064	19.177		1129.520		0.000	421.763
Substructure & Foundation -Portion 2															
Solid Return wall	1		35.24	881.10	142.738	53.298	35.532	-2.50	-2202.750	-88.831	1481.75	910.090	5.95	5242.545	339.828
Shaft above HFL	1	25	62.842	1571.053	254.511	95.034	63.356	0.915	1436.971	57.949	1481.319	1512.811	0.000	0.000	564.884
Shaft below HFL	1	15	23.282	349.225	-21.681	-8.096	-5.397	0.865	302.172	-4.670	1478.968	-77.899	0.000	0.000	-29.088
Back filling above HFL over heel slab	1	20	555.981	11119.625	0.000	0.000	0.000	-2.500	-27799.063	0.000	1483.012	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	178.269	1782.688	0.000	0.000	0.000	-2.677	-4771.823	0.000	1477.547	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	123.168	1231.675				2.378	2928.771				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	68.750	1031.250				-2.042	-2105.469				0.000	0.000	
Toe slab	1	15	47.500	712.500				3.033	2161.250				0.000	0.000	
portion between heel & toe	1	15	22.500	337.500				0.850	286.875				0.000	0.000	
Vertical component of active earth pressure	1			1658.638				-5.250	-8707.847				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			527.096				-5.250	-2767.254				0.000	0.000	
				21502.118	424.131	158.370	105.580		-41238.166	-35.552		2621.454		5242.545	978.852
Total =				25179.685	545.293	366.660	244.440		-38521.014	76.018		3750.975		5262.345	3209.986

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
918.56	8997.27	0.00	0.00	0.00	0.00	175.86	1974.44
496.73	3423.55	0.00	0.00	0.00	0.00	185.48	1278.35
5131.83	27331.00						
218.65	1426.64						
6765.76	41178.46	0.00	0.00	0.00	0.00	361.34	3252.80

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1301.22	12745.41	0.00	0.00
725.77	5020.44	0.00	0.00
3851.664	18894.723		
1920.245	12654.413		
0.000	0.000		
0.000	0.000		
7798.89	49314.98	0.00	0.00

Summary of Restoring Forces

Vertical Load	26622.700	kN
Moment	-175641.471	kNm

-244.440

-76.018

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxL2	MLS due to Fv
Superstructure				2457.673	98.458		-10813.761	-433.214
Substructure & Foundation -Portion 1				883.952	27.145		-4049.096	-125.254
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	35.244	792.990	31.979	-7.75	-6145.673	-247.838
Shaft above HFL	0.9	25	62.842	1413.947	57.021	-4.335	-6129.951	-247.204
Shaft below HFL	0.9	15	23.282	314.302	-7.744	-4.385	-1378.132	33.957
Back filling above HFL over heel slab	0.9	20	555.981	10007.663	0.000	-7.750	-77559.384	0.000
Back filling below HFL over heel slab	0.9	10	178.269	1604.419	0.000	-7.927	-12717.839	0.000
Front Filling ovetoe	0.9	10	123.168	1108.508		-2.872	-3183.771	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	68.750	928.125		-7.292	-6767.578	
Toe slab	0.9	15	47.500	641.250		-2.217	-1421.438	
portion between heel & toe	0.9	15	22.500	303.750		-4.400	-1336.500	
Vertical component of active earth pressure	0.9			1492.774		-10.500	-15674.125	
Vertical component of dynamic increment of earth pressure	0.9			474.386		-10.500	-4981.057	
				19351.906	92.135		-138626.03	-514.744
Total =				22693.531	217.739		-153488.89	-1073.213

-217.739

1073.213

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLS due to FL
	121.163		1129.520
1.0	142.738	1481.751	910.090
1.0	254.511	1481.319	1512.811
1.0	-34.567	1478.968	-124.198
1.0	0.000	1483.012	0.000
1.0	0.000	1477.547	0.000
	362.682		2298.703
	483.845		3428.223

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	867.477	163.047	1485.170	8496.937	1809.372
due to Substructure	1	545.293	203.613		3750.975	1400.615
due to Earth pressure	1	5276.828			25952.789	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1301.215	1485.170	12745.405
due to Substructure	1.5	725.768		5142.335
due to Active Earth pressure	1	4004.305		17538.425
due to dynamic Earth pressure	1.5	1908.783		12621.547
		7940.072		48047.712

Summary of Forces For SBC

	Downward	Upward	
P	25424.125	24935.245	kN
ML	-244.295	-396.331	kNm
MT	8472.331	8472.331	kNm

Forces along Long. Axis

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
867.48	8496.94	0.00	0.00	0.00	0.00	163.05	1809.37
545.29	3750.97	0.00	0.00	0.00	0.00	203.61	1400.61
5276.83	25952.79						
6689.60	38200.70	0.00	0.00	0.00	0.00	366.66	3209.99

Forces along Long. Axis

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1301.22	12745.41	0.00	0.00
725.77	5142.33	0.00	0.00
4004.305	17538.425		
1908.783	12621.547		
7940.07	48047.71	0.00	0.00

Summary of Restoring Forces

Vertical Load	22475.793	kN
Moment	-154562.104	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure	0.20			2695.398		163.047	108.698		2291.088	92.393				19.800	1809.372
Forces from Substructure				22484.287	545.293	203.613	135.742		-40812.102	-16.375		3750.975		5242.545	1400.615
CWLL-Max. Reaction case				58.13		3.517	2.344	0.850	49.414	1.993	1488.255		0.950	55.227	45.293
Vertical component of LL Surcharge				71.55				-5.250	-375.630				0.000	0.000	
Vertical component of dynamic increment LL Surcharge				29.58				-5.250	-155.307				0.000	0.000	
Total =				25338.949	545.293	370.176	246.784		-39002.537	78.011		3750.975		5317.572	3255.280
-246.784															
-78.011															

MT	8572.852	8572.852	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x α_h x P (kN)	FT = α_h x P (kN)	Fv = 0.3 x α_v x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				2695.398		543.490	108.698		2291.088	92.393				19.800	6031.238
Substructure & Foundation -Portion 1				982.169	36.349	150.808	30.162		426.064	19.177		338.856		0.000	1405.877
Substructure & Foundation -Portion 2				25997.875	112.670	467.457	93.491		-39978.039	-38.681		688.209		5242.545	2855.304
Total =				29675.442	149.019	1161.754	232.351		-37260.886	72.889		1027.065		5262.345	10292.420
										-232.351		-72.889			

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Superstructure				2457.673	98.458		-10813.76	-433.214
Substructure & Foundation -Portion 1				883.952	27.145		-4049.10	-125.254
Substructure & Foundation -Portion 2				23487.934	81.255		-159756.24	-463.901
Total =				26829.559	206.859		-174619.10	-1022.370
					-206.859			1022.370

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x α_h x P (kN)	C.g. of Force (m)	MLs due to FL
	121.163		1129.520
	362.682		2217.437
	483.845		3346.957

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.991	543.490	1485.170	2585.796	6031.238
due to Substructure	1	149.019	618.265		1027.065	4261.181
due to Earth pressure	1	5131.827			27330.999	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.99	2585.80	0.00	0.00	0.00	0.00	543.49	6031.24
149.02	1027.06	0.00	0.00	0.00	0.00	618.26	4261.18
5131.83	27331.00						
5544.84	30943.86	0.00	0.00	0.00	0.00	1161.75	10292.42

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		390.365	1485.170	3823.622
due to Substructure	1.5	725.768		5020.43567
due to Active Earth pressure	1	3851.664		18894.7229
due to dynamic Earth pressure	1.5	576.073		3796.32402
		5543.870		31535.104

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
390.36	3823.62	0.00	0.00
725.77	5020.44	0.00	0.00
3851.664	18894.723		
576.073	3796.324		
5543.87	31535.10	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	29907.793	29443.092	kN
ML	-6244.137	-6389.916	kNm
MT	15554.765	15554.765	kNm

Summary of Restoring Forces

Vertical Load	26622.700	kN
Moment	-175641.471	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Total =	29977.876	240.895	-37556.28	80.151	5463.618
		-240.895		-80.151	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	311.323	586.210	1485.170	3049.410	6581.471
due to Substructure	1	149.019	618.265		1027.065	4261.181
due to Earth pressure	1	5131.827			27330.999	
due to Live load surcharge	0.2	218.649			1426.642	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		390.36	1485.170	3823.622
due to Substructure	1.5	217.73		1506.131
due to Active Earth pressure	1	3851.66		18894.723
due to dynamic Earth pressure	1.5	576.07		3796.324
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		5035.83		28020.799

Summary of Forces For SBC

	Downward	Upward	
P	30218.771	29736.981	kN
ML	-4642.012	-4802.315	kNm
MT	16306.271	16306.271	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Superstructure	2695.398	108.698	2291.088	92.393	19.800
Substructure & Foundation -Portion 1	982.169	30.162	426.064	19.177	0.000
Substructure & Foundation -Portion 2	21502.118	105.580	-41238.166	-35.552	5242.545
Total =	25179.685	244.440	-38521.014	76.018	5262.345
		-244.440		-76.018	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L2}$	MLs due to F_v
Total =	26829.559	206.859	-174619.10	-1022.370
		-206.859		1022.370

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
311.32	3049.41	0.00	0.00	0.00	0.00	586.21	6581.47
149.02	1027.06	0.00	0.00	0.00	0.00	618.26	4261.18
5131.83	27331.00						
218.65	1426.64						
5810.82	32834.12	0.00	0.00	0.00	0.00	1204.47	10842.65

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
390.36	3823.62	0.00	0.00
217.73	1506.13	0.00	0.00
3851.664	18894.723		
576.073	3796.324		
0.000	0.000		
0.000	0.000		
5035.83	28020.80	0.00	0.00

Summary of Restoring Forces

Vertical Load	26622.700	kN
Moment	-175641.471	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	Long. Ecc. (e_{L2}) @ Toe (m)	ML = $P \times e_{L2}$	MLs due to F_v
Superstructure	2457.673	98.458		-10813.761	-433.214
Substructure & Foundation	883.952	27.145		-4049.096	-125.254
Substructure & Foundation	19351.906	92.135		-138626.034	-514.744
Total =	22693.531	217.739		-153488.891	-1073.213
		-217.739			1073.213

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.99	543.49	1485.17	2585.80	6031.24
due to Substructure	1	163.59	678.71		1125.29	4668.72
due to Earth pressure	1	5276.83			25952.79	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		390.365	1485.170	3823.62
due to Substructure	1.5	217.730		1542.70
due to Active Earth pressure	1	4004.305		17538.42
due to dynamic Earth pressure	1.5	572.635		3786.46
		5185.035		26691.21

Summary of Forces For SBC

	Downward	Upward	
P	25424.125	24935.245	kN
ML	-8781.118	-8933.154	kNm
MT	15962.300	15962.300	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	$ML = P \times eL1$	MLs due to F_v	$MT = P \times eT$
Total =	25338.949	246.784	-39002.54	78.011	5317.572
		-246.784		-78.011	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	311.323	555.21147	1485.170	3049.410	6182.21558
due to Substructure	1	163.588	678.709		1125.292	4668.71652
due to Earth pressure	1	5276.828			25952.789	
due to Live load surcharge	0.2	244.151			1531.304	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		390.36	1485.170	3823.62
due to Substructure	1.5	217.73		1542.70
due to Active Earth pressure	1	4004.31		17538.42
due to dynamic Earth pressure	1.5	572.64		3786.46
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		5185.04		26691.21

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
263.99	2585.80	0.00	0.00	0.00	0.00	543.49	6031.24
163.59	1125.29	0.00	0.00	0.00	0.00	678.71	4668.72
5276.83	25952.79						
5704.41	29663.88	0.00	0.00	0.00	0.00	1222.20	10699.95

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
390.36	3823.62	0.00	0.00
217.73	1542.70	0.00	0.00
4004.305	17538.425		
572.635	3786.464		
5185.04	26691.21	0.00	0.00

Summary of Restoring Forces

Vertical Load	22475.793	kN
Moment	-154562.104	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	$ML = P \times eL2$	MLs due to F_v
Total =	22693.531	217.739		-153493.29	-1073.213
		-217.739			1073.213

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
311.32	3049.41	0.00	0.00	0.00	0.00	555.21	6182.22
163.59	1125.29	0.00	0.00	0.00	0.00	678.71	4668.72
5276.83	25952.79						
244.15	1531.30						
5995.89	31658.79	0.00	0.00	0.00	0.00	1233.92	10850.93

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
390.36	3823.62	0.00	0.00
217.73	1542.70	0.00	0.00
4004.305	17538.425		
572.635	3786.464		
0.000	0.000		
0.000	0.000		
5185.04	26691.21	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	25585.734	25092.165	kN
ML	-7265.731	-7421.753	kNm
MT	16168.504	16168.504	kNm

Summary of Restoring Forces

Vertical Load	22475.793	kN
Moment	-154566.504	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1488.255 -	1475.375)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1488.255 -	1475.375)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	29245.010	-14795.139	5262.345	306.478	177.650	267.988	139.160	306.478	139.160	6290.242	18446.627	2.93	29823.419	169816.33	5.69
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	30632.167	-9600.611	6268.711	298.112	214.515	252.261	168.664	298.112	168.664	7453.317	18446.627	2.47	37459.579	172914.27	4.62
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	24689.192	-17000.885	5262.345	281.371	133.336	242.880	94.846	281.371	94.846	6290.242	15576.461	2.48	27788.972	149061.93	5.36
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	25337.602	-11818.522	5538.479	264.758	161.849	224.248	121.339	264.758	121.339	7597.642	15801.839	2.08	36495.993	152442.59	4.18
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	29907.793	2063.488	8350.071	249.422	267.390	188.348	206.315	267.390	188.348	7798.892	18635.890	2.39	49314.977	175641.47	3.56
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	30218.771	3702.329	8716.414	245.996	278.234	182.242	214.480	278.234	182.242	7798.892	18635.890	2.39	49314.977	175641.47	3.56
								SAFE	SAFE			SAFE			SAFE

Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	25424.125	-244.295	8472.331	225.756	223.629	163.787	161.659	225.756	161.659	7940.072	15733.055	1.98	48047.712	154562.10	3.22
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	25585.734	1307.807	8572.852	220.597	231.985	157.893	169.281	231.985	157.893	7940.072	15733.055	1.98	48047.712	154566.50	3.22
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	29907.793	-6244.137	15554.765	311.940	257.570	198.168	143.798	311.940	143.798	5543.870	18635.890	3.36	31535.104	175641.47	5.57
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	30218.771	-4642.012	16306.271	310.083	269.662	190.814	150.394	310.083	150.394	5035.832	18635.890	3.70	28020.799	175641.47	6.27
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	25424.125	-8781.118	15962.300	290.315	213.853	173.562	97.100	290.315	97.100	5185.035	15733.055	3.03	26691.211	154562.10	5.79
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	25585.734	-7265.731	16168.504	285.703	222.436	167.442	104.175	285.703	104.175	5185.035	15733.055	3.03	26691.211	154566.50	5.79
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

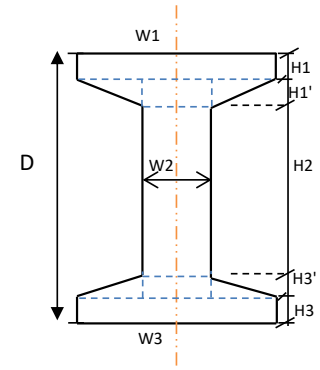
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	300
W3	=	800

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	700	100	35000
1' (Rect.)	300	100	30000
2 (Rect.)	300	950	285000
3' (Tri.)	500	150	37500
3' (Rect.)	300	150	45000
3 (Rect.)	800	250	200000
			782500



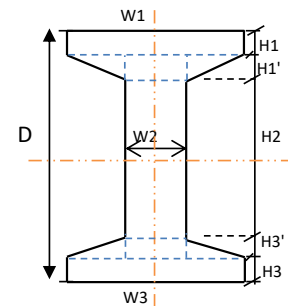
Total Area of Section A = 0.783 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	77
H2	=	1123
H3'	=	0
H3	=	250
W1	=	1000
W2	=	463
W3	=	463

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	538	77	20636
1' (Rect.)	463	77	35513
2 (Rect.)	463	1123	519487
3' (Tri.)	0	0	0
3' (Rect.)	463	0	0
3 (Rect.)	463	250	115625
			841261



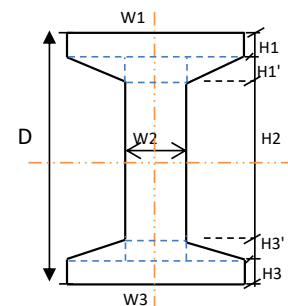
Total Area of Section A = 0.841 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

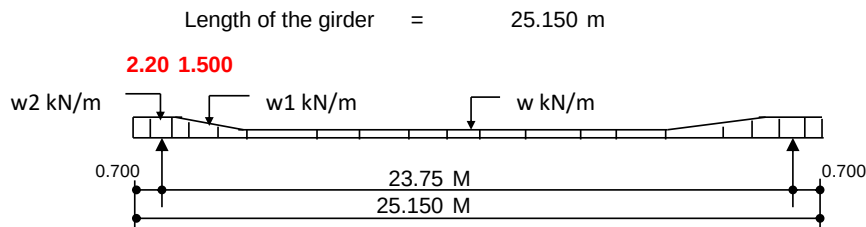
D	=	1600
H1	=	150
H1'	=	54
H2	=	1396
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	54	10045
1' (Rect.)	625	54	33482
2 (Rect.)	625	1396	872768
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066295



Total Area of Section A = 1.066 m²

1.0 Self weight of Inner Girder & Outer Girder



UDL , w	=	19.56 kN/m
Avg. , w ₁	=	21.03 kN/m
UDL , w ₂	=	26.66 kN/m
Supp. Reaction , R	=	263.81 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	

UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder

Supp. Reaction , R	=	224.778 kN	For Outer Girder
	=	207.488 kN	For Inner Girder

weight of deck slab has been Increase by 5 %

Supp. Reaction , R	=	236.017 kN	For Outer Girder
	=	217.862 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

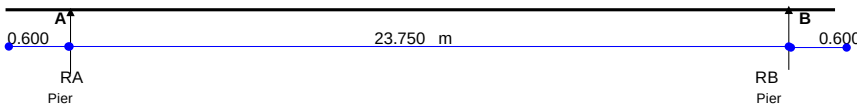
4.0 Weight due to wearing coat

Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	
UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder
Supp. Reaction , R	=	76.079 kN	For Outer Girder
	=	82.995 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)

Maximum Live Load Reaction Case**Due to Class A 1-Lane , 2-Lanes, 3-Lanes & 4-Lanes**Considering **114.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **23.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **24.95 m**

Nos. of Lane for design purpose = **3 LANE**

Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

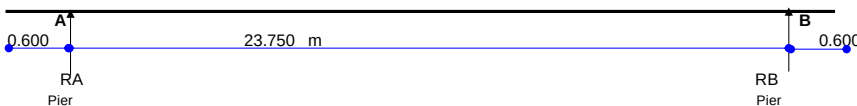
Ist Train	Total Load	500.0kN				
					392.345	107.655
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	392.345	107.655

Due to Class 70R WheeledConsidering **170.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **23.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **24.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

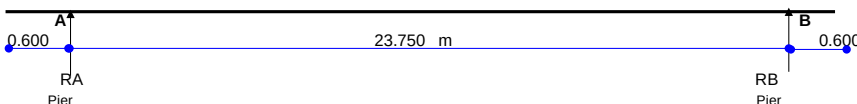
Ist Train	Total Load	1000.0kN				
					809.524	190.476
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	809.524	190.476

Due to Class A 1-Lane/2-Lane (along with Class 70R)Considering **68.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **23.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **24.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

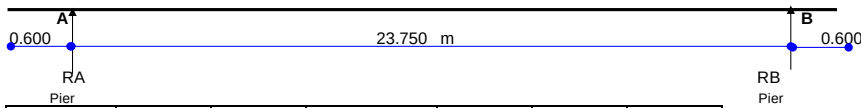
Ist Train	Total Load	554.0kN				
					341.524	212.476
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
27.000	1.100	18.800	18.200	491.400		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	341.524	212.476

Due to Class SV LoadingConsidering **180.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = 23.75 m
 Projection beyond c.L. of bearing at Abut. end = 0.600 m
 Projection beyond c.L. of bearing at far end = 0.600 m
 Total length including Overhangs = 24.95 m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	3420.0kN				
180.000		0.000	-0.600	-108.000	1926.189	1493.811
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		
180.000	1.500	9.000	8.400	1512.000		
180.000	1.500	10.500	9.900	1782.000		
180.000	1.500	12.000	11.400	2052.000		
180.000	1.500	13.500	12.900	2322.000		
180.000	1.500	15.000	14.400	2592.000		
180.000	1.500	16.500	15.900	2862.000		
180.000	1.500	18.000	17.400	3132.000		
180.000	1.500	19.500	18.900	3402.000		
180.000	1.500	21.000	20.400	3672.000		
180.000	1.500	22.500	21.900	3942.000		
180.000	1.500	24.000	23.400	4212.000		
0.000	1.500	0.000	-0.600	0.000		
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	1926.189	1493.811

Deck Width = 12.500 m

transverse eccentricity of superstructure about c.g of abutment and foundation = 0 m

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	1926.189 KN	Corr. MT	= 577.857 kNm
Minimum Reaction (RB)	=	1493.811 KN	Corr. MT	= 448.143 kNm

1-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-0.9	0	0	=	4.45 m
Maximum Reaction (RA)	=	392.345	KN	Corr. MT	=	1745.936	kNm		
Minimum Reaction (RB)	=	107.655	KN	Corr. MT	=	479.064	kNm		

2-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-0.85	0	=	2.70 m
Maximum Reaction (RA)	=	784.691 KN	Corr. MT	=		2118.664 kNm			
Minimum Reaction (RB)	=	215.309 KN	Corr. MT	=		581.336 kNm			

3-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	0.95 m
Maximum Reaction (RA)	=	1177.036 KN	Corr. MT	=	1118.184 kNm				
Minimum Reaction (RB)	=	322.964 KN	Corr. MT	=	306.816 kNm				

4-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-0.8 m
Maximum Reaction (RA)	=	1569.381 KN	Corr. MT	=	-1255.505 kNm					
Minimum Reaction (RB)	=	430.619 KN	Corr. MT	=	-344.495 kNm					

1- 70RW

Transverse Ecc.	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m
Maximum Reaction (RA)	=	809.524 KN	Corr. MT	=	2554			
Minimum Reaction (RB)	=	190.476 KN	Corr. MT	=	601			

1-CLASS A + 1-70RW

SECTION 11000											
Transverse Ecc.	for Class A	=	6.25	-0.5	-0.4	-0.9	=	4.45 m			
	for 70RW	=	6.25	-0.5	-0.4	-1.8	-0.25	-1.8	-1.395	=	0.105
Maximum Reaction (RA)	=	1151.048 KN	Corr. MT	=	341.52 x	4.45 +	809.52 x	0.105	=	1605	
Minimum Reaction (RB)	=	402.952 KN	Corr. MT	=	212.476	4.45 +	190.476	0.105	=	966	

2-CLASS A + 1-70RW

<u>CORNER AT E-PORT</u>												
Transverse Ecc.	=	for 1st Class A for 2nd Class A for 70RW	= = =	4.45 m 4.45 2.7	-0.90 -0.85	-0.85	0 -1.8	= -0.25	2.7 m -1.2	-1.395	=	-2.795
Maximum Reaction (RA)	=	1492.573 KN		Corr. MT	=	341.52 x	4.45 + +	341.52 x 809.52 x	2.700 -2.795		=	179
Minimum Reaction (RB)	=	615.427 KN		Corr. MT	=	212.476	4.45 + +	212.48 x 190.48 x	2.700 -2.795		=	987

1- 70RW + 1-CLASS A

Transverse Ecc.		for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m	
		for Class A	=	6.25	-0.5	-7.25	-0.25	-0.9 =	-2.650 m	
Maximum Reaction (RA)	=	1151.048 KN	Corr. MT	=	809.52 x	3.155 +	341.52 x	-2.650	=	1649
Minimum Reaction (RB)	=	402.952 KN	Corr. MT	=	190.476	3.155 +	212.476	-2.650	=	38

1- 70RW + 2-CLASS A

Transverse Ecc.	for 70RW	=	3.155 m							
	for 1st Class	=	-2.650 m							
	for 2nd Class	=	-2.650	-0.9	-0.85	=		-4.400 m		
Maximum Reaction (RA)	=	1492.573 KN	Corr. MT	=	809.52 x	3.155 +	341.52 x	-2.650	=	146
					+		341.52 x	-4.400		
Minimum Reaction (RB)	=	615.427 KN	Corr. MT	=	190.476	3.155 +	212.476	-2.650	=	-897
					+		212.48 x	-4.400		

2- 70RW

Transverse Ecc.	for 1-70RW for 2-70RW	= =	3.16 m 3.155	-1.395	-1.2	-1.40	=	-0.835	m	
Maximum Reaction (RA)	=	1619.048 KN	Corr. MT	=	809.52 x	3.155 +	809.52 x	-0.835	=	1878
Minimum Reaction (RB)	=	380.952 KN	Corr. MT	=	190.476	3.155 +	190.476	-0.835	=	442

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	392.345	1745.936
2-CLASS A	784.691	2118.664
3-CLASS A	1177.036	1118.184
4-CLASS A	0.000	0.000
1- 70RW	809.524	2554.049
1-CLASS A + 1-70RW	1151.048	1604.783
1- 70RW + 1-CLASS A	1151.048	1649.010
2-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 2-CLASS A	0.000	0.000
2- 70RW	0.000	0.000

Reaction RB, , (kN)	Corr. MT (kNm)
107.655	479.064
215.309	581.336
322.964	306.816
0.000	0.000
190.476	600.951
402.952	965.517
402.952	37.890
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 1177.036 KN
 Reaction RB = 322.964 KN

& Corresponding moment = 1118.184 kNm
 & Corresponding moment = 306.816 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 809.524 KN
 Reaction RB = 190.476 KN

& Max moment = 2554.049 kNm
 & Max moment = 600.951 kNm

Stability Check Summary

Description	P _{max} (kN/m ²)	P _{min} (kN/m ²)	Sliding	Overturnin g	Shear (Heel slab)	Shear (Toe slab)
Normal Dry Case						
Case 1 : DL+SIDL-Normal Dry Case	306.48	139.16	2.93	5.69	0.075	0.723
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	298.11	168.66	2.47	4.62		
Permissible	422	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Normal HFLCase						
Case 3 : DL+SIDL-Normal HFL Case	281.37	94.85	2.48	5.36		
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	264.76	121.34	2.08	4.18		
Permissible	422	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic Dry Case						
Case 5 : DL+SIDL-Long. Seismic Dry Case	267.39	188.35	2.39	3.56		
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	278.23	182.24	2.39	3.56		
Permissible	527	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic HFL Case						
Case 7 : DL+SIDL-Long. Seismic HFL Case	225.756	161.659	1.98	3.22		
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	231.985	157.893	1.98	3.22		
Permissible	527	0	1.00	1.00		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic Dry Case						
Case 9 : DL+SIDL-Trans. Seismic Dry Case	311.940	143.798	3.36	5.57		
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	310.083	150.394	3.70	6.27		
Permissible	527	0	1	1		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic HFL Case						
Case 11 : DL+SIDL-Trans. Seismic HFL Case	290.315	97.100	3.03	5.79		
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	285.703	104.175	3.03	5.79		
Permissible	527	0	1.00	1.00	4.192	4.192
Remarks	O.K	O.K	O.K	O.K	O.K	O.K

Reinforcement summary

Type of reinforcement	Area of steel required	Area of steel provided						
Heel Slab								
Main steel at bottom face	2035	25	mm bar @	200	mm c/c (i.e.)	4909	mm ²	(O.K)
		25	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	982	12	mm bar @	100	mm c/c (i.e.)	1131	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 20 Legged bar @	100	mm c/c (i.e.)			
Toe Slab								
Main steel at bottom face	4900	32	mm bar @	100	mm c/c (i.e.)	10053	mm ²	(O.K)
		16	mm bar @	100	mm c/c (i.e.)			
Distribution reinforcement	2011	20	mm bar @	100	mm c/c (i.e.)	3142	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 28 Legged bar @	100	mm c/c (i.e.)			

Abutment Shaft								
Vertical steel at earth face	8896	32	mm bar @	100	mm c/c (i.e.)	16085	mm ²	(O.K)
		32	mm bar @	100	mm c/c (i.e.)			
Vertical steel at non-earth face	1440	20	mm bar @	100	mm c/c (i.e.)	3142	mm ²	(O.K)
Distribution steel	4807	20	mm 2 Legged bar @	100	mm c/c (i.e.)	6283	mm ²	(O.K)
Abutment Cap								
Top Longitudinal Reinf.	1106	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Bottom Longitudinal Reinf.	1106	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Side Face Longitudinal Reinf.		12	mm dia bar	200	mm c/c (i.e.)			
Transverse Reinf.	2.19E+06	12	mm 2 Legged bar @	150	mm c/c (i.e.)	2.77E+06	mm ³ /m	(O.K)
Corbel Reinforcement								
Primary tensile reinf.	2106	20	mm bar @	125	mm c/c (i.e.)	2513	mm ²	(O.K)
Horizontal Stirrups	198.48	12	mm 2 Legged bar @	in 3 Vertical Layers		679	mm ² /m	(O.K)
Vertical Stirrups	526.40	10	mm 2 Legged bar @	200	mm c/c (i.e.)	785	mm ³ /m	(O.K)

***DESIGN OF PIER P4 TO P5 AND
ITS FOUNDATION
FOR MAJOR BRIDGE AT CH- 16+463***

This Document Contains Design Calculations of Pier with Open Foundation

1. SALIENT FEATURES OF THE BRIDGE

Skew angle : 0 degree 0 Radian

<u>SPAN DETAILS</u>	<u>Left span</u>		<u>Right span</u>		<u>Unit</u>
	<u>Right</u>	<u>Skew</u>	<u>Right</u>	<u>Skew</u>	
C/C Expansion joint	= 25.000	25.000	25.000	25.00	m
C/C Expansion joint to C.L. brg	= 0.625	0.625	0.625	0.63	m
Expansion gap	= 50	50	50	50	mm
C/C bearing	= 23.750	23.750	23.750	23.750	m

SUPERSTRUCTURE DETAILS

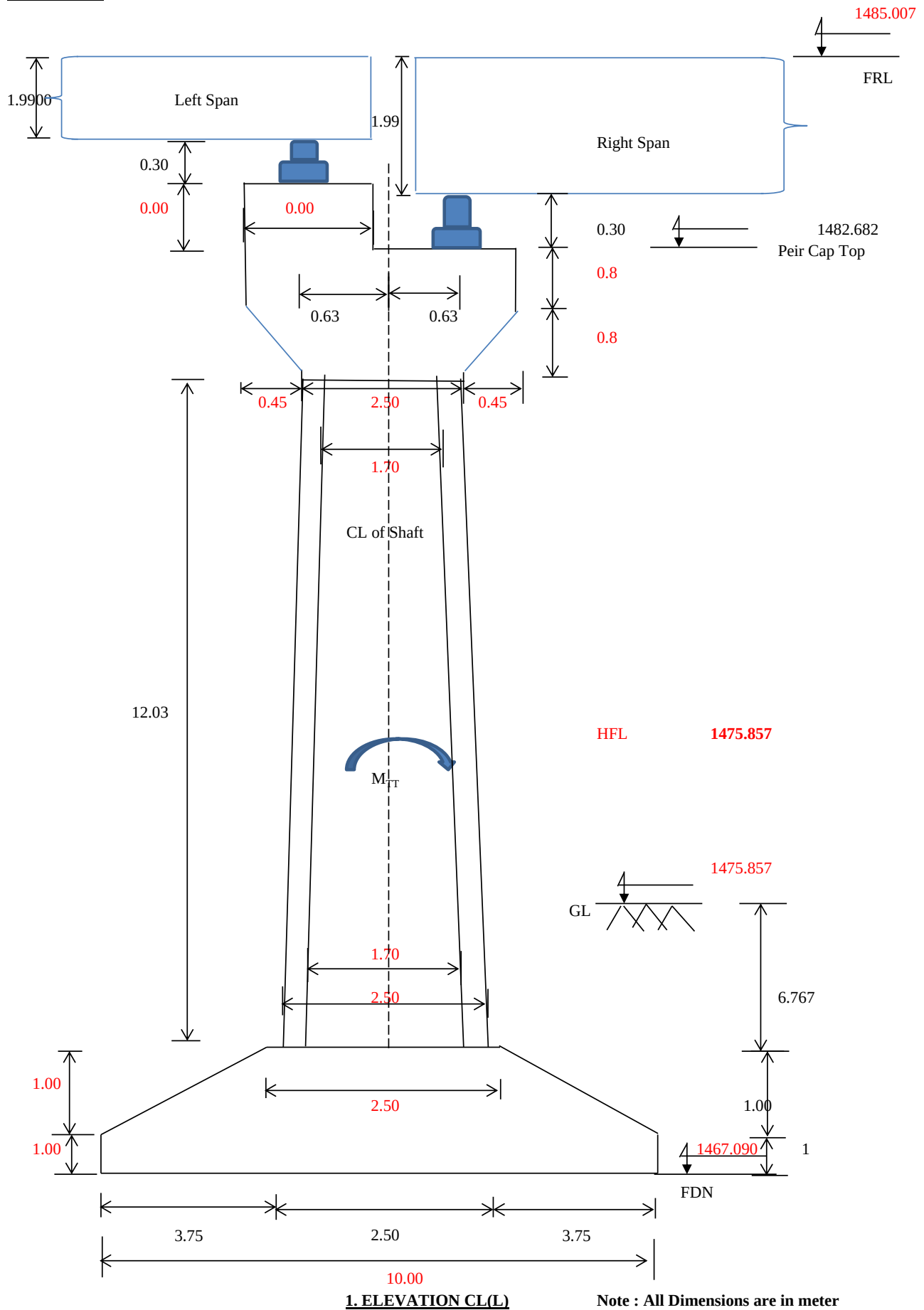
	Left Span	Right Span
Overall deck width of bridge	= 12.50 m	12.50 m
Carriage way width	= 11.50 m	11.50 m
Crash barrier width	= 0.45 m	0.45 m
Height of crash barrier	= 1.00 m	1.00 m
C.g of Crash barrier from bottom	= 0.40 m	0.40 m
Cross-slope	= 7.00 %	7.00 %
Wearing coat thickness	= 65 mm	65 mm
Nos. of Bearing	= 4	4
Spacing of Bearing	= 3.00 m	3.00 m
Depth of Girder	= 1.600 m	1.600 m
Depth of slab	= 0.22 m	0.22 m
Depth of superstructure	= 1.820 m	1.820 m
Bearing + Pedestal height	= 0.30 m	0.30 m
Width of main girder at top	= 1.00 m	1.00 m
Width of cross girder	= 0.40 m	0.40 m
Nos. of cross-girders	= 2	2
FRL to Soffit	= 1.99 m	1.99 m
Type of Bearing	OR SPHERICAL BR	

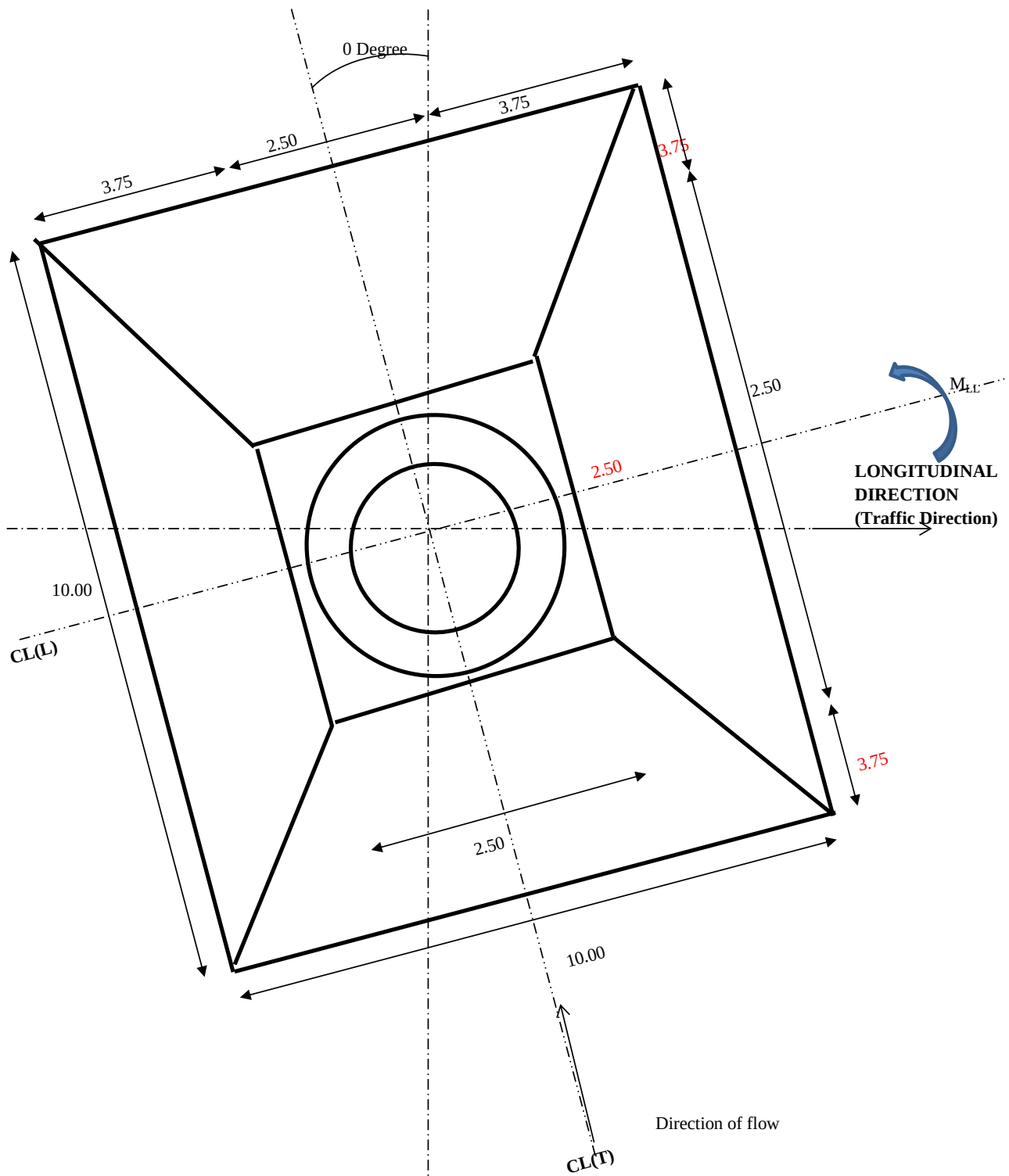
LEVEL DETAILS

Formation level	= 1485.007 m
Pier cap top level	= 1482.682 m
HFL	= 1475.857 m
Ground level	= 1475.857 m
MSL (Normal)/Rock Level	= 1475.857 m
MSL (Seismic)/Rock Level	= 1475.857 m
Pier shaft bottom/Footing top level	= 1469.055 m
Founding level	= 1467.090 m

DIMENSION DETAILS

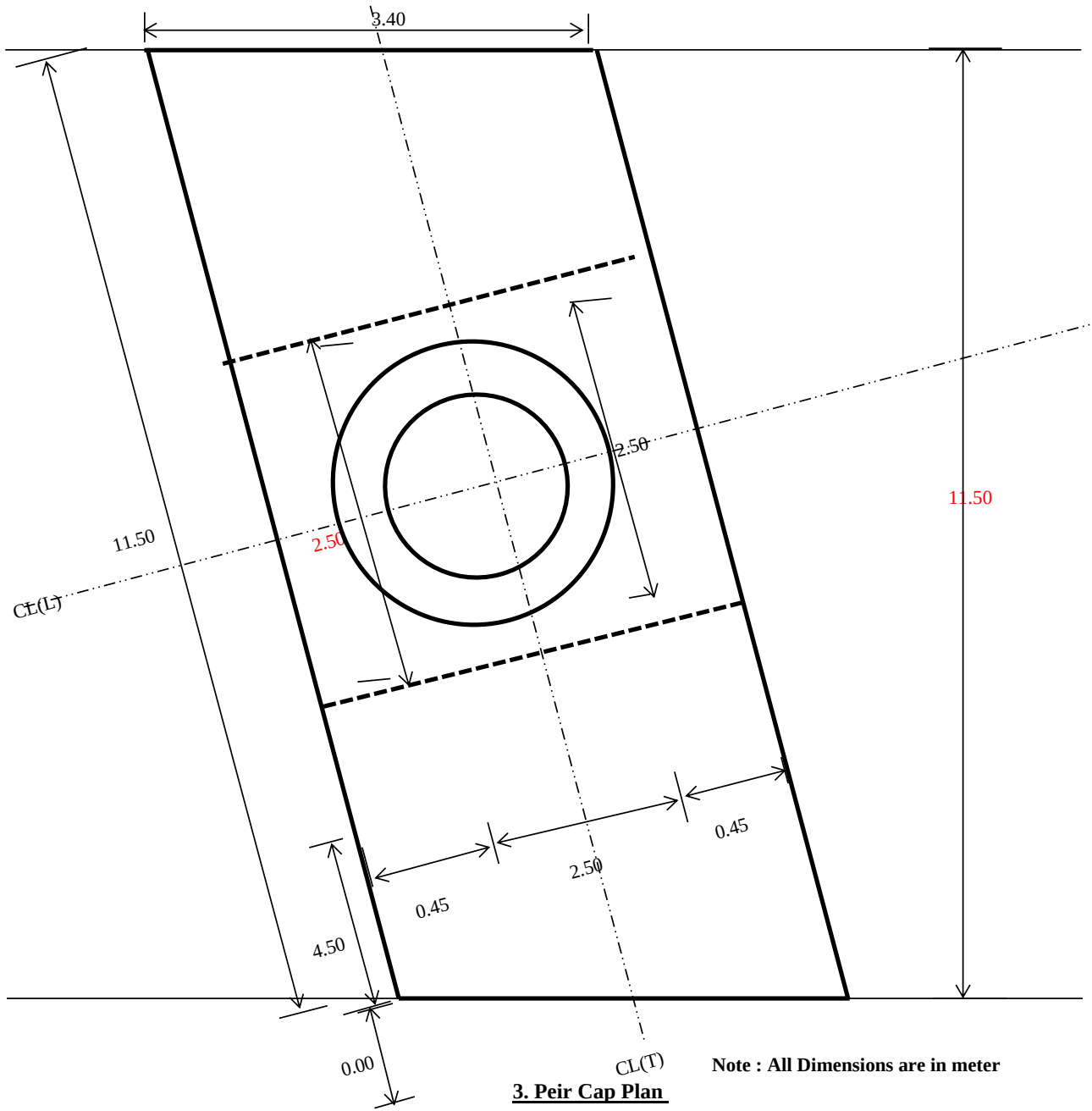
Pier cap:	Right	Skew
Width of pier cap at top	= 3.40 m	3.40 m
Width of pier cap at bottom	= 2.50 m	2.50 m
Length of pier cap at top	= 11.50 m	11.50 m
Length of pier cap at bottom	= 2.50 m	2.50 m
Thickness of uniform portion	= 0.80 m	0.80 m
Thickness of tapered portion	= 0.80 m	0.80 m
Pier shaft:		
Width of pier at top	= 2.50 m	2.50 m
Width of pier at bottom	= 2.50 m	2.50 m
Length of pier (rectangle portion)	= 2.50 m	2.50 m
Length of pier(end circular portion Top)	= 1.25 m	1.25 m
Length of pier (end circular portion Bottom)	= 1.25 m	1.25 m
Total length of pier shaft at top	= 5.00 m	5.00 m
Total length of pier shaft at bottom	= 5.00 m	5.00 m
Footing:		
Width of footing at top	= 2.50 m	2.50 m
Width of footing at bottom	= 10.00 m	10.00 m
Length of footing	= 10.00 m	10.00 m
Thickness at tip	= 1.00 m	1.00 m
Maximum thickness	= 2.00 m	2.00 m

DIAGRAMS:



Note : All Dimensions are in meter

2. PLAN SHOWING FOOTING & PIER SHAFT



2. MATERIALS DETAILS**CONCRETE:**

Grade of concrete	fck	=	M 35	
Mean value of concrete compressive strength, fcm		=	M 45 (28 days strength from table 6.5 of IRC:112)	
Design concrete compressive strength, fcd		=	0.447 x fck	(Basic and Seismic combination)
		=	15.633 Mpa	
Secant modulus of elasticity, Ecm		=	32000 Mpa	(from table 6.5 of IRC:112)
Mean axial tensile strength fctm		=	2.8 Mpa	(from table 6.5 of IRC:112)
Density of concrete		=	2.5 t/m ³	

SOIL :

Density dry		=	2 t/m ³	
Density Saturated		=	2.3 t/m ³	
Soil density (submerged)		=	2 t/m ³	
Angle of Internal friction ϕ		=	35	
Cohesion c		=	0	
Gross bearing capacity		=	35.00 t/m ²	(Normal case)
		=	43.75 t/m ²	(Seismic case)

CREEP COEFFICIENT:

(Table 6.5 of IRC:112)

Pier shaft cross-sectional area	Ac	=	11.159 m ²	
Perimeter in contact with atmosphere, u		=	12.854 m	
Notional size, ho = 2Ac/u		=	1736 mm	
Age of concrete at the time of loading to		=	90 days	
	ta	=	25550 days	
	f(α 90)	=	1.2 (Table 6.9 IRC 112)	
Hence creep co-efficient for pier shaft		=	1.32 (increased by 10%)	
Creep co-efficient for foundation		=	1.0 (as ho = α for foundation)	
Hence modulus of elasticity for long term loading, Ecm'		=	13793 Mpa	Ecm' = Ecm / (1 + f)

REINFORCING STEEL:

Grade of reinforcement	fy	=	Fe 500 Mpa	
Modulus of elasticity	Es	=	200000 Mpa	

3. PERMISSIBLE STRESSES**MAXIMUM PERMISSIBLE STRESS IN CONCRETE:**

Rare combination		=	0.48 x fck	
		=	16.8 Mpa	
Quasi permanent combination		=	0.36 x fck	
		=	12.6 Mpa	

MAXIMUM PERMISSIBLE STRESS IN STEEL:

Rare Combination		=	300 Mpa	
Permissible crack width	Wk,max	=	0.3 mm	

4. SEISMIC PARAMETERS

Seismic zone		=	IV	
Type of soil		=	Type2(Medium)	
Zone factor	Z	=	0.24	
Importance factor	I	=	1.20	
Response Reduction Factor(Long)	R	=	3	
Response Reduction Factor(Trans)	R	=	3	

5. ENVIRONMENTAL PARAMETERS

Relative humidity		=	80%	
Condition of exposure		=	Moderate	

6.OTHER PARAMETERS

Mean velocity of water current flow		=	0.00 m/s	
Design Speed	V	=	50 Kmph	
Radius of Curvature	R	=	0 m	

7.LOADS FROM SUPERSTRUCTURES

Angle of Skew	=	0 °	=	0.000 Radian
No. of Bearing	=		4.00	
		Right		Skew
c/c of bearing in Transverse Direction	=	3.000	m	3.000 m
c/c bearing on Longitudinal Direction	=	1.250	m	1.250 m
Impact Factor for Design of Pier Cap				= 1.24

Left Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	2159.25	200.00								
Total Load	2359.25		318.15	0.000	0.00	0.00	0.00	0.00	0.00	0.00
Resultant Transverse Ecc.	0.000		0.000	0.000	0.950	1.210	1.000	1.000	0.300	0.300

Right Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	2159.25	200.00								
Total Load	2359.25		318.15	0.00	1109.88	240.12	0.00	0.00	1890.78	1709.22
Resultant Transverse Ecc.	0.000		0.000	0.000	0.300	0.300	1.000	1.000	0.300	0.300

8. VOLUME OF SUBSTRUCTURE**PIER CAP****Uniform portion**

No.		L(m)	B(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	11.500	0.000	0.000	0.000	1.700	0.000	13.627	15.63
2	1	11.500	3.400	0.800	31.280	0.000	0.000	13.227	15.23

Tapered portion

2a	Rectangle	2.500	3.400	0.800	6.800	0.000	0.000	12.427	14.427
	Triangle	4.500	3.400	0.800	12.240	0.000	0.000	12.560	14.560
Total				Inc. 10%	55.352	0.000	0.000	11.779	13.597

PIER SHAFT**Above HFL****Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	5.260	13.881	0.000	0.000	9.397	11.397
2	0	2.500	0.000	5.260	0.000	-1.250	0.000	8.520	10.520
3	0	2.500	0.000	5.260	0.000	1.250	0.000	8.520	10.520

Cylinder/Conical Frustum portion

4	0.000	0.850	1.25	5.260	0.000	0.000	0.000	9.397	11.397
Total					13.881	0.000	0.000	9.397	11.397

Below HFL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	6.767	17.858	0.000	0.000	3.384	5.384
2	0	2.500	0.000	6.767	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	6.767	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.250	0.850	6.767	0.000	0.000	0.000	3.384	5.384
Total					17.858	0.000	0.000	3.383	5.383

Above LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	5.260	13.881	0.000	0.000	9.397	11.397
2	0.0	2.500	0.000	5.260	0.000	-0.850	0.000	8.520	10.520
3	0.0	2.500	0.000	5.260	0.000	0.850	0.000	8.520	10.520

Cylinder/Conical Frustum portion

4	0.000	0.850	0.850	5.260	0.000	0.000	0.000	9.251	11.251
Total					13.881	0.000	0.000	9.397	11.397

Below LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	6.767	17.858	0.000	0.000	3.384	5.384
2	0	2.500	0.000	6.767	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	6.767	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.25	0.850	6.767	0.000	0.000	0.000	2.850	4.850
Total					17.858	0.000	0.000	3.384	5.384

FOOTING

No.		L	B	H	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	2.500	2.00	50.000	0.000	0.000	-1.000	1.000
2	1	10.000	3.75	1.500	56.250	-2.917	0.000	-1.222	0.778
3	1	10.000	3.750	1.500	56.250	2.917	0.000	-1.222	0.778
Total					162.5	0.000	0.000	-1.154	0.846

EARTH ABOVE FOOTING

No.		L	B	H	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	10.000	8.767	876.700	0.000	0.000	2.384	4.384
2	-1	Shaft Below LBL			-17.858	0.000	0.000	3.384	5.384
3	-1	Footing			-162.500	0.000	0.000	-1.154	0.846
Total					696.342	0.000	0.000	3.183	5.183

9. CALCULATION OF SEISMIC FORCE

$$\text{Time Period (T)} = 2 \sqrt{\frac{D}{1000 F}}$$

$$\begin{aligned} D &= \text{Approximate DL of super-structure \& LL in Tonne (DL+20\% LL)} \\ &= 5354.80 \text{ KN} + 221.977 \text{ KN} \\ &= 5577 \text{ KN} \end{aligned}$$

$$F =$$

Horizontal force in KN required to be applied at center of mass of super-structure for 1 mm deflection at the top of pier / abutment for the earthquake in the transverse direction; and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

$$F = \frac{6 E I}{x^2(3L-x)} \quad \text{for 1 mm deflection at } x$$

$$E = 32000 \text{ N/mm}^2$$

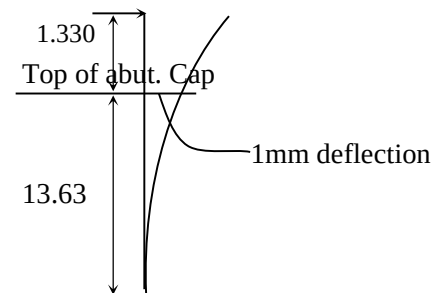
Column Cross-Section

$$D1 = 2.50 \text{ m (Average Outer Dia)}$$

$$D2 = 1.70 \text{ m (Average Inner Dia)}$$

$$I_{TT} = 1.13E+12 \text{ mm}^4 \quad 1.5E+12$$

$$I_{LL} = 1.13E+12 \text{ mm}^4$$



$$\text{Force required in Long. Dir. } F_L = 43 \text{ KN}$$

$$\text{Force required in Trans. Dir. } F_T = 37 \text{ KN}$$

$$\text{Time Period -Longitudinal Seismic Case} = 0.721 \text{ seconds}$$

$$\text{Time Period -Transverse Seismic Case} = 0.772 \text{ seconds}$$

$$S_a/g \text{ -Longitudinal Seismic Case} = 1.886$$

$$S_a/g \text{ -Transverse Seismic Case} = 1.761$$

$$\text{Seismic Zone} = \text{IV}$$

$$\text{Zone Factor } Z = 0.24$$

$$\text{Importance Factor } I = 1.20$$

$$\begin{aligned} \text{Response reduction Factor } R &= 3.00 \quad \text{Longitudinal} \\ R &= 3.00 \quad \text{Transverse} \end{aligned}$$

$$\text{Avg response acceleration } S_a/g = 1.886 \quad \text{Longitudinal}$$

$$\text{Avg response acceleration } S_a/g = 1.761 \quad \text{Transverse}$$

$$\text{Elastic Seismic Coefficient} = \frac{Z}{2} \times (S_a/g) \times (I/R)$$

$$\text{In longitudinal direction} = 0.272 / R$$

$$\text{In Transverse direction} = 0.254 / R$$

			Substructure	Foundation
Enhancement factor				1.35
Design Seismic coefficient	Long. Ah		0.091	0.122
	Trans. At		0.085	0.114
	Vert. Av		0.060	0.081

CALCULATION OF SEISMIC FORCES

SEISMIC COMPONENT OF SUPER-STRUCTURE DL, SIDL, WC & FPLL :

Loads & Their Lever arm

Description	LEFT			RIGHT		
	W	eVS	eVF	W	eVS	eVF
	Tonnes	(m)	(m)		(m)	(m)
Total Super-Structure DL	431.85	15.25	17.25	431.85	15.25	17.25
Total Super-Structure SIDL	40.00	16.32	18.32	40.00	16.32	18.32
Total Super-Structure WC	63.63	15.95	17.95	63.63	15.95	17.95
Total Super-Structure FPLL	0.00	15.95	17.95	0.00	15.95	17.95
Total	535.48	15.42	17.42	535.48	14.20	16.05

* Bearing level	13.93	15.93		13.93	15.93

SEISMIC COMPONENT OF SUB-STRUCTURE :

Description	W	eVS	eVF
	Tonnes	(m)	(m)
Total weight of shaft and pier cap	217.73	9.68	11.56
Total weight of foundation	406.25	-1.15	0.85

SEISMIC COMPONENT OF LIVE LOAD(70R+Class A) :

Description	W	W	eVS	eVF
	Tonnes	Tonnes	(m)	(m)
	70R + Class A	70R + Class A		
Maximum Live Load	110.99	0.00	17.12	19.117
Minimum Live Load	24.01	0.00	17.12	19.117

Summary of Seismic force at shaft and foundation

Super structure

Dead load - Super structure about shaft	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		48.47		675.076	
Seismic Transverse direction			45.27		670.31
Seismic Vertical Direction	32.31				

Dead load - Super structure about foundation	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		65.44		1042.23	
Seismic Transverse direction			61.117		1022.59
Seismic Vertical Direction	43.63				

Dead load Left span Dislodged- about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		32.72		521.114	
Seismic Transverse direction			30.56		490.39
Seismic Vertical Direction	21.81				

Sub structure

Dead load - Sub structure about Shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		19.71		190.74	
Seismic Transverse direction			18.41		178.14
Seismic Vertical Direction	13.14				

Dead load - Sub structure about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		26.6071		307.64	
Seismic Transverse direction			24.85		287.32
Seismic Vertical Direction	17.74				

Live Load - Max**70R + Class A**

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			1.88		32.12
Seismic Vertical Direction	1.34				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.53		48.43
Seismic Vertical Direction	1.81				

70R

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			1.88		32.12
Seismic Vertical Direction	1.34				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.53		48.43
Seismic Vertical Direction	1.81				

Live Load - Min**70R + Class A**

Live load - Min shaft	about	V	H L	H T	M TT	M LL
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.41		6.95
Seismic Vertical Direction		0.29				

Live load - Min foundation	about	V	H L	H T	M TT	M LL
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.55		10.48
Seismic Vertical Direction		0.39				

70R

Live load - Min shaft	about	V	H L	H T	M TT	M LL
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.41		6.95
Seismic Vertical Direction		0.29				

Live load - Min foundation	about	V	H L	H T	M TT	M LL
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.55		10.48
Seismic Vertical Direction		0.39				

10. HORIZONTAL FORCES

Consider Left span free and Right Span fix on pier

BOTH SPAN POSITION CONDITION(70R + Class A)

1. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= \text{Class 70R on First two Lane } 0.2 \times 100 + \text{Class A on third Lane } 0.05 \times 54 \\ &+ 0.1 \times 100 + 0.05 \times 54 \\ F_h &= 35.4 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 24.01) \\ &= 14.59 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 32.29 \text{ Ton} \\ F_{hr} &= 32.29 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 32.29 - 13.39 \\ &= 18.90 \text{ Ton} \end{aligned}$$

2. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= \text{Class 70R on First two Lane } 0.2 \times 100 + \text{Class A on third Lane } 0.05 \times 54 \\ F_h &= 22.7 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 110.99) \\ &= 18.94 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 30.29 \text{ Ton} \\ F_{hr} &= 30.29 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 30.29 - 13.39 \\
 &= 16.90 \text{ Ton}
 \end{aligned}$$

3. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 13.39 \text{ Ton} \\
 F_{hl} &= 13.39 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= \frac{\text{Class 70R on First two Lane}}{0.2} \times 20 + \frac{\text{Class A on third Lane}}{0.05} \times 10.8 \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force(Shaft)} &= 24.2362359 \text{ Ton} \\
 \text{Seismic Horizontal force(foundation)} &= 32.7189185 \text{ Ton} \\
 \text{Total Horizontal force for Shaft}(F_h) &= 28.7762359 \text{ Ton} \\
 \text{Total Horizontal force for foundation}(F_h) &= 37.2589185 \text{ Ton} \\
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{4.80} \right) \\
 &= 13.627107 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \right) \text{ and } F_h - \mu (R_g + R_q)$$

$$\begin{aligned}
 F_{hr} \text{ for shaft} &= 30.29 \text{ Ton} \\
 F_{hr} \text{ for foundation} &= 34.53 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 30.29 - 13.39 \\
 &= 16.90 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 34.53 - 13.39 \\
 &= 21.14 \text{ Ton}
 \end{aligned}$$

4. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 13.39 \text{ Ton} \\
 F_{hl} &= 13.39 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= \frac{\text{Class 70R on First two Lane}}{0.2} \times 20 + \frac{\text{Class A on third Lane}}{0.05} \times 10.8 \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force(Shaft)} &= 24.2362359 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\
 \text{Seismic Horizontal force(foundation)} &= 32.7189185 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \%})
 \end{aligned}$$

Total Horizontal force for Shaft(Fh) = 28.7762359 Ton inc.)
 Total Horizontal force for foundation(Fh) = 37.2589185 Ton

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{22.20} \right)$$

$$= 14.496873 \text{ Ton}$$

Max of (Fh/2 + $\mu (Rg + Rq)$ and Fh - $\mu (Rg + Rq)$)

Fhr for shaft = 31.15 Ton
 Fhr for foundation = 35.40 Ton

Horizontal force for shaft
 = Fhr - Fhl
 = 31.15 - 13.39
 = 17.77 Ton

Horizontal force for foundation
 = Fhr - Fhl
 = 35.40 - 13.39
 = 22.01 Ton

BOTH SPAN POSITION CONDITION(70R)

5. Normal Max Reaction on Fix End

Left Span

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right)$$

$$= 13.39 \text{ Ton}$$

Fhl = 13.39 Ton

Right Span

Beaking Force = $0.2 \times \frac{\text{Class 70R on First two Lane}}{100} + 0.05 \times \frac{\text{Class A on third Lane}}{0.00}$

Fh = 20 Ton

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{24.01} \right)$$

$$= 14.59 \text{ Ton}$$

Max of (Fh/2 + $\mu (Rg + Rq)$ and Fh - $\mu (Rg + Rq)$)

Fhr = 24.59 Ton
 Fhr = 24.59 Ton

Horizontal force
 = Fhr - Fhl
 = 24.59 - 13.39
 = 11.20 Ton

6. Normal Min Reaction on Fix End

Left Span

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right)$$

$$= 13.39 \text{ Ton}$$

$$F_{hl} = 13.39 \text{ Ton}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 100 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 20 \text{ Ton}$$

$$\mu (R_g + R_q) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{110.99} \right) = 18.94 \text{ Ton}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$= 28.94 \text{ Ton}$$

$$F_{hr} = 28.94 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 28.94 - 13.39 \\ &= 15.55 \text{ Ton} \end{aligned}$$

7. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 20 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 4 \text{ Ton}$$

$$\text{Seismic Horizontal force (Shaft)} = 24.2362359 \text{ Ton}$$

$$\text{Seismic Horizontal force (foundation)} = 32.7189185 \text{ Ton}$$

$$\text{Total Horizontal force for Shaft (Fh)} = 28.2362359 \text{ Ton}$$

$$\text{Total Horizontal force for foundation (Fh)} = 36.7189185 \text{ Ton}$$

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{4.80} \right) \\ &= 13.627107 \text{ Ton} \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$F_{hr} \text{ for shaft} = 29.75 \text{ Ton}$$

$$F_{hr} \text{ for foundation} = 33.99 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 29.75 - 13.39 \\ &= 16.36 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 33.99 - 13.39 \end{aligned}$$

$$= 20.60 \text{ Ton}$$

8. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= 0.2 \times \frac{\text{Class 70R on First two Lane}}{20} + 0.05 \times \frac{\text{Class A on third Lane}}{0} \\ F_h &= 4 \text{ Ton} \\ \text{Seismic Horizontal force(Shaft)} &= 24.2362359 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\ \text{Seismic Horizontal force(foundation)} &= 32.7189185 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.}) \\ \text{Total Horizontal force for Shaft}(F_h) &= 28.2362359 \text{ Ton} \\ \text{Total Horizontal force for foundation}(F_h) &= 36.7189185 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{22.20} \right) \\ &= 14.496873 \text{ Ton} \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned} F_{hr} \text{ for shaft} &= 30.61 \text{ Ton} \\ F_{hr} \text{ for foundation} &= 34.86 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 30.61 - 13.39 \\ &= 17.23 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 34.86 - 13.39 \\ &= 21.47 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(Special Vehicle)

9. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{170.92} \right) \\ &= 21.93 \text{ Ton} \\ F_{hr} &= 21.93 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 21.93 - 13.39 \\ &= 8.55 \text{ Ton} \end{aligned}$$

10. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 22.84 \text{ Ton} \\ F_{hr} &= 22.84 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 22.84 - 13.39 \\ &= 9.45 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(NO LIVE LOAD)

11. Normal No Reaction

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hr} &= 13.39 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 13.39 - 13.39 \\ &= 0.00 \text{ Ton} \end{aligned}$$

ONE SPAN DISLODGED CONDITION

12. Normal Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.38699 \text{ Ton} \\ F_{hr} &= 13.38699 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 13.38699 - 0.000 \\
 &= 13.38699 \text{ Ton}
 \end{aligned}$$

13. Seismic Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= \text{Class 70R on First two Lane} + \text{Class A on third Lane} \\
 &= 0.2 \times 0 + 0.05 \times 0
 \end{aligned}$$

$$F_h = 0 \text{ Ton}$$

$$\text{Seismic Horizontal force(Shaft)} = 24.2362359 \text{ Ton} \quad (\text{Including safty factor 1.5})$$

$$\text{Seismic Horizontal force(foundation)} = 32.7189185 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.})$$

$$\text{Total Horizontal force for Shaft}(F_h) = 24.2362359 \text{ Ton}$$

$$\text{Total Horizontal force for foundation}(F_h) = 32.7189185 \text{ Ton}$$

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 13.38699 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q) \right)$$

$$F_{hr} \text{ for shaft} = 25.51 \text{ Ton}$$

$$F_{hr} \text{ for foundation} = 29.75 \text{ Ton}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 25.51 - 0.00 \\
 &= 25.51 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 29.75 - 0.00 \\
 &= 29.75 \text{ Ton}
 \end{aligned}$$

Summary of forces			
Cases		Horizontal force(Ton)	
		Shaft	Foundation
BOTH SPAN POSITION CONDITION(70R + Class A)			
	1. Normal Max Reaction on Fix End	18.90	18.90
	2. Normal Min Reaction on Fix End	16.90	16.90
	3. Seismic Max Reaction on Fix End	16.90	21.14
	4. Seismic Min Reaction on Fix End	17.77	22.01
BOTH SPAN POSITION CONDITION(70R)			
	5. Normal Max Reaction on Fix End	11.20	11.20
	6. Normal Min Reaction on Fix End	15.55	15.55
	7. Seismic Max Reaction on Fix End	16.36	20.60
	8. Seismic Min Reaction on Fix End	17.23	21.47
BOTH SPAN POSITION CONDITION(Special Vehicle)			
	9. Normal Max Reaction on Fix End	8.55	8.55
	10. Normal Min Reaction on Fix End	9.45	9.45
BOTH SPAN POSITION CONDITION(NO LIVE LOAD)			
	11. Normal No Reaction	0.00	0.00
ONE SPAN DISLOGED CONDITION			
	12. Normal Span Dislodge Condition	13.39	13.39
	13. Seismic Span Dislodge Condition	25.51	29.75

11.A CALCULATION OF CENTRIFUGAL FORCE

As per clause 212 of IRC:6-2014

$$C = \frac{WV^2}{127R}$$

70R + Class A

Maxium Reaction				
Live Load	W	=	110.99	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0.00	17.12	19.12

Minimum Reaction				
Live Load	W	=	0.00	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0	17.12	19.12

70R

Maxium Reaction				
Live Load	W	=	110.99	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	17.12	19.12

Minimum Reaction				
Live Load	W	=	24.01	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	17.12	19.12

Special Vehicle**Maxium Reaction**

Live Load	W	=	189.08	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	17.12	19.12

Minimum Reaction

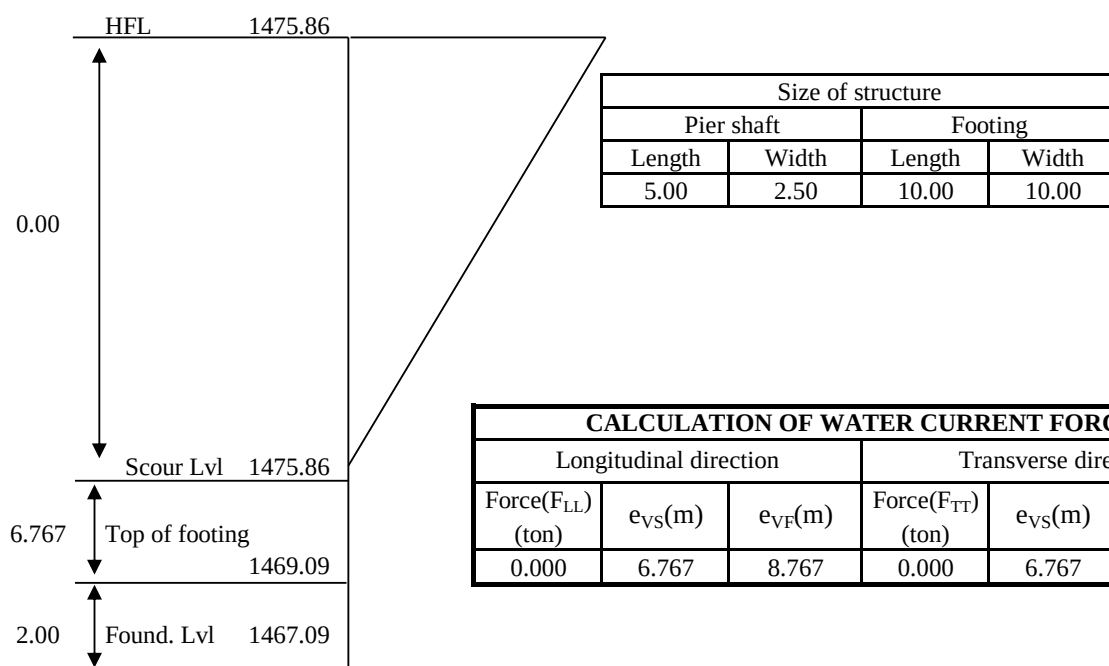
Live Load	W	=	170.92	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

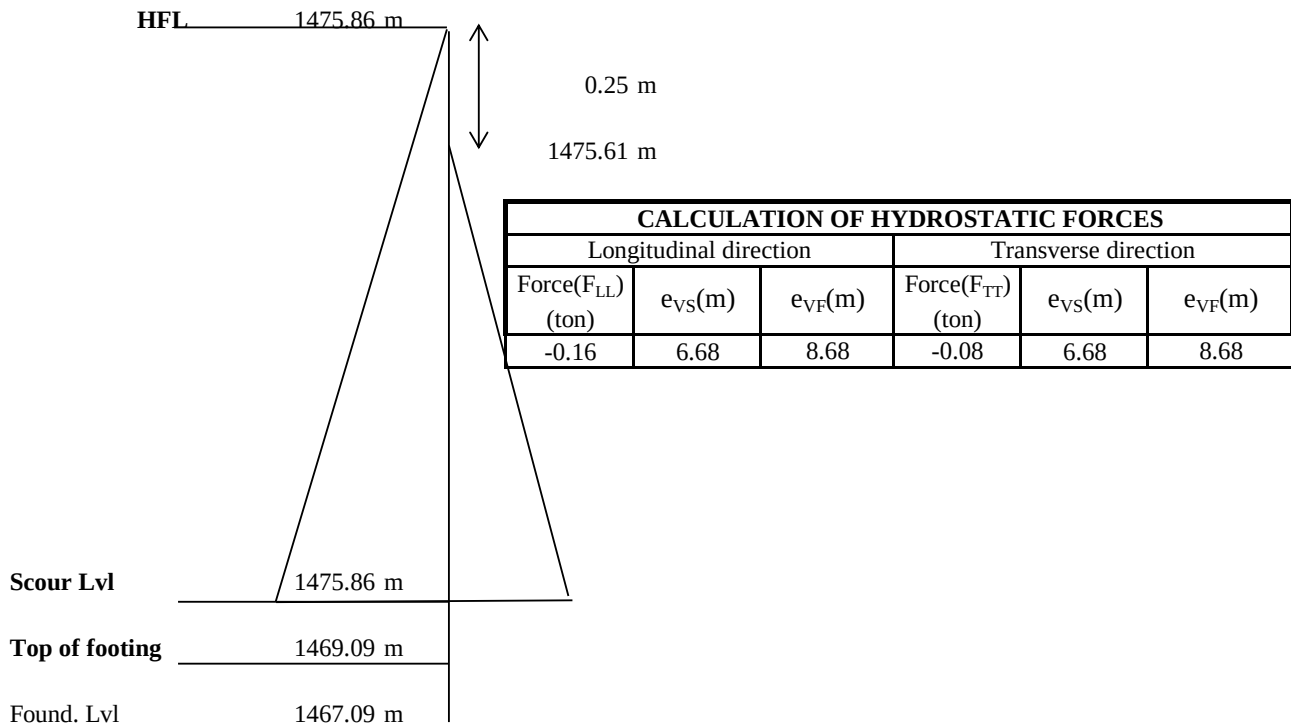
$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	17.12	19.12

12. WATER CURRENT FORCES**NORMAL CASE****Water current forces**

HFL	=	1475.857 m	
MSL (NORMAL)	=	1475.857 m	
Water current pressure	P	=	52 KV ² Kg/m ²
For piers with square ends	K	=	1.5
For piers with semicircular ends	K	=	0.66
Mean velocity of water current	V _{mean}	=	0.00 m/s
Max. velocity of water current	V	=	$\sqrt{2}$ x 0.00
		=	0.00 m/s
Angle of variation		=	20 degree
Water current pressure at HFL	(Trans direction)	=	0.00 t/m ² 52 K (vcosq) ²
Water current pressure at HFL	(Long direction)	=	0.00 t/m ² 53 K (vsinq) ²

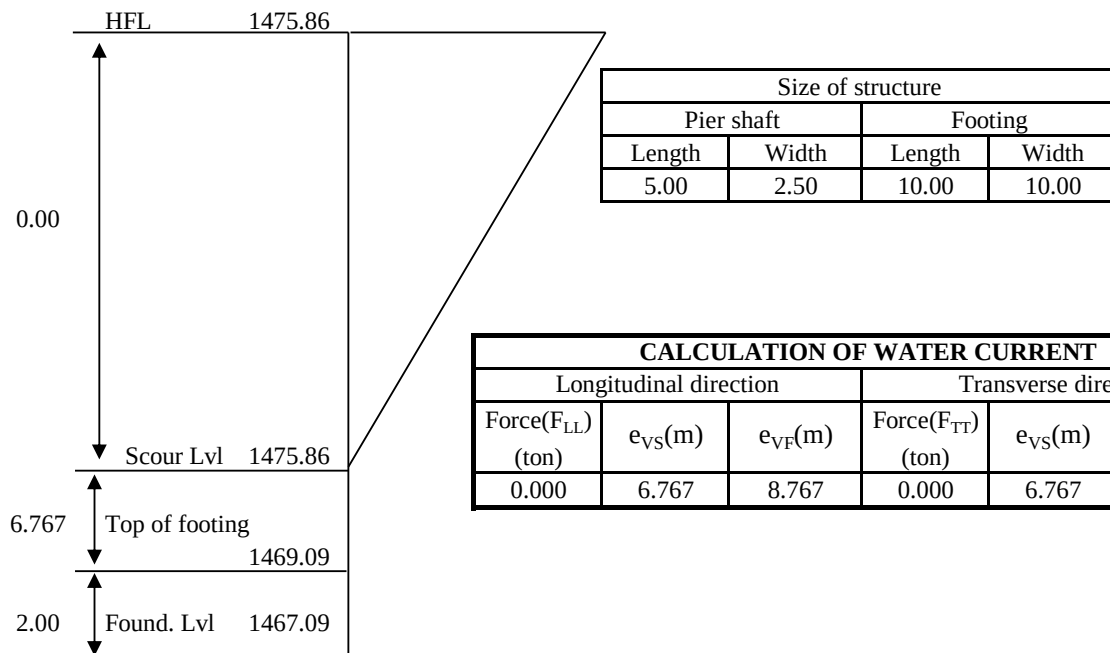
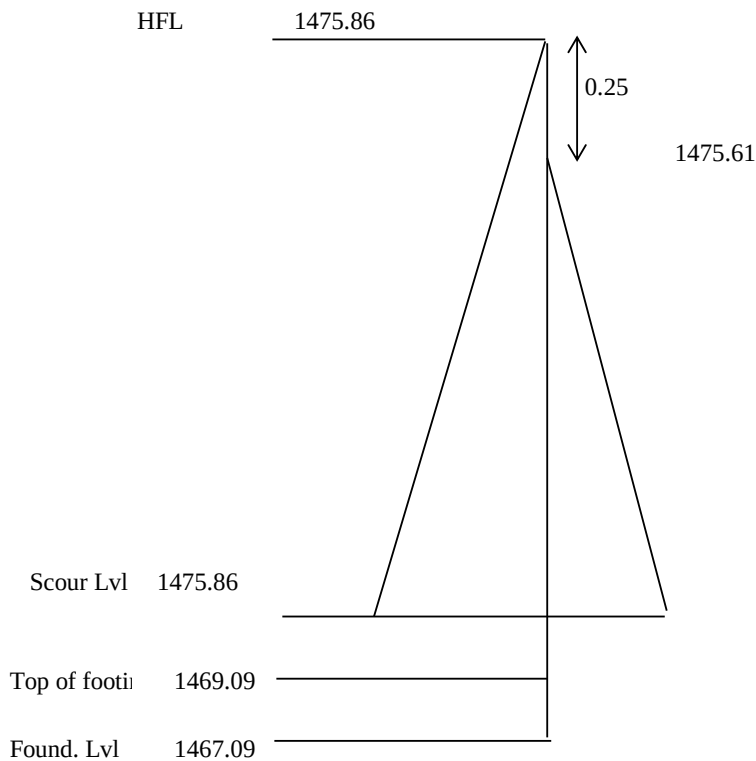
**Pressure diagram (Normal case)**

Static Force

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	6.767	8.767	0.000	6.767	8.767

SEISMIC CASE

HFL	=	1475.857 m		
MSL (SEISMIC)	=	1475.86 m		
Water current pressure	P	=	52 KV ²	Kg/m ²
For piers with square ends	K	=	1.5	
For piers with semicircular ends	K	=	0.66	
Mean velocity of water current	V_{mean}	=	0.00 m/s	
Max. velocity of water current	V	=	$\sqrt{2}$ x 0.00	
		=	0.00 m/s	
Angle of variation		=	20 degree	
Water current pressure at HFL	(Trans direction)	=	0.00 t/m ²	52 K (vcosq) ²
Water current pressure at HFL	(Long direction)	=	0.00 t/m ²	53 K (vsinq) ²

**Pressure diagram (seismic case)****Static Force**

CALCULATION OF HYDROSTATIC FORCES					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
-0.16	6.68	8.68	-0.08	6.68	8.68

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	6.767	8.767	0.000	6.767	8.767

13. LOAD COMBINATION DEFINATION

PIER SHAFT DESIGN

ULS

LC -1 ULS NORMAL LWL MAX REACTION
 LC -2 ULS NORMAL LWL SPAN DISLOGED

LC - 5 ULS MAX REACTION $1X + 0.3Y + 0.3Z$
 LC - 6 ULS MAX REACTION $0.3X + 1Y + 0.3Z$

LC - 9 ULS NORMAL SV MAX REACTION

SLS

LC -1 RARE NORMAL MAX REACTION
 LC -2 RARE NORMAL SPAN DISLOGED

LC -5 QP NORMAL

LC -7 QP NORMAL SPAN DISLOGED

LC -9 RARE NORMAL SV MAX REACTION

Footing design

LC - 1 ULS NORMAL MAX REACTION(70R +Class A)
 LC - 2 ULS NORMAL SPAN DISLOGED

LC - 5 ULS NORMAL MAX REACTION(70R)

LC - 7 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R + \text{Class A})$
 LC - 8 ULS SPAN DISLOGED $1X + 0.3Y + 0.3Z$
 LC - 9 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{Class A})$
 LC - 10 ULS SPAN DISLOGED $0.3X + 1Y + 0.3Z$

LC - 15 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R)$
 LC - 16 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 19 ULS NORMAL LWL SV MAX REACTION

BASE PRESSRE

LC - 1 NORMAL MAX REACTION(70R+Class A)

LC - 2 NORMAL SPAN DISLOGED

LC - 5 0

LC - 6 SEISMIC SPAN DISLODGED $1X + 0.3Y + 0.3Z$

LC - 7 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{class A})$

LC - 8 SEISMIC LWL SPAN DISLODGED $0.3X + 1Y + 0.3Z$

LC - 13 SEISMIC NORMAL MAX REACTION(70R)

LC - 15 SEISMIC MAX REACTION $1X + 0.3Y + 0.3Z(70R)$

LC - 16 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 13 NORMAL LWL SV MAX REACTION

14.A DESIGN FORCES FOR PIER SHAFT(ULS CASES)**LC -1 ULS NORMAL LWL MAX REACTION**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.453	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1.5	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1.5	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	1.5	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1.5	0.9
11	Centifugal force				0.000			17.117	0.0	0.000	1.5	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
17	Horizontal Force(F _{LL})				18.901			13.927	263.228		1.5	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1.5	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1208.76	488.49	44.95

LC -2 ULS NORMAL LWL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1.35	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1.5	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1.5	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	1.5	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1.5	0.9
11	Centifugal force				0.000			17.117	0.000	0.000	1.5	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
17	Horizontal Force(F _{LL})				13.387			13.927	186.441		1.5	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1.5	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
668.11	513.52	0.00

LC - 5 ULS MAX REACTION 1X + 0.3Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.63	0.000		-147.453	0.000	1.35	
2	Surfacing			31.815		-0.63	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.63	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.63	0.950		0.000	0.000	0.2	1
5	LL (Class-A)			0.000		-0.63	1.000		0.000	0.000	0.2	1
	Right Span											
6	DL (superstructure)			235.925		0.63	0.000		147.453	0.000	1.35	
7	Surfacing			31.815		0.63	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.63	0.000		0.000	0.000	0.2	
9	LL (70R)			110.988		0.63	0.300		69.368	33.3	0.2	1
10	LL (Class-A)			0.000		0.63	1.000		0.000	0.0	0.2	1
11	Centifugal force				0.000			17.117	0.0	0.0	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.000		0.000	0.000	1.35	
17	Horizontal Force(F _{LL})				16.898			13.927	235.342		1.000	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1.000	
18	Seismic											
	Super structure											
	Longitudinal				48.472				675.08		1.5	
	Transverse				45.272					670.30872	0.45	
	Vertical			32.315							0.45	
	Sub structure											
	Longitudinal				19.709				190.74		1.5	
	Transverse				18.408					178.14383	0.45	
	Vertical			13.139							0.45	
a)	Max Live Load											
	Longitudinal										1.5	
	Transverse				1.877					32.12321	0.45	
	Vertical			1.340							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
1085.535	1547.94	402.92

LC - 6 ULS MAX REACTION 0.3X +1Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.63	0.00		-147.45	0.00	1.35	
2	Surfacing			31.815		-0.63	0.00		-19.88	0.00	1.75	
3	FLL			0.000		-0.63	0.00		0.00	0.00	0.2	
4	LL (70R)			0.000		-0.63	0.95		0.00	0.00	0.2	1
5	LL (Class-A)			0.000		-0.63	1.00		0.00	0.00	0.2	1
	Right Span											
6	DL (superstructure)			235.925		0.63	0.00		147.45	0.00	1.35	
7	Surfacing			31.815		0.63	0.00		19.88	0.00	1.75	
8	FLL			0.000		0.63	0.00		0.00	0.00	0.2	
9	LL (70R)			110.988		0.63	0.30		69.37	33.30	0.2	1
10	LL (Class-A)			0.000		0.63	1.00		0.00	0.00	0.2	1
11	Centifugal force				0.000			17.117	0.0	0.000	0.2	
12	Pier cap	55.352	2.500	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.500	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.500	44.644		0.000	0.000		0.000	0.000	1.35	
17	Horizontal Force(F _{LL})				16.898			13.927	235.342		1.000	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1.000	
18	Seismic											
	Super structure											
	Longitudinal				48.472				675.08		0.45	
	Transverse				45.272					670.309	1.5	
	Vertical			32.315							0.45	
	Sub structure											
	Longitudinal				19.709				190.74		0.45	
	Transverse				18.408					178.144	1.5	
	Vertical			13.139							0.45	
a)	Max Live Load											
	Longitudinal										0.45	

Transverse				1.877					32.123	1.5	
Vertical			1.340							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
1085.535	638.83	1327.52

LC - 9 ULS NORMAL SV MAX REACTION

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.453	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
	Right Span											
5	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1.35	
6	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
7	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
8	LL			189.078		0.625	0.300		118.174	56.723	1	1
9	Centifugal force				0.000			17.117	0.000		1	
10	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
11	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
12	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
15	Horizontal Force(F _{LL})				8.546			13.927	119.021		1.5	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1.5	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1231.36	296.71	56.72

14.B DESIGN FORCES FOR PIER SHAFT(SLS CASES)**LC -1 RARE NORMAL MAX REACTION**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.453	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.2	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.2	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centrifugal force				0.000			17.117	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			6.767	0.000		1	
	Water Current Force(F _{TT})				0.000			6.767		0.000	1	
17	Horizontal Force(F _{LL})				18.901			13.927	263.228		1	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
876.92	325.66	29.97

LC-2 RARE NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.2	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.2	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force(F _{LL})				0.000			17.117	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			6.767	0.000		1	
	Water Current Force(F _{TT})				0.000			6.767		0.000	1	
17	Horizontal Force(F _{LL})				13.387			13.927	186.441		1	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
491.83	357.75	0.00

LC -5 QP NORMAL

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.453	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.2	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.2	
8	FLL			0.000		0.625	0.000		0.000	0.000	0	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	0	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centifugal force				0.000			17.117	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			6.767	0.000		0	
	Water Current Force(F _{TT})				0.000			6.767		0.000	0	
17	Horizontal Force(F _{LL})				0.000			13.927	0.000		0	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	0	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
765.93	0.00	0.00

LC -7 QP NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.2	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.2	
8	FLL			0.000		0.625	0.000		0.000	0.000	0	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centifugal force				0.000			17.117	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0	
16	Water Current Force(F _{LL})				0.000			6.767	0.000		0	
	Water Current Force(F _{TT})				0.000			6.767		0.000	0	
17	Horizontal Force(F _{LL})				13.387			13.927	186.441		1	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
491.83	357.75	0.00

LC -9 RARE NORMAL SV MAX REACTION

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.453	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.2	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
	Right Span											
5	DL (superstructure)			235.925		0.625	0.000		147.453	0.000	1	
6	Surfacing			31.815		0.625	0.000		19.884	0.000	1.2	
7	FLL			0.000		0.625	0.000		0.000	0.000	1	
8	LL			189.078		0.625	0.300		118.174	56.723	1	1
9	Centifugal force				0.000			17.117	0.000		1	
10	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
11	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
12	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
13	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
14	Water Current Force(F _{LL})				0.000			6.767	0.000		1	
	Water Current Force(F _{TT})				0.000			6.767		0.000	1	
15	Horizontal Force(F _{LL})				8.546			13.927	119.021		1	
	Horizontal Force(F _{TT})				0.000			13.927		0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
955.01	237.20	56.72

15. DESIGN FORCES FOR FOUNDATION**LC -1 ULS NORMAL MAX REACTION(70R +Class A)**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors (Design)	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1.5	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1.5	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1.5	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1.5	0.9
11	Centifugal force				0.000			19.117	0.0	0.000	1.5	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				18.901			15.927	301.03		1.5	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1.5	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1.35	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1.35	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
3620.68	545.19	44.95

LC -2 ULS NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors (Design)	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1.35	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1.5	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1.5	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	1.5	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1.5	0.9
11	Centrifugal force				0.000			19.117	0.000	0.000	1.5	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				13.387			15.927	213.215		1.5	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1.5	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1.35	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1.35	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
3096.67	553.68	0.00

LC -5 ULS NORMAL MAX REACTION(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors (Design)	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1.5	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1.5	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centrifugal force				0.000			19.117	0.0	0.000	1.5	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				11.201			15.927	178.39		1.5	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1.5	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1.35	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1.35	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1740.551	361.23	44.95

LC - 7 ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R + Class A)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0.2	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0.2	
11	Centrifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.352	2.500	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.500	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.500	44.644		0.000	0.000		0.000	0.000	1.35	
15	Buoyancy	0.000	1.000	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				21.140			15.927	336.69		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.00	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				65.438				1042.2		1.5	
	Transverse				61.117					1022.6	0.45	
	Vertical			43.625							0.45	
	Sub structure											
	Longitudinal				26.607				307.6		1.5	
	Transverse				24.850					287.32285	0.45	
	Vertical			17.74							0.45	
a)	Max Live Load											
	Longitudinal										1.5	
	Transverse				2.534					48.433382	0.45	
	Vertical			1.808							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
3521.519	2375.36	619.33

LC - 8 ULS SPAN DISLODGED 1X + 0.3Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1.35	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.00	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1.35	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1.35	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1.35	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				29.746			15.927	473.77		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				32.719				521.11		1.5	
	Transverse				30.558					490.38645	0.45	
	Vertical			21.813							0.45	
	Sub structure											
	Longitudinal				26.607				307.6		1.5	
	Transverse				24.850					287.32285	0.45	
	Vertical			17.738							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
3114.465	1751.70	349.97

LC - 9 ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R +Class A)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0.2	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0.2	
11	Centifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1.35	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1.35	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1.35	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				21.140			15.927	336.69		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				65.438				1042.2		0.45	
	Transverse				61.117					1022.585	1.5	
	Vertical			43.625							0.45	
	Sub structure											
	Longitudinal				26.607				307.6		0.45	
	Transverse				24.850					287.323	1.5	
	Vertical			17.738							0.45	
a)	Max Live Load											
	Longitudinal										0.45	
	Transverse				2.534					48.433	1.5	
	Vertical			1.808							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
3521.519	958.00	2059.89

LC - 10 ULS SPAN DISLODGED 0.3X + 1Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1.35	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centrifugal force				0.000			19.117	0.00	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1.35	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1.35	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1.35	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				29.746			15.927	473.77		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				32.719				521.11		0.45	
	Transverse				30.558					490.38645	1.5	
	Vertical			21.813							0.45	
	Sub structure											
	Longitudinal				26.607				307.6		0.45	
	Transverse				24.850					287.32285	1.5	
	Vertical			17.738							0.45	

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
3114.465	881.51	1166.56

LC -15 ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.352	2.500	138.380		0.000	0.000		0.000	0.000	1.35	
13	Pier (above LBL)	13.881	2.500	34.702		0.000	0.000		0.000	0.000	1.35	
14	Pier (below LBL)	17.858	2.500	44.644		0.000	0.000		0.000	0.000	1.35	
15	Buoyancy	0.000	1.000	0.000		0.000	0.000		0.000	0.000	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				20.600			15.927	328.09		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.00	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				65.438				1042.2		1.5	
	Transverse				61.117					1022.6	0.45	
	Vertical			43.625							0.45	
	Sub structure											
	Longitudinal				26.607				307.6		1.5	
	Transverse				24.850					287.32285	0.45	
	Vertical			17.74							0.45	
a)	Max Live Load											
	Longitudinal										1.5	
	Transverse				2.534					48.43	0.45	
	Vertical			1.808							0.45	
b)	Min Live Load											
	Longitudinal										1.5	1
	Transverse				0.548					10.478285	0.45	0.3
	Vertical			0.391							0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	MTT (tm)	MLL(tm)
3521.519	2366.76	619.33

LC -16 ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VS} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	0.2	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
8	FLL			0.000		0.625	0.000		0.000	0.000	0.2	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1.35	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1.35	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1.35	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	0.15	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				20.600			15.927	328.09		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1.35	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1.35	
20	Seismic											
	Super structure											
	Longitudinal				65.438			1042.2			0.45	
	Transverse				61.117					1022.585	1.5	
	Vertical			43.625							0.45	
	Sub structure											
	Longitudinal				26.607			307.6			0.45	
	Transverse				24.850					287.323	1.5	
	Vertical			17.738							0.45	
a)	Max Live Load											
	Longitudinal										0.45	
	Transverse				2.534					48.433	1.5	
	Vertical			1.808							0.45	
b)	Min Live Load											
	Longitudinal										0.45	0.3

	Transverse				0.548					10.478285	1.5	1
	Vertical			0.391							0.45	0.3
SUMMARY OF FACTORED FORCES												
V(ton)		MTT (tm)	MLL(tm)									
3521.519		949.40	2059.89									

LC - 19 ULS NORMAL SV MAX REACTION

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors (Design)	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1.35	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1.75	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1.5	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
Right Span												
5	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1.35	
6	Surfacing			31.815		0.625	0.000		19.884	0.000	1.75	
7	FLL			0.000		0.625	0.000		0.000	0.000	1.5	
8	LL			189.078		0.625	0.300		118.174	56.723	1	1
9	Centifugal force				0.000			19.117	0.0	0.000	1	
10	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1.35	
11	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1.35	
12	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1.35	
13	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	0.15	
14	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
15	Horizontal Force(F _{LL})				8.546			15.927	136.11		1.5	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1.5	
16	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1.35	
17	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1.35	

SUMMARY OF FACTORED FORCES

V(ton)	M_{TT} (tm)	M_{LL}(tm)
3659.92	322.34	56.72

16. DESIGN FORCES FOR BASE PRESSURE**LC -1 NORMAL MAX REACTION(70R+Class A)**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			19.117	0.0	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				18.901			15.927	301.03		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
2652.03	363.46	29.97

LC -2 NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			19.117	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				13.387			15.927	213.215		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
2284.40	380.55	0.00

380.552 0.000

LC - 6 SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.00	0.000	0	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	

	Water Current Force(F_{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F_{LL})				29.746			15.927	473.77		1	
	Horizontal Force(F_{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				32.719				521.11		1	
	Transverse				22.636					363.24922	0.3	
	Vertical			16.157							0.3	
	Sub structure											
	Longitudinal				26.607				307.6		1	
	Transverse				24.850					287.32	0.3	
	Vertical			17.738							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M_{TT} (tm)	M_{LL} (tm)
2294.569	1322.41	195.17

LC - 8 SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0.2	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0.2	
11	Centifugal force				0.000			19.117	0.00	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				29.746			15.927	473.77		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				0.000			0.00			0.3	
	Transverse				22.636					363.24922	1	
	Vertical			16.157							0.3	
	Sub structure											
	Longitudinal				26.607				307.6		0.3	
	Transverse				24.850					287.32285	1	
	Vertical			17.738							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
2294.569	585.95	650.57

LC -13 SEISMIC NORMAL MAX REACTION(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centifugal force				0.000			19.117	0.0	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				11.201			15.927	178.39		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
2652.03	240.82	29.97

LC - 15 SEISMIC MAX REACTION 1X + 0.3Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.352	2.500	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	13.881	2.500	34.702		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	17.858	2.500	44.644		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1.000	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.000		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				20.600			15.927	328.09		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.00	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				65.438			1042.2			1	
	Transverse				61.117					1022.6	0.3	
	Vertical			43.625							0.3	
	Sub structure											
	Longitudinal				26.607			307.6			1	
	Transverse				24.850					287.32	0.3	
	Vertical			17.74							0.3	
a)	Max Live Load											
	Longitudinal										1	
	Transverse				2.534					48.43	0.3	
	Vertical			1.808							0.3	

b)	Min Live Load											
	Longitudinal										1.5	1
	Transverse				0.548					10.478285	0.45	0.3
	Vertical			0.391							0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
2593.342	1691.83	415.58

LC - 16 SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			19.117	0.0	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	13.88	2.50	34.70		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	17.86	2.50	44.64		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			8.767	0.00		1	
	Water Current Force(F _{TT})				0.000			8.767		0.000	1	
17	Horizontal Force(F _{LL})				20.600			15.927	328.09		1	
	Horizontal Force(F _{TT})				0.000			15.927		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	696.34	2.00	1392.68		0.00	0.00		0.00	0.00	1	
20	Seismic											

	Super structure											
	Longitudinal				65.438				1042.2		0.3	
	Transverse				61.117				1022.6		1	
	Vertical		43.625								0.3	
	Sub structure											
	Longitudinal				26.607				307.6		0.3	
	Transverse				24.850				287.32		1	
	Vertical		17.738								0.3	
a)	Max Live Load											
	Longitudinal										0.3	
	Transverse				2.534				48.43		1	
	Vertical		1.808								0.3	
b)	Min Live Load											
	Longitudinal										0.45	0.3
	Transverse				0.548				10.5		1.5	1
	Vertical		0.391								0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
2593.342	746.92	1380.72

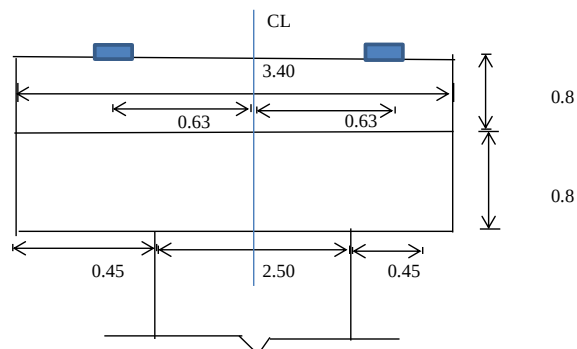
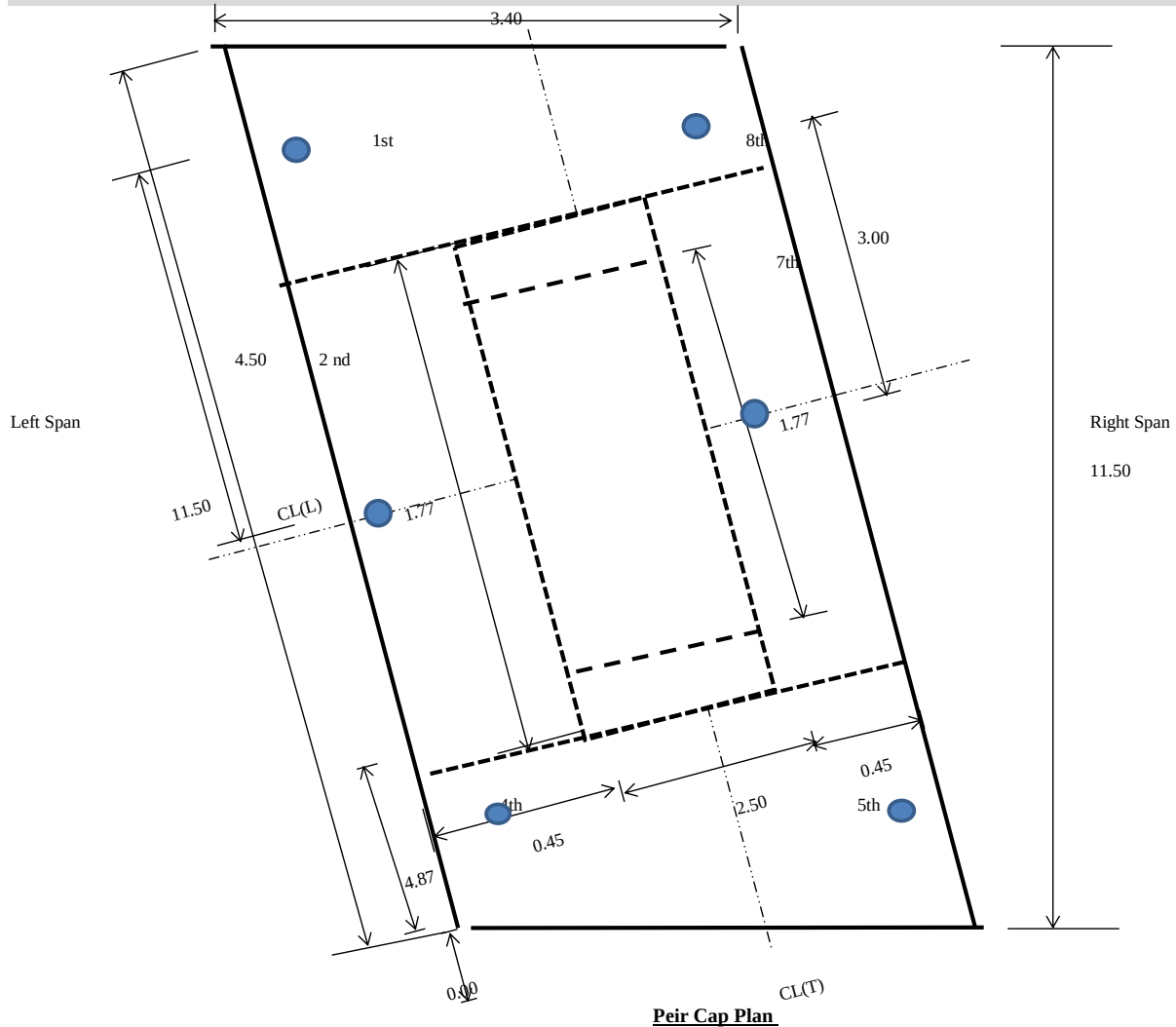
LC - 18 NORMAL SV MAX REACTION

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
	Right Span											
5	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
6	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
7	FLL			0.000		0.625	0.000		0.000	0.000	1	
8	LL			189.078		0.625	0.300		118.174	56.723	1	1
9	Centifugal force				0.000			19.117	0.0	0.000	1	
10	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
11	Pier (above LBL)	13.881	2.5	34.702		0.000	0.000		0.000	0.000	1	

12	Pier (below LBL)	17.858	2.5	44.644		0.000	0.000		0.000	0.000	1	
13	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
14	Water Current Force(F_{LL})				0.000			8.767	0.000		1	
	Water Current Force(F_{TT})				0.000			8.767		0.000	1	
15	Horizontal Force(F_{LL})				8.546			15.927	136.11		1	
	Horizontal Force(F_{TT})				0.000			15.927		0.000	1	
16	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
17	Earth Over Footing	696.34	2	1392.685		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M_{TT} (tm)	M_{LL} (tm)
2741.22	254.29	56.72

17. DESIGN OF PIER CAP IN TRANSVERSE DIRECTION

Lever arm for 1st bearing
 Lever arm for 8th bearing
 width of wquivalentsquare
 Overall depth
 Clear cover to main reinforcement
 Dia of main reinforcement
 Dia of transverse reinforcement
 Effective depth
 (Considering two layer reinforcement)

Depth at effective depth
 Effective depth at effective depth

Hence ,

1st bearing
 8th bearing

a1	=	3.616	m	2nd brg.=	0.000	m
a8	=	2.116	m	7th brg =	0.000	m
	=	1.768	m			
h	=	1.6	m			
c	=	50	mm			
f	=	32	mm			
f	=	16	mm			
d	=	1.486	m			
D	=	1.356	m			
d'	=	0.8	m			
a1/d	=	2.433457	m			
a8/d	=	1.424035	m			

>1.0 Design as cantilever
 >1.0 Design as cantilever

1. CALCULATION OF FORCES**1.1 . SELF WEIGHT OF PIER CAP**Density of concrete = 25 KN/m³

Description	Unit	At face of Pier Shaft	At deff.		
Volumn of straight portion(Top)	m ³	13.236	9.194		
Weight of straight portion	KN	330.896	229.8479		
Lever arm	m	2.433	1.690		
Torsional eccentricity	m	0	0		
Volumn of straight portion(Bottom)	m ³	0.996	0		
Weight of straight portion		24.896	0		
Lever arm		0.183	0		
Torsional eccentricity		0	0		
Volumn of tapered portion	m ²	6.618	7.790		
Weight of tapered portion	KN	165.448	194.7528		
Lever arm	m	1.622	1.1267		
Torsional eccentricity	m	0	0		
Total Weight	KN	521.240	424.601		
Lever arm	m	2.07	1.43		
Shear force at the section	KN	521.240	424.601		
BM at the section	KN-m	1078.01	607.89		
Torsional moment at the section	KN-m	0	0		

1.2. LOAD FROM SUPERSTRUCTURE

Left Span Dominating						
Description	Left Span(Brg 2)			Right Span(Brg 7)		
	V(KN)	Eccentricity for moment	Eccentricity for Torsion	V(KN)	Eccentricity for moment	Eccentricity for Torsion
DL	589.81	0.000	0.625	589.81	0.000	-0.625
Surfacing	79.54	0.000	0.625	79.54	0.000	-0.625
FPLL	0.00	0.000	0.625	0.00	0.000	-0.625
Class A	0.00	0.000	0.625	0.00	0.000	-0.625
70R	0.00	0.000	0.625	0.00	0.000	-0.625

Description	Shear at face of support	Shear at deff	oment at fa	Torsion
DL	1179.63	1179.625	0.000	0.000
Surfacing	159.07	159.074	0.000	0.000
FPLL	0.00	0.0002	0.00	0.000
Class A	0.00	0	0.00	0
70R	0.00	0	0.00	0

DESIGN MOMENT AND SHEAR

Description	Left Span(Brg 1)			Right Span(Brg 8)		
	V(KN)	Eccentricity for moment	Eccentricity for Torsion	V(KN)	Eccentricity for moment	Eccentricity for Torsion
DL including Crashbarrier	589.81	3.616	0.625	589.81	2.116	-0.625
Surfacing	79.54	3.616	0.625	79.54	2.116	-0.625
FPLL	0.00	3.616	0.625	0.00	2.116	-0.625
3L Class A	0.00	3.616	0.625	0.00	2.116	-0.625
70R	0.00	3.616	0.625	443.95	2.116	-0.625

Design Forces Normal Case				
Description	Shear at face of support	Shear at deff	Moment at face	Torsion
Due to Cap Self weight	521.240	424.601	1078.01	0
DL including Crashbarrier	2359.25	2359.25	3380.94	0.00
Surfacing	318.15	318.15	455.92	0.00
FPLL	0.00	0.00	0.00	0.00
3L Class A Including Impact	0.00	0.00	0.00	0.00
70R Including Impact	551.22	684.40	1166.44	344.51

Design Seismic forces				
Av = 0.06034773				
Description	Shear at face of support	Shear at deff	Moment at face	Torsion
Due to Cap Self weight	31	25.62	65	0.00
DL including Crashbarrier	142	142.38	204	0.00
Surfacing	19	19.20	28	0.00
FPLL	0	0.00	0	0.00
Class A(20% LL) Excluding Impact	0	0.00	0	0.00
70R Excluding Impact(20 % LL)	5	6.65	11	3.35

Load Combination 1 : Normal

Description	Factor for Strength(Basic)	Factored Shear at deff kN	Factored moment at face kNm	Factored Torsion(KNm)
Due to Cap Self weight	1.35	573.21	1455.31	0
DL(Super Structure+Crashbarrier)	1.35	3184.99	4564.27	0
Surfacing	1.75	556.76	797.87	0
FPLL	1.5	0.00	0.00	0
Class A	1.5	0.00	0.00	0
70R	1.5	1026.60	1749.66	517
Total		5342	8567	517

Load Combination 2 : Seismic

Description	Factor for Strength(Seismic)	Factored Shear at deff kN	Factored moment at face kNm	Factored Torsion(KNm)
Due to Cap Self weight	1.35	573	1455	0
DL(Super Structure+Crashbarrier)	1.35	3185	4564	0
Surfacing	1.75	557	798	0
FPLL	0.2	0	0	0
Class A	0.2	0	0	0
70R	0.2	137	233	69
Seismic Load				
Due to Cap Self weight	1.5	38	98	0
DL(Super Structure+Crashbarrier)	1.5	214	306	0
Surfacing	1.5	29	41	0
FPLL	1.5	0	0	0
Class A	1.5	0	0	0
70R	1.5	10	17	5
Total		4743	7513	74

Design Forces		
Factored Shear at deff kN	Factored moment at face KNm	Factored Torsion(KNm)
5342	8567	517

DESIGN AS CANTILEVER

2.1 Design for flexure

Mu	=	8567 KNm
b	=	3400 mm
Fck	=	35 N/mm2
Fcd	=	15.63 N/mm2
Fyk	=	500 N/mm2
Fyd	=	435 N/mm2
Es	=	200000 N/mm2
xu,lim/d	=	0.617
$R_{lim} = M_{u,lim}/bd^2$		5.803

Effective depth required	=	0.659 m
$R = Mu/bd^2$	=	1.141
Ast required	=	13795 mm2
Minimum Ast required	=	7281 mm2
Governing Ast in Flexure	=	13795 mm2
Ast from torsion	=	4268 mm2
Total Ast required	=	18063 mm2

Provided	32	dia at	30	Nos	24127 mm2	>	18063 mm2	OK
	0	dia at	30	Nos				
	0	dia at	30	Nos				

2.2 Design for Shear

Design Shear	=	5341553 N
Shear due to torsion	=	157159 N
Total Shear	=	5498712 N

Design shear resistance of the member with shear reinforcement is given by:

Asw	=	855	mm2				
fywd	=	435	Mpa				
v1	=	0.53					
z	=	720.00	mm				
acw	=	1					
q	=	21.80	°	Tan(q)	=	0.4000	
				Cot(q)	=	2.5002	
Provided outer	16	dia	2	legged	at spacing	100	mm
Inner	12	dia	4	legged	at spacing	100	mm

$V_{RD,S}$	=	6687972 N					
$V_{RD,Max}$	=	7023681.621 N					
V_{RD}	=	Min(6687972,7023682)	6687972 N	>	5498712 N	Ok	

Minimum Shear Reinforcement

$$r_{\min} = \frac{0.072 \sqrt{f_{ck}}}{f_{yk}}$$

$$= \frac{0.001}{0.001}$$

As per clause 10.3.3.5 of IRC:112-2011 minimum shear reinforcement required

$$A_{sw,\min} = r_{\min} \times s \times b_w$$

$$A_{sw,\min} = 289.651 \text{ mm}^2 < 854.51 \text{ mm}^2 \quad \text{Ok}$$

$$\text{Maximum Spacing} = 0.75 \times d' = 600 \text{ mm} > 100 \text{ mm} \quad \text{Ok}$$

2.3 Design for Torsion

Torsional Inertia of section

$$b = 3.4 \text{ m}$$

$$h = 1.6 \text{ m}$$

$$I_{\text{tor}} = \frac{3b^3d^3}{10(b^2+d^2)} = 3.420450085 \text{ m}^4$$

Ultimate Torsion Moment , Ted	kNm	517
Total Area of section , A	m2	5.44
Total Perimeter of section , u	m	10
Effective cover	mm	114
tef,i = max(A / u, 2*eff. Cover)	m	0.54
Area of section , Ak	m2	1.18
Perimeter of section , uk	m	5.65
Shear stress due to pure Torsion, tt,i = TE _d / (2 .Ak. tef,i)	N/mm2	0.40
Shear force due to torsion =VE _d ,i = tt,i . tef . Zi	kN	157.16
Mean compr. stress at c.g. sec. scp = NE _d / Ac ≤ 0.2 fcd	N/mm2	0
Coefficient acw=1+scp /fcd		1
strength reduction factor for section cracked in shear = n = 0.6 [1 - fck/310]		0.532
cot θ		2.50
θ smallest angle before concrete crushing starts hence	deg	21.8
Maximum Torsional Moment TR _{d,max} = 2 . n . acw. fcd .Ak. tef. sinq . cosq	kNm	3695.22
Torsional Long. Reinf. Asl = TE _d . cot q*s /(2 . Ak. fyd)	mm2	4267.60
Total torsional moment, TR _{d,max}	kNm	3695.22
Total shear force due to torsion, V _{edi}	kN	157.16
Design shear force, Vu	kN	5342
Total shear Force , VE _d =Vu	kN	5498.71
Maximum shear capacity VR _{d,max}	kN	7024
TE _d /TR _{d,max} + VE _d / VR _{d,max} ≤ 1.0		0.88
		ok

$$\tau_{t,i} = \frac{T_{Ed}}{2 A_k t_{ef,i}} \quad \text{Eq. 10.46}$$

$$T_{Rd,\max} = 2v\alpha_{cw}f_{cd}A_k t_{ef,i} \sin \theta \cos \theta \quad \text{Eq. 10.48}$$

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2 A_k} \cot \theta \quad \text{Eq. 10.49}$$

$$V_{Rd,\max} = \alpha_{cw} b_w z v_1 \frac{f_{cd}}{(\cot \theta + \tan \theta)} \quad \text{Eq. 10.8}$$

18. DESIGN OF PIER SHAFT

Grade of Concrete	:	35 MPa		
Grade of Steel	:	500 MPa		
ym	:	1.5 (1.5 for Basic & Seismic combination, 1.2 for accidental combination)		
ys	:	1.15 (1.15 for basic and Seismic combination and 1.0 for accidental combination)		
Es	:	200000 MPa		
Clear Cover for outer surface	:	50 mm		
Clear cover for Inner surface	:	50 mm		
fyd	:	434.8 MPa		
fcd	:	15.63 MPa		
fcm	:	45 MPa		
fctm	:	2.77 MPa		
Ecm	:	32000 MPa		
ec2	:	0.002		
ecu2	:	0.0035		
n	:	2		
Outer Diameter	:	2500 mm	Inner Diameter	: 1700 mm
Thickness	:	400 mm	Inner Radius	: 850 mm
Outer Radius	:	1250 mm		
Dia of Bar in First layer	:	32 mm	Area of bar	804 mm ²
Dia of Bar in Second layer	:	25 mm	Area of bar	490.8734375 mm ²
Spacing b/w Layer	:			
Effective Cover for outer layer	:	66 mm		
Effective Cover for Inner layer	:	62.5 mm		
C/C distance between layer	:	272 mm		
Radius of outer layer	:	1184 mm		
Radius of Inner layer	:	912.5 mm		
Number of Bar in outer layer	:	40		
Number of bar in inner layer	:	40		
Spacing in outer Side	:	186 mm		
Spacing in inner Side	:	143 mm		

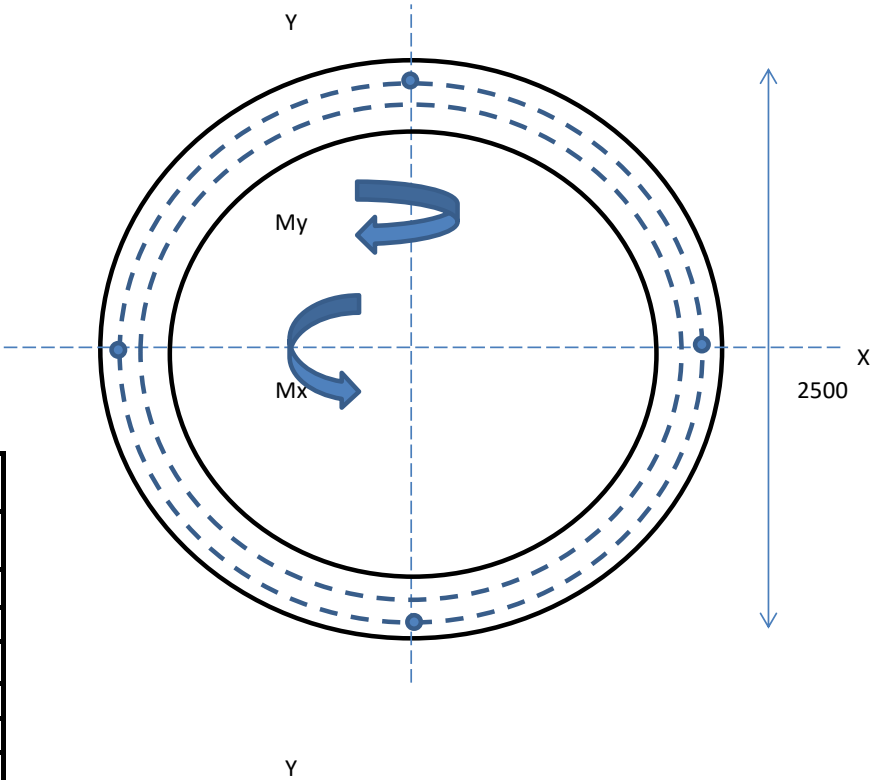
As	:	51804.82	mm ²
p	:	1.963	%
p/fck	:	0.056	
d'/D	:	0.03	

number of division of circular section(outer)
number of division of circular section(inner)

: 100
: 100

ULS CASE

Load Combination	P	M _{LL}	M _{TT}	M _R
	KN	KNm	KNm	KNm
LC1	12087.62	449.50	4884.89	11152.84
LC2	6681.05	0.00	5135.20	9273.25
LC5	10855.35	4029.18	15479.38	22319.3
LC6	10855.35	13275.23	6388.32	21056.4
LC9	12313.58	567.23	2967.06	9230.7



Final Moments with Slenderness effect

Load Combination	P	M _{LL}	M _{TT}	M _R
	KN	KNm	KNm	KNm
LC1	12087.62	3553.93	7989.32	8744.12
LC2	6681.05	1895.34	7044.69	7295.21
LC5	10855.35	6462.98	18024.49	19148.2
LC6	10855.35	15672.24	9166.99	18156.3

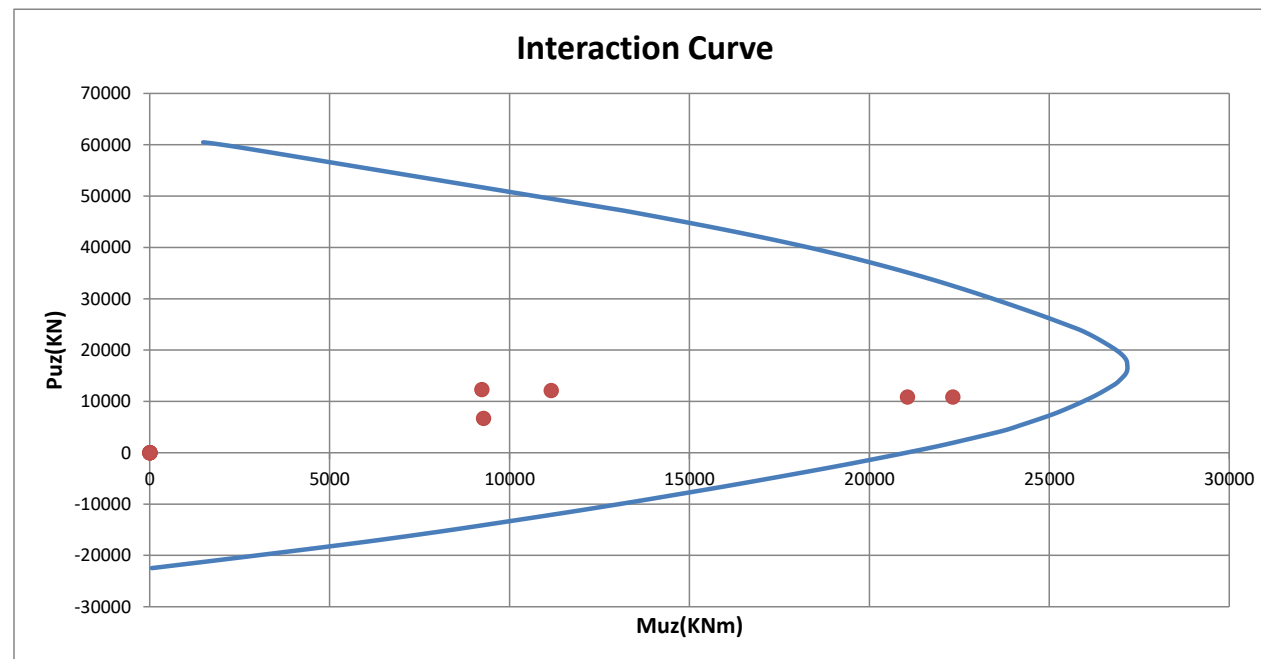
SLS CASE**Final Moments with Slenderness effect**

Load Combination	P	M _{LL}	M _{TT}	M _R
	KN	KNm	KNm	KNm
LC1	8769.20	299.67	3256.59	3270.35
LC2	4918.29	0.00	3577.55	3577.55
LC3	9550.10	567.23	2371.95	2438.83

Peir	:	P2
Level	:	2

Grade of Concrete	:	35 MPa
Grade of Steel	:	500 MPa
ym	:	1.5
ys	:	1.15
Es	:	200000 MPa
Clear Cover for outer surface	:	50 mm
Clear cover for Inner surface	:	50 mm
Outer Diameter	:	2500 mm
Thickness	:	400 mm
Dia of Bar in Outer Side	:	32 mm
Dia of Bar in Inner Side	:	25 mm
Number of Bar in Outer Side	:	40
Number of bar in Inner Side	:	40
Spacing of bar in outer side	:	186 mm
Spacing of bar in inner side	:	143 mm

As	:	51804.82	mm ²
p	:	1.963	%
p/fck	:	0.056	
d'/D	:	0.026	



Load Case	Pu	Mux	Muy	$Mur=(Mux^2+Muy^2)^{0.5}$	Muz	Mur/Muz	Check
	KN	KNm	KNm	KNm	KNm		
1	12087.6	3553.9	7989.3	8744.123	26523.438	0.330	Ok
2	6681.1	1895.3	7044.7	7295.206	24767.412	0.295	Ok
3							
4							
5	10855.3	6463.0	18024.5	19148.165	26197.026	0.731	Ok
6	10855.3	15672.2	9167.0	18156.348	26197.026	0.693	Ok
7							

SLS CHECK OF PIER SHAFT USING OYASIS

Section : Elastic Properties

1 : Hollow Circular

Effective properties of the section, ignoring reinforcement.

Geometric Centroid	y	0.0mm
	z	0.0mm
Area		2.639E+6mm ²
Second Moments of Area	I _{yy}	1.507E+12mm ⁴
	I _{zz}	1.507E+12mm ⁴
	I _{yz}	0.0mm ⁴
Principal Second Moments of Area	I _{uu}	1.507E+12mm ⁴
	I _{zz}	1.507E+12mm ⁴
	Angle	0.0°
Shear Area Factor	k _y	0.5000
	k _z	0.5000
Torsion Constant		3.015E+12mm ⁴
Section Modulus	Z _y	1.206E+9mm ³
	Z _z	1.206E+9mm ³
Plastic Modulus	Z _{py}	1.785E+9mm ³
	Z _{pz}	1.785E+9mm ³
Radius of Gyration	R _y	755.8mm
	R _z	755.8mm

Properties of gross section, including reinforcement.

Geometric Centroid	y	-6.721E-6mm
	z	5.949E-6mm
EA		87.99E+6kN
EI	E _{yy}	50.40E+6kNm ²
	E _{zz}	50.40E+6kNm ²
	E _{yz}	-0.1541kNm ²
Principal EI	E _{uu}	50.40E+6kNm ²
	E _{zz}	50.40E+6kNm ²
	Angle	-88.92°

Maximum compressive force	N _u	65680.kN
Strain at N _{max}		0.003500
Moment at ref. pt. for N _{max}	M _{yy}	0.04610kNm
	M _{zz}	0.2277kNm

Note: N_{max} is the maximum compressive force which can be carried by the section.
This is calculated by applying a constant strain across the entire section, using ultimate material properties.

Material : Concrete

1 : M35

Name M35
 (Modified)
 Cube fck 35.00N/mm2
 Strength
 Tensile fcr 4.141N/mm2
 Strength
 Weight Normal
 Weight
 Density r 2.400t/m3
 Elastic E 29580.N/mm2
 Modulus
 Poisson's n 0.2000
 Ratio
 Coeff. a 10.00E-6/°C
 Thermal
 Expansion
 Partial gm 1.500
 Safety
 Factor
 Maximum 0.003500
 Strain
 ULS Parabola
 Compression rectangle
 Curve
 ULS Tension No-tension
 Curve
 SLS Linear
 Compression
 Curve
 SLS Tension BS8110 Part
 Curve 2
 Design a 0.6700
 strength
 factor
 Aggregate 0.0mm
 Size

Material : Rebar

0.388889

Type Steel rebar
 Strength fy 500.0N/mm2
 Elastic Modulus E 200000.N/mm2
 Hardening Modulus Eh 0.0N/mm2
 Density r 7.850t/m3
 Poisson's Ratio n 0.3000
 Coeff. Thermal Expansion a 12.00E-6/°C
 Ductility Normal
 Partial Safety Factor gm 1.150
 gme 1.000
 Maximum Strain eu 0.004174

Maximum Strain eud 0.004174
 Stress/Strain Curve Elastic-plastic

Loading

Applied loads

Load Case	N	Myy	Mzz	Note
	[kN]	[kNm]	[kNm]	
1	10620.	7964.	14000.	
2	6079.	4088.	9529.	
3	9341.	8793.	17480.	
4	9341.	15290.	11150.	
5	7701.	905.6	4931.	
6	4479.	0.0	3744.	
7	8568.	577.9	2894.	

Section 1 Details

2.21% reinforcement in section 1 (Hollow Circular). Check this against code requirements.

SLS Loads Analysis Summary

Case Secant Neutral Neutral k at M0

El	Axis	Axis	
	Angle	Depth	
	[kNm ²]	[°]	[mm] [m]
1	50.39E+6	-79.59	2130. 179.3E-12
2	50.39E+6	-90.00	1936. 172.5E-12
3	50.39E+6	-78.71	2913. 177.2E-12

Moment summary for SLS loads

Effective centroid is reported relative to the reference point.

Cracking moment is for short term loading

Case	Eff. Centroid	N	M	Mu	M/Mu	Mcr	Note
	y z						
	[mm] [mm]	[kN]	[kNm]	[kNm]		[kNm]	

Maxima

3	-329.7E-9	8.099E-6	8568.	2952.	26370.	0.1119	6044.
1	-329.7E-9	8.099E-6	7701.	5014.	25920.	0.1934	5657.
3	-329.7E-9	8.099E-6	8568.	2952.	26370.	0.1119	6044.

Minima

2	-329.7E-9	8.099E-6	4479.	3744.	24010.	0.1560	4170.
3	-329.7E-9	8.099E-6	8568.	2952.	26370.	0.1119	6044.
2	-329.7E-9	8.099E-6	4479.	3744.	24010.	0.1560	4170.

Section Material Stresses/Strains at SLS Loads

Case	Point	Coordinates		Strain	Stress	Notes
		y	z			
		[mm]	[mm]	[-]	[N/mm ²]	
Maxima						
1	13	1220.	278.4	211.9E-6	6.268	
1	13	1220.	278.4	211.9E-6	6.268	
Minima						
2	43	-1251.	14.92E-6	-42.08E-6	-1.245	
2	43	-1251.	14.92E-6	-42.08E-6	-1.245	

Reinforcement Stresses/Strains at SLS Loads

Case	Bar	Coordinates		Strain	Stress	Notes
		y	z			
		[mm]	[mm]	[-]	[N/mm ²]	
Maxima						
1	11	1166.	205.6	205.3E-6	41.06	500
1	11	1166.	205.6	205.3E-6	41.06	500
Minima						
2	35	-1183.	41.32	-37.02E-6	-7.404	500
2	35	-1183.	41.32	-37.02E-6	-7.404	500

Crack Widths at SLS Loads

Crack widths calculated at 20mm intervals

Note: Cover to controlling bar measured to different side from crack location

Case 1

Crack Numbers

6 to 7 15 to 23 29 to 32 40 to 41 51 to 52 60 to 68 74 to 77 85 to 86 96 to 97 105 to 113 119 to 122

Warning: Cmin < control bar diameter. Crack width not valid

Case 2

Crack Numbers

157 to 158 174 to 188 204 to 218

Note: Cover to controlling bar measured to different side from crack location

Case 2

Crack Numbers

7 to 8 16 to 24 30 to 33 41 to 42 52 to 53 61 to 69 75 to 78 86 to 87 97 to 98 106 to 114 120 to 123
 131 to 132
 142 to 143
 151 to 156
 159 to 160
 173
 179
 179 to 184
 189 to 190
 203
 209
 209 to 214
 219 to 220

Case Face Point Coordinates Strain Control Bar acr Cover h x Crack

		Width											
		y	z			cmin	From						
		[mm]	[mm]			[mm]	[mm]			[mm]	[mm]		
Maxima													
2	42	76	-1249.	-40.26	-41.91E-6		35	88.79	49.61	Face	43	2503.	1936. 0.009806

Confinement Reinforcement of Pier Shaft:

Width of shaft	Outer Dia	=	2500 mm	Average
Length of shaft	Inner dia	=	1700 mm	Average

Clear cover to earth face	=	50 mm
Clear cover to other face	=	50 mm

Confined area of concrete	=	2513274 mm ²

N _{ED}	=	1209 Tonne
A _c	=	2638937.8 mm ²
f _{ck}	=	35 Mpa
f _{cd}	=	15.63 Mpa
f _{yd}	=	435 Mpa

Normalized axial force

η_k	=	Max	N _{ED} /A _c f _{ck}

η_k	=	0.131	>	0.064
			<	0.30

As	Long Bar	804.2	mm ²
S As		1608.5	mm ²
St		150.0	mm
At		151	mm ²
Hence Provide	16 mm	Link Bar to each Long bar	@ 150 mm C/C
Provided reinforcement		201.1	mm ²

Extent of confinement	=	Depth of section within the plane of bending
	=	2.50 m

Required volumetric ratio of transverse reinforcement

$\omega_{w,req}$	=	0.37 A _c /A _{cc} η_k + 0.13 f _{yd} /f _{cd} (ρ_L -0.01)
A _c	=	Area of gross concrete section
	=	2.6 m ²

A _{cc}	=	Confined core concrete area of the section within the outside dia of loop.
	=	2.51 m ²

ρ_L	=	Reinforcement ratio of the longitudinal reinforcement
	=	A _s /A _{cc}

Area of steel provided	A _s	=	1608 mm ²
ρ_L		=	0.00064 mm ²

$\omega_{w,req}$	=	0.07118

Minimum Confining Reinforcement

$\omega_{w,d}$	=	Max($\omega_{w,req}$, 0.18)

$$= 0.1800$$

Volumetric ratio of transverse reinforcement

$$\rho_w = \omega_{wd} f_{cd} / f_{yd} = 0.00647$$

Volumetric ratio of transverse reinforcement

$$\rho_w = A_{sw} / b S_L$$

Area of the spiral or hoop bar

$$A_{sw} = \rho_w * b * S_L$$

$$b = \text{Dimension of the core perpendicular to the direction of confinement.} = 2050 \text{ mm}$$

$$S_L = \text{Spacing of hoops or ties in longitudinal direction (in vertical direction)}$$

$$\leq 1/5 * \text{smallest dimension of confined core}$$

$$= 0.2 \times 2050 = 410 \text{ mm}$$

$$= 410 \text{ mm}$$

$$A_{sw} = 1327 \text{ mm}^2$$

Provide $\begin{matrix} 25 \\ 20 \end{matrix}$ dia $\begin{matrix} 2 \\ 2 \end{matrix}$ Leg @ $\begin{matrix} 100 \\ 100 \end{matrix}$ mm c/c

$$\text{Area of reinforcement provided at one section} = 1610 \text{ mm}^2$$

OK

Transverse distance b/w hoops legs or supplementary cross ties :

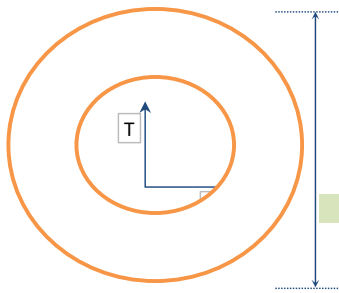
$$S_T = \text{Min} \left\{ \begin{array}{l} 1/3 * \text{smallest dimension of confined core} \\ 200 \text{ mm} \end{array} \right.$$

$$= \text{Min} \left\{ \begin{array}{l} 0.33 \times 2050 \\ 200 \text{ mm} \end{array} \right. = 683 \text{ mm}$$

$$= 200 \text{ mm}$$

CHECK FOR PIER SHAFT SECOND ORDER FORCES:

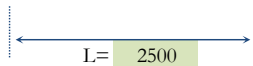
(IRC 112 / clause 11.2.1)



Ac	=	2.64E+06	mm ²	Outer Diameter of Pier shaft	=
As	=	51805	mm ²	Inner Diameter of Pier shaft	=
fcđ	=	15.6	Mpa	Gross Area of c/s A _g	=
f _{yd}	=	435	Mpa	Moment of inertia, I _g	=

Slenderness criteria

Moment of Inertia about TT axis	I _{TT}	=	1.51E+12	mm ⁴
Moment of Inertia about LL axis	I _{LL}	=	1.51E+12	mm ⁴
Radius of gyration along LL axis	i _L	=	$\sqrt{I_{TT}/A}$	
		=	756	mm
Radius of gyration along TT axis	i _T	=	$\sqrt{I_{LL}/A}$	
		=	756	mm



Clear Height of compression member	l _o	=	13.63	m
Effective length of column along L-L	l _{eL}	=	2.3 * l _o	
		=	31.34	m
Slenderness ratio along L-L axis	λ _L	=	l _{eL} / i _L	
		=	41.47	
Effective Length of column along TT -axis	l _{eT}	=	2.3 * l _o	
		=	31.34	m
Slenderness ratio along TT axis	λ _T	=	l _{eT} / i _T	
		=	41.47	

Finding Limiting Value of Slenderness Ratio.

$$\lambda_{lim} = 20 A B C / \sqrt{n}$$

$\phi(\infty, t_o)$	=	1.32	Creep for abutment shaft
$M_{O_{Eqp}}/M_{O_{Ed}}$	=	Ratio of BM in Quasi Permanent LC of SLS to BM in Design LC of ULS	
ϕ_{ef}	=	$\phi(\infty, t_o) * M_{O_{Eqp}}/M_{O_{Ed}}$	
A	=	$1/ (1+0.2 \phi_{ef})$	
ω	=	$A_s f_{yd} / (A_c f_{cd})$	
	=	0.546	
B	=	$\sqrt{ (1 + 2\omega)}$	
	=	1.446	
r_m	=	M_{o1} / M_{o2}	(Ratio of First order moments at two ends of members)
	=	1	
C	=	$1.7 - r_m$	
	=	0.7	
n	=	$N_{ED} / (A_c f_{cd})$	
	=	$N_{ED} / 4125.53947 \text{ */ } (N_{ED} \text{ in Tonne})$	

SUMMARY OF FORCES AT BOTTOM OF SHAFT:					
S.N.	ULS FORCES			SLS (QP LC)	
	N _{ED}	M _{ELL}	M _{ETT}	M _{ELL}	M _{ETT}
	Tonne	Tm	Tm	Tm	Tm
LC1	1209	45	488	30	326
LC2	668	0	514	0	358
LC5	1086	403	1548	0	0
LC6	1086	1328	639	0	0

CHECK FOR SECOND ORDER EFFECT (along LL Axis)					
Mo _{Eqp} /Mo _{Ed}	φ _{ef}	Λ	n	λ _{lim}	Second Order Effect
					(λ < λ _{lim}) : Ignore
					(λ > λ _{lim}) : Consider
0.67	0.88	0.85	0.29	31.81	Consider Second order Effect
0.00	0.00	1.00	0.16	50.32	Ignore second order effect
0.00	0.00	1.00	0.26	39.47	Consider Second order Effect
0.00	0.00	1.00	0.26	39.47	Consider Second order Effect

Load Case	CHECK FOR SECOND ORDER EFFECT (along TT Axis)							
	Mo _{Eqp} / Mo _{Ed}	ϕ _{ef}	Λ	n	λ _{lim}	Second Order Effect		
						λ < λ _{lim} : Ignore λ > λ _{lim} : Consider		
LC1	0.67	0.88	0.85	0.29	31.81	Consider Second order Effect		
LC2	0.70	0.92	0.84	0.16	42.50	Ignore second order effect		
LC5	0.00	0.00	1.00	0.26	39.47	Consider Second order Effect		
LC6	0.00	0.00	1.00	0.26	39.47	Consider Second order Effect		

Second Order Moment (M2)

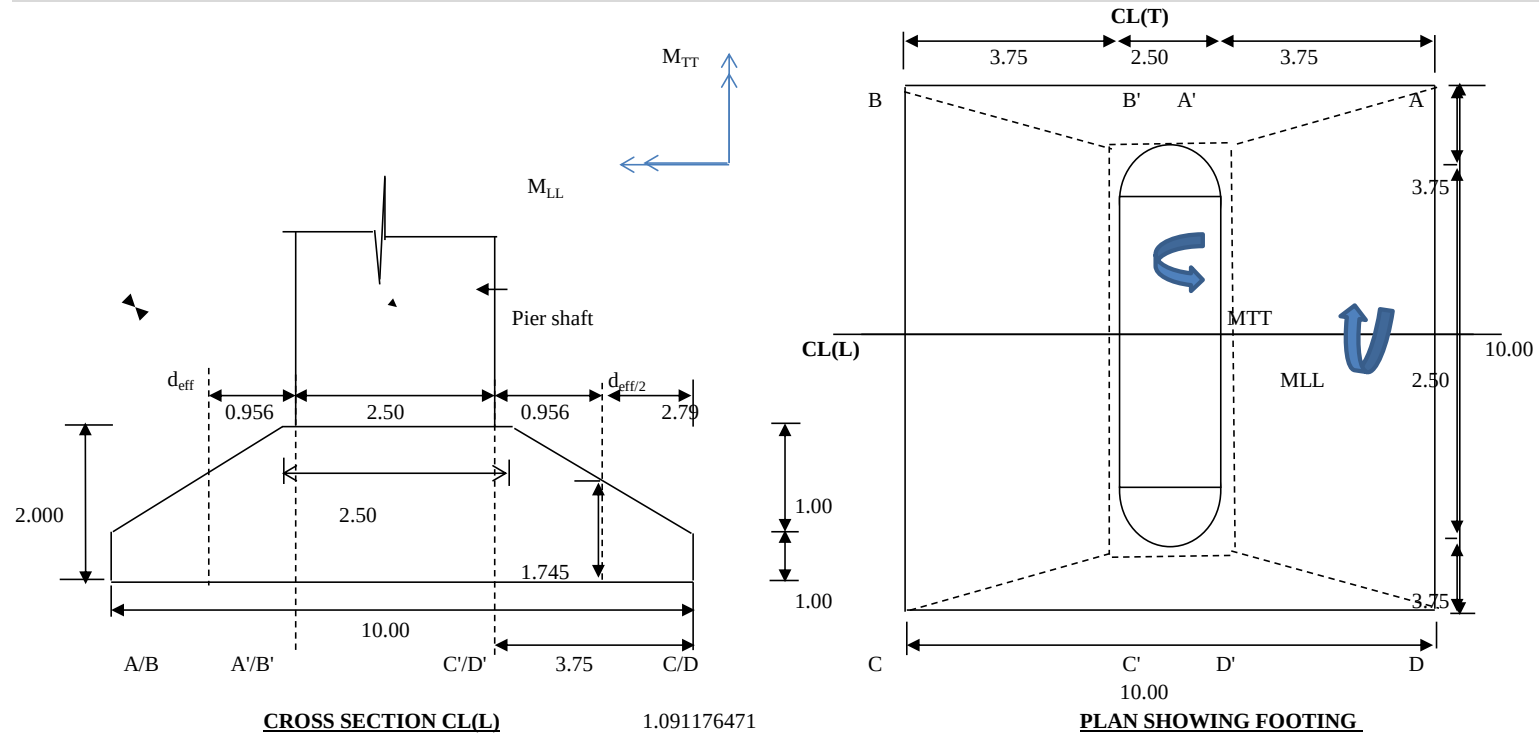
fcd	1.56E+01	N/mm2	Ac	2.64E+06		As	5.18E+04	mm2	fyd	434.78	Es	200000.00	N/mm2
fck	35.00		h	2500.00	mm	dia of main ba	32.00	mm	eyd	0.00217	c	10.00	
Leff long.	31342	mm	Leff trans	31342.10	mm	effective cove	78.00		is - radius of gyration of reinf			1172.00	

LOAD CASES	N _{Ed}	Kr = (nu - n)/(nu-nbal)			Kr	K _ø = 1 + βø _{ef} ≥ 1						K _ø		ε _{yd} /(0.45 d)	1/ro	1/r = Kr K _ø 1/ro	Deflectio n (e ₂) = (1/r)* (l _{exx} ² /c)	M2 = N _{Ed} x e ₂	
		n= N _{Ed} / (Ac fcd)	nu = 1+ (As fyd/Ac fcd)	nbal		ø _{ef} = ø (inf, to) * MoEqp/MoEd				β = 0.35 + fck/200- λ/150				d = h/2 + is			mm		kNm
						ø (inf, to)	M _{oEqp, long}	M _{oEd, Long.}	ø _{ef}	λ = Lx/B	β								
Normal Dry Case (Max)	12087.62	0.29	1.55	0.40	1.09	1.20	299.67	449.50	0.80	41.47	0.25	1.20	>1 ok	2422.00	1.99E-06	2.61E-06	256.83	3104.43	
Normal Dry Case (Min)	6681.05	0.16	1.55	0.40	1.21	1.20	299.67	449.50	0.80	41.47	0.25	1.20	>1 ok	2422.00	1.99E-06	2.89E-06	283.69	1895.34	
Longitudinal Seismic Case (Max)	10855.35	0.26	1.55	0.40	1.12	1.20	299.67	4029.18	0.09	41.47	0.25	1.02	>1 ok	2422.00	1.99E-06	2.28E-06	224.20	2433.80	
Longitudinal Seismic Case (Min)	10855.35	0.26	1.55	0.40	1.12	1.20	299.67	13275.23	0.03	41.47	0.25	1.01	>1 ok	2422.00	1.99E-06	2.25E-06	220.81	2397.01	

Second Order Moment (M2) ALONG Transverse Axis

LOAD CASES	N _{Ed}	Kr = (nu - n)/(nu-nbal)			Kr	K _θ = 1 + βθ _{ef} ≥ 1						K _θ		ε _{yd} /(0.45 d)	1/ro	1/r = Kr K _θ 1/ro	Deflectio n (e ₂) = (1/r)* (l _{eyy} ² /c)	M2 = N _{Ed} x e ₂
		n= N _{Ed} / (Ac fcd)	nu = 1+ (As fyd/Ac fcd)	nbal		θ _{ef} = θ (inf, to) * MoEqp/MoEd				β = 0.35 + fck/200- λ/150				d = h/2 + is = B/2+dia/4			mm	
						θ (inf, to)	M _{oEqp,trans}	M _{oEd,trans}	θ _{ef}	λ = Ly/B	β							
Normal Dry Case (Max)	12087.62	0.29	1.55	0.40	1.09	1.20	3256.59	4884.89	0.80	41.47	0.25	1.20	>1 ok	2422.00	1.99E-06	2.61E-06	256.83	3104.43
Normal Dry Case (Min)	6681.05	0.16	1.55	0.40	1.21	1.20	3577.55	5135.20	0.84	41.47	0.25	1.21	>1 ok	2422.00	1.99E-06	2.91E-06	285.81	1909.49
Longitudinal Seismic Case (Max)	10855.35	0.26	1.55	0.40	1.12	1.20	3577.55	15479.38	0.28	41.47	0.25	1.07	>1 ok	2422.00	1.99E-06	2.39E-06	234.46	2545.11
Longitudinal Seismic Case (Min)	10855.35	0.26	1.55	0.40	1.12	1.20	3577.55	6388.32	0.67	41.47	0.25	1.17	>1 ok	2422.00	1.99E-06	2.61E-06	255.97	2778.67

19A. DESIGN OF FOOTING LONGITUDINAL DIRECTION



PROPERTIES OF BASE					
Area	=	10.00	10.00	=	100.00 m ²
I _{LL}	=	10.00	10.00	+	100.00 X 0.00 ² = 833.33 m ⁴
			12		
I _{TT}	=	10.00	10.00	+	100.00 X 0.00 ² = 833.33 m ⁴
			12		

COORDINATES OF BASE CORNERS		
Edges	L(m)	T(m)
A	5.00	5.00
A'	1.25	5.00
B	-5.00	5.00
B'	-1.25	5.00
C	-5.00	-5.00
C'	-1.25	-5.00
D	5.00	-5.00
D'	1.25	-5.00

Base pressure = $P/A + M_{LL} \cdot T/I_{LL} + M_{TT} \cdot L/I_{TT}$

DESIGN FORCES

Lc. No.	Descriptions	V(Tonne)	M _{TT} (Tm)	M _{LL} (Tm)	MR
	ULS				
Normal					
LC - 1	ULS NORMAL MAX REACTION(70R +Class A)	3620.68	545.19	44.95	547.041
LC - 2	ULS NORMAL SPAN DISLOGED	3096.67	553.68	0.00	553.681
LC - 3	0	0.00	0.00	0.00	0
LC - 4	0	0.00	0.00	0.00	0
LC - 5	ULS NORMAL MAX REACTION(70R)	1740.55	361.23	44.95	364.02
LC - 6	0	0.00	0.00	0.00	0
SEISMIC					
LC - 7	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R + Class A)	3521.52	2375.36	619.33	2454.77
LC - 8	ULS SPAN DISLODGED 1X + 0.3Y + 0.3Z	3114.46	1751.70	349.97	1786.31
LC - 9	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R +Class A)	3521.52	958.00	2059.89	2271.76
LC - 10	ULS SPAN DISLODGED 0.3X + 1Y + 0.3Z	3114.46	881.51	1166.56	1462.16
LC - 11	0	0.00	0.00	0.00	0
LC - 12	0	0.00	0.00	0.00	0
LC - 13	0	0.00	0.00	0.00	0
LC - 14	0	0.00	0.00	0.00	0
LC - 15	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R)	3521.52	2366.76	619.33	2446.45
LC - 16	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R)	3521.52	949.40	2059.89	2268.15
LC - 17	0	0.00	0.00	0.00	0
LC - 18	0	0.00	0.00	0.00	0
					0
LC - 19	ULS NORMAL SV MAX REACTION	0.00	0.00	0.00	0
LC - 20	0	0.00	0.00	0.00	0

Lc. No	Descriptions	Gross Base pressures at critical points (Tm ²)							
		A	A'	B'	B	D	D'	C'	C
	ULS								
LC - 1	ULS NORMAL MAX REACTION(70R +Class A)	39.75	37.29	35.66	33.21	39.21	36.75	35.12	32.67
LC - 2	ULS NORMAL SPAN DISLOGED	34.29	31.80	30.14	27.64	34.29	31.80	30.14	27.64
LC - 5	ULS NORMAL MAX REACTION(70R)	19.84	18.22	17.13	15.51	19.30	17.68	16.59	14.97
LC - 7	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R + Class A)	53.18	42.49	35.37	24.68	45.75	35.06	27.94	17.25
LC - 8	ULS SPAN DISLODGED 1X + 0.3Y + 0.3Z	43.75	35.87	30.62	22.73	39.56	31.67	26.42	18.53
LC - 9	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R +Class A)	53.32	49.01	46.14	41.83	28.60	24.29	21.42	17.11
LC - 10	ULS SPAN DISLODGED 0.3X + 1Y + 0.3Z	43.43	39.47	36.82	32.85	29.43	25.47	22.82	18.86
LC - 15	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R)	53.13	42.48	35.38	24.73	45.70	35.05	27.95	17.30
LC - 16	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R)	53.27	49.00	46.15	41.88	28.55	24.28	21.43	17.16
LC - 19	ULS NORMAL SV MAX REACTION	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

LONGITUDINAL DIRECTION**Downward pressure due to overburden (T/m²)**

LWL Case	Location			Load factor ULS		HFL Case	Location			Load factor ULS	
	A'/D'	A/D					A'/D'	A/D			
Due to Earth filling	13.53	15.534		1.35		Due to Earth filling	13.53	15.534		1.35	
Footing	5	2.5		1.35		Footing	5	2.5		1.35	
Total ULS	25.02	24.3459				Total ULS	25.0209	24.35			

Calculation of forces for Design of Bottom Reinforcement in Longitudinal Direction											
Lc. No	Net Base pressures at critical points							Design Forces			
	A'	D'	A	D	Avg along A'D'	Avg Along AD	d _{eff} /2	BM _{face}	SF _{face}	BM _{deff}	Sfdeff/2
ULS											
LC - 1	12.27	11.73	15.40	14.86	12.00	15.13	12.80	99.06	50.88	56.02	39.02
LC - 2	6.78	6.78	9.94	9.94	6.78	9.94	7.58	62.49	31.35	35.73	24.48
LC - 5	-6.80	17.68	-4.50	-5.04	5.44	-2.25	3.48	2.19	5.97	-1.34	1.71
LC - 7	17.47	10.04	28.84	21.41	13.76	25.12	16.66	150.00	72.90	87.02	58.36
LC - 8	10.85	6.65	19.41	15.21	8.75	17.31	10.93	101.65	48.86	59.25	39.45
LC - 9	23.99	-0.73	28.98	4.26	12.00	16.62	13.17	106.01	53.65	60.37	41.61
LC - 10	14.45	0.45	19.09	5.09	7.45	12.09	8.63	74.11	36.63	42.67	28.94
LC - 15	17.46	10.03	28.79	21.35	13.74	25.07	16.63	149.73	72.78	86.86	58.25
LC - 16	23.98	-0.74	28.93	4.21	11.99	16.57	13.16	105.75	53.54	60.21	41.52
LC - 19	-25.02	-25.02	-24.35	-24.35	-12.51	-12.17	-12.42	-86.38	-46.28	-47.83	-34.36

SUMMARY OF DESIGN FORCES**Normal case**

Max design Bending moment	=	Face of support	At effective depth/2
		99 Tm/m	56 Tm/m
Max design Shear force	=	51 T/m	39 T/m

Seismic case

Max design Bending moment	=	150 Tm/m	87 Tm/m
Max design Shear force	=	73 T/m	58 T/m

DESIGN OF REINFORCEMENT IN LONGITUDINAL DIRECTION (Traffic direction)

Design Bending moment	M_{ED}	=	1500.0 Tm
Total depth of section at face of support	D	=	2.000 m
Total depth of section at half of effective depth	d	=	1.745 m
Effective Width,	"beff"	=	10.000 m
Clear cover		=	75 mm
Diameter of reinforcement provided		=	25 mm
Effective depth of section at face of support	D'	=	1.913 m
Effective depth of section at D'/2 from face of support	d'	=	1.658 m
Grade of concrete	fck	=	35 Mpa
Grade of steel	fyk	=	500 Mpa

MAIN STEEL :

	Ast	=	3927 mm ² /m		
Hence depth of Neutral axis	X_u	=	$\frac{0.87fykAst}{0.362f_{ck} b}$		
		=	135 mm		
Limiting depth of Neutral axis	$X_{u,max}$	=	$\frac{0.0035}{0.0035+0.87fyk/Es}$		
	D'	=	0.617		
	$X_{u,max}$	=	1180 mm	>	135 mm Underreinforced
Area of steel Calculated		=	$M_{ED}/(0.87f_y(d-0.416x_u))$	=	1858 mm ² /m
	f_{ctm}	=	2.77 Mpa		
Ast Minimum	=	Max of		=	2756 mm ² /m

Ast Required	=	Max(1858,2756)	=	2756 mm ² /m
Provide	25 mm @ 125 c/c			
+	0 mm @ 125 c/c			ALTERNATE
=	3927 mm ² /m	>	2756 mm ² /m	OK

DISTRIBUTION STEEL :

$$\begin{aligned} \text{Minimum Steel Required} &= 20\% \text{ of main steel} = 689.0064 \text{ mm}^2/\text{m} \\ \text{Provide } 20 \text{ mm } @ 100 \text{ c/c} &= 3142 \text{ mm}^2/\text{m} > 689 \text{ mm}^2/\text{m} \quad \text{OK} \end{aligned}$$

As per clause 16.6.1 of IRC:112-2011

As per cl. 10.3.2 of IRC:112-2011 shear resistance of section i.e. Shear capacity

$$\text{Design Shear force} = 58.36 \text{ Tonne/m}$$

$$\text{Design Bending Moment} = 87.02 \text{ Tm/m}$$

$$\text{Tan}(\text{Beta}) = 0.27$$

$$V_{\text{net}} = 44.36 \text{ T/m}$$

Max Shear Capacity of section

$$\begin{aligned} v &= 0.6 * (1 - f_{ck} / 310) * f_{ck} \text{ in Mpa} \\ &= 0.5322581 \end{aligned}$$

$$\begin{aligned} f_{cd} &= 0.447 * f_{ck} \\ &= 15.63 \text{ Mpa} \end{aligned}$$

$$\begin{aligned} V_{\text{RDC, max}} &= 0.5 b_w d v f_{cd} \\ &= 690 \text{ Tonne} > 58.36 \text{ Tonne} \quad \text{OK} \end{aligned}$$

$$\begin{aligned} V_{\text{Rdc}} &= \text{Max of } 0.12 K (80 r_1 f_{ck})^{0.33} + 0.150 x s_{cp} b_w x d \\ &\quad \text{and } (u_{\min} + 0.15 s_{cp}) b_w x d \end{aligned}$$

$$\begin{aligned} \text{Here } K &= \text{Min. of } 1 + 200/d \text{ Where } d \text{ is depth in mm} \\ &\quad \text{and } 2.0 \end{aligned}$$

$$\begin{aligned} r_1 &= \text{Min. of } A_s / (b_w x d) \\ &\quad \text{and } 0.02 \end{aligned} \quad r_1 = \text{Reinforcement ratio for longitudinal reinforcement.}$$

$$s_{cp} = \text{Min. of } N_{ed}/A_c \quad s_{cp} = \text{Compressive stress in the concrete from axial load or prestressing.}$$

(Mpa)

$$\text{and } 0.20 f_{cd} \quad N_{ed} = \text{design value of applied axial force.}$$

$$A_c = \text{cross-sectional area of concrete}$$

$$u_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$$

In the above case

$$K = 1.3$$

$$r_1 = 0.0024$$

$$s_{cp} = 0$$

$$u_{min} = 0.2868$$

$$V_{Rdc} = \text{Max of } 500383 \text{ N or } 50.04 \text{ T/m}$$

$$\text{and } 475421 \text{ N or } 47.54 \text{ T/m}$$

Hence $V_{Rdc} = 50.04 \text{ T/m}$ Shear reinf required

$< 58.36 \text{ T/m}$

SHEAR RESISTANCE WITH VERTICAL SHEAR REINFORCEMENT

As per clause 10.2.3 of IRC:112-2011 design shear force V_{NS} shall be taken at the face of support for checking crushing of concrete.

$$V_{NS} = V_{ED} - V_{pd} - V_{ccd} - V_{td}$$

Where $V_{ED} =$ Design shear force from sectional analysis.

$V_{pd} =$ Shear force due to prestressing tendons.

$V_{ccd} =$ Design value of max shear component of the force in the compression area in the case of inclined compression chord.

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

As per clause 10.3.3.2 of IRC:112-2011 the shear resistance V_{Rd} is

$$V_{Rd} = \text{Min of } V_{Rds} = \frac{A_{sw} \times z \times f_{ywd} \times \cot(q)}{s}$$

$$\text{and } V_{Rd,max} = a_{cw} \times b_w \times z \times v_1 \times f_{cd} \cot(q) + \tan(q)$$

Here A_{SW} = Cross-sectional area of the shear reinforcement.

s = Spacing of the stirrups

f_{ywd} = Design yield strength of the shear reinforcement = $0.8 f_{yk}$

$v1$ = Strength reduction factor for concrete cracked in shear $0.6 \times (1 - f_{ck}/310)$

= 0.6 for $f_{ck} \leq 80$ Mpa

= $0.9 - f_{ck}/250 > 0.5$ for $f_{ck} > 80$ Mpa

z = lever arm can be taken as $0.9d$ for RC section and to be calculated for PSC section.

a_{cw} = coefficient taking account of the state of the stress in the compression chord.

= 1 where $s_{cp} = 0.0$ (Mean compressive stress in the concrete.)

f_{cd} = Design value of concrete compressive strength. $g_m = 1.5$

= $a_{cc} \times f_{ck} / g_m$ $a_{cc} = 0.67$

= 15.63333 Mpa

In the above case:

f_{ywd} = 435 Mpa

$v1$ = 0.532258

z = 1491.75 mm

a_{cw} = 1

q = 21.8^0 $\tan(q) = 0.40$

$\cot(q) = 2.50$

Providing shear reinforcement

$10 \quad 4 \text{ L} \quad @ \quad s = 200 \text{ c/c} = 314.16 \text{ mm}^2$

$VR_{d,s} = 2547176.7 \text{ N}$ or 255 T/m

$V_{Rd,max} = 4280056 \text{ N}$ or 428 T/m

$VR_d = \text{Min}(255, 428) = 255 \text{ T/m} > 44.36 \text{ T/m}$ **Ok**

Minimum Shear Reinforcement

$r_{min} = 0.072 f_{ck}$

$= \frac{f_{yk}}{0.001}$

As per clause 10.3.3.5 of IRC:112-2011 minimum shear reinforcement required

$A_{sw,min} = r_{min} \times s \times b_w$

$A_{sw,min} = 170.383 \text{ mm}^2 < 314.16 \text{ mm}^2$ **Ok**

Maximum Spacing $= 0.75 \times d' = 1243 \text{ mm} > 200 \text{ mm}$ **Ok**

PUNCHING SHEAR CHECK AT COLUMN FACE

Axial Force at the base of column	=	12313.580	kN
Pedestal dimension	=	2500.000	mm
Foundation depth at face of pier	=	2000.000	mm
deff	=	1912.500	mm
a, dist. From periphery of the column to control	=	0.000	mm
Uo at column face	=	7853.982	mm
Area under perimeter	=	4.909	m2
Weight of footing under consider perimeter	=	245.437	kN
Upward force due to base pressure	=	1764.056	kN
Net Shear Force	=	10549.524	kN
grade of concrete	=	35	Mpa
β	=	2.367	
Eccentricity	=	1812.572	mm
$V_{ED,red} = \beta V_{ED,red} / U_o d$	=	1.662	Mpa
Concrete Shear capacity $V_{Rd} = [0.12K(80\rho_l f_{ck})^{1/3}]2d$	=	4.160	Mpa
Checks	=	OK	

Axial Force at the base of column	=	12313.580	kN
Column dia	=	2500.000	mm
Foundation depth at face of pier	=	2000.000	mm
deff	=	1912.500	mm
a, dist. From periphery of the column to control	=	1000.000	mm
Uo at column face	=	14137.167	mm
Area under perimeter	=	15.904	m2
Weight of footing under consider perimeter	=	795.216	kN
Depth of Soil under consider perimeter	=	8.767	m
Weight of soil under cons. Perimeter	=	1993.447	kN
Total downward force	=	2788.662	kN
Upward force due to base pressure	=	5715.542	kN

Axial Load = 12313.58 KN
 Moment = 22319.25 KNM
 Stress = 359.37 KN/m2

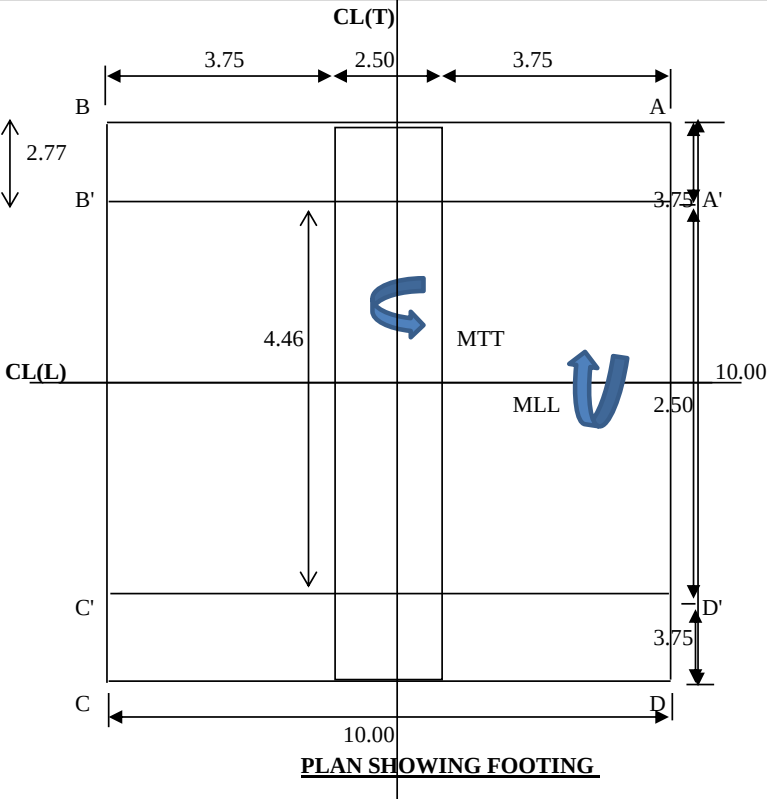
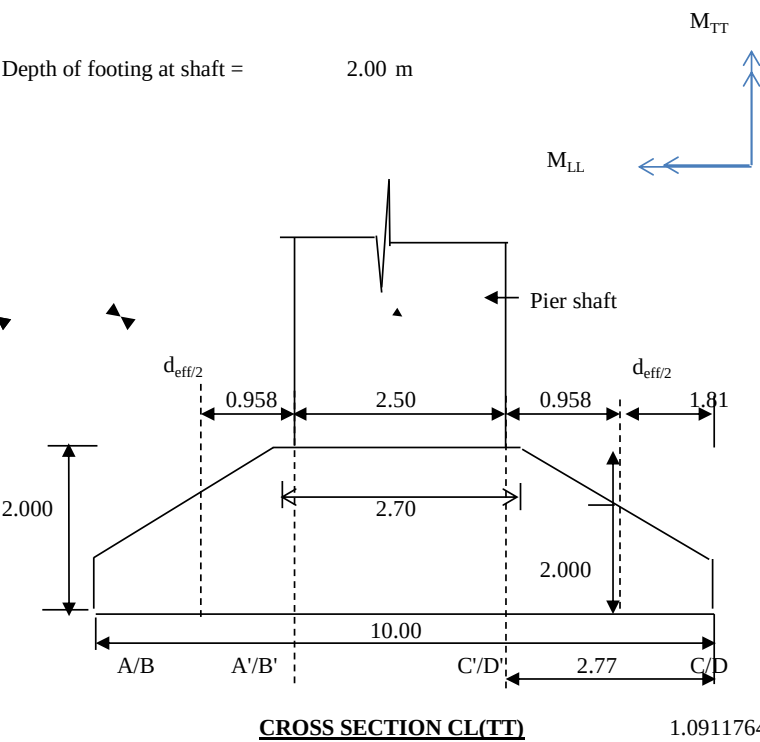
Depth at fa = 2000.000
 depth at de = 1466.6667
 depth at 2 deff = 933.33333

Net Shear Force	=	9386.700	kN	Location of control perimeter	distance from face of pier	depth of footing along Long	depth of footing along Trans	depth at control perimeter , d = (dx + dy)/2
grade of concrete	=	35	Mpa	face of pedestal	1000	2000	2000	2000
β	=	1.759		deff or extreme edge	1913	1490	1490	1490
Eccentricity	=	1812.572	mm	2 deff or extreme edge	3750	1000	1000	1000
$V_{ED,red} = \beta V_{ED,red} / UO \cdot d$	=	0.611	Mpa					
Concrete Shear capacity $V_{Rd} = [0.12K(80\rho_l \cdot f_{ck})^{1/3}]2d$	=	1.141	Mpa					
$K = 1 + \sqrt{200/d} \leq 2$	=	1.323						
$V_{min} = 0.031K \wedge 1.5 f_{ck}^{0.5}$	=	0.279	Mpa					
Area of steel $A1 = \sqrt{Ax \times Ay}$	=	4495.881						
$A1/b/d$	=	0.002						
$\rho_l = A_l / bd \leq 0.02$	=	0.002						
Checks	=	OK						
Shear Resistance (At control perimeter d)								
Control perimeter distance , a	=	1912.500	mm					
Axial Force at the base of column	=	12313.580	kN					
Pier Diameter	=	2500.000	mm					
Foundation depth at deff	=	1490.000	mm					
deff at control perimeter	=	1402.500	mm					
Uo at deff from column face	=	19870.574	mm					
Area under perimeter	=	31.420	m2					
Weight of footing under consider perimeter	=	1571.017	kN					
Depth of Soil under consider perimeter	=	8.767	m					
Weight of soil under cons. Perimeter	=	3938.226	kN					
Total downward force	=	5509.243	kN					

Upward force due to base pressure	=	11291.547	kN
Net Shear Force	=	6531.277	kN
grade of concrete	=	35	Mpa
$\beta = 1 + 0.6 \pi (e + (D+4d))$	=	1.540	
Eccentricity	=	1812.572	mm
$V_{ED,red} = \beta V_{ED,red} / u_o d$	=	0.361	Mpa
Concrete Shear capacity $V_{Rd} = [0.12K(80\rho_l f_{ck})^{1/3}]2d$	=	0.456	Mpa
$K = 1 + \sqrt{200/d} \leq 2$	=	1.378	
$V_{min} = 0.031K^{0.5} f_{ck}^{0.5}$	=	0.297	Mpa
$\rho_l = A_l / bd \leq 0.02$	=	0.002	
Checks	=	OK	
Shear Resistance (At d to 2d)			
Control perimeter distance , a	=	3750.000	mm
Axial Force at the base of column	=	12313.580	kN
Moment at the base of column	=	22319.251	KNM
Pier Diameter	=	2500.000	mm
Foundation depth at face of pier	=	2000.000	mm
Eff depth at face of pier	=	1912.500	
Foundation depth at control perimeter	=	1000.000	mm
deff	=	912.500	mm
$U_l = \pi(2a + \text{Dia of column})$ (critical)	=	31415.927	mm
Area under perimeter	=	78.540	m2
Weight of footing under consider perimeter	=	3926.991	kN
Depth of Soil under consider perimeter	=	8.767	m
Weight of soil under cons. Perimeter	=	9844.181	kN
Total downward force	=	13771.171	kN
Upward force due to base pressure	=	28224.897	kN
Net Shear Force	=	-2140.146	kN
Eccentricity	=	1812.572	mm
$\beta = 1 + 0.6 \pi (e + (D+4d))$	=	1.342	mm
Punching shear stress, $v_{Ed} = \beta V_{Ed} / U_l d$	=	-0.131	Mpa

Concrete Shear capacity $V_{Rd} = [0.12K\{80\rho_l f_{ck}\}^{1/3}]2d$	=	0.161	Mpa
$K = 1 + \sqrt{200/d} \leq 2$	=	1.468	
$V_{min} = 0.031K \wedge 1.5 f_{ck}^{0.5}$	=	0.326	Mpa
$\rho_l = A_l / bd \leq 0.02$	=	0.002	
Checks	=	OK	

19B. DESIGN OF FOOTING TRANSVERSE DIRECTION



PROPERTIES OF BASE

Area = 10.00 x 10.00 = 100.00 m²

$I_{LL} = 10.00 \times \frac{10.00^3}{12} + 100.00 \times 0.00^2 = 833.33 \text{ m}^4$

$I_{TT} = 10.00 \times \frac{10.00^3}{12} + 100.00 \times 0.00^2 = 833.33 \text{ m}^4$

COORDINATES OF BASE CORNERS

Edges	L(m)	T(m)
A	5.00	5.00
A'	5.00	2.23
B	-5.00	5.00
B'	-5.00	2.23
C	-5.00	-5.00
C'	-5.00	-2.23
D	5.00	-5.00
D'	5.00	-2.23

Base pressure =	$P/A + M_{LL} * T/I_{LL} + M_{TT} * L/I_{TT}$

DESIGN FORCES

Lc. No.	Descriptions	V(Tonne)	M _{TT} (Tm)	M _{LL} (Tm)
ULS				
Normal				
LC - 1	ULS NORMAL MAX REACTION(70R +Class A)	3620.68	545.19	44.95
LC - 2	ULS NORMAL SPAN DISLOGED	3096.67	553.68	0.00
LC - 5	ULS NORMAL MAX REACTION(70R)	1740.55	361.23	44.95
SEISMIC				
LC - 7	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R + Class A)	3521.52	2375.36	619.33
LC - 8	ULS SPAN DISLODGED 1X + 0.3Y + 0.3Z	3114.46	1751.70	349.97
LC - 9	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R +Class A)	3521.52	958.00	2059.89
LC - 10	ULS SPAN DISLODGED 0.3X + 1Y + 0.3Z	3114.46	881.51	1166.56
LC - 15	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R)	3521.52	2366.76	619.33
LC - 16	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R)	3521.52	949.40	2059.89
LC - 19	ULS NORMAL SV MAX REACTION	3659.92	322.34	56.72

Lc. No	Descriptions	Gross Base pressures at critical points (Tm ²)							
		A	A'	B'	B	D	D'	C'	C
	ULS								
LC - 1	ULS NORMAL MAX REACTION(70R +Class A)	39.75	39.60	33.06	33.21	39.21	39.36	32.82	32.67
LC - 2	ULS NORMAL SPAN DISLOGED	34.29	34.29	27.64	27.64	34.29	34.29	27.64	27.64
LC - 5	ULS NORMAL MAX REACTION(70R)	19.84	19.69	15.36	15.51	19.30	19.45	15.12	14.97
LC - 7	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R + Class A)	53.18	51.13	22.62	24.68	45.75	47.81	19.30	17.25
LC - 8	ULS SPAN DISLODGED 1X + 0.3Y + 0.3Z	43.75	42.59	21.57	22.73	39.56	40.72	19.70	18.53
LC - 9	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R +Class A)	53.32	46.48	34.98	41.83	28.60	35.45	23.95	17.11
LC - 10	ULS SPAN DISLODGED 0.3X + 1Y + 0.3Z	43.43	39.56	28.98	32.85	29.43	33.31	22.73	18.86
LC - 15	ULS MAX REACTION 1X + 0.3Y + 0.3Z(70R)	53.13	51.07	22.67	24.73	45.70	47.76	19.36	17.30
LC - 16	ULS MAX REACTION 0.3X + 1Y + 0.3Z(70R)	53.27	46.43	35.04	41.88	28.55	35.40	24.00	17.16
LC - 19	ULS NORMAL SV MAX REACTION	38.87	38.69	34.82	35.01	38.19	38.38	34.51	34.32

TRANSVERSE DIRECTION**Downward pressure due to overburden (T/m²)**

LWL Case	Location			Load factor ULS		HFL Case	Location			Load factor ULS	
	A'/B'	A/B					A'/B'	A/B			
Due to Earth filling	13.53	13.53		1.35		Due to Earth filling	13.53	13.53		1.35	
Footing	5.00	5.00		1.35		Footing	5.00	5.00		1.35	
Total ULS	25.02	25.0209				Total ULS	25.0209	25.02			

Calculation of forces for Design of Bottom Reinforcement in Longitudinal Direction											
Lc. No	Net Base pressures at critical points							Design Forces			
	A'	B'	A	B	Avg along A'B'	Avg Along AD	d _{eff} /2	BM _{face}	SF _{face}	Bmdeff/2	Sfdeff/2
ULS											
LC - 1	14.58	8.04	14.73	8.18	11.31	11.46	11.36	43.70	31.51	18.73	20.65
LC - 2	9.27	2.62	9.27	2.62	5.95	5.95	5.95	22.78	16.46	9.75	10.77
LC - 5	-5.33	-9.66	-5.18	-9.51	-2.66	-2.59	-2.64	-10.02	-7.27	-4.27	-4.73
LC - 7	26.11	-2.40	28.16	-0.34	13.05	14.08	13.41	52.64	37.56	22.72	24.89
LC - 8	17.57	-3.45	18.73	-2.29	8.79	9.37	8.99	35.15	25.13	15.15	16.62
LC - 9	21.46	9.96	28.30	16.81	15.71	22.55	18.08	77.68	52.96	34.53	36.79
LC - 10	14.54	3.96	18.41	7.83	9.25	13.12	10.59	45.33	30.96	20.13	21.47
LC - 15	26.05	-2.35	28.11	-0.29	13.03	14.06	13.38	52.54	37.49	22.67	24.84
LC - 16	21.41	10.01	28.25	16.86	15.71	22.55	18.08	77.68	52.96	34.53	36.79
LC - 19	13.66	9.80	13.85	9.98	11.73	11.92	11.80	45.43	32.73	19.47	21.47

SUMMARY OF DESIGN FORCES

Normal case

Max design Bending moment = 45 Tm/m 19 Tm/m

Max design Shear force = 33 T/m 21 T/m

Seismic case

Max design Bending moment = 78 Tm/m 35 Tm/m

Max design Shear force = 53 T/m 37 T/m

DESIGN OF REINFORCEMENT IN TRANSVERSE DIRECTION (Perpendicular to Traffic direction)

Design Bending moment	M_{ED}	=	776.8 Tm
Total depth of section at face of support	D	=	2.000 m
Total depth of section at half of effective depth	d	=	2.000 m
Effective Width,	"beff"	=	3.438 m
Clear cover		=	75 mm
Diameter of reinforcement provided		=	20 mm
Effective depth of section at face of support	D'	=	1.915 m
Effective depth of section at D'/2 from face of support	d'	=	1.915 m
Grade of concrete	fck	=	35 Mpa
Grade of steel	fyk	=	500 Mpa

MAIN STEEL :

$$\begin{aligned}
 A_{st} &= 3142 \text{ mm}^2/\text{m} \\
 \text{Hence depth of Neutral axis } X_u &= \frac{0.87 f_{yk} A_{st}}{0.362 f_{ck} b} \\
 &= 108 \text{ mm} \\
 \text{Limiting depth of Neutral axis } \frac{X_{u_{max}}}{D'} &= \frac{0.0035}{0.0035 + 0.87 f_{yk} / E_s} \\
 &= 0.617 \\
 X_{u_{max}} &= 1181 \text{ mm} > 108 \text{ mm} \quad \text{Underreinforced} \\
 \text{Area of steel Calculated} &= M_{ED} / (0.87 f_y (d - 0.416 x_u)) = 2778 \text{ mm}^2/\text{m} \\
 f_{ctm} &= 2.77 \text{ Mpa} \\
 d_{eff} &= 1915 \text{ mm} \\
 A_{st \text{ Minimum}} &= \text{Max of } \left[0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} \right] \quad \left[0.0013 \cdot b_t \cdot d_{eff} \right] = 2760 \text{ mm}^2/\text{m} \\
 A_{st \text{ Required}} &= \text{Max}(2778, 2760) = 2778 \text{ mm}^2/\text{m} \\
 \text{Provide } & \begin{array}{l} 20 \text{ mm @ } 100 \text{ c/c} \\ + \quad 0 \text{ mm @ } 100 \text{ c/c} \end{array} \quad \text{ALTERNATE} \\
 &= 3142 \text{ mm}^2/\text{m} > 2778 \text{ mm}^2/\text{m} \quad \text{OK}
 \end{aligned}$$

As per cl. 10.3.2 of IRC:112-2011 shear resistance of section i.e. Shear capacity

$$\text{Design Shear force} = 36.79 \text{ T/m}$$

$$V_{\text{net}} = 36.79 \text{ T/m}$$

Max Shear Capacity of section

$$v = \frac{0.6 * (1 - f_{ck} / 310)}{f_{ck} \text{ in Mpa}} = 0.5322581$$

$$f_{cd} = 0.447 * f_{ck} = 15.63 \text{ Mpa}$$

$$V_{\text{RDC, max}} = 0.5 b_w d v f_{cd} = 797 \text{ Tonne} > 36.79 \text{ Tonne} \quad \text{OK}$$

$$V_{\text{Rdc}} = \text{Max of} \left\{ \begin{array}{l} \left[0.12 K (80 r_1 f_{ck})^{0.33} + 0.150 s_{cp} \right] b_w \times d \\ \text{and} \quad (u_{\min} + 0.15 s_{cp}) b_w \times d \end{array} \right.$$

$$\text{Here } K = \text{Min. of} \left\{ \begin{array}{l} 1 + \sqrt{200/d} \quad \text{Where } d \text{ is depth in mm} \\ \text{and} \quad 2.0 \end{array} \right.$$

$$r_1 = \text{Min. of} \left\{ \begin{array}{l} \frac{A_{sl}}{b_w \times d} \\ \text{and} \quad 0.02 \end{array} \right. \quad r_1 = \text{Reinforcement ratio for longitudinal reinforcement.}$$

$$s_{cp} = \text{Min. of} \left\{ \begin{array}{l} N_{ed} / A_c \\ \text{and} \quad 0.20 f_{cd} \end{array} \right. \quad \begin{array}{l} s_{cp} = \text{Compressive stress in the concrete from axial load or prestressing.} \\ N_{ed} = \text{design value of applied axial force.} \\ A_c = \text{cross-sectional area of concrete} \end{array}$$

$$u_{\min} = 0.031 K^{3/2} f_{ck}^{1/2}$$

In the above case

$$K = 1.3$$

$$r1 = 0.0016$$

$$s_{cp} = 0$$

$$u_{min} = 0.2791$$

$$V_{Rdc} = \begin{cases} \text{Max of} & 502888 \text{ N} & \text{or} & 50.29 \text{ t} \\ \text{and} & 534549 \text{ N} & \text{or} & 53.45 \text{ t} \end{cases}$$

$$\text{Hence } V_{Rdc} = 53.45 \text{ t} \\ > 36.79 \text{ t}$$

Shear reinf not required

The plan view shows a rectangular footing with a width of 10.00 units and a length of 10.00 units. The corners are labeled B (top-left), C (bottom-left), A (top-right), and D (bottom-right). A vertical dashed line passes through the center, labeled 'T' at both ends. A horizontal dashed line passes through the center, labeled 'L' at both ends. A blue curved arrow labeled 'MTT' indicates a counter-clockwise moment about the vertical axis. A blue curved arrow labeled 'MLL' indicates a clockwise moment about the horizontal axis. A black arrow labeled 'TRAFFIC DIRECTION' points to the right above the footing. Dimensions of 10.00 are marked for both the width and length.

PROPERTIES OF BASE					
Area	=	10.00	x	10.00	= 100.00 m ²
I _{LL}	=	10.00	x	$\frac{10.00^3}{12}$	= 833.33 m ⁴
I _{TT}	=	10.00	x	$\frac{10.00^3}{12}$	= 833.33 m ⁴

Edges	L(m)	T(m)
A	5.00	5.00
B	-5.00	5.00
C	-5.00	-5.00
D	5.00	-5.00

$$\text{Base pressure} = P/A + M_{LL} * x/I_{LL} + M_{TT} * z / I_{TT}$$

Sr. No.	Description	Forces at founding levels			Base pressure at footing corners in T/m ²			
		V(Tonne)	M _{TT} (Tm)	M _{LL} (Tm)	A	B	C	D
LC -1	NORMAL MAX REACTION(70R+Class A)	2652.03	363.46	29.97	28.88	24.52	24.16	28.52
LC -2	NORMAL SPAN DISLOGED	2284.40	380.55	0.00	25.13	20.56	20.56	25.13
LC -6	SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z	2294.57	1322.41	195.17	32.05	16.18	13.84	29.71
LC -7	SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R + class A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LC -8	SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z	2294.57	585.95	650.57	30.36	23.33	15.53	22.56
LC -13	SEISMIC NORMAL MAX REACTION(70R)	2652.03	240.82	29.97	28.15	25.26	24.90	27.79

LC - 15	SEISMIC MAX REACTION $1X + 0.3Y + 0.3Z(70R)$	2593.34	1691.83	415.58	38.58	18.28	13.29	33.59
LC - 16	SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R)$	2593.34	746.92	1380.72	38.70	29.74	13.17	22.13
LC - 18	NORMAL SV MAX REACTION	2741.22	254.29	56.72	29.28	26.23	25.55	28.60

Sr. No.	Average pressure along BC	Average pressure along AD	Average pressure along AB	Average pressure along CD	Effective Length along TT	Effective Width along LL	Effective Area	I_{LL}	I_{TT}
	T/m ²	T/m ²	T/m ²	T/m ²	m	m	m ²	m ⁴	m ⁴
LC -1	24.34	28.70	26.70	26.34	10.00	10.00	100.00	833.33	833.33
LC -2	20.56	25.13	22.84	22.84	10.00	10.00	100.00	833.33	833.33
LC -6	15.01	30.88	24.12	21.77	10.00	10.00	100.00	833.33	833.33
LC -8	19.43	26.46	26.85	19.04	10.00	10.00	100.00	833.33	833.33
LC -13	25.08	27.97	26.70	26.34	10.00	10.00	100.00	833.33	833.33
LC -15	15.78	36.08	28.43	23.44	10.00	10.00	100.00	833.33	833.33
LC -16	21.45	30.41	34.22	17.65	10.00	10.00	100.00	833.33	833.33
LC -18	25.89	28.94	27.75	27.07	10.00	10.00	100.00	833.33	833.33

Sr. No.	M _{TT}	M _{LL}	A'		B'		C'		D'		Base Pressure(T/m ²)			
	Tm	Tm	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	T/m ²	T/m ²	T/m ²	T/m ²
LC -1	363.46	29.97	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	28.88	24.52	24.16	28.52
LC -2	380.55	0.00	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	25.13	20.56	20.56	25.13
LC -6	1322.41	195.17	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	32.05	16.18	13.84	29.71
LC -8	585.95	650.57	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	30.36	23.33	15.53	22.56
LC -13	240.82	29.97	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	28.15	25.26	24.90	27.79
LC -15	1691.83	415.58	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	38.58	18.28	13.29	33.59
LC -16	746.92	1380.72	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	38.70	29.74	13.17	22.13
LC -18	254.29	56.72	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	29.28	26.23	25.55	28.60

Minimum Effective Area Allowed Normal case = 0.8 x 100 = 80.00 m²

Minimum Effective Area Allowed Seismic case = 0.67 x 100 = 67.000 m²

Minimum Effective Area Normal Case = 100.00 > 80.00 m²

Minimum Effective Area Seismic Case = 100.00 > 67.00 m²

SUMMARY OF BASE PRESSURES

NORMAL CASE

Maximum base pressure = 29.28 t/m² < 35 t/m² **Ok**

Minimum base pressure = 20.56 t/m² > 0 t/m² **Ok**

SEISMIC CASE

Maximum base pressure = 38.70 t/m² < 43.75 t/m² **Ok**

Minimum base pressure = 13.17 t/m² > 0 t/m² **Ok**

***DESIGN OF PIER P1 TO P3 AND
ITS FOUNDATION
FOR MAJOR BRIDGE AT CH- 16+463***

This Document Contains Design Calculations of Pier with Open Foundation

1. SALIENT FEATURES OF THE BRIDGE

Skew angle : 0 degree 0 Radian

<u>SPAN DETAILS</u>	<u>Left span</u>		<u>Right span</u>		<u>Unit</u>
	Right	Skew	Right	Skew	
C/C Expansion joint	= 25.000	25.000	25.000	25.00	m
C/C Expansion joint to C.L. brg	= 0.625	0.625	0.625	0.63	m
Expansion gap	= 50	50	50	50	mm
C/C bearing	= 23.750	23.750	23.750	23.750	m

SUPERSTRUCTURE DETAILS

	Left Span	Right Span
Overall deck width of bridge	= 12.50 m	12.50 m
Carriage way width	= 11.50 m	11.50 m
Crash barrier width	= 0.45 m	0.45 m
Height of crash barrier	= 1.00 m	1.00 m
C.g of Crash barrier from bottom	= 0.40 m	0.40 m
Cross-slope	= 7.00 %	7.00 %
Wearing coat thickness	= 65 mm	65 mm
Nos. of Bearing	= 4	4
Spacing of Bearing	= 3.00 m	3.00 m
Depth of Girder	= 1.600 m	1.600 m
Depth of slab	= 0.22 m	0.22 m
Depth of superstructure	= 1.820 m	1.820 m
Bearing + Pedestal height	= 0.30 m	0.30 m
Width of main girder at top	= 1.00 m	1.00 m
Width of cross girder	= 0.40 m	0.40 m
Nos. of cross-girders	= 2	2
FRL to Soffit	= 1.99 m	1.99 m
Type of Bearing	OR SPHERICAL BR	

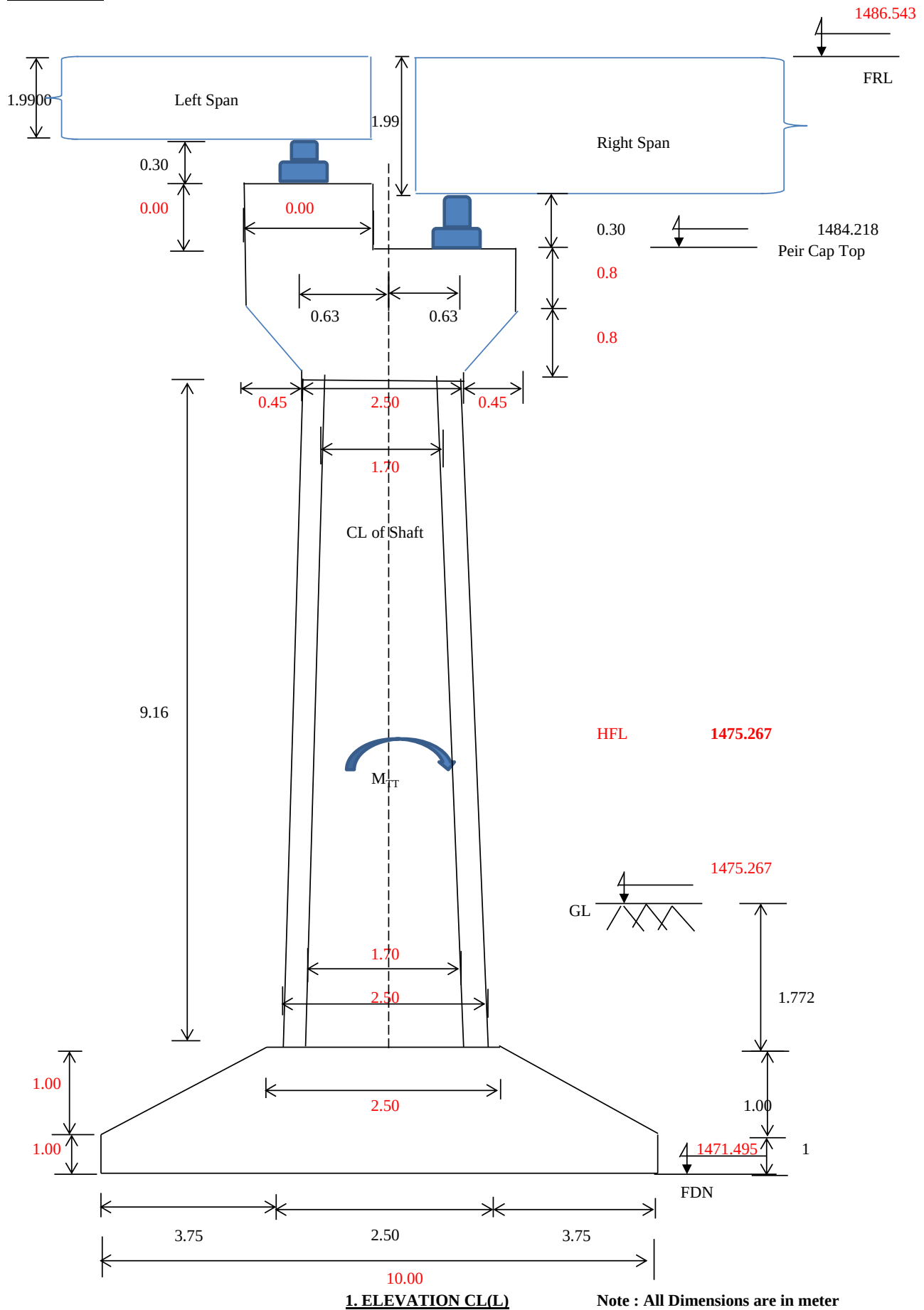
LEVEL DETAILS

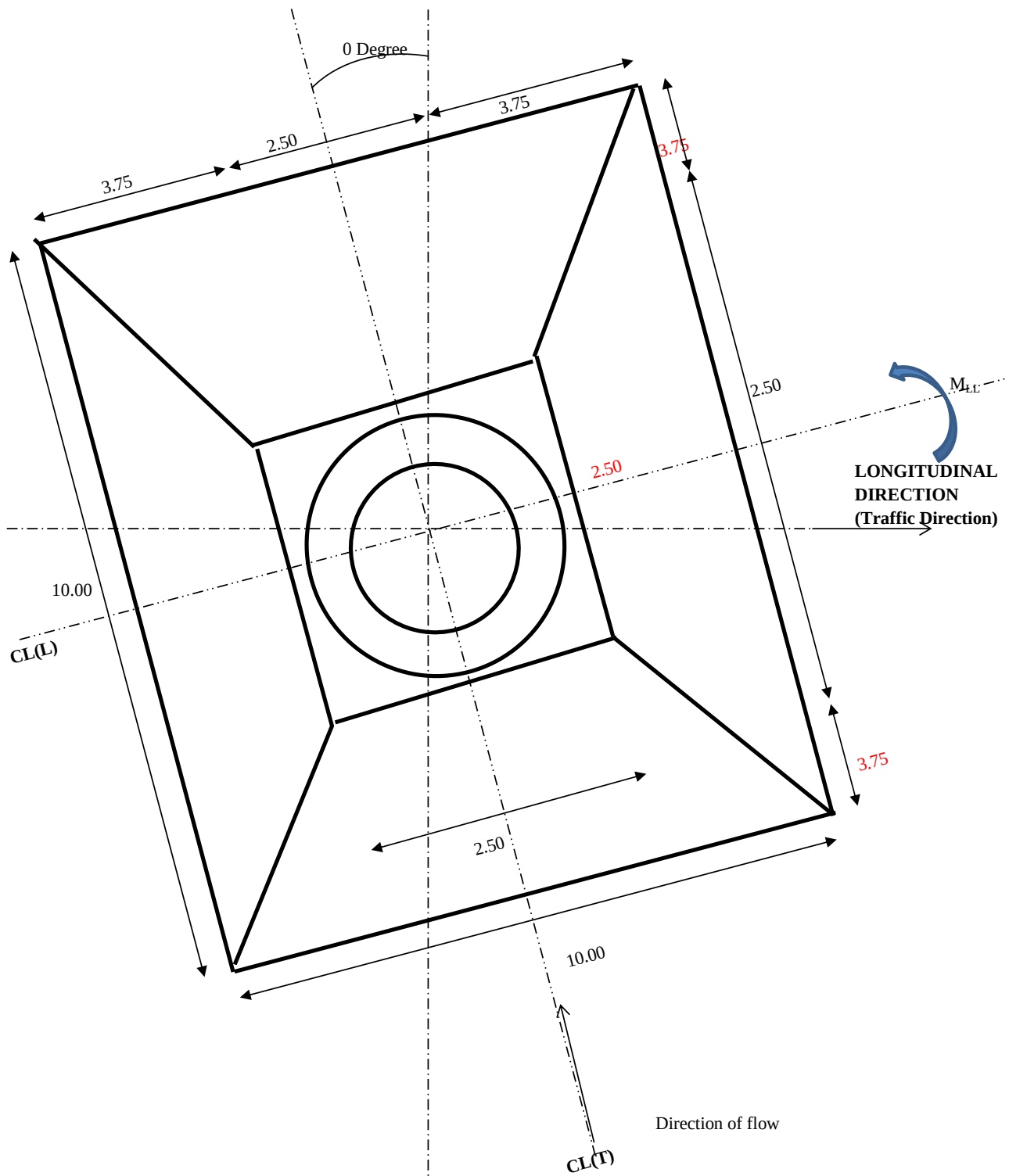
Formation level	= 1486.543 m
Pier cap top level	= 1484.218 m
HFL	= 1475.267 m
Ground level	= 1475.267 m
MSL (Normal)/Rock Level	= 1475.267 m
MSL (Seismic)/Rock Level	= 1475.267 m
Pier shaft bottom/Footing top level	= 1473.460 m
Founding level	= 1471.495 m

DIMENSION DETAILS

Pier cap:	Right	Skew
Width of pier cap at top	= 3.40 m	3.40 m
Width of pier cap at bottom	= 2.50 m	2.50 m
Length of pier cap at top	= 11.50 m	11.50 m
Length of pier cap at bottom	= 2.50 m	2.50 m
Thickness of uniform portion	= 0.80 m	0.80 m
Thickness of tapered portion	= 0.80 m	0.80 m
Pier shaft:		
Width of pier at top	= 2.50 m	2.50 m
Width of pier at bottom	= 2.50 m	2.50 m
Length of pier (rectangle portion)	= 2.50 m	2.50 m
Length of pier(end circular portion Top)	= 1.25 m	1.25 m
Length of pier (end circular portion Bottom)	= 1.25 m	1.25 m
Total length of pier shaft at top	= 5.00 m	5.00 m
Total length of pier shaft at bottom	= 5.00 m	5.00 m
Footing:		
Width of footing at top	= 2.50 m	2.50 m
Width of footing at bottom	= 10.00 m	10.00 m
Length of footing	= 10.00 m	10.00 m
Thickness at tip	= 1.00 m	1.00 m
Maximum thickness	= 2.00 m	2.00 m

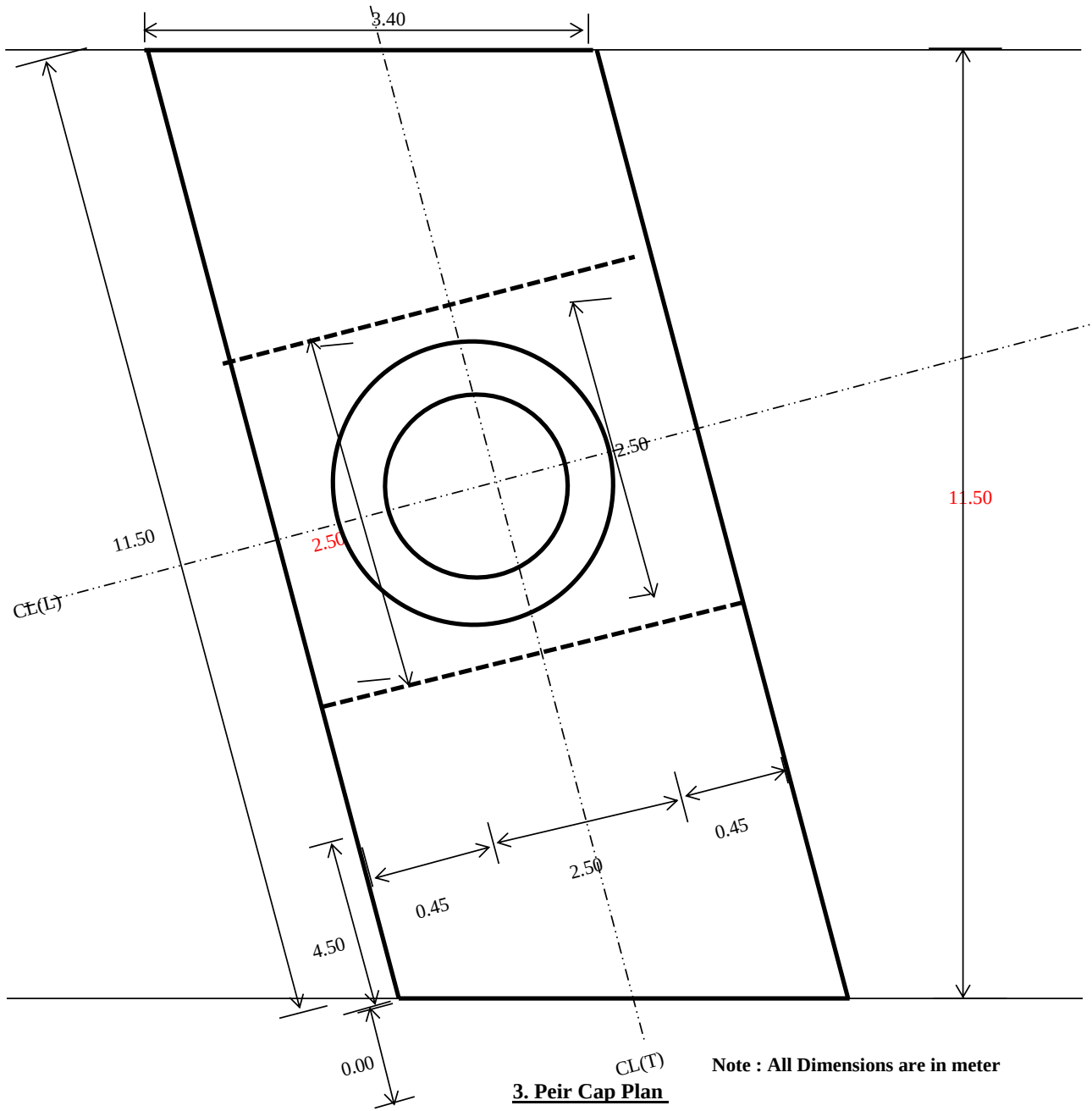
DIAGRAMS:





Note : All Dimensions are in meter

2. PLAN SHOWING FOOTING & PIER SHAFT



2. MATERIALS DETAILS**CONCRETE:**

Grade of concrete	fck	=	M 35	
Mean value of concrete compressive strength, fcm		=	M 45 (28 days strength from table 6.5 of IRC:112)	
Design concrete compressive strength, fcd		=	0.447 x fck	(Basic and Seismic combination)
		=	15.633 Mpa	
Secant modulus of elasticity, Ecm		=	32000 Mpa	(from table 6.5 of IRC:112)
Mean axial tensile strength fctm		=	2.8 Mpa	(from table 6.5 of IRC:112)
Density of concrete		=	2.5 t/m ³	

SOIL :

Density dry		=	2 t/m ³	
Density Saturated		=	2.3 t/m ³	
Soil density (submerged)		=	2 t/m ³	
Angle of Internal friction ϕ		=	35	
Cohesion c		=	0	
Gross bearing capacity		=	35.00 t/m ²	(Normal case)
		=	43.75 t/m ²	(Seismic case)

CREEP COEFFICIENT:

(Table 6.5 of IRC:112)

Pier shaft cross-sectional area	Ac	=	11.159 m ²	
Perimeter in contact with atmosphere, u		=	12.854 m	
Notional size, ho = 2Ac/u		=	1736 mm	
Age of concrete at the time of loading to		=	90 days	
	ta	=	25550 days	
	f(α 90)	=	1.2 (Table 6.9 IRC 112)	
Hence creep co-efficient for pier shaft		=	1.32 (increased by 10%)	
Creep co-efficient for foundation		=	1.0 (as ho = α for foundation)	
Hence modulus of elasticity for long term loading, Ecm'		=	13793 Mpa	Ecm' = Ecm / (1 + f)

REINFORCING STEEL:

Grade of reinforcement	fy	=	Fe 500 Mpa	
Modulus of elasticity	Es	=	200000 Mpa	

3. PERMISSIBLE STRESSES**MAXIMUM PERMISSIBLE STRESS IN CONCRETE:**

Rare combination		=	0.48 x fck	
		=	16.8 Mpa	
Quasi permanent combination		=	0.36 x fck	
		=	12.6 Mpa	

MAXIMUM PERMISSIBLE STRESS IN STEEL:

Rare Combination		=	300 Mpa	
Permissible crack width	W _{k,max}	=	0.3 mm	

4. SEISMIC PARAMETERS

Seismic zone		=	IV	
Type of soil		=	Type2(Medium)	
Zone factor	Z	=	0.24	
Importance factor	I	=	1.20	
Response Reduction Factor(Long)	R	=	3	
Response Reduction Factor(Trans)	R	=	3	

5. ENVIRONMENTAL PARAMETERS

Relative humidity		=	80%	
Condition of exposure		=	Moderate	

6.OTHER PARAMETERS

Mean velocity of water current flow		=	0.00 m/s	
Design Speed	V	=	50 Kmph	
Radius of Curvature	R	=	0 m	

7.LOADS FROM SUPERSTRUCTURES

Angle of Skew	=	0 °	=	0.000 Radian
No. of Bearing	=		4.00	
		Right		Skew
c/c of bearing in Transverse Direction	=	3.000 m		3.000 m
c/c bearing on Longitudinal Direction	=	1.250 m		1.250 m
Impact Factor for Design of Pier Cap			=	1.24

Left Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	2159.25	200.00								
Total Load	2359.25		318.15	0.000	0.00	0.00	0.00	0.00	0.00	0.00
Resultant Transverse Ecc.	0.000		0.000	0.000	0.950	1.210	1.000	1.000	0.300	0.300

Right Span										
	Dead Load+SIDL(KN)		Surfacing(KN)	FLL(KN)	Live Load (70R + Class A) max	Live Load (70R + Class A) min	Live Load(3L Class A) max	Live Load(3L Class A) min	Live Load(SV Load) max	Live Load(SV Load) min
	DL	SIDL								
	2159.25	200.00								
Total Load	2359.25		318.15	0.00	1109.88	240.12	0.00	0.00	1890.78	1709.22
Resultant Transverse Ecc.	0.000		0.000	0.000	0.300	0.300	1.000	1.000	0.300	0.300

8. VOLUME OF SUBSTRUCTURE**PIER CAP****Uniform portion**

No.		L(m)	B(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	11.500	0.000	0.000	0.000	1.700	0.000	10.758	12.76
2	1	11.500	3.400	0.800	31.280	0.000	0.000	10.358	12.36

Tapered portion

2a	Rectangle	2.500	3.400	0.800	6.800	0.000	0.000	9.558	11.558
	Triangle	4.500	3.400	0.800	12.240	0.000	0.000	9.691	11.691
Total				Inc. 10%	55.352	0.000	0.000	9.171	10.989

PIER SHAFT**Above HFL****Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	7.386	19.491	0.000	0.000	5.465	7.465
2	0	2.500	0.000	7.386	0.000	-1.250	0.000	4.234	6.234
3	0	2.500	0.000	7.386	0.000	1.250	0.000	4.234	6.234

Cylinder/Conical Frustum portion

4	0.000	0.850	1.25	7.386	0.000	0.000	0.000	5.465	7.465
Total					19.491	0.000	0.000	5.465	7.465

Below HFL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	1.772	4.676	0.000	0.000	0.886	2.886
2	0	2.500	0.000	1.772	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	1.772	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.250	0.850	1.772	0.000	0.000	0.000	0.886	2.886
Total					4.676	0.000	0.000	0.886	2.886

Above LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	7.386	19.491	0.000	0.000	5.465	7.465
2	0.0	2.500	0.000	7.386	0.000	-0.850	0.000	4.234	6.234
3	0.0	2.500	0.000	7.386	0.000	0.850	0.000	4.234	6.234

Cylinder/Conical Frustum portion

4	0.000	0.850	0.850	7.386	0.000	0.000	0.000	5.260	7.260
Total					19.491	0.000	0.000	5.465	7.465

Below LBL**Rectangular/Triangular Portion**

No.		L/R1(m)	B/R2(m)	H(m)	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	2.500	1.700	1.772	4.676	0.000	0.000	0.886	2.886
2	0	2.500	0.000	1.772	0.000	0.000	0.000	0.000	0.000
3	0	2.500	0.000	1.772	0.000	0.000	0.000	0.000	0.000

Cylinder/Conical Frustum portion

4	0.000	1.25	0.850	1.772	0.000	0.000	0.000	0.746	2.746
Total					4.676	0.000	0.000	0.886	2.886

FOOTING

No.		L	B	H	V(m ³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	2.500	2.00	50.000	0.000	0.000	-1.000	1.000
2	1	10.000	3.75	1.500	56.250	-2.917	0.000	-1.222	0.778
3	1	10.000	3.750	1.500	56.250	2.917	0.000	-1.222	0.778
Total					162.5	0.000	0.000	-1.154	0.846

EARTH ABOVE FOOTING

No.		L	B	H	V(m³)	e _{LL} (m)	e _{TT} (m)	e _{VS} (m)	e _{VF} (m)
1	1	10.000	10.000	3.772	377.200	0.000	0.000	-0.114	1.886
2	-1	Shaft Below LBL			-4.676	0.000	0.000	0.886	2.886
3	-1	Footing			-162.500	0.000	0.000	-1.154	0.846
Total					210.024	0.000	0.000	0.668	2.668

9. CALCULATION OF SEISMIC FORCE

$$\text{Time Period (T)} = 2 \sqrt{\frac{D}{1000 F}}$$

$$\begin{aligned} D &= \text{Approximate DL of super-structure \& LL in Tonne (DL+20\% LL)} \\ &= 5354.80 \text{ KN} + 221.977 \text{ KN} \\ &= 5577 \text{ KN} \end{aligned}$$

$$F =$$

Horizontal force in KN required to be applied at center of mass of super-structure for 1 mm deflection at the top of pier / abutment for the earthquake in the transverse direction; and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

$$F = \frac{6 E I}{x^2(3L-x)} \quad \text{for 1 mm deflection at } x$$

$$E = 32000 \text{ N/mm}^2$$

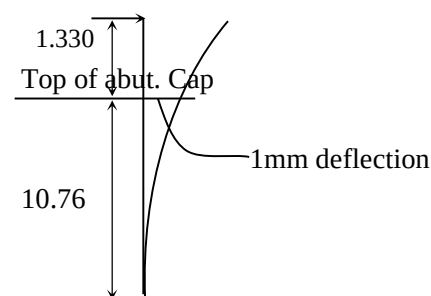
Column Cross-Section

$$D1 = 2.50 \text{ m (Average Outer Dia)}$$

$$D2 = 1.70 \text{ m (Average Inner Dia)}$$

$$I_{TT} = 1.13E+12 \text{ mm}^4 \quad 1.5E+12$$

$$I_{LL} = 1.13E+12 \text{ mm}^4$$



$$\text{Force required in Long. Dir. } F_L = 87 \text{ KN}$$

$$\text{Force required in Trans. Dir. } F_T = 74 \text{ KN}$$

$$\text{Time Period -Longitudinal Seismic Case} = 0.506 \text{ seconds}$$

$$\text{Time Period -Transverse Seismic Case} = 0.551 \text{ seconds}$$

$$S_a/g \text{ -Longitudinal Seismic Case} = 2.500$$

$$S_a/g \text{ -Transverse Seismic Case} = 2.469$$

$$\text{Seismic Zone} = \text{IV}$$

$$\text{Zone Factor } Z = 0.24$$

$$\text{Importance Factor } I = 1.20$$

$$\begin{aligned} \text{Response reduction Factor } R &= 3.00 && \text{Longitudinal} \\ R &= 3.00 && \text{Transverse} \end{aligned}$$

$$\text{Avg response acceleration } S_a/g = 2.500 \quad \text{Longitudinal}$$

$$\text{Avg response acceleration } S_a/g = 2.469 \quad \text{Transverse}$$

$$\text{Elastic Seismic Coefficient} = \frac{Z}{2} \times (S_a/g) \times (I/R)$$

$$\text{In longitudinal direction} = 0.360 / R$$

$$\text{In Transverse direction} = 0.356 / R$$

			Substructure	Foundation
Enhancement factor				1.35
Design Seismic coefficient	Long. Ah		0.120	0.162
	Trans. At		0.119	0.160
	Vert. Av		0.080	0.108

CALCULATION OF SEISMIC FORCES

SEISMIC COMPONENT OF SUPER-STRUCTURE DL, SIDL, WC & FPLL :

Loads & Their Lever arm

Description	LEFT			RIGHT		
	W	eVS	eVF	W	eVS	eVF
	Tonnes	(m)	(m)		(m)	(m)
Total Super-Structure DL	431.85	12.38	14.38	431.85	12.38	14.38
Total Super-Structure SIDL	40.00	13.45	15.45	40.00	13.45	15.45
Total Super-Structure WC	63.63	13.08	15.08	63.63	13.08	15.08
Total Super-Structure FPLL	0.00	13.08	15.08	0.00	13.08	15.08
Total	535.48	12.55	14.55	535.48	11.54	13.39

* Bearing level	11.06	13.06		11.06	13.06

SEISMIC COMPONENT OF SUB-STRUCTURE :

Description	W	eVS	eVF
	Tonnes	(m)	(m)
Total weight of shaft and pier cap	198.80	7.78	9.65
Total weight of foundation	406.25	-1.15	0.85

SEISMIC COMPONENT OF LIVE LOAD(70R+Class A) :

Description	W	W	eVS	eVF
	Tonnes	Tonnes	(m)	(m)
	70R + Class A	70R + Class A		
Maximum Live Load	110.99	0.00	14.25	16.248
Minimum Live Load	24.01	0.00	14.25	16.248

Summary of Seismic force at shaft and foundation

Super structure

Dead load - Super structure about shaft	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		64.26		710.560	
Seismic Transverse direction			63.47		764.45
Seismic Vertical Direction	42.84				

Dead load - Super structure about foundation	V	H L	H T	M TT	M LL
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		86.75		1132.75	
Seismic Transverse direction			85.682		1196.97
Seismic Vertical Direction	57.83				

Dead load Left span Dislodged- about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		43.37		566.376	
Seismic Transverse direction			42.84		573.76
Seismic Vertical Direction	28.92				

Sub structure

Dead load - Sub structure about Shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		23.86		185.48	
Seismic Transverse direction			23.56		183.21
Seismic Vertical Direction	15.90				

Dead load - Sub structure about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		32.2054		310.74	
Seismic Transverse direction			31.81		306.92
Seismic Vertical Direction	21.47				

Live Load - Max**70R + Class A**

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.63		37.49
Seismic Vertical Direction	1.78				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			3.55		57.71
Seismic Vertical Direction	2.40				

70R

Live load - Max about shaft	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			2.63		37.49
Seismic Vertical Direction	1.78				

Live load - Max about foundation	V	H L	H T	M T T	M L L
	Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction	Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction			3.55		57.71
Seismic Vertical Direction	2.40				

Live Load - Min**70R + Class A**

Live load - Min shaft	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.57		8.11
Seismic Vertical Direction		0.38				

Live load - Min foundation	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.77		12.49
Seismic Vertical Direction		0.52				

70R

Live load - Min shaft	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.57		8.11
Seismic Vertical Direction		0.38				

Live load - Min foundation	about	V	H L	H T	M T T	M L L
		Tonnes	Tonnes	Tonnes	Tonne meter	Tonne meter
Seismic longitudinal direction		Not to be considered - IRC:6 219.5.2				
Seismic Transverse direction				0.77		12.49
Seismic Vertical Direction		0.52				

10. HORIZONTAL FORCES

Consider Left span free and Right Span fix on pier

BOTH SPAN POSITION CONDITION(70R + Class A)

1. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= \text{Class 70R on First two Lane } 0.2 \times 100 + \text{Class A on third Lane } 0.05 \times 54 \\ &+ 0.1 \times 100 + 0.05 \times 54 \\ F_h &= 35.4 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 24.01) \\ &= 14.59 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 32.29 \text{ Ton} \\ F_{hr} &= 32.29 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 32.29 - 13.39 \\ &= 18.90 \text{ Ton} \end{aligned}$$

2. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= \text{Class 70R on First two Lane } 0.2 \times 100 + \text{Class A on third Lane } 0.05 \times 54 \\ F_h &= 22.7 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (Rg + Rq) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 110.99) \\ &= 18.94 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Max of } (F_h/2 + \mu (Rg + Rq) \text{ and } F_h - \mu (Rg + Rq)) \\ &= 30.29 \text{ Ton} \\ F_{hr} &= 30.29 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 30.29 - 13.39 \\
 &= 16.90 \text{ Ton}
 \end{aligned}$$

3. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\
 &= 13.39 \text{ Ton} \\
 F_{hl} &= 13.39 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= 0.2 \times \text{Class 70R on First two Lane } 20 + 0.05 \times \text{Class A on third Lane } 10.8 \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force(Shaft)} &= 32.1287767 \text{ Ton} \\
 \text{Seismic Horizontal force(foundation)} &= 43.3738485 \text{ Ton} \\
 \text{Total Horizontal force for Shaft}(F_h) &= 36.6687767 \text{ Ton} \\
 \text{Total Horizontal force for foundation}(F_h) &= 47.9138485 \text{ Ton} \\
 \mu (R_g + R_q) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 4.80) \\
 &= 13.627107 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned}
 F_{hr} \text{ for shaft} &= 34.23 \text{ Ton} \\
 F_{hr} \text{ for foundation} &= 39.85 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 34.23 - 13.39 \\
 &= 20.84 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 39.85 - 13.39 \\
 &= 26.47 \text{ Ton}
 \end{aligned}$$

4. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times (\text{Total DL } 215.925 + \text{Total SIDL load } 51.81 + \text{CWLL } 0.00) \\
 &= 13.39 \text{ Ton} \\
 F_{hl} &= 13.39 \text{ Ton}
 \end{aligned}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= 0.2 \times \text{Class 70R on First two Lane } 20 + 0.05 \times \text{Class A on third Lane } 10.8 \\
 F_h &= 4.54 \text{ Ton} \\
 \text{Seismic Horizontal force(Shaft)} &= 32.1287767 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\
 \text{Seismic Horizontal force(foundation)} &= 43.3738485 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \%})
 \end{aligned}$$

Total Horizontal force for Shaft(Fh) = 36.6687767 Ton inc.)
 Total Horizontal force for foundation(Fh) = 47.9138485 Ton

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{22.20} \right)$$

$$= 14.496873 \text{ Ton}$$

Max of (Fh/2 + $\mu (Rg + Rq)$ and Fh - $\mu (Rg + Rq)$)

Fhr for shaft = 35.10 Ton
 Fhr for foundation = 40.72 Ton

Horizontal force for shaft
 = Fhr - Fhl
 = 35.10 - 13.39
 = 21.71 Ton

Horizontal force for foundation
 = Fhr - Fhl
 = 40.72 - 13.39
 = 27.34 Ton

BOTH SPAN POSITION CONDITION(70R)

5. Normal Max Reaction on Fix End

Left Span

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right)$$

$$= 13.39 \text{ Ton}$$

Fhl = 13.39 Ton

Right Span

Beaking Force = $0.2 \times \frac{\text{Class 70R on First two Lane}}{100} + 0.05 \times \frac{\text{Class A on third Lane}}{0.00}$

Fh = 20 Ton

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{24.01} \right)$$

$$= 14.59 \text{ Ton}$$

Max of (Fh/2 + $\mu (Rg + Rq)$ and Fh - $\mu (Rg + Rq)$)

Fhr = 24.59 Ton
Fhr = 24.59 Ton

Horizontal force
 = Fhr - Fhl
 = 24.59 - 13.39
 = 11.20 Ton

6. Normal Min Reaction on Fix End

Left Span

$$\mu (Rg + Rq) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right)$$

$$= 13.39 \text{ Ton}$$

$$F_{hl} = 13.39 \text{ Ton}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 100 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 20 \text{ Ton}$$

$$\mu (R_g + R_q) = 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{110.99} \right) = 18.94 \text{ Ton}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q) \right)$$

$$= 28.94 \text{ Ton}$$

$$F_{hr} = 28.94 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 28.94 - 13.39 \\ &= 15.55 \text{ Ton} \end{aligned}$$

7. Seismic Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\text{Beaking Force} = \frac{\text{Class 70R on First two Lane}}{0.2} \times 20 + \frac{\text{Class A on third Lane}}{0.05} \times 0$$

$$F_h = 4 \text{ Ton}$$

$$\text{Seismic Horizontal force (Shaft)} = 32.1287767 \text{ Ton}$$

$$\text{Seismic Horizontal force (foundation)} = 43.3738485 \text{ Ton}$$

$$\text{Total Horizontal force for Shaft (Fh)} = 36.1287767 \text{ Ton}$$

$$\text{Total Horizontal force for foundation (Fh)} = 47.3738485 \text{ Ton}$$

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{4.80} \right) \\ &= 13.627107 \text{ Ton} \end{aligned}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q) \right)$$

$$F_{hr} \text{ for shaft} = 33.69 \text{ Ton}$$

$$F_{hr} \text{ for foundation} = 39.31 \text{ Ton}$$

$$\begin{aligned} \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 33.69 - 13.39 \\ &= 20.30 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 39.31 - 13.39 \end{aligned}$$

$$= 25.93 \text{ Ton}$$

8. Seismic Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \text{Beaking Force} &= 0.2 \times \frac{\text{Class 70R on First two Lane}}{20} + 0.05 \times \frac{\text{Class A on third Lane}}{0} \\ F_h &= 4 \text{ Ton} \\ \text{Seismic Horizontal force(Shaft)} &= 32.1287767 \text{ Ton} \quad (\text{Including safty factor 1.5}) \\ \text{Seismic Horizontal force(foundation)} &= 43.3738485 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.}) \\ \text{Total Horizontal force for Shaft}(F_h) &= 36.1287767 \text{ Ton} \\ \text{Total Horizontal force for foundation}(F_h) &= 47.3738485 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{22.20} \right) \\ &= 14.496873 \text{ Ton} \end{aligned}$$

$$\text{Max of } (F_h/2 + \mu (R_g + R_q) \text{ and } F_h - \mu (R_g + R_q))$$

$$\begin{aligned} F_{hr} \text{ for shaft} &= 34.56 \text{ Ton} \\ F_{hr} \text{ for foundation} &= 40.18 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\ &= 34.56 - 13.39 \\ &= 21.17 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\ &= 40.18 - 13.39 \\ &= 26.80 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(Special Vehicle)

9. Normal Max Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{170.92} \right) \\ &= 21.93 \text{ Ton} \\ F_{hr} &= 21.93 \text{ Ton} \end{aligned}$$

$$\begin{aligned} \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 21.93 - 13.39 \\ &= 8.55 \text{ Ton} \end{aligned}$$

10. Normal Min Reaction on Fix End

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 22.84 \text{ Ton} \\ F_{hr} &= 22.84 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 22.84 - 13.39 \\ &= 9.45 \text{ Ton} \end{aligned}$$

BOTH SPAN POSITION CONDITION(NO LIVE LOAD)

11. Normal No Reaction

Left Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hl} &= 13.39 \text{ Ton} \end{aligned}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.39 \text{ Ton} \\ F_{hr} &= 13.39 \text{ Ton} \\ \text{Horizontal force} &= F_{hr} - F_{hl} \\ &= 13.39 - 13.39 \\ &= 0.00 \text{ Ton} \end{aligned}$$

ONE SPAN DISLODGED CONDITION

12. Normal Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned} \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \text{CWLL} \right) \\ &= 13.38699 \text{ Ton} \\ F_{hr} &= 13.38699 \text{ Ton} \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force} &= F_{hr} - F_{hl} \\
 &= 13.38699 - 0.000 \\
 &= 13.38699 \text{ Ton}
 \end{aligned}$$

13. Seismic Span Dislodge Condition

Consider left Span dislodge

Left Span

$$F_{hl} = 0 \text{ Ton}$$

Right Span

$$\begin{aligned}
 \text{Beaking Force} &= \text{Class 70R on First two Lane} + \text{Class A on third Lane} \\
 &= 0.2 \times 0 + 0.05 \times 0
 \end{aligned}$$

$$F_h = 0 \text{ Ton}$$

$$\text{Seismic Horizontal force(Shaft)} = 32.1287767 \text{ Ton} \quad (\text{Including safty factor 1.5})$$

$$\text{Seismic Horizontal force(foundation)} = 43.3738485 \text{ Ton} \quad (\text{Including safty factor 1.5 and 35 \% inc.})$$

$$\text{Total Horizontal force for Shaft}(F_h) = 32.1287767 \text{ Ton}$$

$$\text{Total Horizontal force for foundation}(F_h) = 43.3738485 \text{ Ton}$$

$$\begin{aligned}
 \mu (R_g + R_q) &= 0.05 \times \left(\frac{\text{Total DL}}{215.925} + \frac{\text{Total SIDL load}}{51.81} + \frac{\text{CWLL}}{0.00} \right) \\
 &= 13.38699 \text{ Ton}
 \end{aligned}$$

$$\text{Max of } \left(\frac{F_h}{2} + \mu (R_g + R_q) \right) \text{ and } F_h - \mu (R_g + R_q)$$

$$F_{hr} \text{ for shaft} = 29.45 \text{ Ton}$$

$$F_{hr} \text{ for foundation} = 35.07 \text{ Ton}$$

$$\begin{aligned}
 \text{Horizontal force for shaft} &= F_{hr} - F_{hl} \\
 &= 29.45 - 0.00 \\
 &= 29.45 \text{ Ton}
 \end{aligned}$$

$$\begin{aligned}
 \text{Horizontal force for foundation} &= F_{hr} - F_{hl} \\
 &= 35.07 - 0.00 \\
 &= 35.07 \text{ Ton}
 \end{aligned}$$

Summary of forces			
Cases		Horizontal force(Ton)	
		Shaft	Foundation
BOTH SPAN POSITION CONDITION(70R + Class A)			
	1. Normal Max Reaction on Fix End	18.90	18.90
	2. Normal Min Reaction on Fix End	16.90	16.90
	3. Seismic Max Reaction on Fix End	20.84	26.47
	4. Seismic Min Reaction on Fix End	21.71	27.34
BOTH SPAN POSITION CONDITION(70R)			
	5. Normal Max Reaction on Fix End	11.20	11.20
	6. Normal Min Reaction on Fix End	15.55	15.55
	7. Seismic Max Reaction on Fix End	20.30	25.93
	8. Seismic Min Reaction on Fix End	21.17	26.80
BOTH SPAN POSITION CONDITION(Special Vehicle)			
	9. Normal Max Reaction on Fix End	8.55	8.55
	10. Normal Min Reaction on Fix End	9.45	9.45
BOTH SPAN POSITION CONDITION(NO LIVE LOAD)			
	11. Normal No Reaction	0.00	0.00
ONE SPAN DISLOGED CONDITION			
	12. Normal Span Dislodge Condition	13.39	13.39
	13. Seismic Span Dislodge Condition	29.45	35.07

11.A CALCULATION OF CENTRIFUGAL FORCE

As per clause 212 of IRC:6-2014

$$C = \frac{WV^2}{127R}$$

70R + Class A

Maxium Reaction				
Live Load	W	=	110.99	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0.00	14.25	16.25

Minimum Reaction				
Live Load	W	=	0.00	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0.00	t	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.25	16.25

70R

Maxium Reaction				
Live Load	W	=	110.99	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.25	16.25

Minimum Reaction				
Live Load	W	=	24.01	t
Design Speed	V	=	50	Kmph
Radius of Curvature	R	=	0	m
C	=	0	KN	

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.25	16.25

Special Vehicle**Maxium Reaction**

Live Load	W	=	189.08	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.25	16.25

Minimum Reaction

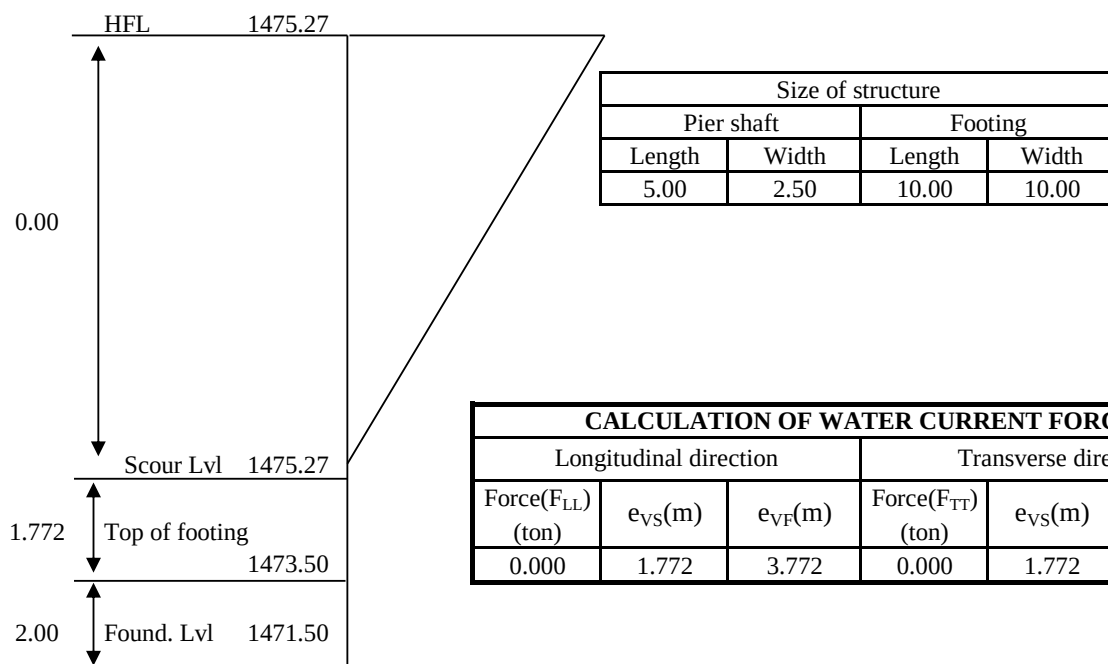
Live Load	W	=	170.92	t
Design Speed	V	=	5	Kmph
Radius of Curvature	R	=	0	m

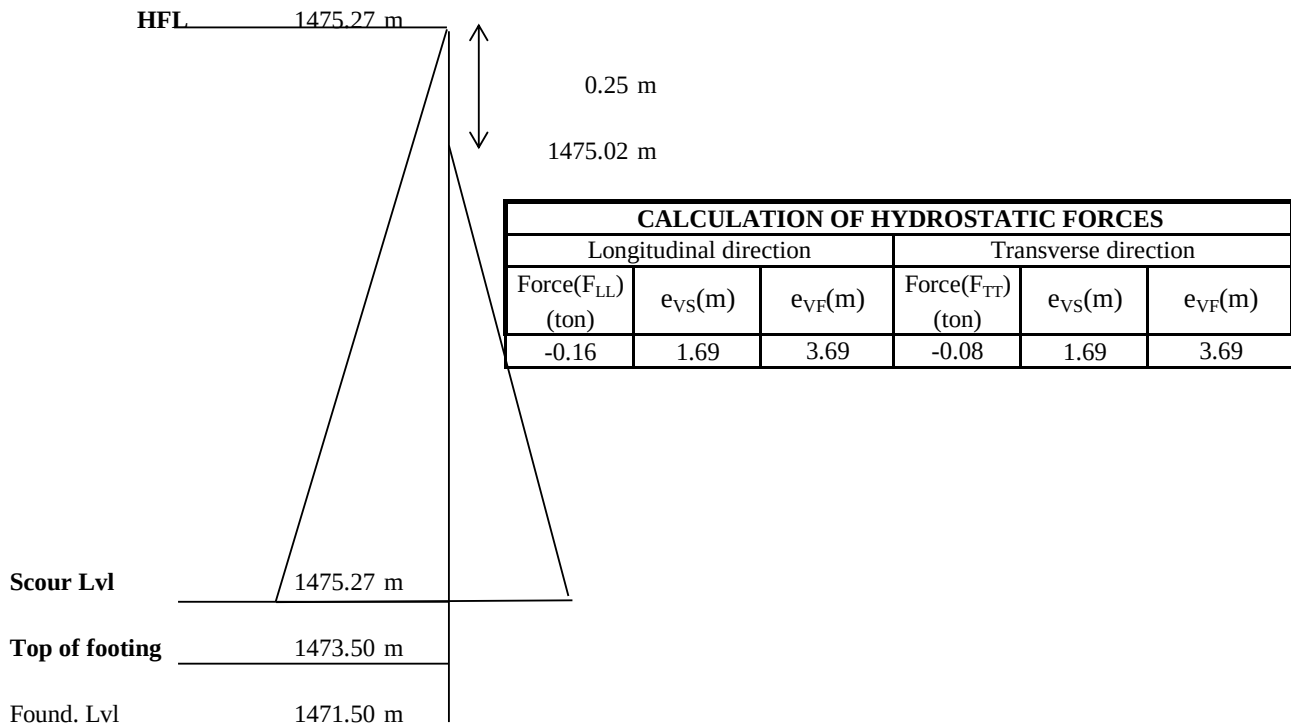
$$C = 0 \text{ KN}$$

C (t)	e _{VS} (m)	e _{VF} (m)
0	14.25	16.25

12. WATER CURRENT FORCES**NORMAL CASE****Water current forces**

HFL	=	1475.267	m
MSL (NORMAL)	=	1475.267	m
Water current pressure	P	=	52 KV ² Kg/m ²
For piers with square ends	K	=	1.5
For piers with semicircular ends	K	=	0.66
Mean velocity of water current	V _{mean}	=	0.00 m/s
Max. velocity of water current	V	=	$\sqrt{2}$ x 0.00
		=	0.00 m/s
Angle of variation		=	20 degree
Water current pressure at HFL	(Trans direction)	=	0.00 t/m ² 52 K (vcosq) ²
Water current pressure at HFL	(Long direction)	=	0.00 t/m ² 53 K (vsinq) ²

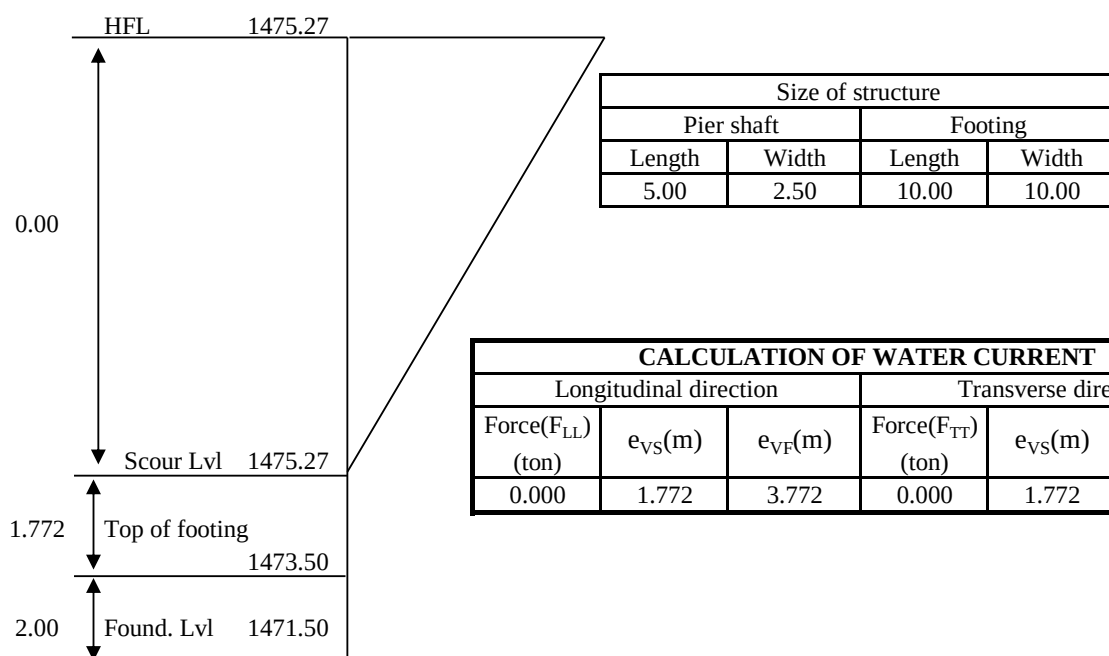
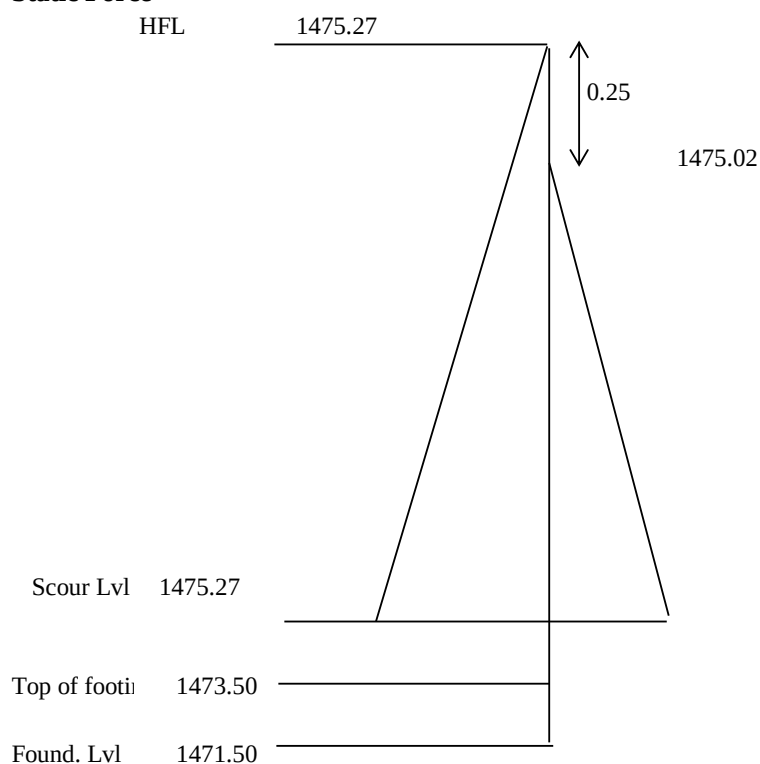
**Pressure diagram (Normal case)**

Static Force

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	1.772	3.772	0.000	1.772	3.772

SEISMIC CASE

HFL	=	1475.267 m		
MSL (SEISMIC)	=	1475.27 m		
Water current pressure	P	=	52 KV ²	Kg/m ²
For piers with square ends	K	=	1.5	
For piers with semicircular ends	K	=	0.66	
Mean velocity of water current	V_{mean}	=	0.00 m/s	
Max. velocity of water current	V	=	$\sqrt{2} \times 0.00$	
		=	0.00 m/s	
Angle of variation		=	20 degree	
Water current pressure at HFL (Trans direction)		=	0.00 t/m ²	52 K (vcosq) ²
Water current pressure at HFL (Long direction)		=	0.00 t/m ²	53 K (vsinq) ²

**Pressure diagram (seismic case)****Static Force**

CALCULATION OF HYDROSTATIC FORCES					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
-0.16	1.69	3.69	-0.08	1.69	3.69

Governing					
Longitudinal direction			Transverse direction		
Force(F_{LL}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$	Force(F_{TT}) (ton)	$e_{VS}(m)$	$e_{VF}(m)$
0.000	1.772	3.772	0.000	1.772	3.772

13. LOAD COMBINATION DEFINATION**PIER SHAFT DESIGN****ULS**

LC -1 ULS NORMAL LWL MAX REACTION
 LC -2 ULS NORMAL LWL SPAN DISLOGED

LC -5 ULS MAX REACTION $1X + 0.3Y + 0.3Z$
 LC -6 ULS MAX REACTION $0.3X + 1Y + 0.3Z$

LC -9 ULS NORMAL SV MAX REACTION

SLS

LC -1 RARE NORMAL MAX REACTION
 LC -2 RARE NORMAL SPAN DISLOGED

LC -5 QP NORMAL

LC -7 QP NORMAL SPAN DISLOGED

LC -9 RARE NORMAL SV MAX REACTION

Footing design

LC -1 ULS NORMAL MAX REACTION(70R +Class A)
 LC -2 ULS NORMAL SPAN DISLOGED

LC -5 ULS NORMAL MAX REACTION(70R)

LC -7 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R + \text{Class A})$
 LC -8 ULS SPAN DISLOGED $1X + 0.3Y + 0.3Z$
 LC -9 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{Class A})$
 LC -10 ULS SPAN DISLOGED $0.3X + 1Y + 0.3Z$

LC -15 ULS MAX REACTION $1X + 0.3Y + 0.3Z(70R)$
 LC -16 ULS MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 19 ULS NORMAL LWL SV MAX REACTION

BASE PRESSRE

LC - 1 NORMAL MAX REACTION(70R+Class A)

LC - 2 NORMAL SPAN DISLOGED

LC - 5 0

LC - 6 SEISMIC SPAN DISLODGED $1X + 0.3Y + 0.3Z$

LC - 7 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R + \text{class A})$

LC - 8 SEISMIC LWL SPAN DISLODGED $0.3X + 1Y + 0.3Z$

LC - 13 SEISMIC NORMAL MAX REACTION(70R)

LC - 15 SEISMIC MAX REACTION $1X + 0.3Y + 0.3Z(70R)$

LC - 16 SEISMIC MAX REACTION $0.3X + 1Y + 0.3Z(70R)$

LC - 13 NORMAL LWL SV MAX REACTION

16. DESIGN FORCES FOR BASE PRESSURE**LC -1 NORMAL MAX REACTION(70R+Class A)**

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			16.248	0.0	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	19.491	2.5	48.728		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	4.676	2.5	11.690		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.000		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				18.901			13.058	246.80		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	210.02	2	420.048		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1660.47	309.23	29.97

LC -2 NORMAL SPAN DISLOGED

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	1	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	1	0.9
11	Centifugal force				0.000			16.248	0.000	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	19.491	2.5	48.728		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	4.676	2.5	11.690		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.000		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				13.387			13.058	174.807		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	210.02	2	420.048		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1292.84	342.14	0.00

342.145 0.000

LC - 6 SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			16.248	0.00	0.000	0	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	19.49	2.50	48.73		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	4.68	2.50	11.69		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.00		1	

	Water Current Force(F_{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F_{LL})				35.074			13.058	458.00		1	
	Horizontal Force(F_{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	210.02	2.00	420.05		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				43.374				566.38		1	
	Transverse				31.734					425.01085	0.3	
	Vertical			21.419							0.3	
	Sub structure											
	Longitudinal				32.205				310.7		1	
	Transverse				31.810					306.92	0.3	
	Vertical			21.470							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M_{TT} (tm)	M_{LL} (tm)
1305.703	1354.99	219.58

LC - 8 SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			0.000		-0.625	0.000		0.000	0.000	1	
2	Surfacing			0.000		-0.625	0.000		0.000	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0.2	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			0.000		0.625	0.300		0.000	0.000	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0.2	
11	Centifugal force				0.000			16.248	0.00	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	19.49	2.50	48.73		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	4.68	2.50	11.69		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.00		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				35.074			13.058	458.00		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	210.02	2.00	420.05		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				0.000			0.00			0.3	
	Transverse				31.734					425.01085	1	
	Vertical			21.419							0.3	
	Sub structure											
	Longitudinal				32.205				310.7		0.3	
	Transverse				31.810					306.92139	1	
	Vertical			21.470							0.3	

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1305.703	571.10	731.93

LC -13 SEISMIC NORMAL MAX REACTION(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
Left Span												
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	1	0.9
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	0.9
Right Span												
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	1	0.9
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	0.9
11	Centifugal force				0.000			16.248	0.0	0.000	1	
12	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	19.491	2.5	48.728		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	4.676	2.5	11.690		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.000		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				11.201			13.058	146.26		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
19	Earth Over Footing	210.02	2	420.048		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M _{TT} (tm)	M _{LL} (tm)
1660.47	208.69	29.97

LC - 15 SEISMIC MAX REACTION 1X + 0.3Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.368	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			16.248	0.0	0.000	0.2	
12	Pier cap	55.352	2.500	138.380		0.000	0.000		0.000	0.000	1	
13	Pier (above LBL)	19.491	2.500	48.728		0.000	0.000		0.000	0.000	1	
14	Pier (below LBL)	4.676	2.500	11.690		0.000	0.000		0.000	0.000	1	
15	Buoyancy	0.000	1.000	0.000		0.000	0.000		0.000	0.000	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.000		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				25.927			13.058	338.56		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.00	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	210.02	2.00	420.05		0.00	0.00		0.00	0.00	1	
20	Seismic											
	Super structure											
	Longitudinal				86.748			1132.8			1	
	Transverse				85.682					1197.0	0.3	
	Vertical			57.832							0.3	
	Sub structure											
	Longitudinal				32.205				310.7		1	
	Transverse				31.810					306.92	0.3	
	Vertical			21.47							0.3	
a)	Max Live Load											
	Longitudinal										1	
	Transverse				3.552					57.71	0.3	
	Vertical			2.397							0.3	

b)	Min Live Load											
	Longitudinal										1.5	1
	Transverse				0.768					12.485365	0.45	0.3
	Vertical			0.519							0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1607.353	1795.92	476.82

LC - 16 SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R)

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	0	0
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL (70R)			0.000		-0.625	0.950		0.000	0.000	0.2	
5	LL (Class-A)			0.000		-0.625	1.000		0.000	0.000	0	
	Right Span											
6	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
7	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
8	FLL			0.000		0.625	0.000		0.000	0.000	1	
9	LL (70R)			110.988		0.625	0.300		69.3677	33.296	0.2	
10	LL (Class-A)			0.000		0.625	1.000		0.000	0.000	0	
11	Centifugal force				0.000			16.248	0.0	0.000	0.2	
12	Pier cap	55.35	2.50	138.38		0.00	0.00		0.00	0.00	1	
13	Pier (above LBL)	19.49	2.50	48.73		0.00	0.00		0.00	0.00	1	
14	Pier (below LBL)	4.68	2.50	11.69		0.00	0.00		0.00	0.00	1	
15	Buoyancy	0.00	1.00	0.00		0.00	0.00		0.00	0.00	1	
16	Water Current Force(F _{LL})				0.000			3.772	0.00		1	
	Water Current Force(F _{TT})				0.000			3.772		0.000	1	
17	Horizontal Force(F _{LL})				25.927			13.058	338.56		1	
	Horizontal Force(F _{TT})				0.000			13.058		0.000	1	
18	Footing	162.50	2.50	406.25		0.00	0.00		0.00	0.00	1	
19	Earth Over Footing	210.02	2.00	420.05		0.00	0.00		0.00	0.00	1	
20	Seismic											

	Super structure											
	Longitudinal				86.748				1132.8		0.3	
	Transverse				85.682				1197.0		1	
	Vertical		57.832								0.3	
	Sub structure											
	Longitudinal				32.205				310.7		0.3	
	Transverse				31.810				306.92		1	
	Vertical		21.470								0.3	
a)	Max Live Load											
	Longitudinal										0.3	
	Transverse				3.552				57.71		1	
	Vertical		2.397								0.3	
b)	Min Live Load											
	Longitudinal										0.45	0.3
	Transverse				0.768				12.5		1.5	1
	Vertical		0.519								0.45	0.3

SUMMARY OF FACTORED FORCES		
V(ton)	M _{TT} (tm)	M _{LL} (tm)
1607.353	785.48	1586.99

LC - 18 NORMAL SV MAX REACTION

Sr. No.	Description	Volume (m ³)	Unit weight (t/m ³)	Vertical Load (V in ton)	Horizontal Load(ton)	e _L (m)	e _T (m)	e _{VF} (m)	M _{TT} (tm)	M _{LL} (tm)	Load factors	Lane Reduction Factor
	Left Span											
1	DL (superstructure)			235.925		-0.625	0.000		-147.45	0.000	1	
2	Surfacing			31.815		-0.625	0.000		-19.884	0.000	1	
3	FLL			0.000		-0.625	0.000		0.000	0.000	1	
4	LL			0.000		-0.625	0.300		0.000	0.000	1	1
	Right Span											
5	DL (superstructure)			235.925		0.625	0.000		147.45	0.000	1	
6	Surfacing			31.815		0.625	0.000		19.884	0.000	1	
7	FLL			0.000		0.625	0.000		0.000	0.000	1	
8	LL			189.078		0.625	0.300		118.174	56.723	1	1
9	Centifugal force				0.000			16.248	0.0	0.000	1	
10	Pier cap	55.352	2.5	138.380		0.000	0.000		0.000	0.000	1	
11	Pier (above LBL)	19.491	2.5	48.728		0.000	0.000		0.000	0.000	1	

12	Pier (below LBL)	4.676	2.5	11.690		0.000	0.000		0.000	0.000	1	
13	Buoyancy	0.000	1	0.000		0.000	0.000		0.000	0.000	1	
14	Water Current Force(F_{LL})				0.000			3.772	0.000		1	
	Water Current Force(F_{TT})				0.000			3.772		0.000	1	
15	Horizontal Force(F_{LL})				8.546			13.058	111.59		1	
	Horizontal Force(F_{TT})				0.000			13.058		0.000	1	
16	Footing	162.50	2.5	406.250		0.000	0.000		0.000	0.000	1	
17	Earth Over Footing	210.02	2	420.048		0.00	0		0.000	0.000	1	

SUMMARY OF FACTORED FORCES

V(ton)	M_{TT} (tm)	M_{LL} (tm)
1749.65	229.77	56.72

PROPERTIES OF BASE					
Area	=	10.00	x	10.00	= 100.00 m ²
I _{LL}	=	10.00	x	$\frac{10.00^3}{12}$	= 833.33 m ⁴
I _{TT}	=	10.00	x	$\frac{10.00^3}{12}$	= 833.33 m ⁴

Edges	L(m)	T(m)
A	5.00	5.00
B	-5.00	5.00
C	-5.00	-5.00
D	5.00	-5.00

$$\text{Base pressure} = P/A + M_{LL} * x/I_{LL} + M_{TT} * z/I_{TT}$$

Sr. No.	Description	Forces at founding levels			Base pressure at footing corners in T/m ²			
		V(Tonne)	M _{TT} (Tm)	M _{LL} (Tm)	A	B	C	D
LC -1	NORMAL MAX REACTION(70R+Class A)	1660.47	309.23	29.97	18.64	14.93	14.57	18.28
LC -2	NORMAL SPAN DISLOGED	1292.84	342.14	0.00	14.98	10.88	10.88	14.98
LC -6	SEISMIC SPAN DISLODGED 1X + 0.3Y + 0.3Z	1305.70	1354.99	219.58	22.50	6.24	3.61	19.87
LC -7	SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R + class A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LC -8	SEISMIC LWL SPAN DISLODGED 0.3X + 1Y + 0.3Z	1305.70	571.10	731.93	20.88	14.02	5.24	12.09
LC -13	SEISMIC NORMAL MAX REACTION(70R)	1660.47	208.69	29.97	18.04	15.53	15.17	17.68

LC - 15	SEISMIC MAX REACTION 1X + 0.3Y + 0.3Z(70R)	1607.35	1795.92	476.82	29.71	8.16	2.44	23.99
LC - 16	SEISMIC MAX REACTION 0.3X + 1Y + 0.3Z(70R)	1607.35	785.48	1586.99	30.31	20.88	1.84	11.26
LC - 18	NORMAL SV MAX REACTION	1749.65	229.77	56.72	19.22	16.46	15.78	18.53

Sr. No.	Average pressure along BC	Average pressure along AD	Average pressure along AB	Average pressure along CD	Effective Length along TT	Effective Width along LL	Effective Area	I _{LL}	I _{TT}
	T/m ²	T/m ²	T/m ²	T/m ²	m	m	m ²	m ⁴	m ⁴
LC -1	14.75	18.46	16.78	16.42	10.00	10.00	100.00	833.33	833.33
LC -2	10.88	14.98	12.93	12.93	10.00	10.00	100.00	833.33	833.33
LC -6	4.93	21.19	14.37	11.74	10.00	10.00	100.00	833.33	833.33
LC -8	9.63	16.48	17.45	8.67	10.00	10.00	100.00	833.33	833.33
LC -13	15.35	17.86	16.78	16.42	10.00	10.00	100.00	833.33	833.33
LC -15	5.30	26.85	18.93	13.21	10.00	10.00	100.00	833.33	833.33
LC -16	11.36	20.79	25.60	6.55	10.00	10.00	100.00	833.33	833.33
LC -18	16.12	18.88	17.84	17.16	10.00	10.00	100.00	833.33	833.33

Sr. No.	M _{TT}	M _{LL}	A'		B'		C'		D'		Base Pressure(T/m ²)			
	Tm	Tm	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	L(m)	T(m)	T/m ²	T/m ²	T/m ²	T/m ²
LC -1	309.23	29.97	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	18.64	14.93	14.57	18.28
LC -2	342.14	0.00	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	14.98	10.88	10.88	14.98
LC -6	1354.99	219.58	5.00	5	-5.00	5	-5.00	-5	5.00	-5.00	22.50	6.24	3.61	19.87
LC -8	571.10	731.93	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	20.88	14.02	5.24	12.09
LC -13	208.69	29.97	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	18.04	15.53	15.17	17.68
LC -15	1795.92	476.82	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	29.71	8.16	2.44	23.99
LC -16	785.48	1586.99	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	30.31	20.88	1.84	11.26
LC -18	229.77	56.72	5.00	5.00	-5.00	5.00	-5.00	-5.00	5.00	-5.00	19.22	16.46	15.78	18.53

Minimum Effective Area Allowed Normal case = 0.8 x 100 = 80.00 m²

Minimum Effective Area Allowed Seismic case = 0.67 x 100 = 67.000 m²

Minimum Effective Area Normal Case = 100.00 > 80.00 m²

Minimum Effective Area Seismic Case = 100.00 > 67.00 m²

SUMMARY OF BASE PRESSURES

NORMAL CASE

Maximum base pressure = 19.22 t/m² < 35 t/m² **Ok**

Minimum base pressure = 10.88 t/m² > 0 t/m² **Ok**

SEISMIC CASE

Maximum base pressure = 30.31 t/m² < 43.75 t/m² **Ok**

Minimum base pressure = 1.84 t/m² > 0 t/m² **Ok**

***DESIGN OF PRECAST POST TENSIONED
GIRDER AND CAST IN SITU DECK SLAB
TYPE SUPERSTRUCTURE***

**EFFECTIVE SPAN - 23.75 m
DEGREE OF SKEW - 0 deg
OVERALL DECK WIDTH –12.5 m**

DESIGN DATA AND SALIENT FEATURES OF SUPERSTRUCTURE

1. DESCRIPTION OF STRUCTURE

* Span Length (c/c of bearing)	=	23.750	m	
* Overhange of Girder at each end	=	0.350	m	
* Width of Exp. Joint	=	50	mm	
* Overhange of Deck Slab at each end	=	0.600	m	
* Span Length (c/c of Expansion Joint)	=	25.00	m	
* Skew Angle	=	0	degree	
* Width of Carriageway	=	11.50	m	
* Width of Footpath	=	0.00	m	both side
* Overall Deck Width	=	12.50	m	
* Cross Slope of superstructure	=	2.50	%	(bidirectional)
* Camber	=	5.00	%	(bidirectional)
* Nos. of Main Longitudinal Girders	=	4	Nos.	
* c/c Distance of Longitudinal Girder	=	3.000	m	
* Dist. bet. c/c outer girder and edge of deck	=	1.75	m	
* Depth of Longitudinal Girder	=	1.600	m	
* Thickness of Deck Slab	=	0.220	m	
* Overall Depth of Superstructure	=	1.820	m	
* Type of Bearings	=	SPHERICAL BEARING		
* Type of Expansion Joints	=	Strip Seal Type		
* Width of Crash Barrier on either side	=	0.5	m	
* Thickness of Wearing coat	=	65	mm	

2. MATERIAL

* Type of cement	=	Ordinary portland cement	s =	0.25
* Relative Humidity of ambient climate R_H	=	80 %		
* Average ambient temperature during curing T	=	25 deg		
* Grade of Concrete For Precast girder	=	M- 45		
* Grade of Concrete For Deck Slab	=	M- 45		
* Grade of H.Y.S.D. Reinforcement	=	Fe- 500		
* Modulus of elasticity of concrete , E_{cm}	=	3.40E+04	N/mm ²	
* Modulus of elasticity of Reinforcement , E_s	=	2.00E+05	N/mm ²	
* Modulus of elasticity of prestress , E_p	=	1.95E+05	N/mm ²	
* Prestressing Steel	=	12 DP 13 (12 Strands of 12.7 mm Nominal Dia.) 7- PLY Class 2 Conforming To of IS 14268 (Low Relaxation Strands with Corrugated HDPE Sheathing)		
* Outer Dia. Of Duct	=	75	mm	(to be confirmed from supplier)
* Type of Sheathing	=	Corrugated HDPE		
* Friction Coeff. (μ)	=	0.17	per radians	

* Wobble Coeff. (k)	=	0.002	per meter
* Jacking Force	=	0.765 %	of UTS (refer clause 7.9.2 (1) of IRC:112-2011)
* Shear analysis	=	As per stipulation of IRC :112-2011	
* Diameter of aggregate	=	20	mm

3. CONSTRUCTION SEQUENCE

The construction schedule for typical span is taken as following:

Age of the Girder	Activity
0 Days	Casting of Long. I-Girder at Casting Yard with portion of Diaphragm
7 Days	Transfer of Prestress of 1st Stage Prestressing(using one end prestressing)
7 Days -- 28 Days	Girder age between I stage and IInd Stage Prestressing
28 Days	Transfer of Prestress of IInd Stage Prestressing
28 Days -- 35 Days	Position the I-Girder on permanent bearings. Erect shuttering for RCC deck slab supported from bottom bulb of girders.
35 Days	Casting of Deck Slab along with balance portion of Diaphragms
49 Days	Average Age of Girder When Composite action takes place
49 Days -- 63 Days	Fixing the Crash Barrier and laying of Wearing Coat
63 Days	Average Age of Girder When SIDL acts on it
Infinity	Average Age of Girder When CWLL acts on it

4. LOADING

DEAD LOAD (DL)

* Unit Weight of Precast Concrete Girder and deck slab	=	25	kN/m ³
* Unit Weight of Green Concrete	=	26	kN/m ³

SUPER IMPOSED DEAD LOAD (SIDL)

- * 65 mm thick wearing coat intensity with additional overlay has been taken as 2.2 kN/m²
- * Weight of crash barrier has been taken as 8 kN/m

Footpath Live Load (FPLL) on Superstructure

Refer clause 206.3 of IRC:6-2014

P1	=	0	kg/m ²				
Effective span L	=	23.75	m				
Intensity of FPLL , P	=	P1	-	(40x L - 300)/9			
	=	0	-	72.22	=	-72.22	kg/m ² = -0.72 kN/m ²
UDL	=	-0.72	x	0.00	=	0.000	kN/m

CARRIAGEWAY LIVE LOAD (CWLL)

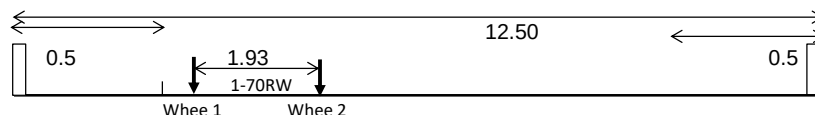
Superstructure meant for 3-lane carriageway has been designed for governing effect of CWLL loading as specified in IRC:6-2017 with appropriate lane reduction factor and suitable allowances for impact so placed on the span to produce maximum stresses.

The following CWLL combination have been considered in the design. The design is then carried out for the most critical CWLL Combination.

* 1-70R Wheel (2 Lane)	* 1-70R Wheel (2 Lane) + 1-CLASS A
* 2-Class A	* 3-Class A
	* SV LOAD

1) 1-70RW

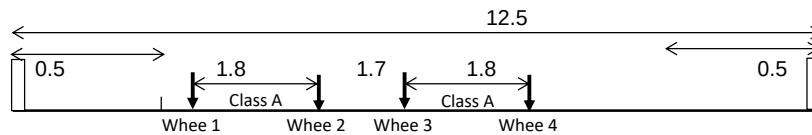
Load position of 1- 70RW



$$\text{Wheel 1} = 0.5 + 1.2 + 0.43 = 2.13 \text{ m}$$

$$\text{Wheel 2} = 2.13 + 1.93 = 4.06 \text{ m}$$

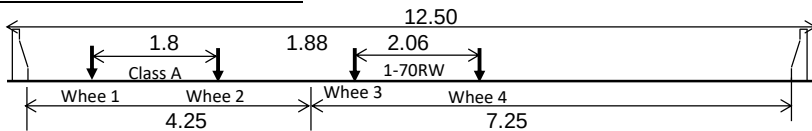
2) 2-Class A



$$\begin{aligned} \text{Wheel 1} &= 0.5 + 0.15 + 0.25 = 0.90 \text{ m} \\ \text{Wheel 2} &= 0.9 + 1.8 = 2.7 \text{ m} \\ \text{Wheel 3} &= 2.7 + 1.7 = 4.4 \text{ m} \\ \text{Wheel 4} &= 4.4 + 1.8 = 6.2 \text{ m} \end{aligned}$$

3) 1- Lane Class A + 1-70RW

Load position of 1-Lane Class A + 1- 70RW



no other vehicle permitted in this zone

From left Side

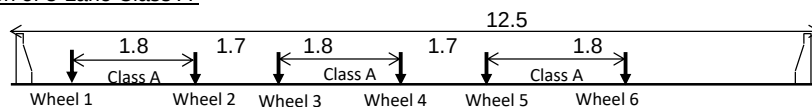
$$\begin{aligned} \text{Wheel 1} &= 0.5 + 0.15 + 0.25 = 0.9 \text{ m} \\ \text{Wheel 2} &= 0.9 + 1.8 = 2.7 \text{ m} \\ \text{Wheel 3} &= 2.7 + 1.88 = 4.58 \text{ m} \\ \text{Wheel 4} &= 4.58 + 2.06 = 6.64 \text{ m} \end{aligned}$$

From Right Side

$$\begin{aligned} \text{Wheel 4} &= 0.5 + 1.2 + 0.43 = 2.13 \text{ m} \\ \text{Wheel 3} &= 2.13 + 2.06 = 4.19 \text{ m} \\ \text{Wheel 2} &= 7.75 + 0.25 = 8 \text{ m} \\ \text{Wheel 1} &= 8 + 1.8 = 9.8 \text{ m} \end{aligned}$$

4) 3- Lane Class A

Load position of 3-Lane Class A



$$\begin{aligned} \text{Wheel 1} &= 0.5 + 0.15 + 0.25 = 0.9 \text{ m} \\ \text{Wheel 2} &= 0.9 + 1.8 = 2.7 \text{ m} \\ \text{Wheel 3} &= 2.7 + 1.7 = 4.4 \text{ m} \\ \text{Wheel 4} &= 4.4 + 1.8 = 6.2 \text{ m} \\ \text{Wheel 5} &= 6.2 + 1.7 = 7.9 \text{ m} \\ \text{Wheel 6} &= 7.9 + 1.8 = 9.7 \text{ m} \end{aligned}$$

5. TEMPERATURE EFFECT

The climate condition is assumed to be "MODERATE". The temperature variation of $\pm 25^\circ\text{C}$ shall be considered for the design of bearings and expansion joints. Temperature gradients given in IRC 6:2017 has also been considered to evaluated the effect on structure due to daily changes in temperature.

Coefficient of thermal expansion $= 1.2 \times 10^{-5}$ as per IRC: 6-2017

6. TEMPERATURE GRADIENT

Temperature gradient has been considered as per clause 215.3 of IRC:6-2017

7. STRUCTURAL ANALYSIS

7.1 SECTION PROPERTIES

Uncracked sectional properties have been used for analysis. The effective flange widths for sectional properties for the longitudinal girders and cross diaphragms have been determined in accordance with IRC:112-2011

7.2 ANALYSIS FOR LONGITUDINAL GIRDER AND DIAPHRAGMS

Superstructure has been modeled as an equivalent grillage mesh consisting of longitudinal and transverse members. Analysis of Self weight of girder and green concrete of slab has been carried out by single line model and the analysis of SIDL and CWLL has been carried out by grillage model.

STAAD-Pro Software , which is widely accepted for structural analysis,has been used for analysis.

For the design of longitudinal girders , moments and shear forces has been detremined at the crittical section . The design of various components has been carried out by design excel sheets. End diaphragm has been designed for DL+SIDL load in Jack up condition.

7.3 ANALYSIS OF DECK SLAB

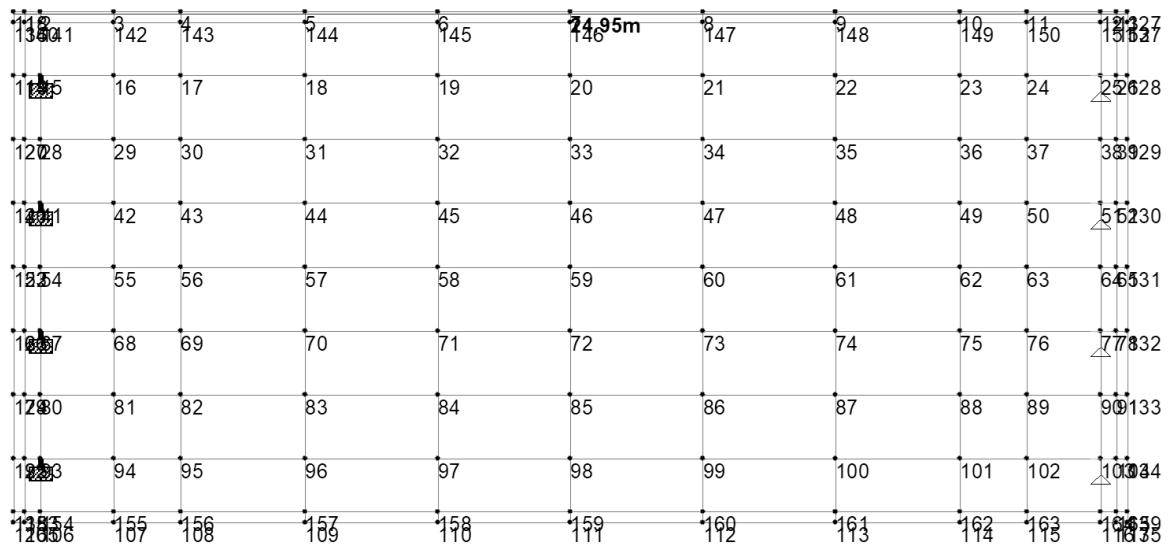
Deck slab has been analysed along transverse direction as a continuous beam with overhangs supported on longitudinal girders and diaphragms. Live load effects shall be taken based on effective width method.

7.4 PROVISIONS OF EMERGENCY CABLES

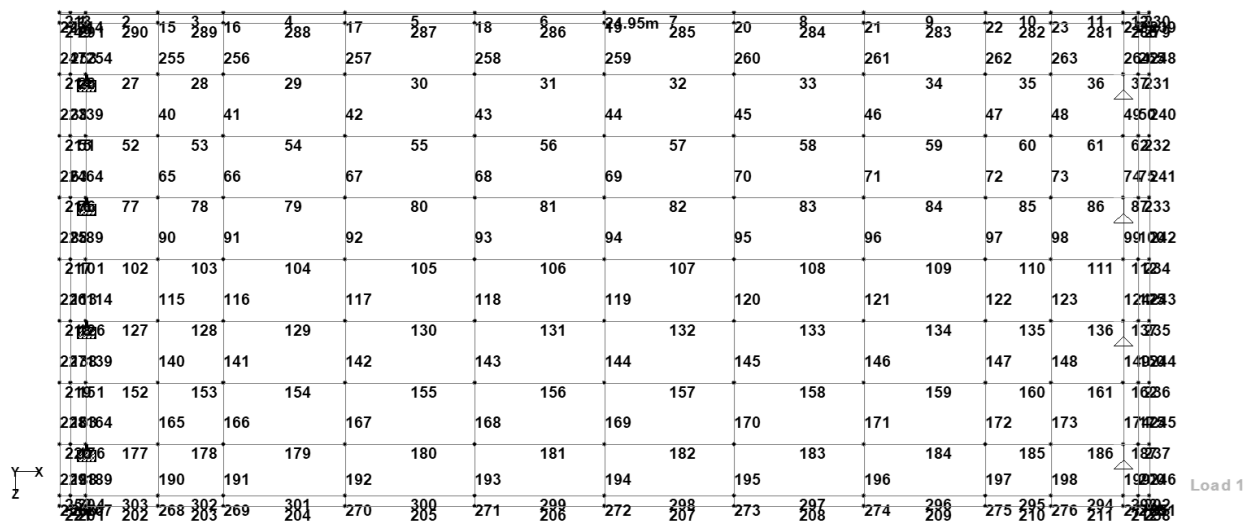
In addition to design requirements, additional cables/strands symmetrically placed in structure, capable of generating prestressing force about 4 % of total design prestressing force in the structure shall be provided.Only those cables which are required to make up the deficiency shall be stressed and the remaining cables/strands will be pulled out and the cable holes shall be epoxy grouted.

7.5 REFERENCE IRC CODES USED IN DESIGN

- * IRC:6-2017 Section II, Load and stress
- * IRC:112-2011 code of practice of concrete road bridges (Limit state Design)
- * IRC:83 (Part IV)-2000 Standard specification and code of practice for road bridges/ section IX-bearing /Part:IV-SPHERICAL bearings



GRILLAGE SHOWING JOINT NUMBERS



GRILLAGE SHOWING MEMBER NUMBERS

SECTIONAL PROPERTIES OF OUTER GIRDER

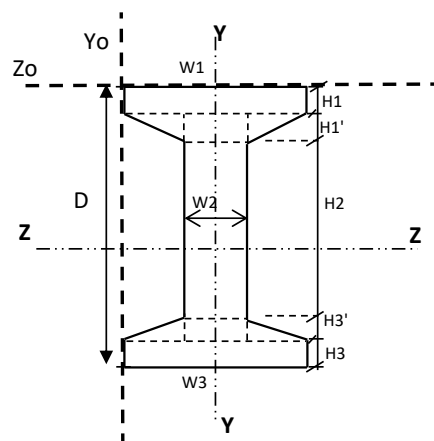
1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

For Section 6.1375m TO 18.0125m From Jacking End

(Dimension in mm)

D	=	1600			
H1	=	150	W1	=	1000
H1'	=	100			
H2	=	950	W2	=	275
H3'	=	150			
H3	=	250	W3	=	700
			Perimeter , u	=	5672.30

Yo, Zo - Reference Axis



1a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Izz=Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	8.438E+08	2.813E+08	1.125E+09
1' (Tri.)	725	100	36250	183	6.646E+06	1.218E+09	2.014E+07	1.239E+09
1' (Rect.)	275	100	27500	200	5.500E+06	1.100E+09	2.292E+07	1.123E+09
2 (Rect.)	275	950	261250	725	1.894E+08	1.373E+11	1.965E+10	1.570E+11
3' (Tri.)	425	150	31875	1300	4.144E+07	5.387E+10	3.984E+07	5.391E+10
3' (Rect.)	275	150	41250	1275	5.259E+07	6.706E+10	7.734E+07	6.713E+10
3 (Rect.)	700	250	175000	1475	2.581E+08	3.807E+11	9.115E+08	3.816E+11
			723125		5.650E+08	6.42.E+11	2.100E+10	6.631E+11

Total Area of Section	A	=	0.723 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.781 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.819 m	
Moment of Inertia of Section	Iz	=	0.222 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.284 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.271 m ³	

1b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
1 (Rect.)	1000	150	150000	0	0	0	1.25E+10	1.25E+10
1' (Tri.)	725	100	36250	258.333	9.36E+06	2.42E+09	2.65E+08	2.68E+09
1' (Rect.)	275	100	27500	0	0	0	1.73E+08	1.73E+08
2 (Rect.)	275	950	261250	0	0	0	1.65E+09	1.65E+09
3' (Tri.)	425	150	31875	208.333	6.64E+06	1.38E+09	8.00E+07	1.46E+09
3' (Rect.)	275	150	41250	0	0	0	2.60E+08	2.60E+08
3 (Rect.)	700	250	175000	0	0	0	7.15E+09	7.15E+09
								2.59E+10

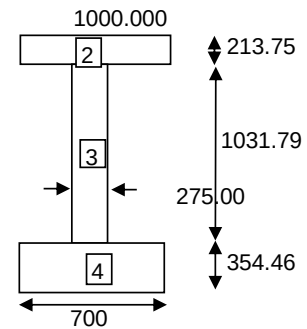
Moment of Inertia of Section Iyy = 0.026 m⁴

1c. Calculation of Sectional Properties about X-X Axis

Average Thk. Of top flange = 213.750 mm
 Average Thk. Of bottom flange = 354.464 mm

$$I_{xx} = \frac{3 \cdot b^3 \cdot d^3}{10(b^2 + d^2)}$$

Portion no.	b mm	d mm	$\frac{3 \cdot b^3 \cdot d^3}{10(b^2 + d^2)}$
2	1000	213.75	2.80E+09
3	275	1031.79	6.01E+09
4	700	354.46	7.44E+09



Moment of Inertia of Precast Girder only $I_{xx} = 1.63E+10 \text{ mm}^4$

1d . First moment of Area Above Centroid of Composite Section (Ay)p**Property of Girder for Mid Span**

Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)p mm ³
1 (Rect.)	1000	150	150000	263.154	3.95E+07
1' (Tri.)	725	100	36250	154.821	5.61E+06
1' (Rect.)	275	100	27500	138.154	3.80E+06
2 (Rect.)	275	88.154	24242	44.077	1.07E+06
$\Sigma (Ay)p$					5.00E+07

2. SECTIONAL PROPERTIES OF COMPOSITE SECTION-MID SPAN**For Section 6.1375m TO 18.0125m From Jacking End****FOR OUTER GIRDER**

Thickness of Deck Slab = 220 mm

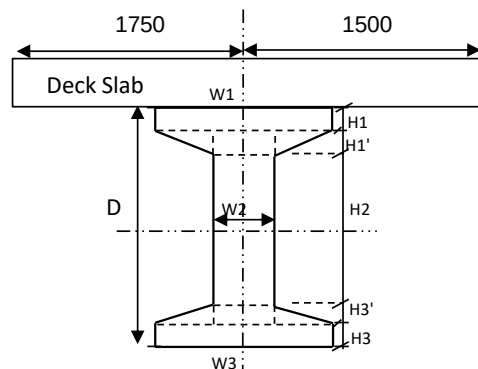
Effective Width of Flange

$$\begin{aligned} bw &= b = 3.250 \text{ m} \\ &= w2 = 275 \text{ mm} \\ &= b1 = 1612.5 \text{ mm} \\ &= b2 = 1362.5 \text{ mm} \\ &= lo = 23.75 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{effective width of cantilever on one end} &= \min (0.2 \times b1 + 0.1 \times lo , 0.2 \times lo , b1) \\ \min (2.6975 , 4.75 , 1.6125) &= 1.6125 \\ \text{beff, 1} &= 1.6125 \end{aligned}$$

$$\begin{aligned} \text{effective width of cantilever on other end} &= \min (0.2 \times b2 + 0.1 \times lo , 0.2 \times lo , b2) \\ \min (2.6475 , 4.75 , 1.3625) &= 1.363 \\ \text{beff, 2} &= 1.363 \end{aligned}$$

$$\begin{aligned} \text{Total effective width of flange beff} &= 1.613 + 1.3625 + 0.275 \\ &= 3.250 \text{ m} \end{aligned}$$



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

with $b_{eff,i} = 0.2b_i + 0.1l_o \leq 0.2l_o$

2a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
Slab	3250	220	715000	110.00	7.865E+07	8.652E+09	2.884E+09	1.154E+10
Girder			723125	1001.27	7.240E+08	7.250E+11	2.2176E+11	9.467E+11
			1438125		8.027E+08	7.336E+11	2.246E+11	9.583E+11

Total Area of Section $A = 1.438 \text{ m}^2$

c.g. of Section from Top of Slab	Yts	=	0.558 m	
c.g. of Section from Top of Girder	Ytg	=	0.338 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.262 m	
Moment of Inertia of Section	Iz	=	0.510 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.914 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.509 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.404 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6940.000 mm	
Total perimeter of composite girder		=	12612.297 mm	

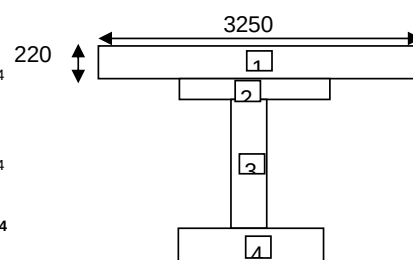
2b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
Slab	3250	220	715000	125.00	8.938E+07	1.117E+10	6.293E+11	6.405E+11
Girder			723125	0.00	0.000E+00	0.000E+00	2.5873E+10	2.587E+10
			1438125		8.938E+07	1.117E+10	6.552E+11	6.664E+11

Moment of Inertia of Section $I_{yy} = 0.666 \text{ m}^4$

2c. Calculation of Sectional Properties about X-X Axis

Moment of Inertia of Precast Girder	I_{xx}	=	1.63E+10 mm ⁴
Moment of Inertia of Deck Slab	I_{xxslab}	=	$b * d^3/6$
		=	5.77E+09 mm ⁴
Total Ixx for Composite Girder		=	2.20E+10 mm⁴

**2d. First moment of Area Above Centroid of Composite Section (Ay)c****Property of Composite Girder For Mid Span**

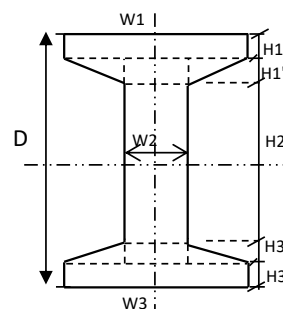
Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)c mm ³
Slab	3250	220	715000	448.154	3.20E+08
1 (Rect.)	1000	150	150000	263.154	3.95E+07
1' (Tri.)	725	100	72500	154.821	1.12E+07
1' (Rect.)	275	100	27500	138.154	3.80E+06
2 (Rect.)	275	88	24242.467	44.077	1.07E+06
			$\Sigma (Ay)c$		3.76E+08

3. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

For Section 1.85m TO 3.35m & For Section 20.8m TO 22.3 m From Jacking End

(Dimension in mm)

D	=	1600		
H1	=	150	W1	= 1000
H1'	=	71		
H2	=	1054	W2	= 488
H3'	=	75		
H3	=	250	W3	= 700
			Perimeter , u	= 5400.37



Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	8.438E+08	2.813E+08	1.125E+09
1' (Tri.)	513	71	18114	174	3.144E+06	5.457E+08	5.029E+06	5.507E+08
1' (Rect.)	488	71	34461	185	6.387E+06	1.184E+09	1.435E+07	1.198E+09
2 (Rect.)	488	1054	513976	747.84	3.844E+08	2.875E+11	4.761E+10	3.351E+11
3' (Tri.)	213	75	7969	1325	1.056E+07	1.399E+10	2.490E+06	1.399E+10
3' (Rect.)	488	75	36563	1313	4.799E+07	6.298E+10	1.714E+07	6.300E+10
3 (Rect.)	700	250	175000	1475	2.581E+08	3.807E+11	9.115E+08	3.816E+11
			936083		7.218E+08	7.48.E+11	4.884E+10	7.966E+11

Total Area of Section	A	=	0.936 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.771 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.829 m	
Moment of Inertia of Section	Iz	=	0.240 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.311 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.290 m ³	

First moment of Area Above Centroid of Composite Section (Ay)p

Property of Girder at Section L/8

Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)p mm ³
1 (Rect.)	1000	150	150000	314.549	4.72E+07
1' (Tri.)	513	71	36228.45	215.986	7.82E+06
1' (Rect.)	488	71	34461.21	204.204	7.04E+06
2 (Rect.)	488	169	82318.88	84.430	6.95E+06
			Σ (Ay)p		6.90E+07

4. SECTIONAL PROPERTIES OF COMPOSITE SECTION -VARYING PORTION

For Section 1.85m TO 3.35m & For Section 20.8m TO 22.3 m From Jacking End

FOR OUTER GIRDER

Thickness of Deck Slab = 220 mm

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
Slab	3250	220	715000	110	7.865E+07	8.652E+09	2.884E+09	1.154E+10
Girder			936083	991.11	9.278E+08	9.195E+11	2.400E+11	1.159E+12
			1651083		1.006E+09	9.282E+11	2.428E+11	1.171E+12

Total Area of Section	A	=	1.651 m ²	
c.g. of Section from Top of Slab	Yts	=	0.610 m	
c.g. of Section from Top of Girder	Ytg	=	0.390 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.210 m	
Moment of Inertia of Section	Iz	=	0.558 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.915 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.431 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.461 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6940.000 mm	
Total perimeter of composite girder		=	12340.372 mm	

First moment of Area Above Centroid of Composite Section (Ay)c

Property of Composite Girder at Section L/8

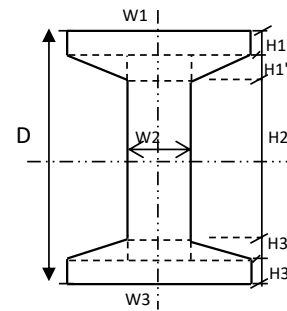
Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)c mm ³
Slab	3250	220	715000.00	499.549	3.57E+08
1 (Rect.)	1000	150	150000.00	314.549	4.72E+07
1' (Tri.)	513	71	36228.45	215.986	7.82E+06
1' (Rect.)	488	71	34461.21	204.204	7.04E+06
2 (Rect.)	488	169	82318.88	84.430	6.95E+06
			Σ (Ay)c		4.26E+08

5. SECTIONAL PROPERTIES OF GIRDERS (NEAR SUPPORT)

For Section 0m TO 0.2m & For Section 23.95m TO 24.15 m From Jacking End

(Dimension in mm)

D	=	1600			
H1	=	150	W1	=	1000
H1'	=	41			
H2	=	1159	W2	=	700
H3'	=	0.00			
H3	=	250	W3	=	700
Perimeter , u			=	5128.45	



5a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	8.438E+08	2.813E+08	1.125E+09
1' (Tri.)	300	41	6207	164	1.017E+06	1.665E+08	5.904E+05	1.671E+08
1' (Rect.)	700	41	28966	171	4.944E+06	8.439E+08	4.133E+06	8.480E+08
2 (Rect.)	700	1159	811034	770.69	6.251E+08	4.817E+11	9.073E+10	5.725E+11
3' (Tri.)	0	0	0	1350	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3' (Rect.)	700	0	0	1350	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3 (Rect.)	700	250	175000	1475	2.581E+08	3.807E+11	9.115E+08	3.816E+11
			1171207		9.004E+08	8.64.E+11	9.193E+10	9.562E+11

Total Area of Section	A	=	1.171 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.769 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.831 m	
Moment of Inertia of Section	Iz	=	0.264 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.343 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.318 m ³	

5b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
1 (Rect.)	1000	150	150000	0	0	0	1.25E+10	1.25E+10
1' (Tri.)	300	41	6206.90	400.000	2.48E+06	9.93E+08	7.76E+06	1.00E+09
1' (Rect.)	700	41	28965.52	0	0	0	1.18E+09	1.18E+09
2 (Rect.)	700	1159	811034.48	0	0	0	3.31E+10	3.31E+10
3' (Tri.)	0	0	0.00	350.000	0.00E+00	0.00E+00	0.00E+00	0.00E+00
3' (Rect.)	700	0	0.00	0	0	0	0.00E+00	0.00E+00
3 (Rect.)	700	250	175000.00	0	0	0	7.15E+09	7.15E+09
								5.49E+10

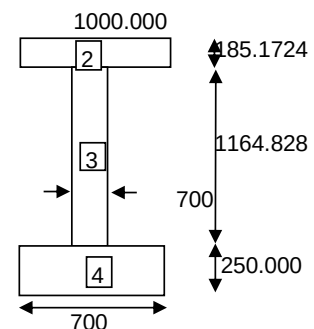
Moment of Inertia of Section Iyy = 0.0549 m⁴

5c. Calculation of Sectional Properties about X-X Axis

Average Thk. Of top flange	=	185.172 mm
Average Thk. Of bottom flange	=	250.000 mm

$$I_{xx} = \frac{3.b^3.d^3}{10(b^2+d^2)}$$

Portion no.	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
2	1000	185.172	1.84E+09
3	700	1164.828	8.81E+10
4	700	250.000	2.91E+09



Moment of Inertia of Precast Girder only Ixx = 9.28E+10 mm⁴

5d. First moment of Area Above Centroid of Composite Section (Ay)p**Property of Girder Near Support**

Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)p mm ³
1 (Rect.)	1000	150	150000	360.658	5.41E+07
1' (Tri.)	300	41	12413.79	271.865	3.37E+06
1' (Rect.)	700	41	28965.52	264.969	7.67E+06
2 (Rect.)	700	244	170995.28	122.139	2.09E+07
$\Sigma (Ay)p$					8.60E+07

6. SECTIONAL PROPERTIES OF COMPOSITE SECTION (NEAR SUPPORT)**For Section 0m TO 0.2m & For Section 23.95m TO 24.15 m From Jacking End****FOR OUTER GIRDER**

Thickness of Deck Slab = 220.00 mm
 Depth of Girder = 1600.00 mm

6a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Isself (mm ⁴)	Ip=Isself+A.Yt ² (mm ⁴)
Slab	3250	220	715000	110	7.865E+07	8.652E+09	2.884E+09	1.154E+10
Girder			1171207	988.77	1.158E+09	1.145E+12	2.640E+11	1.409E+12
			1886207		1.237E+09	1.154E+12	2.669E+11	1.421E+12

Total Area of Section A = 1.886 m²
 c.g. of Section from Top of Slab Yts = 0.656 m
 c.g. of Section from Top of Girder Ytg = 0.436 m
 c.g. of Section from Bottom of Girder Ybg = 1.164 m
 Moment of Inertia of Section Iz = 0.610 m⁴ (Ip - A Yt²)
 Section Modulus w.r.to Top of Slab Zts = 0.930 m³
 Section Modulus w.r.to Top of Girder Ztg = 1.400 m³
 Section Modulus w.r.to Bottom of Girder Zbg = 0.524 m³
 Perimeter of Deck slab excluding top flange surface,uslab = 6940.000 mm
 Total perimeter of composite girder = 12068.447 mm

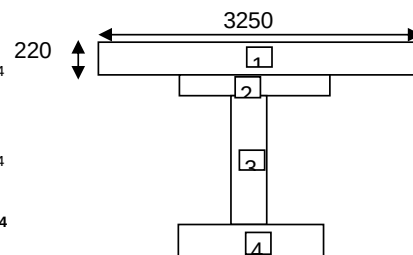
6b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Isself (mm ⁴)	Iyy=Isself+A.z ² (mm ⁴)
Slab	3250	220	715000	125.00	8.938E+07	1.117E+10	6.293E+11	6.405E+11
Girder			1171207	0.00	0.000E+00	0.000E+00	5.4947E+10	5.495E+10
			1886207		8.938E+07	1.117E+10	6.843E+11	6.955E+11

Moment of Inertia of Section Iyy = 0.695 m⁴

6c. Calculation of Sectional Properties about X-X Axis

Moment of Inertia of Precast Girder Ixx = 9.28E+10 mm⁴
 Moment of Inertia of Deck Slab Ixxslab = b * d³/6 = 5.77E+09 mm⁴
Total Ixx for Composite Girder = 9.86E+10 mm⁴



6d. First moment of Area Above Centroid of Composite Section (Ay)c**Property of Composite Girder Near Support**

Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)c mm ³
Slab	3250	220	715000	545.658	3.90E+08
1 (Rect.)	1000	150	150000	360.658	5.41E+07
1' (Tri.)	300	41	12413.79	271.865	3.37E+06
1' (Rect.)	700	41	28965.52	264.969	7.67E+06
2 (Rect.)	700	244	170995.28	122.139	2.09E+07
$\Sigma (Ay)c$					4.76E+08

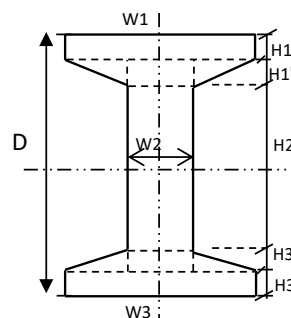
SECTIONAL PROPERTIES OF INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

For Section 6.1375m TO 18.0125m From Jacking End

(Dimension in mm)

D	=	1600			
H1	=	150	W1	=	1000
H1'	=	100			
H2	=	950	W2	=	275
H3'	=	150			
H3	=	250	W3	=	700
Perimeter , u			=	5672.30	



1a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	843750000	2.813E+08	1.125E+09
1' (Tri.)	725	100	36250	183	6.646E+06	1218402778	2.014E+07	1.239E+09
1' (Rect.)	275	100	27500	200	5.500E+06	1100000000	2.292E+07	1.123E+09
2 (Rect.)	275	950	261250	725	1.894E+08	1.3732E+11	1.965E+10	1.570E+11
3' (Tri.)	425	150	31875	1300	4.144E+07	53868750000	3.984E+07	5.391E+10
3' (Rect.)	275	150	41250	1275	5.259E+07	67057031250	7.734E+07	6.713E+10
3 (Rect.)	700	250	175000	1475	2.581E+08	3.80734E+11	9.115E+08	3.816E+11
			723125		5.650E+08	6.42.E+11	2.100E+10	6.631E+11

Total Area of Section	A	=	0.723 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.781 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.819 m	
Moment of Inertia of Section	Iz	=	0.222 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.284 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.271 m ³	

1b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
1 (Rect.)	1000	150	150000	0	0	0	1.25E+10	1.25E+10
1' (Tri.)	725	100	36250	258.333	9.36E+06	2.42E+09	2.65E+08	2.68E+09
1' (Rect.)	275	100	27500	0	0	0	1.73E+08	1.73E+08
2 (Rect.)	275	950	261250	0	0	0	1.65E+09	1.65E+09
3' (Tri.)	425	150	31875	208.333	6.64E+06	1.38E+09	8.00E+07	1.46E+09
3' (Rect.)	275	150	41250	0	0	0	2.60E+08	2.60E+08
3 (Rect.)	700	250	175000	0	0	0	7.15E+09	7.15E+09
								2.59E+10

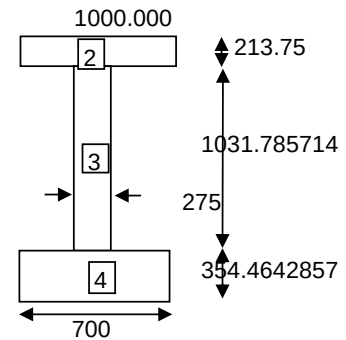
Moment of Inertia of Section	Iyy	=	0.026 m ⁴

1c. Calculation of Sectional Properties about X-X Axis

Average Thk. Of top flange = 213.75
 Average Thk. Of bottom flange = 354.46429

$$I_{xx} = \frac{3.b^3.d^3}{10(b^2+d^2)}$$

Portion no.	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
2	1000	213.75	2.80E+09
3	275	1031.786	6.01E+09
4	700	354.4643	7.44E+09



Moment of Inertia of Precast Girder only $I_{xx} = 1.63E+10 \text{ mm}^4$

2. SECTIONAL PROPERTIES OF COMPOSITE SECTION-MID SPAN

For Section 6.1375m TO 18.0125m From Jacking End

FOR INNER GIRDER

Thickness of Deck Slab = 220 mm

Effective Width of Flange

is least of the following

a) Inner Girder [Cl. 305.15.2 IRC 21]

- i) $L_o/5 + W_2 = 5.03 \text{ m}$
 ii) Contributory width of deck slab = 3.00 m

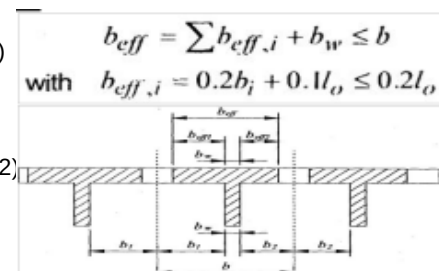
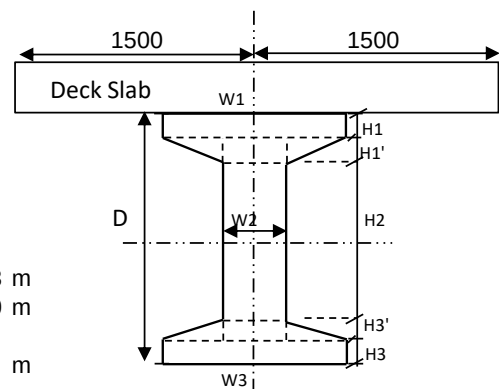
Governing width of flange = m

bw	=	b	=	3.000 m
	=	w2	=	275 mm
	=	b1	=	1362.5 mm
	=	b2	=	1362.5 mm
	=	lo	=	23.75 m

effective width of cantilever on one end = $\min (0.2 \times b1 + 0.1 \times lo, 0.2 \times lo, b1)$
 $\min (2.6475, 4.75, 1.3625) = 1.3625$
 $b_{eff, 1} = 1.3625$

effective width of cantilever on other end = $\min (0.2 \times b2 + 0.1 \times lo, 0.2 \times lo, b2)$
 $\min (2.6475, 4.75, 1.3625) = 1.363$
 $b_{eff, 2} = 1.363$

Total effective width of flange $b_{eff} = 1.363 + 1.3625 + 0.275 = 3.00 \text{ m}$



2a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
Slab	3000	220	660000	110.00	7.260E+07	7.986E+09	2.662E+09	1.065E+10
Girder			723125	1001.27	7.240E+08	7.250E+11	2.2176E+11	9.467E+11
			1383125		7.966E+08	7.330E+11	2.244E+11	9.574E+11

Total Area of Section	A	=	1.383 m ²	
c.g. of Section from Top of Slab	Yts	=	0.576 m	
c.g. of Section from Top of Girder	Ytg	=	0.356 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.244 m	
Moment of Inertia of Section	Iz	=	0.499 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.866 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.400 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.401 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6440.000 mm	
Total perimeter of composite girder		=	12112.297 mm	

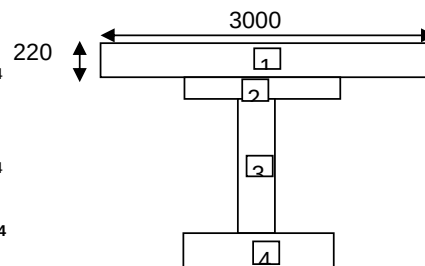
2b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
Slab	3000	220	660000	0.00	0.000E+00	0.000E+00	4.950E+11	4.950E+11
Girder			723125	0.00	0.000E+00	0.000E+00	2.5873E+10	2.587E+10
			1383125		0.000E+00	0.000E+00	5.209E+11	5.209E+11

Moment of Inertia of Section	Iyy	=	0.521 m ⁴

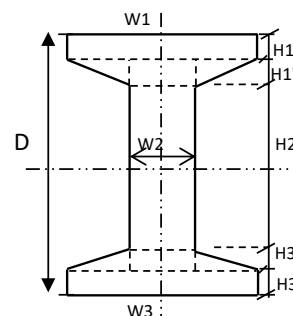
2c. Calculation of Sectional Properties about X-X Axis

Moment of Inertia of Precast Girde Ixx	=	1.63E+10 mm ⁴
Moment of Inertia of Deck Slab Ixxslab	=	b * d ³ /6
	=	5.32E+09 mm ⁴
Total Ixx for Composite Girder	=	2.16E+10 mm ⁴

**3. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION****For Section 1.85m TO 3.35m & For Section 20.8m TO 22.3 m From Jacking End**

(Dimension in mm)

D	=	1600		
H1	=	150	W1	= 1000
H1'	=	71		
H2	=	1054	W2	= 488
H3'	=	75		
H3	=	250	W3	= 700



Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	8.438E+08	2.813E+08	1.125E+09
1' (Tri.)	513	71	18114	174	3.144E+06	5.457E+08	5.029E+06	5.507E+08
1' (Rect.)	488	71	34461	185	6.387E+06	1.184E+09	1.435E+07	1.198E+09
2 (Rect.)	488	1054	513976	747.84	3.844E+08	2.875E+11	4.761E+10	3.351E+11
3' (Tri.)	213	75	7969	1325	1.056E+07	1.399E+10	2.490E+06	1.399E+10
3' (Rect.)	488	75	36563	1313	4.799E+07	6.298E+10	1.714E+07	6.300E+10
3 (Rect.)	700	250	175000	1475	2.581E+08	3.807E+11	9.115E+08	3.816E+11
			936083		7.218E+08	7.48.E+11	4.884E+10	7.966E+11

Total Area of Section	A	=	0.936 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.771 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.829 m	
Moment of Inertia of Section	Iz	=	0.240 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.311 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.290 m ³	

4. SECTIONAL PROPERTIES OF COMPOSITE SECTION -VARYING PORTION

For Section 1.85m TO 3.35m & For Section 20.8m TO 22.3 m From Jacking End

FOR INNER GIRDER

Thickness of Deck Slab = 220 mm

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	I _p =Iself+A.Yt ² (mm ⁴)
Slab	3000	220	660000	110	7.260E+07	7.986E+09	2.662E+09	1.065E+10
Girder			936083	991.11	9.278E+08	9.195E+11	2.400E+11	1.159E+12
			1596083		1.000E+09	9.275E+11	2.426E+11	1.170E+12

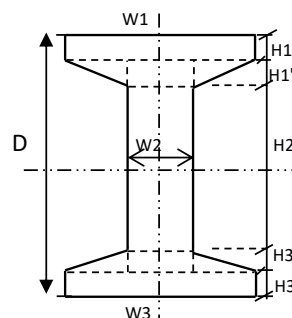
Total Area of Section	A	=	1.596 m ²	
c.g. of Section from Top of Slab	Yts	=	0.627 m	
c.g. of Section from Top of Girder	Ytg	=	0.407 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.193 m	
Moment of Inertia of Section	Iz	=	0.543 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.867 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.335 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.455 m ³	

5. SECTIONAL PROPERTIES OF GIRDERS (NEAR SUPPORT)

For Section 0m TO 0.2m & For Section 23.95m TO 24.15 m From Jacking End

(Dimension in mm)

D	=	1600			
H1	=	150	W1	=	1000
H1'	=	41			
H2	=	1159	W2	=	700
H3'	=	0			
H3	=	250	W3	=	700
Perimeter , u			=	5128.45	



5a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	I _p =Iself+A.Yt ² (mm ⁴)
1 (Rect.)	1000	150	150000	75	1.125E+07	8.438E+08	2.813E+08	1.125E+09
1' (Tri.)	300	41	6207	164	1.017E+06	1.665E+08	5.904E+05	1.671E+08
1' (Rect.)	700	41	28966	171	4.944E+06	8.439E+08	4.133E+06	8.480E+08
2 (Rect.)	700	1159	811034	770.69	6.251E+08	4.817E+11	9.073E+10	5.725E+11
3' (Tri.)	0	0	0	1350	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3' (Rect.)	700	0	0	1350	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3 (Rect.)	700	250	175000	1475	2.581E+08	3.807E+11	9.115E+08	3.816E+11
			1171207		9.004E+08	8.64.E+11	9.193E+10	9.562E+11

Total Area of Section	A	=	1.171 m ²	
c.g. of Section from Top of Girder	Ytg	=	0.769 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.831 m	
Moment of Inertia of Section	Iz	=	0.264 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Girder	Ztg	=	0.343 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.318 m ³	

1b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
1 (Rect.)	1000	150	150000	0	0	0	1.25E+10	1.25E+10
1' (Tri.)	300	41	6206.897	400.000	2.48E+06	9.93E+08	7.76E+06	1.00E+09
1' (Rect.)	700	41	28965.52	0	0	0	1.18E+09	1.18E+09
2 (Rect.)	700	1159	811034.5	0	0	0	3.31E+10	3.31E+10
3' (Tri.)	0	0	0.00	350.000	0.00E+00	0.00E+00	0.00E+00	0.00E+00
3' (Rect.)	700	0	0.00	0	0	0	0.00E+00	0.00E+00
3 (Rect.)	700	250	175000	0	0	0	7.15E+09	7.15E+09
								5.49E+10

Moment of Inertia of Section $I_{yy} = 0.0549 \text{ m}^4$

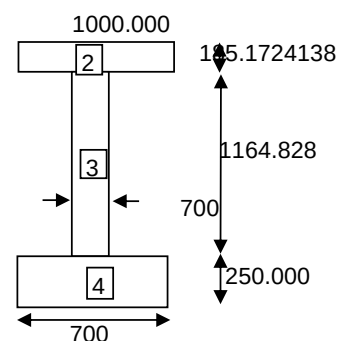
5c. Calculation of Sectional Properties about X-X Axis

Average Thk. Of top flange = 185.172 mm

Average Thk. Of bottom flange = 250.000 mm

$$I_{xx} = \frac{3.b^3.d^3}{10(b^2+d^2)}$$

Portion no.	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
2	1000	185.172	1.84E+09
3	700	1164.828	8.81E+10
4	700	250.000	2.91E+09



Moment of Inertia of Precast Girder only $I_{xx} = 9.28E+10 \text{ mm}^4$

6. SECTIONAL PROPERTIES OF COMPOSITE SECTION (NEAR SUPPORT)**For Section 0m TO 0.2m & For Section 23.95m TO 24.15 m From Jacking End**

FOR INNER GIRDER

Thickness of Deck Slab = 220.00 mm

Depth of Girder = 1600.00 mm

6a. Calculation of Sectional Properties about Major Axis Z-Z

Element	Width (mm)	Height (mm)	A (mm ²)	Yt (mm)	A.Yt (mm ³)	A.Yt ² (mm ⁴)	Iself (mm ⁴)	Ip=Iself+A.Yt ² (mm ⁴)
Slab	3000	220	660000	110	7.260E+07	7.986E+09	2.662E+09	1.065E+10
Girder			1171207	988.77	1.158E+09	1.145E+12	2.640E+11	1.409E+12
			1831207		1.231E+09	1.153E+12	2.667E+11	1.420E+12

Total Area of Section A = 1.831 m²

c.g. of Section from Top of Slab Yts = 0.672 m

c.g. of Section from Top of Girder Ytg = 0.452 m

c.g. of Section from Bottom of Girder Ybg = 1.148 m

Moment of Inertia of Section Iz = 0.593 m⁴ (Ip - A Yt²)Section Modulus w.r.to Top of Slab Zts = 0.882 m³Section Modulus w.r.to Top of Girder Ztg = 1.311 m³Section Modulus w.r.to Bottom of Girder Zbg = 0.516 m³

Perimeter of Deck slab excluding top flange surface, uslab = 6440.000 mm

Total perimeter of composite girder = 11568.447 mm

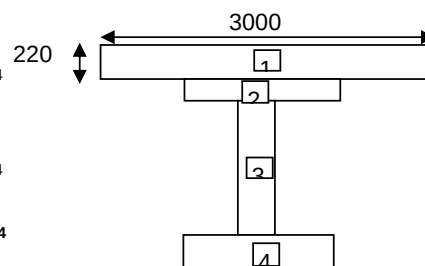
6b. Calculation of Sectional Properties about Minor Axis Y-Y

Element	Width (mm)	Height (mm)	A (mm ²)	z (mm)	A.z (mm ³)	A.z ² (mm ⁴)	Iself (mm ⁴)	Iyy=Iself+A.z ² (mm ⁴)
Slab	3000	220	660000	0.00	0.000E+00	0.000E+00	4.950E+11	4.950E+11
Girder			1171207	0.00	0.000E+00	0.000E+00	5.4947E+10	5.495E+10
			1831207		0.000E+00	0.000E+00	5.499E+11	5.499E+11

Moment of Inertia of Section Iyy = 0.550 m⁴**6c. Calculation of Sectional Properties about X-X Axis**Moment of Inertia of Precast Girde Ixx = 9.28E+10 mm⁴

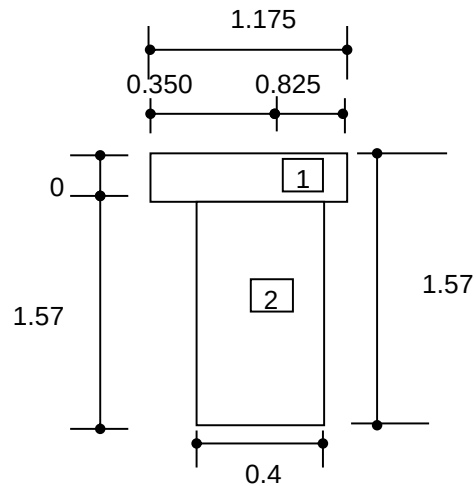
Moment of Inertia of Deck Slab Ixxslab = $b * d^3/6$

= 5.32E+09 mm⁴

Total Ixx for Composite Girder = 9.81E+10 mm⁴

SECTIONAL PROPERTIES OF END DIAPHRAGM

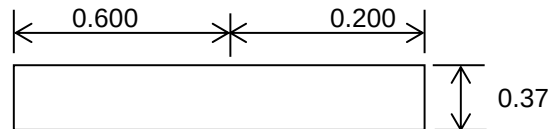
$$\begin{aligned}
 & \boxed{I_z} \\
 \text{Area (Ax)} &= \begin{array}{rcl} 1.175 & \times & 0 \\ + & 1.57 & \times & 0.4 \end{array} \\
 &= 0.628 \text{ m}^2 \\
 y_b &= 0.785 \text{ m} \\
 y_t &= 0.785 \text{ m}
 \end{aligned}$$



Element	$I_{self} = bd^3/12$ m^4	A_y^2 m^4
1	0.000E+00	0
2	1.290E-01	7.7407E-33
	1.290E-01	7.7407E-33

$$\boxed{I_z = 1.290E-01 \text{ m}^4}$$

I_y			I_x			
$y_l = 0.3500 \text{ m}$			Element	b m	d m	$\frac{3 \cdot b^3 \cdot d^3}{10 \cdot (b^2 + d^2)}$
$y_r = 0.825 \text{ m}$			1	1.175	0	0.000E+00
			2	0.4	1.57	2.831E-02
Element	$I_{self} = bd^3/12$ m^4	A_y^2 m^4	$\boxed{I_x = 2.83E-02 \text{ m}^4}$			
1	0.000E+00	0.00E+00				
2	8.373E-03	0.00E+00				
	8.373E-03	0.00E+00				
$I_y = 8.373E-03 \text{ m}^4$						

SECTIONAL PROPERTIES OF CROSS MEMBER OF DECK SLAB**1)**

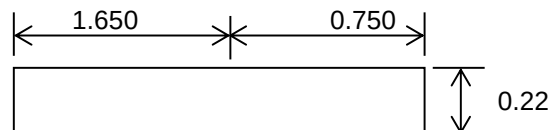
Area Ax =

$$0.800 \quad \times \quad 0.37 \quad = \quad 0.296 \text{ m}^2$$

$$I_x = \frac{0.8 \times 0.37^3}{6} = 0.006754 \text{ m}^4$$

$$I_y = \frac{0.37 \times 0.8^3}{12} = 0.015787 \text{ m}^4$$

$$I_z = \frac{0.8 \times 0.37^3}{12} = 0.003377 \text{ m}^4$$

2)

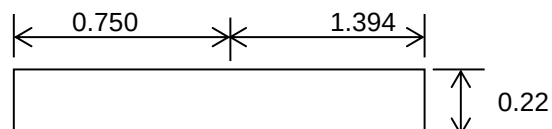
Area Ax =

$$2.400 \quad \times \quad 0.22 \quad = \quad 0.528 \text{ m}^2$$

$$I_x = \frac{2.4 \times 0.22^3}{6} = 0.004259 \text{ m}^4$$

$$I_y = \frac{0.22 \times 2.4^3}{12} = 0.253440 \text{ m}^4$$

$$I_z = \frac{2.4 \times 0.22^3}{12} = 0.002130 \text{ m}^4$$

3)

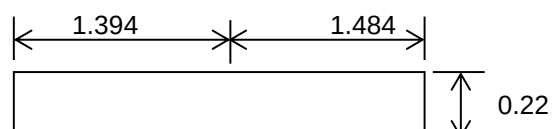
Area Ax =

$$2.144 \quad \times \quad 0.22 \quad = \quad 0.47168 \text{ m}^2$$

$$I_x = \frac{2.144 \times 0.22^3}{6} = 0.003805 \text{ m}^4$$

$$I_y = \frac{0.22 \times 2.144^3}{12} = 0.180682 \text{ m}^4$$

$$I_z = \frac{2.144 \times 0.22^3}{12} = 0.001902 \text{ m}^4$$

4)

Area Ax =

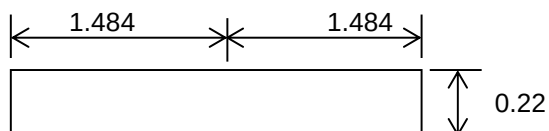
$$2.878 \quad \times \quad 0.22 \quad = \quad 0.63316 \text{ m}^2$$

$$I_x = \frac{2.878}{6} \times \frac{0.22^3}{12} = 0.005107 \text{ m}^4$$

$$I_y = \frac{0.22}{12} \times \frac{2.878^3}{6} = 0.437033 \text{ m}^4$$

$$I_z = \frac{2.878}{12} \times \frac{0.22^3}{6} = 0.002554 \text{ m}^4$$

5)



Area Ax =

$$2.968 \quad \times \quad 0.22 \quad = \quad 0.65296 \text{ m}^2$$

$$I_x = \frac{2.968}{6} \times \frac{0.22^3}{12} = 0.005267 \text{ m}^4$$

$$I_y = \frac{0.22}{12} \times \frac{2.968^3}{6} = 0.479328 \text{ m}^4$$

$$I_z = \frac{2.968}{12} \times \frac{0.22^3}{6} = 0.002634 \text{ m}^4$$

Superimposed Dead Load (SIDL)

The SIDL is applied on the continuous unit and it comprises of following items:-

- (i) RCC Crash Barriers
- (ii) RCC Railing
- (iii) Wearing coat

RCC Crash Barrier

Cross sectional area	=	0.3068 m ²	
Weight of crash barrier	=	0.3068 x 25	
	=	7.670 kN/m	
		say	8.00 kN/m

Wearing coat

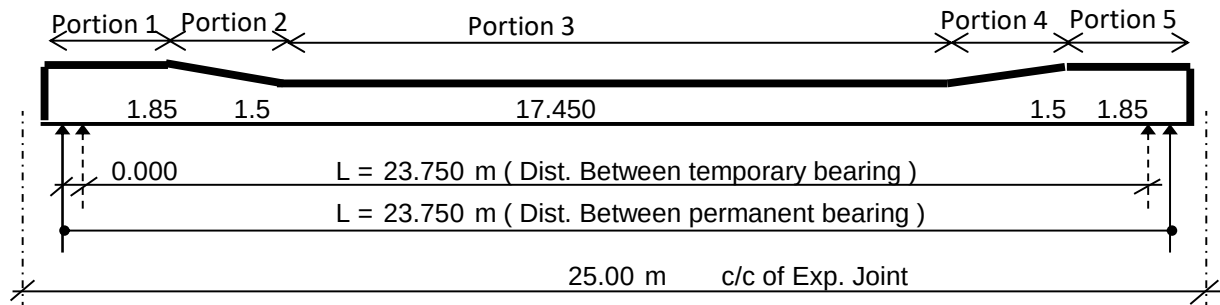
Thickness of wearing coat	=	0.065 m
Load Intensity	=	22 x 0.065
	=	1.43 kN/m ²

However, for design purpose, the wearing coat load intensity is taken as 2.2 kN/m² to account for future overlaying.

Therefore Wearing coat load intensity =	2.2	kN/m ²
UDL over outer Girder	=	6.325 kN/m
UDL over Inner Girder	=	6.6 kN/m

SECTIONS CONSIDERED IN DESIGN

Length of Girder	=	24.450	m
Pocket depth at anchorage end	=	0.150	m
Distance between anchorage end	=	24.150	m
Effective depth deff	=	1.700	m



section		Distance from jacking end	Section falls in portion of
Jacking End	-0.200	0.00	Portion 1
Left bearing	0.000 L	0.20	Portion 1
0.069 L	0.069 L	1.85	Portion 2
0.133 L	0.133 L	3.35	Portion 3
2/8L	0.250 L	6.14	Portion 3
3/8L	0.375 L	9.11	Portion 3
4/8L	0.500 L	12.08	Portion 3
5/8L	0.625 L	15.04	Portion 3
6/8L	0.750 L	18.01	Portion 3
0.867 L	0.867 L	20.80	Portion 4
0.931 L	0.931 L	22.30	Portion 5
Right bearing	1.000 L	23.95	Portion 5
Dead end		24.15	Portion 5

CALCULATION OF MOMENTS AND SHEARS

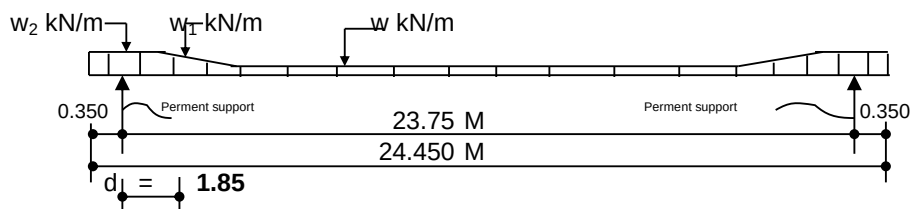
(FOR INTERMEDIATE GIRDERS)

Analysis for Self Wt. of Girder is done by taking it as an individual girder while analysis for SIDL, FPLL and LL are done by GRID analysis using STAAD-Pro.

1.0 Self weight of Inner Girder

Length of the girder = 24.450 m

2.00 1.500



Distance Between Support, L = **23.75 m**
Overhang = **0.350 m**

UDL , w = 18.08 kN/m
Avg. , w_1 = 23.68 kN/m
UDL , w_2 = 29.28 kN/m
Supp. Reaction , R = **251.81 KN**

a) Moment at	x =	-0.200 m	Jacking End
=	0.00	kNm	Corresp. Shear = 241.56 kN
b) Moment at	x =	0.000 L (sec. 6-6)	
=	-1.79	kNm	Corresp. Shear = 241.56 kN
c) Moment at	x =	0.069 L	
=	356.93	kNm	Corresp. Shear = 193.25 kN
d) Moment at	x =	0.133 L	
=	620.16	kNm	Corresp. Shear = 157.73 kN
e) Moment at	x =	0.250 L	
=	989.61	kNm	Corresp. Shear = 107.34 kN
f) Moment at	x =	0.375 L	
=	1228.60	kNm	Corresp. Shear = 53.67 kN
g) Moment at	x =	0.500 L	
=	1308.27	kNm	Corresp. Shear = 0.00 kN
h) Moment at	x =	0.625 L	
=	1228.60	kNm	Corresp. Shear = 53.67 kN
i) Moment at	x =	0.750 L	
=	989.61	kNm	Corresp. Shear = 107.34 kN

j) Moment at	x =	0.867 L		
=	620.16	kNm	Corresp. Shear =	157.73 kN
k) Moment at	x =	0.931 L		
=	356.93	kNm	Corresp. Shear =	193.25 kN
L) Moment at	x =	1.000 L		
=	-1.79	kNm	Corresp. Shear =	241.56 kN
M) Moment at	x =	Dead end		
=	0.00	kNm	Corresp. Shear =	241.56 kN

2.0 Green weight of cast-in-situ deck slab

Effective Width of Slab	=	3000 mm
Thickness of Slab	=	220 mm
Profile correction course thickness for $c\epsilon$	=	118.75 mm
Avg. Thickness of Wedge over girder	=	12.50 mm
Weight of green concrete	=	26 kN/m ³
Temporary Live Load on Deck Slab	=	2 kN/m ²

UDL due to deck slab weight	=	17.16 kN/m
UDL due to additional weight	=	10.238 kN/m
UDL due to Temp. Live load	=	6 kN/m
		33.398 kN/m

UDL , w = 33.398 kN/m (On entire length of girder)

Avg. , w₁ = 33.398 kN/m

UDL , w₂ = 33.398 kN/m

Supp. Reaction , R = **408.284 kN**

a) Moment at	x =	-0.200 m		
=	0.00	kNm	Corresp. Shear =	396.60 kN
b) Moment at	x =	0.000 L		
=	-2.05	kNm	Corresp. Shear =	396.60 kN
c) Moment at	x =	0.069 L		
=	606.87	kNm	Corresp. Shear =	341.49 kN
d) Moment at	x =	0.133 L		
=	1081.54	kNm	Corresp. Shear =	291.39 kN
e) Moment at	x =	0.250 L		
=	1764.04	kNm	Corresp. Shear =	198.30 kN
f) Moment at	x =	0.375 L		
=	2205.57	kNm	Corresp. Shear =	99.15 kN

g) Moment at	x =	0.500 L		
=	2352.74 kNm		Corresp. Shear =	0.00 kN
h) Moment at	x =	0.625 L		
=	2205.57 kNm		Corresp. Shear =	99.15 kN
i) Moment at	x =	0.750 L		
=	1764.04 kNm		Corresp. Shear =	198.30 kN
j) Moment at	x =	0.867 L		
=	1081.54 kNm		Corresp. Shear =	291.39 kN
k) Moment at	x =	0.931 L		
=	606.87 kNm		Corresp. Shear =	341.49 kN
L) Moment at	x =	1.000 L		
=	-2.05 kNm		Corresp. Shear =	396.60 kN
M) Moment at	x =	Dead end L		
=	0.00 kNm		Corresp. Shear =	396.60 kN

3.0 Shuttering Laod

As per clause 4.2.2.2.2 of IRC:87-1984 superimposed loads

$$= 1.5 \text{ kN/m}^2 = 0.45 \text{ kN/m}$$

$$\text{UDL , } w = 0.450 \text{ kN/m (On entire length of girder)}$$

$$\text{Avg. , } w_1 = 0.450$$

$$\text{UDL , } w_2 = 0.450 \text{ kN/m}$$

$$\text{Supp. Reaction , } R = 5.501 \text{ kN}$$

a) Moment at	x =	-0.200 m		
=	0.00 kNm		Corresp. Shear =	5.34 kN
b) Moment at	x =	0.000 L		
=	-0.03 kNm		Corresp. Shear =	5.34 kN
c) Moment at	x =	0.069 L		
=	8.18 kNm		Corresp. Shear =	4.60 kN
d) Moment at	x =	0.133 L		
=	14.57 kNm		Corresp. Shear =	3.93 kN
e) Moment at	x =	0.250 L		
=	23.77 kNm		Corresp. Shear =	2.67 kN
f) Moment at	x =	0.375 L		

=	29.72	kNm		Corresp. Shear =	1.34	kN
g) Moment at	x	=	0.500	L		
=	31.70	kNm		Corresp. Shear =	0.00	kN
h) Moment at	x	=	0.625	L		
=	29.72	kNm		Corresp. Shear =	1.34	kN
i) Moment at	x	=	0.750	L		
=	23.77	kNm		Corresp. Shear =	2.67	kN
j) Moment at	x	=	0.867	L		
=	14.57	kNm		Corresp. Shear =	3.93	kN
k) Moment at	x	=	0.931	L		
=	8.18	kNm		Corresp. Shear =	4.60	kN
L) Moment at	x	=	1.000	L		
=	-0.03	kNm		Corresp. Shear =	5.34	kN
M) Moment at	x	=	Dead end	L		
=	0.00	kNm		Corresp. Shear =	5.34	kN

CALCULATION OF MOMENTS AND SHEARS

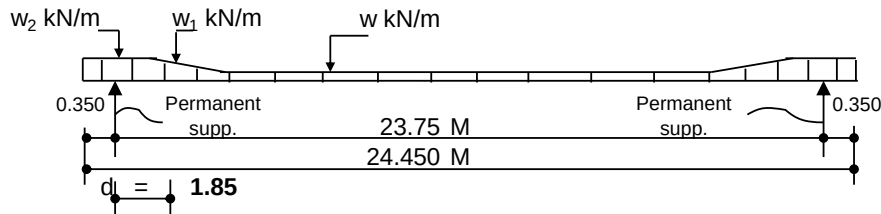
(FOR OUTER GIRDERS)

Analysis for Self Wt. of Girder is done by taking it as an individual girder while analysis for SIDL, FPLL and LL are done by GRID analysis using STAAD-Pro.

1.0 Self Weight of Outer Girder

Length of the girder = 24.450 m

2.000 1.50



Distance Between Temp. Support, L = 23.75 m
Overhang = 0.350 m

UDL , w = 18.08 kN/m
Avg. , w₁ = 23.68 kN/m
UDL , w₂ = 29.28 kN/m
Supp. Reaction , R = 251.81 kN

a) Moment at	x =	-0.200 m		
=	0.00	kNm	Corresp. Shear =	241.56 kN
b) Moment at	x =	0.000 L		
=	-1.79	kNm	Corresp. Shear =	241.56 kN
c) Moment at	x =	0.069 L		
=	356.93	kNm	Corresp. Shear =	193.25 kN
d) Moment at	x =	0.133 L		
=	620.16	kNm	Corresp. Shear =	157.73 kN
e) Moment at	x =	0.250 L		
=	989.61	kNm	Corresp. Shear =	107.34 kN
f) Moment at	x =	0.375 L		
=	1228.60	kNm	Corresp. Shear =	53.67 kN
g) Moment at	x =	0.500 L		
=	1308.27	kNm	Corresp. Shear =	0.00 kN
h) Moment at	x =	0.625 L		
=	1228.60	kNm	Corresp. Shear =	53.67 kN
i) Moment at	x =	0.750 L		
=	989.61	kNm	Corresp. Shear =	107.34 kN
j) Moment at	x =	0.867 L		

=	620.16	kNm	Corresp. Shear =	157.73	kN
k) Moment at	x	=	0.931	L	
=	356.93	kNm	Corresp. Shear =	193.25	kN
L) Moment at	x	=	1.000	L	
=	-1.79	kNm	Corresp. Shear =	241.56	kN
M) Moment at	x	=	Dead end		
=	0.00	kNm	Corresp. Shear =	241.56	kN

2.0 Green weight of cast-in-situ deck slab

Effective Width of Slab	=	3250	mm
Thickness of Slab	=	220	mm
Extra average slab thickness for camber	=	118.75	mm
Avg. Thickness of Wedge over girder	=	12.50	mm
Weight of green concrete	=	26	kN/m ³
Temporary Live Load on Deck Slab	=	2	kN/m ²
UDL due to deck slab weight	=	18.59	kN/m
UDL due to additional weight	=	11.091	kN/m
UDL due to Temp. Live load	=	<u>6.5</u>	kN/m
		36.181	kN/m

UDL , w = 36.181 kN/m (On entire length of girder)

Avg. , w₁ = 36.181 kN/m (For avg. thickness of web)

UDL , w₂ = 36.181 kN/m

Supp. Reaction , R = **442.308** kN

a) Moment at	x	=	-0.200	m	
=	0.00	kNm	Corresp. Shear =	429.64	kN
b) Moment at	x	=	0.000	L	
=	-2.22	kNm	Corresp. Shear =	429.64	kN
c) Moment at	x	=	0.069	L	
=	657.45	kNm	Corresp. Shear =	369.95	kN
d) Moment at	x	=	0.133	L	
=	1171.66	kNm	Corresp. Shear =	315.68	kN
e) Moment at	x	=	0.250	L	
=	1911.05	kNm	Corresp. Shear =	214.82	kN
f) Moment at	x	=	0.375	L	
=	2389.36	kNm	Corresp. Shear =	107.41	kN
g) Moment at	x	=	0.500	L	
=	2548.80	kNm	Corresp. Shear =	0.00	kN

h) Moment at	x =	0.625 L		
=	2389.36	kNm	Corresp. Shear =	107.41 kN
i) Moment at	x =	0.750 L		
=	1911.05	kNm	Corresp. Shear =	214.82 kN
j) Moment at	x =	0.867 L		
=	1171.66	kNm	Corresp. Shear =	315.68 kN
k) Moment at	x =	0.931 L		
=	657.45	kNm	Corresp. Shear =	369.95 kN
L) Moment at	x =	1.000 L		
=	-2.22	kNm	Corresp. Shear =	429.64 kN
M) Moment at	x =	Dead end		
=	0.00	kNm	Corresp. Shear =	429.64 kN

3.0 Shuttering Laod

As per clause 4.2.2.2.2 of IRC:87-1984 superimposed loads

$$= 1.5 \text{ kN/m}^2 = 0.45 \text{ kN/m of girder}$$

UDL , w = 0.450 kN/m (On entire length of girder)

Supp. Reaction , R = **5.5013 kN**

a) Moment at	x =	-0.200 m		
=	0.00	kNm	Corresp. Shear =	5.34 kN
b) Moment at	x =	0.000 L		
=	-0.03	kNm	Corresp. Shear =	5.34 kN
c) Moment at	x =	0.069 L		
=	8.18	kNm	Corresp. Shear =	4.60 kN
d) Moment at	x =	0.133 L		
=	14.57	kNm	Corresp. Shear =	3.93 kN
e) Moment at	x =	0.250 L		
=	23.77	kNm	Corresp. Shear =	2.67 kN
f) Moment at	x =	0.375 L		
=	29.72	kNm	Corresp. Shear =	1.34 kN
g) Moment at	x =	0.500 L		
=	31.70	kNm	Corresp. Shear =	0.00 kN
h) Moment at	x =	0.625 L		

= 29.72 kNm Corresp. Shear = 1.34 kN

i) Moment at x = **0.750 L**

= 23.77 kNm Corresp. Shear = 2.67 kN

j) Moment at x = **0.867 L**

= 14.57 kNm Corresp. Shear = 3.93 kN

k) Moment at x = **0.931 L**

= 8.18 kNm Corresp. Shear = 4.60 kN

L) Moment at x = **1.000 L**

= -0.03 kNm Corresp. Shear = 5.34 kN

M) Moment at x = **Dead end**

= 0.00 kNm Corresp. Shear = 5.34 kN

SUMMARY OF BENDING MOMENT AND SHEAR FORCE

LONGITUDINAL INNER GIRDER :

Summary of Shear Force (S.F.) & Bending Moment (B.M.) at Different Section for Long. Inner Girder

SEC.	Deck Slab + Diaphragm +Temp. LL		S.W. of Girder		Construction Live Load		Tor. Due to S.W. Girder+ Deck slab
	S.F.	B.M.	S.F.	B.M.	S.F.	B.M.	
	kN	kNm	kN	kNm	kN	kNm	kNm
Jacking End	396.60	0.00	241.56	0.00	5.34	0.00	0.00
0.000 L	396.60	-2.05	241.56	-1.79	5.34	-0.03	0.00
0.069 L	341.49	606.87	193.25	356.93	4.60	8.18	0.00
0.133 L	291.39	1081.54	157.73	620.16	3.93	14.57	0.00
0.250 L	198.30	1764.04	107.34	989.61	2.67	23.77	0.00
0.375 L	99.15	2205.57	53.67	1228.60	1.34	29.72	0.00
0.500 L	0.00	2352.74	0.00	1308.27	0.00	31.70	0.00
0.625 L	99.15	2205.57	53.67	1228.60	1.34	29.72	0.00
0.750 L	198.30	1764.04	107.34	989.61	2.67	23.77	0.00
0.867 L	291.39	1081.54	157.73	620.16	3.93	14.57	0.00
0.931 L	341.49	606.87	193.25	356.93	4.60	8.18	0.00
1.000 L	396.60	-2.05	241.56	-1.79	5.34	-0.03	0.00
Dead end	396.60	0.00	241.56	0.00	5.34	0.00	0.00

Summary of S.F. & B.M at Different Section for Long. Inner Girder

SEC.	Effect of SIDL 1 (SIDL without Surfacing)			Effect of SIDL 2 (Surfacing)		
	S.F.	B.M.	Torsion	S.F.	B.M.	Torsion
	kN	kNm	kNm	kN	kNm	kNm
Jacking End	0.00	0.00	0.00	1.2	0.3	0.00
0.000 L	17.29	0.00	49.09	82.8	0.8	0.00
0.069 L	17.29	56.32	49.09	82.8	146.8	0.00
0.133 L	13.33	79.40	38.47	72.9	257.8	0.00
0.250 L	5.02	103.01	17.73	52.0	486.7	0.00
0.375 L	3.34	119.15	9.94	29.0	606.7	0.00
0.500 L	1.35	126.14	3.44	6.6	646.8	0.00
0.625 L	3.34	119.17	9.92	28.9	606.9	0.00
0.750 L	5.01	103.04	17.71	52.0	487.2	0.00
0.867 L	13.33	79.38	38.46	72.9	257.8	0.00
0.931 L	17.28	56.32	49.08	82.8	146.8	0.00
1.000 L	17.28	0.00	49.08	82.8	0.8	0.00
Dead end	0.00	0.00	0.00	1.2	0.3	0.00

Summary of S.F. & B.M at Different Section for Long. Inner Girder

SEC.	Effect of Shifting of Support			Due to Footpath Live load		
	S.F.	B.M.	Torsion	S.F.	B.M.	Torsion
	kN	kNm	kNm	kN	kNm	kNm
Jacking End	0.00	0.00	0.00	0.00	0.00	0.00
0.000 L	0.00	0.00	0.00	0.00	0.00	0.00
0.069 L	0.00	0.00	0.00	0.00	0.00	0.00
0.133 L	0.00	0.00	0.00	0.00	0.00	0.00
0.250 L	0.00	0.00	0.00	0.00	0.00	0.00
0.375 L	0.00	0.00	0.00	0.00	0.00	0.00
0.500 L	0.00	0.00	0.00	0.00	0.00	0.00
0.625 L	0.00	0.00	0.00	0.00	0.00	0.00
0.750 L	0.00	0.00	0.00	0.00	0.00	0.00
0.867 L	0.00	0.00	0.00	0.00	0.00	0.00
0.931 L	0.00	0.00	0.00	0.00	0.00	0.00
1.000 L	0.00	0.00	0.00	0.00	0.00	0.00
Dead end	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Max. B.M. & CORRESPONDING S.F. Due to CWLL Without Impact Factor

SEC.	Class A (2-Lanes)		Class 70RW		Class A (3-Lanes)		1-70RW +Class A(1L)		Governing		
	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Torsion	Max. B.M.	Corr. S.F.
	MZ	FY	MZ	FY	MZ	FY	MZ	FY	Mx	MZ	FY
	kNm	kN	kNm	kN	kNm	kN	kNm	kN	kNm	kNm	kN
	LaneReductio	= 1.00	LaneReductio	= 1.00	LaneReductio	= 0.90	LaneReductio	= 0.90			
Jacking End	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.000 L	19.36	236.99	65.66	141.71	10.05	246.67	23.54	173.82	51.53	65.66	141.71
0.069 L	375.12	183.94	174.48	156.10	417.81	194.46	277.83	179.17	42.62	417.81	194.46
0.133 L	663.37	172.85	398.49	187.28	721.83	182.19	542.76	191.04	24.63	721.83	182.19
0.250 L	1247.35	124.04	1129.23	225.45	1364.15	127.17	1295.45	208.24	21.06	1364.15	127.17
0.375 L	1555.10	51.45	1854.04	145.99	1720.05	46.78	1979.31	125.21	6.07	1979.31	125.21
0.500 L	1774.52	3.59	2116.90	11.79	1923.10	1.82	2225.44	20.66	6.49	2225.44	20.66
0.625 L	1480.56	110.41	1868.46	156.79	1671.12	104.42	1989.55	134.37	6.49	1989.55	134.37
0.750 L	988.45	147.38	1229.41	201.68	1149.58	154.34	1380.95	187.31	18.34	1380.95	187.31
0.867 L	416.37	143.25	479.53	210.05	502.28	161.82	613.42	208.85	25.74	613.42	208.85
0.931 L	214.60	136.18	213.11	181.54	265.30	158.95	312.36	200.55	47.04	312.36	200.55
1.000 L	16.00	132.85	69.52	166.24	9.23	157.48	25.59	194.86	56.16	69.52	166.24
Dead end	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Summary of MAX. S.F. & CORRESPONDING B.M Due to CWLL Without Impact Factor

SEC.	Class A (2-Lanes)		Class 70RW		Class A (3-Lanes)		1-70RW +Class A(1L)		Governing		
	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Torsion	Max. S.F.	Corr. B.M.
	FY	MZ	FY	MZ	FY	MZ	FY	MZ	Mx	FY	MZ
	kN	kNm	kN	kNm	kN	kNm	kN	kNm	kNm	kN	kNm
	LaneReductio	= 1.00	LaneReductio	= 1.00	LaneReductio	= 0.90	LaneReductio	= 0.90			
Jacking End	0.00	0.00	381.03	0.00	0.00	0.00	0.00	0.00	33.24	381.03	0.00
0.000 L	74.88	9.17	381.03	81.59	0.00	0.00	0.00	0.00	33.24	381.03	81.59
0.069 L	76.70	120.79	242.54	502.54	0.00	0.00	0.00	0.00	18.36	242.54	502.54
0.133 L	80.51	234.38	190.09	859.11	0.00	0.00	0.00	0.00	15.35	190.09	859.11
0.250 L	87.50	550.26	65.12	1375.36	0.00	0.00	0.00	0.00	1.73	87.50	550.26
0.375 L	87.54	874.26	31.26	1470.67	0.00	0.00	0.00	0.00	4.61	87.54	874.26
0.500 L	53.54	1134.90	106.46	1228.09	0.00	0.00	0.00	0.00	4.61	106.46	1228.09
0.625 L	1.59	1188.39	127.27	904.32	0.00	0.00	0.00	0.00	4.10	127.27	904.32
0.750 L	48.00	1047.49	129.10	521.72	0.00	0.00	0.00	0.00	6.50	129.10	521.72
0.867 L	96.09	677.26	108.47	227.10	0.00	0.00	0.00	0.00	6.17	108.47	227.10
0.931 L	174.74	458.60	93.54	92.67	0.00	0.00	0.00	0.00	9.04	174.74	458.60
1.000 L	282.83	17.38	87.73	48.17	0.00	0.00	0.00	0.00	10.17	282.83	17.38
Dead end	0.00	0.00	87.73	0.00	0.00	0.00	0.00	0.00	10.17	87.73	0.00

Impact Factor with congestion

=

0.174

Summary of B.M. & S.F Due to CWLL With Impact Factor For Long. Inner Girder

SEC.	Governing B.M. Due to CWLL		Governing S.F. Due to CWLL		Torsion Moment
	Max B.M.	Corr. S.F.	Max S.F.	Corr. B.M.	
	kNm	kN	kN	kNm	
Jacking End	0.00	0.00	447.31	0.00	33.24
0.000 L	77.09	166.36	447.31	95.78	51.53
0.069 L	490.49	228.28	284.73	589.95	42.62
0.133 L	847.39	213.89	223.16	1008.55	24.63
0.250 L	1601.44	149.29	102.72	645.98	21.06
0.375 L	2323.61	146.99	102.77	1026.33	6.07
0.500 L	2612.56	24.26	124.97	1441.72	6.49
0.625 L	2335.63	157.74	149.41	1061.63	6.49
0.750 L	1621.17	219.89	151.56	612.47	18.34
0.867 L	720.13	245.18	127.34	266.60	25.74
0.931 L	366.69	235.44	205.14	538.38	47.04
1.000 L	81.61	195.16	332.03	20.40	56.16
Dead end	0.00	0.00	102.99	0.00	10.17

LONGITUDINAL OUTER GIRDER :**Summary of Shear Force (S.F.) & Bending Moment (B.M.) for Long. Outer Girder (Without Composite)**

SEC.	Deck Slab + Diaphragm +Temp. LL		S.W. of Girder		Construction Live Load		Tor. Due to S.W. Girder+ Deck slab
	S.F.	B.M.	S.F.	B.M.	S.F.	B.M.	
	kN	kNm	kN	kNm	kN	kNm	
Jacking End	429.64	0.00	241.56	0.00	5.34	0.00	0.00
0.000 L	429.64	-2.22	241.56	-1.79	5.34	-0.03	0.00
0.069 L	369.95	657.45	193.25	356.93	4.60	8.18	0.00
0.133 L	315.68	1171.66	157.73	620.16	3.93	14.57	0.00
0.250 L	214.82	1911.05	107.34	989.61	2.67	23.77	0.00
0.375 L	107.41	2389.36	53.67	1228.60	1.34	29.72	0.00
0.500 L	0.00	2548.80	0.00	1308.27	0.00	31.70	0.00
0.625 L	107.41	2389.36	53.67	1228.60	1.34	29.72	0.00
0.750 L	214.82	1911.05	107.34	989.61	2.67	23.77	0.00

0.867 L	315.68	1171.66	157.73	620.16	3.93	14.57	0.00
0.931 L	369.95	657.45	193.25	356.93	4.60	8.18	0.00
1.000 L	429.64	-2.22	241.56	-1.79	5.34	-0.03	0.00
Dead end	429.64	0.00	241.56	0.00	5.34	0.00	0.00

Summary of S.F. & B.M at Different Section for Long. Outer Girder

SEC.	Effect of SIDL1 (SIDL without Surfacing)			Effect of SIDL2 (Surfacing)		
	S.F.	B.M.	Torsion	S.F.	B.M.	Torsion
	kN	kNm	kNm	kN	kNm	kNm
Jacking End	91.8	0.0	35.81	29.4	0.0	21.33
0.000 L	91.8	22.6	35.81	29.4	12.2	21.33
0.069 L	77.8	126.6	30.52	25.5	61.6	17.80
0.133 L	61.4	245.2	19.82	19.0	102.0	11.32
0.250 L	37.2	414.5	11.23	11.6	156.5	6.30
0.375 L	12.3	523.5	3.71	3.9	191.6	2.06
0.500 L	12.3	560.0	3.71	3.9	203.2	2.06
0.625 L	12.3	523.5	3.71	3.9	191.6	2.06
0.750 L	37.2	414.5	11.23	11.6	156.5	6.30
0.867 L	61.4	245.2	19.81	19.0	102.0	11.32
0.931 L	77.8	126.6	30.52	25.5	61.6	17.80
1.000 L	91.8	22.6	35.81	29.4	12.2	21.33
Dead end	91.8	0.0	35.81	29.4	0.0	21.33

Summary of S.F. & B.M at Different Section for Long.Outer Girder

SEC.	Effect of Shifting of Support			Due to Footpath Live load		
	S.F.	B.M.	Torsion	S.F.	B.M.	Torsion
	kN	kNm	kNm	kN	kNm	kNm
Jacking End	0.00	0.00	0.00	0.00	0.00	0.00
0.000 L	0.00	0.00	0.00	0.00	0.00	0.00
0.069 L	0.00	0.00	0.00	0.00	0.00	0.00
0.133 L	0.00	0.00	0.00	0.00	0.00	0.00
0.250 L	0.00	0.00	0.00	0.00	0.00	0.00
0.375 L	0.00	0.00	0.00	0.00	0.00	0.00
0.500 L	0.00	0.00	0.00	0.00	0.00	0.00
0.625 L	0.00	0.00	0.00	0.00	0.00	0.00
0.750 L	0.00	0.00	0.00	0.00	0.00	0.00
0.867 L	0.00	0.00	0.00	0.00	0.00	0.00
0.931 L	0.00	0.00	0.00	0.00	0.00	0.00
1.000 L	0.00	0.00	0.00	0.00	0.00	0.00
Dead end	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Max. B.M. & CORRESPONDING S.F. Due to CWLL Without Impact Factor For Long. Outer Girder

SEC.	Class A (2-Lanes)		Class 70RW		Class A (3-Lanes)		1-70RW +Class A(1L)		Governing		
	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Max. B.M.	Corr. S.F.	Torsion	Max. B.M.	Corr. S.F.
	MZ	FY	MZ	FY	MZ	FY	MZ	FY	Mx	MZ	FY
	kNm	kN	kNm	kN	kNm	kN	kNm	kN	kNm	kNm	kN
	LaneReduction	= 1.00	LaneReduction	= 1.00	LaneReduction	= 0.90	LaneReduction	= 0.90			
Jacking End	11.66	231.53	16.23	227.70	0.00	0.00	0.00	0.00	1.48	16.23	227.70
0.000 L	368.16	209.47	359.48	227.70	0.00	0.00	0.00	0.00	1.48	368.16	209.47
0.069 L	364.95	184.18	700.68	228.95	0.00	0.00	0.00	0.00	5.86	700.68	228.95
0.133 L	648.20	169.21	700.68	228.95	0.00	0.00	0.00	0.00	5.86	700.68	228.95
0.250 L	1087.83	114.14	1340.93	230.47	0.00	0.00	0.00	0.00	12.47	1340.93	230.47
0.375 L	1385.80	69.19	1794.99	107.63	0.00	0.00	0.00	0.00	6.66	1794.99	107.63
0.500 L	1608.95	59.34	1949.84	24.15	0.00	0.00	0.00	0.00	4.13	1949.84	24.15
0.625 L	1357.85	128.68	1739.30	106.96	0.00	0.00	0.00	0.00	6.00	1739.30	106.96
0.750 L	951.88	164.98	1246.35	179.86	0.00	0.00	0.00	0.00	10.67	1246.35	179.86
0.867 L	493.04	162.21	689.79	215.52	0.00	0.00	0.00	0.00	6.04	689.79	215.52
0.931 L	249.72	162.21	350.03	225.29	0.00	0.00	0.00	0.00	6.50	350.03	225.29
1.000 L	251.18	160.32	352.35	223.62	0.00	0.00	0.00	0.00	1.74	352.35	223.62
Dead end	13.35	160.32	16.62	223.62	0.00	0.00	0.00	0.00	1.74	16.62	223.62

Summary of MAX. S.F. & CORRESPONDING B.M Due to CWLL Without Impact Factor For Long. Outer Girder

SEC.	Class A (2-Lanes)		Class 70RW		Class A (3-Lanes)		1-70RW +Class A(1L)		Governing		
	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Max. S.F.	Corr. B.M.	Torsion	Max. S.F.	Corr. B.M.
	FY	MZ	FY	MZ	FY	MZ	FY	MZ	Mx	FY	MZ
	kN	kNm	kN	kNm	kN	kNm	kN	kNm	kNm	kN	kNm
	LaneReduction	= 1.00	LaneReduction	= 1.00	LaneReduction	= 0.90	LaneReduction	= 0.90			
Jacking End	123.31	10.61	314.96	10.49	0.00	0.00	0.00	0.00	48.55	314.96	10.49
0.000 L	123.31	192.86	314.96	200.36	0.00	0.00	0.00	0.00	48.55	314.96	200.36
0.069 L	124.85	378.96	314.96	509.19	0.00	0.00	0.00	0.00	48.55	314.96	509.19
0.133 L	127.25	732.85	197.60	864.05	0.00	0.00	0.00	0.00	20.51	197.60	864.05
0.250 L	92.98	1055.77	172.09	1339.44	0.00	0.00	0.00	0.00	11.00	172.09	1339.44
0.375 L	36.31	1199.56	38.43	1535.08	0.00	0.00	0.00	0.00	1.04	38.43	1535.08
0.500 L	20.92	1174.41	65.58	1427.59	0.00	0.00	0.00	0.00	8.41	65.58	1427.59
0.625 L	78.29	979.96	91.54	1146.53	0.00	0.00	0.00	0.00	7.78	91.54	1146.53
0.750 L	103.21	681.87	130.43	766.71	0.00	0.00	0.00	0.00	6.14	130.43	766.71
0.867 L	179.12	440.59	133.93	394.82	0.00	0.00	0.00	0.00	0.05	179.12	440.59
0.931 L	237.44	440.32	131.76	198.60	0.00	0.00	0.00	0.00	5.48	237.44	440.32
1.000 L	237.44	192.86	130.06	200.36	0.00	0.00	0.00	0.00	9.35	237.44	192.86
Dead end	274.43	6.94	130.06	14.25	0.00	0.00	0.00	0.00	9.35	274.43	6.94

Impact Factor = 0.174

Summary of B.M. & S.F Due to CWLL With Impact Factor For Long. Outer Girder

SEC.	Governing B.M. Due to CWLL		Governing S.F. Due to CWLL		Torsion Moment
	Max B.M.	Corr. S.F.	Max S.F.	Corr. B.M.	
	kNm	kN	kN	kNm	kNm
Jacking End	19.05	267.31	369.74	12.32	48.55
0.000 L	432.20	245.91	369.74	235.21	48.55
0.069 L	822.57	268.77	369.74	597.76	48.55
0.133 L	822.57	268.77	231.97	1014.35	20.51
0.250 L	1574.18	270.56	202.02	1572.43	12.47
0.375 L	2107.23	126.35	45.11	1802.11	6.66
0.500 L	2289.01	28.35	76.99	1675.92	8.41
0.625 L	2041.84	125.57	107.46	1345.97	7.78
0.750 L	1463.15	211.15	153.11	900.08	10.67
0.867 L	809.77	253.00	210.27	517.23	6.04
0.931 L	410.92	264.47	278.75	516.91	6.50
1.000 L	413.64	262.52	278.75	226.40	9.35
Dead end	19.52	262.52	322.16	8.14	9.35

Load Factors (As per IRC:6-2014)

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2010

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table 3.3 Partial Safety Factor For Verification of Serviceability Limit State

-Refer Table 3.3 of IRC:6-2010

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.000	1.00	1.00
Prestress , Shrinkage & Creep Effect	1.000	1.00	1.00
CWLL and Associate load and FPLL	1.000	0.75	0.00
Thermal Loads	0.600	0.50	0.00

SUMMARY OF BENDING MOMENT AND SHEAR FORCE DUE TO DIFFERENT LOADING (EXCLUDING PRESTRESS) IN INNER GIRDER

Section	Jacking	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

1 DUE TO SELF WEIGHT OF GIRDER

Bending Moment (kNm)	0.00	-1.79	356.93	620.16	989.61	1228.60	1308.27	1228.60	989.61	620.16	356.93	-1.79	0.00
Shear Force (kN)	241.56	241.56	193.25	157.73	107.34	53.67	0.00	53.67	107.34	157.73	193.25	241.56	241.56

2 DUE TO GREEN WEIGHT OF DECK SLAB AND DIAPHRAGM AND TEMPORARY LIVE LOAD (Applied on Non-Composite Girder)

Bending Moment (kNm)	0.00	-2.05	606.87	1081.54	1764.04	2205.57	2352.74	2205.57	1764.04	1081.54	606.87	-2.05	0.00
Shear Force (kN)	396.60	396.60	341.49	291.39	198.30	99.15	0.00	99.15	198.30	291.39	341.49	396.60	396.60

3 DUE TO SHUTTERING WEIGHT

Bending Moment (kNm)	0.00	-0.03	8.18	14.57	23.77	29.72	31.70	29.72	23.77	14.57	8.18	-0.03	0.00
Shear Force (kN)	5.34	5.34	4.60	3.93	2.67	1.34	0.00	1.34	2.67	3.93	4.60	5.34	5.34

4 DUE TO REMOVAL OF SHUTTERING (Applied on Composite Girder)

Bending Moment (kNm)	0.00	0.03	-8.18	-14.57	-23.77	-29.72	-31.70	-29.72	-23.77	-14.57	-8.18	0.03	0.00
Shear Force (kN)	-5.34	-5.34	-4.60	-3.93	-2.67	-1.34	0.00	-1.34	-2.67	-3.93	-4.60	-5.34	-5.34

5 DUE TO SHIFTING OF SUPPORT (Applied on continuous Composite Girder)

Bending Moment (kNm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Shear Force (kN)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

6 DUE TO SUPERIMPOSED DEAD LOAD -SIDL1 (Applied on Composite Girder)

Bending Moment (kNm)	0.00	0.00	56.32	79.40	103.01	119.15	126.14	119.17	103.04	79.38	56.32	0.00	0.00
Shear Force (kN)	0.00	17.29	17.29	13.33	5.02	3.34	1.35	3.34	5.01	13.33	17.28	17.28	0.00

7 DUE TO SUPERIMPOSED DEAD LOAD -SIDL2 (Applied on Composite Girder)

Bending Moment (kNm)	0.32	0.79	146.84	257.79	486.67	606.69	646.85	606.89	487.18	257.79	146.85	0.79	0.32
Shear Force (kN)	1.17	82.75	82.75	72.94	52.01	28.98	6.63	28.92	51.96	72.94	82.75	82.75	1.17

8 DUE TO FOOTPATH LIVE LOAD (Applied on Composite Girder)

Bending Moment (kNm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Shear Force (kN)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

9 DUE TO CARRIAGEWAY LIVE LOAD (Applied on Composite Girder)

Governing Bending Moment (kNm)	0.00	77.09	490.49	847.39	1601.44	2323.61	2612.56	2335.63	1621.17	720.13	366.69	81.61	0.00
Corresp. Shear Force (kN)	0.00	166.36	228.28	213.89	149.29	146.99	24.26	157.74	219.89	245.18	235.44	195.16	0.00
Governing Shear Force (kN)	447.31	447.31	284.73	223.16	102.72	102.77	124.97	149.41	151.56	127.34	205.14	332.03	102.99
Corresp. Bending Moment (kNm)	0.00	95.78	589.95	1008.55	645.98	1026.33	1441.72	1061.63	612.47	266.60	538.38	20.40	0.00

SUMMARY OF BENDING MOMENT AND SHEAR FORCE DUE TO DIFFERENT LOADING (EXCLUDING PRESTRESS) IN OUTER GIRDER

Section	Jacking	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

1 DUE TO SELF WEIGHT OF GIRDER

Bending Moment (kNm)	0.00	-1.79	356.93	620.16	989.61	1228.60	1308.27	1228.60	989.61	620.16	356.93	-1.79	0.00
Shear Force (kN)	241.56	241.56	193.25	157.73	107.34	53.67	0.00	53.67	107.34	157.73	193.25	241.56	241.56

2 DUE TO GREEN WEIGHT OF DECK SLAB AND DIAPHRAGM AND TEMPORARY LIVE LOAD (Applied on Non-Composite Girder)

Bending Moment (kNm)	0.00	-2.22	657.45	1171.66	1911.05	2389.36	2548.80	2389.36	1911.05	1171.66	657.45	-2.22	0.00
Shear Force (kN)	429.64	429.64	369.95	315.68	214.82	107.41	0.00	107.41	214.82	315.68	369.95	429.64	429.64

3 DUE TO SHUTTERING WEIGHT (CONSTRUCTION LOAD)

Bending Moment (kNm)	0.00	-0.03	8.18	14.57	23.77	29.72	31.70	29.72	23.77	14.57	8.18	-0.03	0.00
Shear Force (kN)	5.34	5.34	4.60	3.93	2.67	1.34	0.00	1.34	2.67	3.93	4.60	5.34	5.34

4 DUE TO REMOVAL OF SHUTTERING (Applied on Composite Girder)

Bending Moment (kNm)	0.00	0.03	-8.18	-14.57	-23.77	-29.72	-31.70	-29.72	-23.77	-14.57	-8.18	0.03	0.00
Shear Force (kN)	-5.34	-5.34	-4.60	-3.93	-2.67	-1.34	0.00	-1.34	-2.67	-3.93	-4.60	-5.34	-5.34

5 DUE TO SHIFTING OF SUPPORT (Applied on continuous Composite Girder)

Bending Moment (kNm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Shear Force (kN)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

6 DUE TO SUPERIMPOSED DEAD LOAD -SIDL1 (Applied on Composite Girder)

Bending Moment (kNm)	0.00	22.62	126.63	245.19	414.50	523.51	560.00	523.50	414.54	245.19	126.63	22.62	0.00
Shear Force (kN)	91.78	91.78	77.80	61.42	37.19	12.30	12.30	12.30	37.20	61.41	77.80	91.78	91.78

7 DUE TO SUPERIMPOSED DEAD LOAD -SIDL2 (Applied on Composite Girder)

Bending Moment (kNm)	0.00	12.24	61.59	101.98	156.51	191.61	203.16	191.60	156.52	101.98	61.59	12.24	0.00
Shear Force (kN)	29.37	29.37	25.55	19.00	11.55	3.89	3.89	3.89	11.55	19.00	25.55	29.37	29.37

8 DUE TO FOOTPATH LIVE LOAD (Applied on Composite Girder)

Bending Moment (kNm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Shear Force (kN)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

9 DUE TO CARRIAGEWAY LIVE LOAD (Applied on Composite Girder)

Governing Bending Moment (kNm)	19.05	432.20	822.57	822.57	1574.18	2107.23	2289.01	2041.84	1463.15	809.77	410.92	413.64	19.52
Corresp. Shear Force (kN)	267.31	245.91	268.77	268.77	270.56	126.35	28.35	125.57	211.15	253.00	264.47	262.52	262.52
Governing Shear Force (kN)	369.74	369.74	369.74	231.97	202.02	45.11	76.99	107.46	153.11	210.27	278.75	278.75	322.16
Corresp. Bending Moment (kNm)	12.32	235.21	597.76	1014.35	1572.43	1802.11	1675.92	1345.97	900.08	517.23	516.91	226.40	8.14

Details of Prestressing Strand Data**(Refer Table 18.4 of IRC:112-2011 , Stress Relieved Class- II Strands)**

No of Prestressing Cables	=	5	
Type of Tendon	=	12 DP 13	
Cross sectional area of each cable, A_s	=	1184.400	mm ²
Minimum Ultimate Tensile Strength (UTS) of a Cable	=	2203.00	kN
Ultimate stress in each cable	=	1860.01	N/mm ²
Maximum permissible force i.e. Jacking End Force	=	0.765	% of UTS

Number of strands in a cable	=	12	
Nominal duct diameter (Outer Diameter)	=	75	mm
Minimum clear cover from concrete surface	=	75	mm
Minimum clear distance between two cable	=	75	mm
Minimum effective cover from concrete surface	=	113	mm
Minimum effective distance between two cable	=	150	mm

Clause 15.3.1.2 , IRC 112-2011

Clause 15.3.1.3 , IRC 112-2011

Modulus of elasticity of prestressing steel, E_{ps}	=	1.95E+05	N/mm ²

For Corrugated HDPE SHEATHING DUCT**(Refer Table 7.1 of IRC:112-2011)**

Wobble coefficient k	=	0.002	per metre
Friction coefficient μ	=	0.17	per radians
Expected slip, s	=	6.0	mm

	Stage of Prestressin g	Nos. of strands to be stressed		Total Strands	area of cables to stressed (mm ²)	Jacking Forces per cable (kN)
No of strands to stressed in cable No 1	=	2	11 out of	12	1085.70	1545
No of strands to stressed in cable No 2	=	2	11 out of	12	1085.70	1545
No of strands to stressed in cable No 3	=	1	10 out of	12	987.00	1404
No of strands to stressed in cable No 4	=	2	10 out of	12	987.00	1404
No of strands to stressed in cable No 5	=	1	10 out of	12	987.00	1404
No of strands to stressed in cable No 6	=	0	0 out of	12	0.00	0

Stage I Prestressing Cable Nos

= 2

Area of Cables stressed in Stage I

= 1974.000 mm²

Maximum Jacking Force in Stage I Cable

= 2808.825 kN

Ultimate Tensile strength of cables in Stage I

= 3671.667 kN

Stage II Prestressing Cable Nos

= 3

Area of Cables stressed in Stage II

= 3158.400 mm²

Maximum Jacking Force in Stage II Cable

= 4494.120 kN

Ultimate Tensile strength of cables in Stage II

= 5874.667 kN

CONFIGURATION OF CABLES

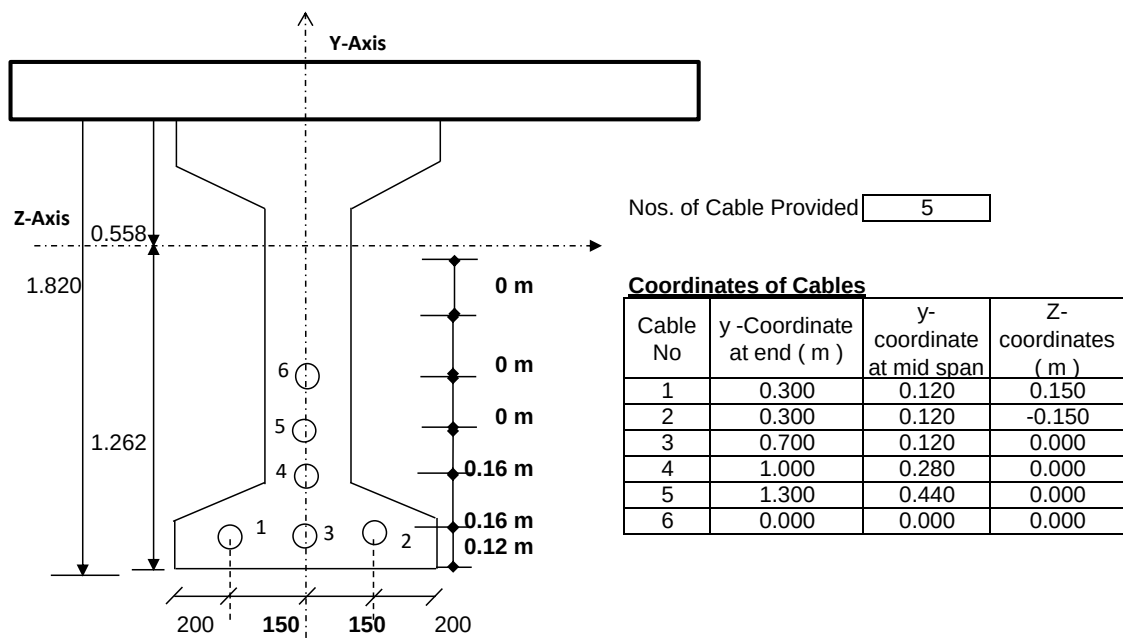


Fig. Section At Mid-Span showing cables in Elevation

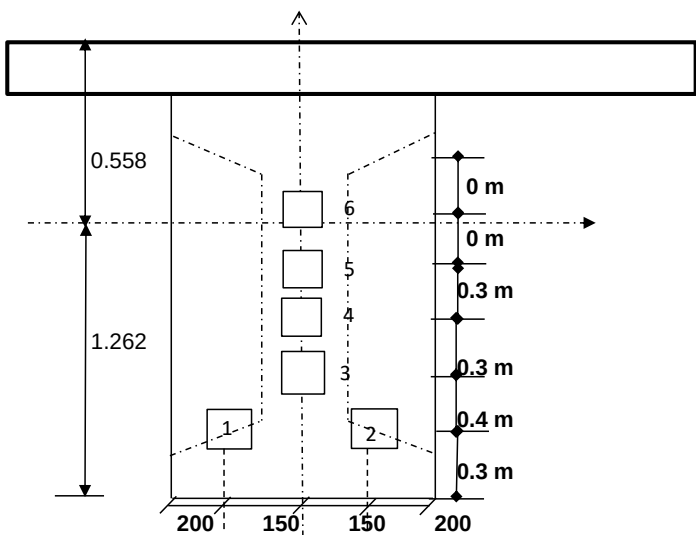


Fig. Section At Jacking End showing cables in Elevation

Calculation of Friction and Slip Loss

C/C of bearing = 23.750 m
 Projected Length of the cable = 24.150 m

Portion A of Cable :- Inclined
 Portion B of Cable :- Parabolic
 Portion c of Cable :- Straight

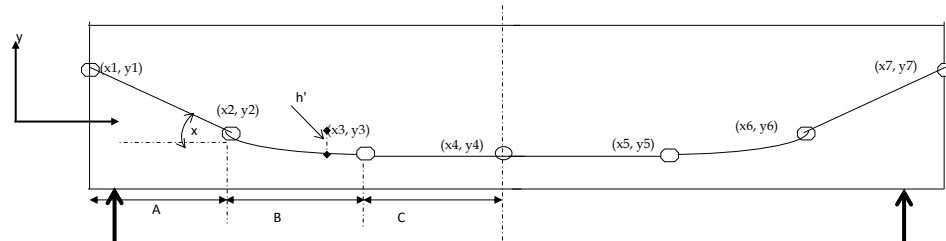


FIG. Showing Cable Profile in Elevation

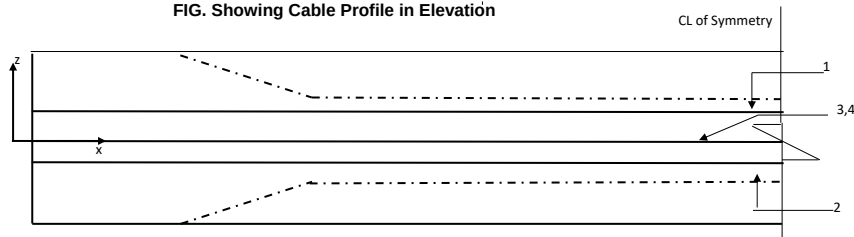


FIG. Showing Cable Profile in Plan

Check for Parabolic curve height h' (IN ELEVATION)						
	Cables	1	2	3	4	5
Coordinate (y2)	h'=	0.115	0.115	0.475	0.589	0.704
Inclined portion (in rad)	theta=	0.065	0.065	0.1051	0.1302	0.1551
parabolic curve	theta=	0.065	0.065	0.1051	0.1302	0.1551
goal seek		0.00	0.000	0.000	0.000	0.000
(deg.)	theta=	3.74	3.74	6.02	7.46	8.89
Length of Curved Portion (L) =		3.500	3.500	9.000	9.000	9.000
	K=h'/L ²	0.009	0.009	0.006	0.007	0.009

CABLE PROFILE

CABLE No.	A = Horr. Length of inclined Portion in m.	B = Horr. Length of parabolic Portion in m.	d = Horr. Length of plan bend. Portion in m.	C = Length of straight Portion in m.	H = Y co-ordinate At jacking end	h3 = Y co-ordinate At mid span	m = plan bending in m.	Jacking force (Po) (kN)	Stressing stage
1	1.000	3.500	0.000	7.575	0.300	0.120	0.000	1544.85	2
2	1.000	3.500	0.000	7.575	0.300	0.120	0.000	1544.85	2
3	1.000	9.000	0.000	2.075	0.700	0.120	0.000	1404.41	1
4	1.000	9.000	0.000	2.075	1.000	0.280	0.000	1404.41	2
5	1.000	9.000	0.000	2.075	1.300	0.440	0.000	1404.41	1
6	0.000	0.000	0.000	12.075	0.000	0.000	0.000	0.00	0

Prestress after Friction and Slip Loss:

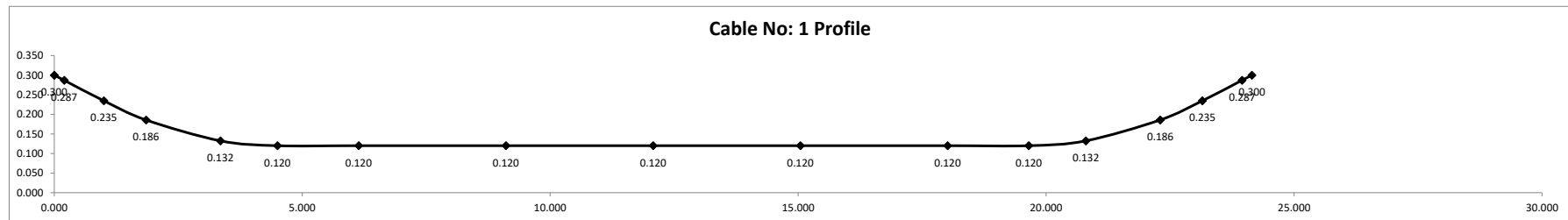
Cable 1

$$\text{Friction loss} = P_o \times \exp(-(\mu x + k \theta)) = P_o \times \text{Friction coeff}$$

Cable Profile	Cable Profile	Sections	x (m)	y(m)	z (m)	θ_v (deg)	θ_H (deg)	θ_s (deg)	cum θ (deg)	L INC (m)	X PRO (m)	Friction coeff.	Force after Friction loss , p (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p' (kN)	p' hor' (kN)
Inclined	Node 1	Jacking End	0.000	0.300	0.150	3.745	0.000	3.745	0.000	0.000	0.000	1.000	1544.854	-	76.634		1391.586	1388.615
		0.000 L	0.200	0.287	0.150	3.745	0.000	3.745	0.000	0.200	0.200	1.000	1544.235	-	76.015	15.265	1392.205	1389.232
	Node 2		1.000	0.235	0.150	3.745	0.000	3.745	0.000	0.802	1.002	0.998	1541.761	-	73.541	59.822	1394.679	1391.701
		0.069 L	1.850	0.186	0.150	2.837	0.000	2.837	0.908	0.851	1.854	0.994	1534.998	-	66.778	59.635	1401.442	1399.724

Parabolic	Node 3	0.133 L	3.350	0.132	0.150	1.232	0.000	1.232	2.513	1.501	3.354	0.986	1523.125	-	54.906	91.263	1413.314	1412.987
			4.500	0.120	0.150	0.000	0.000	0.000	3.745	1.150	4.505	0.980	1514.081	-	45.861	57.941	1422.358	1422.358
Straight	Node 4	0.250 L	6.138	0.120	0.150	0.000	0.000	0.000	3.745	1.638	6.142	0.977	1509.131	-	40.911	71.045	1427.309	1427.309
		0.375 L	9.106	0.120	0.150	0.000	0.000	0.000	3.745	2.969	9.111	0.971	1500.197	-	31.977	108.193	1436.243	1436.243
Straight	Node 5	0.500 L	12.075	0.120	0.150	0.000	0.000	0.000	3.745	2.969	12.080	0.965	1491.316	-	23.096	81.749	1445.124	1445.124
		0.625 L	15.044	0.120	0.150	0.000	0.000	0.000	3.745	2.969	15.048	0.960	1482.487	-	14.267	55.461	1453.952	1453.952
Straight	Node 5	0.750 L	18.013	0.120	0.150	0.000	0.000	0.000	3.745	2.969	18.017	0.954	1473.711	-	5.491	29.329	1462.729	1462.729
			19.650	0.120	0.150	0.000	0.000	0.000	3.745	1.638	19.655	0.951	1468.893	1468.220	0.673	5.047	1467.547	1467.547
Parabolic	Node 6	0.867 L	20.800	0.132	0.150	-1.232	0.000	1.232	4.977	1.150	20.805	0.945	1460.170	-	0.000	0.387	1460.170	1459.833
		0.931 L	22.300	0.186	0.150	-2.837	0.000	2.837	6.582	1.501	22.306	0.938	1448.877	-	0.000	0.000	1448.877	1447.101
Inclined	Node 6		23.150	0.235	0.150	-3.745	0.000	3.745	7.490	0.851	23.157	0.934	1442.521	-	0.000	0.000	1442.521	1439.441
		1.000 L	23.950	0.287	0.150	-3.745	0.000	3.745	7.490	0.802	23.959	0.932	1440.210	-	0.000	0.000	1440.210	1437.135
Inclined	Node 7	Dead end	24.150	0.300	0.150	-3.745	0.000	3.745	7.490	0.200	24.159	0.932	1439.633	-	0.000	0.000	1439.633	1436.559
										24.159		635.137						

Total Length of Cable 1 = 24.159 m

**Summary of Forces after Friction and Slip Losses:****Cable 1****Calculation of Cable extension for Cable No. 1**

Average prestress force after friction loss	=	1491.78 kN
Length of cable	=	24.159 m
Cable extension	=	170.23 mm

Calculation of Slip loss:

Assuming Slip travel upto	19.7387 m from anchorage end	
Prestress force at null point	=	1468.22 kN (After friction loss by Interpolation)
Assume Slip	=	6 mm

Sections	p' _{hor} (kN)	p' _{ver} (kN)	c.g. of cable from soffit (m)	Non-Composite Section		Composite Section	
				c.g. of Non-Composite Section from soffit	Eccentricity of cable	c.g. of Composite Section from soffit	Eccentricity of cable
Jacking End	1388.615	90.891	0.300	0.831	0.531	0.436	0.136
0.000 L	1389.232	90.931	0.287	0.831	0.544	0.436	0.149
0.069 L	1399.724	69.368	0.186	0.829	0.643	0.390	0.204
0.133 L	1412.987	30.389	0.132	0.819	0.686	0.338	0.206
0.250 L	1427.309	0.000	0.120	0.819	0.699	0.338	0.218
0.375 L	1436.243	0.000	0.120	0.819	0.699	0.338	0.218
0.500 L	1445.124	0.000	0.120	0.819	0.699	0.338	0.218
0.625 L	1453.952	0.000	0.120	0.819	0.699	0.338	0.218
0.750 L	1462.729	0.000	0.120	0.819	0.699	0.338	0.218

0.867 L	1459.833	31.396	0.132	0.819	0.686	0.338	0.206
0.931 L	1447.101	71.716	0.186	0.829	0.643	0.390	0.204
1.000 L	1437.135	94.066	0.287	0.831	0.544	0.436	0.149
Dead end	1436.559	94.029	0.300	0.831	0.531	0.436	0.136

Cable Profile		Sections	x (m)	y(m)	z (m)	θ _V (deg)	θ _H (deg)	θ _S (deg)	cum θ (deg)	L _{INC} (m)	X _{PRO} (m)	Friction coeff.	Force after Friction loss , p' (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p' (kN)	p'hor (kN)	
Inclined	Node 1	Jacking End	0.000	0.300	-0.150	3.745	0.000	3.745	0.000	0.000	0.000	1.000	1544.854	-	76.634		1391.586	1388.614	
		0.000 L	0.200	0.287	-0.150	3.745	0.000	3.745	0.000	0.200	0.200	1.000	1544.235	-	76.015	15.265	1392.205	1389.232	
Parabolic	Node 2		1.000	0.235	-0.150	3.745	0.000	3.745	0.000	0.802	1.002	0.998	1541.761	-	73.541	59.822	1394.679	1391.701	
		0.069 L	1.850	0.186	-0.150	2.837	0.000	2.837	0.908	0.851	1.854	0.994	1534.998	-	66.778	59.635	1401.442	1399.724	
		0.133 L	3.350	0.132	-0.150	1.232	0.000	1.232	2.513	1.501	3.354	0.986	1523.125	-	54.906	91.263	1413.314	1412.987	
Straight	Node 3		4.500	0.120	-0.150	0.000	0.000	0.000	3.745	1.150	4.505	0.980	1514.081	-	45.861	57.941	1422.358	1422.358	
		0.250 L	6.138	0.120	-0.150	0.000	0.000	0.000	3.745	1.638	6.142	0.977	1509.131	-	40.911	71.045	1427.309	1427.309	
		0.375 L	9.106	0.120	-0.150	0.000	0.000	0.000	3.745	2.969	9.111	0.971	1500.197	-	31.977	108.193	1436.243	1436.243	
Straight	Node 4		0.500 L	12.075	0.120	-0.150	0.000	0.000	0.000	3.745	2.969	12.080	0.965	1491.316	-	23.096	81.749	1445.124	1445.124
		0.625 L	15.044	0.120	-0.150	0.000	0.000	0.000	3.745	2.969	15.048	0.960	1482.487	-	14.267	55.461	1453.952	1453.952	
		0.750 L	18.013	0.120	-0.150	0.000	0.000	0.000	3.745	2.969	18.017	0.954	1473.711	-	5.491	29.329	1462.728	1462.728	
Parabolic	Node 5		19.650	0.120	-0.150	0.000	0.000	0.000	3.745	1.638	19.655	0.951	1468.892	1468.220	0.673	5.047	1467.547	1467.547	
		0.867 L	20.800	0.132	-0.150	-1.232	0.000	1.232	4.977	1.150	20.805	0.945	1460.170	-	0.000	0.387	1460.170	1459.833	
		0.931 L	22.300	0.186	-0.150	-2.837	0.000	2.837	6.582	1.501	22.306	0.938	1448.877	-	0.000	0.000	1448.877	1447.101	
Inclined	Node 6		23.150	0.235	-0.150	-3.745	0.000	3.745	7.490	0.851	23.157	0.934	1442.521	-	0.000	0.000	1442.521	1439.441	
		1.000 L	23.950	0.287	-0.150	-3.745	0.000	3.745	7.490	0.802	23.959	0.932	1440.210	-	0.000	0.000	1440.210	1437.135	
	Node 7	Dead end	24.150	0.300	-0.150	-3.745	0.000	3.745	7.490	0.200	24.159	0.932	1439.633	-	0.000	0.000	1439.633	1436.559	

Total Length of Cable 2	=	24.159 m

Summary of Forces after Friction and Slip Losses:**Cable 2**

Sections	p' _{hor} (kN)	p' _{ver} (kN)	c.g. of cable from soffit (m)	Non-Composite Section		Composite Section	
				c.g. of Non-Composite Section from soffit	Eccentricity of cable	c.g. of Composite Section from soffit	Eccentricity of cable
Jacking End	1388.614	90.891	0.300	0.831	0.531	0.436	0.136
0.000 L	1389.232	90.932	0.287	0.831	0.544	0.436	0.149
0.069 L	1399.724	69.368	0.186	0.829	0.643	0.390	0.204
0.133 L	1412.987	30.388	0.132	0.819	0.686	0.338	0.206
0.250 L	1427.309	0.000	0.120	0.819	0.699	0.338	0.218
0.375 L	1436.243	0.000	0.120	0.819	0.699	0.338	0.218
0.500 L	1445.124	0.000	0.120	0.819	0.699	0.338	0.218
0.625 L	1453.952	0.000	0.120	0.819	0.699	0.338	0.218
0.750 L	1462.728	0.000	0.120	0.819	0.699	0.338	0.218
0.867 L	1459.833	31.396	0.132	0.819	0.686	0.338	0.206
0.931 L	1447.101	71.716	0.186	0.829	0.643	0.390	0.204
1.000 L	1437.135	94.067	0.287	0.831	0.544	0.436	0.149
Dead end	1436.559	94.029	0.300	0.831	0.531	0.436	0.136

Calculation of Cable extension for Cable No. - - - 2

Average prestress force after friction loss	=	1491.78 kN
Length of cable	=	24.159 m
Cable extension	=	170.23 mm

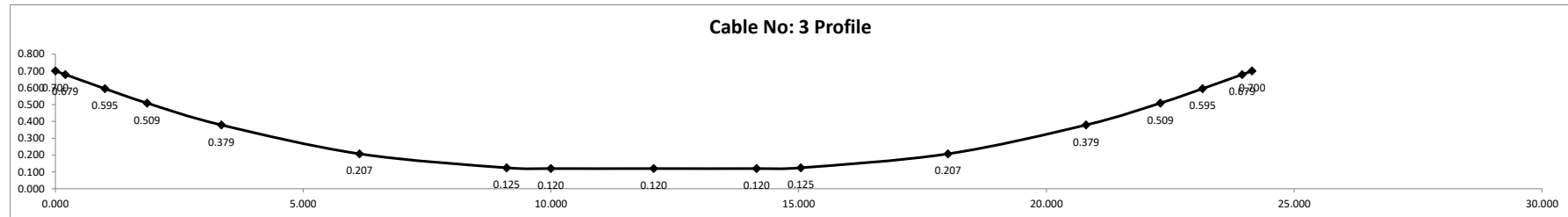
Calculation of Slip loss:

Assuming Slip travel upto	19.7387 m from anchorage end
Prestress force at null point	= 1468.22 kN (After friction loss by Interpolation)
Assume Slip	= 6 mm
Half shaded area s x Es x A / 2	= 635.13 kN
area of force diagram above null point	= 635.14 kN
equat Half shaded area - area of force diagram above null point to zero	= 0.00

Prestress after Friction and Slip Loss:**Cable 3**

Cable Profile		Sections	x (m)	y(m)	z (m)	θ _V (deg)	θ _H (deg)	θ _S (deg)	Cum. θ (deg)	L _{INC} (m)	X _{PRO} (m)	Friction coeff.	Force after Friction loss , p (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p' (kN)	p' _{hor} (kN)
Inclined	Node 1	Jacking End	0.000	0.700	0.000	6.020	0.000	6.020	0.000	0.000	0.000	1.000	1404.413	-	74.282		1255.848	1248.922
	Node 2	0.000 L	0.200	0.679	0.000	6.020	0.000	6.020	0.000	0.201	0.201	1.000	1403.848	-	73.718	14.800	1256.412	1249.484
1.000		0.595	0.000	6.020	0.000	6.020	0.000	6.020	0.000	0.804	1.006	0.998	1401.591	-	71.461	58.071	1258.669	1251.728
0.069 L		1.850	0.509	0.000	5.455	0.000	5.455	0.565	0.854	1.860	0.995	1396.855	-	66.725	58.729	1263.405	1257.684	
0.133 L		3.350	0.379	0.000	4.455	0.000	4.455	1.564	1.506	3.365	0.989	1388.531	-	58.401	93.844	1271.729	1267.886	
0.250 L		6.138	0.207	0.000	2.591	0.000	2.591	3.429	2.793	6.158	0.978	1373.181	-	43.051	141.399	1287.079	1285.763	
Parabolic	Node 3	0.375 L	9.106	0.125	0.000	0.600	0.000	0.600	5.420	2.970	9.128	0.966	1357.007	-	26.877	103.800	1303.253	1303.181
		10.000	0.120	0.000	0.000	0.000	0.000	6.020	0.894	10.022	0.963	1352.174	-	22.044	21.862	1308.086	1308.086	
		Node 4	0.500 L	12.075	0.120	0.000	0.000	0.000	0.000	6.020	2.075	12.097	0.959	1346.575	-	16.445	39.932	1313.686
Straight	Node 5		14.150	0.120	0.000	0.000	0.000	0.000	6.020	2.075	14.172	0.955	1340.998	-	10.868	28.337	1319.262	1319.262
Straight			0.625 L	15.044	0.125	0.000	0.600	0.000	0.600	6.620	0.894	15.066	0.951	1336.222	1330.130	6.092	7.579	1324.038
Parabolic	Node 6	0.750 L	18.013	0.207	0.000	2.591	0.000	2.591	8.611	2.970	18.036	0.940	1320.484	-	0.000	9.043	1320.484	1319.133
		0.867 L	20.800	0.379	0.000	4.455	0.000	4.455	10.475	2.793	20.828	0.930	1305.886	-	0.000	0.000	1305.886	1301.939
		0.931 L	22.300	0.509	0.000	5.455	0.000	5.455	11.475	1.506	22.334	0.924	1298.104	-	0.000	0.000	1298.104	1292.225
		23.150	0.595	0.000	6.020	0.000	6.020	12.040	0.854	23.188	0.921	1293.718	-	0.000	0.000	1293.718	1286.584	
Inclined	Node 7	1.000 L	23.950	0.679	0.000	6.020	0.000	6.020	12.040	0.804	23.993	0.920	1291.638	-	0.000	0.000	1291.638	1284.515
		Dead end	24.150	0.700	0.000	6.020	0.000	6.020	12.040	0.201	24.194	0.919	1291.119	-	0.000	0.000	1291.119	1283.999

577.396Total Length of Cable 3 = **24.194 m**



Calculation of Cable extension for Cable No. - - - 3

Average prestress force after friction loss	=	1347.20 kN
Length of cable	=	24.194 m
Cable extension	=	169.35 mm

Calculation of Slip loss:

Assuming Slip travel upto	16.1929 m from anchorage end
Prestress force at null point	= 1330.13 kN (After friction loss by Interpolation)
Assume Slip	= 6 mm
Half shaded area $s \times E_s \times A / 2$	= 577.40 kN
area of force diagram above null point	= 577.40 kN
equat Half shaded area - area of force diagram above null point to zero	= 0.00

Summary of Forces after Friction and Slip Losses:

Table 3

Sections	p' hor (kN)	p' ver (kN)	c.g. of cable from soffit (m)	Non-Composite Section		Composite Section	
				c.g. of Non-Composite Section from soffit	Eccentricity of cable	c.g. of Composite Section from soffit	Eccentricity of cable
Jacking End	1248.922	131.705	0.700	0.831	0.131	0.436	-0.264
0.000 L	1249.484	131.764	0.679	0.831	0.152	0.436	-0.243
0.069 L	1257.684	120.102	0.509	0.829	0.320	0.390	-0.120
0.133 L	1267.886	98.793	0.379	0.819	0.440	0.338	-0.041
0.250 L	1285.763	58.191	0.207	0.819	0.611	0.338	0.131
0.375 L	1303.181	13.647	0.125	0.819	0.694	0.338	0.213
0.500 L	1313.686	0.000	0.120	0.819	0.699	0.338	0.218
0.625 L	1323.97	13.86	0.12	0.819	0.694	0.338	0.213
0.750 L	1319.13	59.70	0.21	0.819	0.611	0.338	0.131
0.867 L	1301.94	101.45	0.38	0.819	0.440	0.338	-0.041
0.931 L	1292.23	123.40	0.51	0.829	0.320	0.390	-0.120
1.000 L	1284.52	135.46	0.68	0.831	0.152	0.436	-0.243
Dead end	1284.00	135.40	0.70	0.831	0.131	0.436	-0.264

Prestress after Friction and Slip Loss:

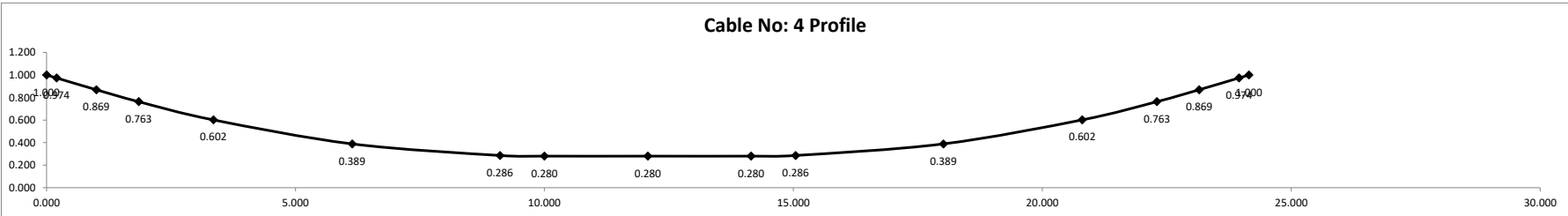
Cable 4

Cable Profile		Sections	x (m)	y(m)	z (m)	θ_v (deg)	θ_H (deg)	θ_s (deg)	Cum. θ (deg)	L INC (m)	X PRO (m)	Friction coeff.	Force after Friction loss , p (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p' (kN)	p' hor (kN)
Inclined	Node 1	Jacking End	0.000	1.000	0.000	7.458	0.000	7.458	0.000	0.000	0.000	1.000	1404.413	-	78.211		1247.990	1237.432
		0.000 L	0.200	0.974	0.000	7.458	0.000	7.458	0.000	0.202	0.202	1.000	1403.846	-	77.645	15.586	1248.556	1237.993
Parabolic	Node 2		1.000	0.869	0.000	7.458	0.000	7.458	0.000	0.807	1.009	0.998	1401.583	-	75.381	61.211	1250.820	1240.238
		0.069 L	1.850	0.763	0.000	6.761	0.000	6.761	0.698	0.857	1.865	0.994	1396.291	-	70.090	61.825	1256.112	1247.377
		0.133 L	3.350	0.602	0.000	5.525	0.000	5.525	1.933	1.509	3.374	0.988	1386.989	-	60.788	98.158	1265.413	1259.534
		0.250 L	6.138	0.389	0.000	3.216	0.000	3.216	4.243	2.796	6.169	0.975	1369.838	-	43.637	145.542	1282.564	1280.545
		0.375 L	9.106	0.286	0.000	0.745	0.000	0.745	6.713	2.971	9.140	0.963	1351.777	-	25.576	102.738	1300.625	1300.515
	Node 3		10.000	0.280	0.000	0.000	0.000	0.000	7.458	0.894	10.034	0.959	1346.385	-	20.183	20.449	1306.018	1306.018

Straight	Node 4	0.500 L	12.075	0.280	0.000	0.000	0.000	0.000	7.458	2.075	12.109	0.955	1340.809	-	14.608	36.096	1311.594	1311.594
Straight	Node 5		14.150	0.280	0.000	0.000	0.000	0.000	7.458	2.075	14.184	0.951	1335.256	-	9.055	24.550	1317.146	1317.146
Parabolic		0.625 L	15.044	0.286	0.000	0.745	0.000	0.745	8.203	0.894	15.077	0.947	1329.929	1326.201	3.728	5.712	1322.473	1322.362
		0.750 L	18.013	0.389	0.000	3.216	0.000	3.216	10.674	2.971	18.048	0.934	1312.395	-	0.000	5.533	1312.395	1310.328
		0.867 L	20.800	0.602	0.000	5.525	0.000	5.525	12.983	2.796	20.844	0.923	1296.166	-	0.000	0.000	1296.166	1290.144
		0.931 L	22.300	0.763	0.000	6.761	0.000	6.761	14.219	1.509	22.352	0.917	1287.531	-	0.000	0.000	1287.531	1278.579
Inclined	Node 6		23.150	0.869	0.000	7.458	0.000	7.458	14.916	0.857	23.209	0.913	1282.670	-	0.000	0.000	1282.670	1271.819
		1.000 L	23.950	0.974	0.000	7.458	0.000	7.458	14.916	0.807	24.016	0.912	1280.602	-	0.000	0.000	1280.602	1269.768
	Node 7	Dead end	24.150	1.000	0.000	7.458	0.000	7.458	14.916	0.202	24.217	0.911	1280.085	-	0.000	0.000	1280.085	1269.256

577.400

Total Length of Cable 4 = 24.217 m



Calculation of Cable extension for Cable No. 4

Average prestress force after friction loss = 1341.56 kN

Length of cable = 24.217 m

Cable extension = 168.81 mm

Calculation of Slip loss:

Assuming Slip travel upto 15.6749 m from anchorage end

Prestress force at null point = 1326.20 kN (After friction loss by Interpolation)

Assume Slip = 6 mm

Half shaded area $s \times E_s \times A / 2$ = 577.40 kN

area of force diagram above null point = 577.40 kN

equat Half shaded area - area of force diagram above null point to zero = 0.00

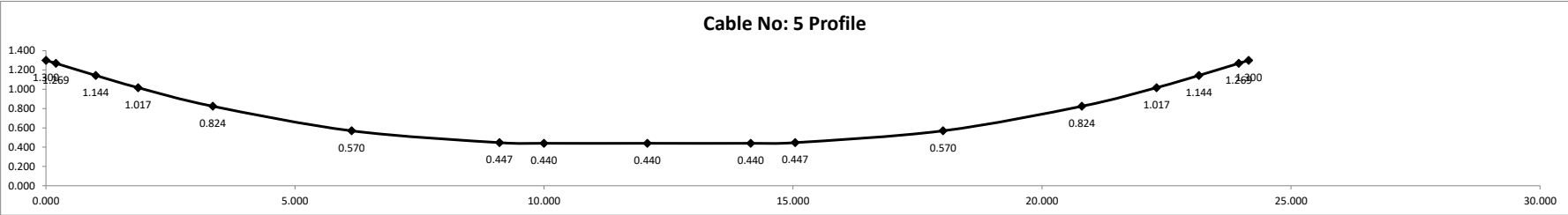
Summary of Forces after Friction and Slip Losses: Cable 4				Non-Composite Section		Composite Section	
Sections	p ' hor (kN)	p ' ver (kN)	c.g. of cable from soffit (m)	c.g. of Non-Composite Section from soffit	Eccentricity of cable	c.g. of Composite Section from soffit	Eccentricity of cable
Jacking End	1237.432	161.991	1.000	0.831	-0.169	0.436	-0.564
0.000 L	1237.993	162.065	0.974	0.831	-0.143	0.436	-0.538
0.069 L	1247.377	147.871	0.763	0.829	0.066	0.390	-0.374
0.133 L	1259.534	121.831	0.602	0.819	0.217	0.338	-0.263
0.250 L	1280.545	71.943	0.389	0.819	0.430	0.338	-0.050
0.375 L	1300.515	16.907	0.286	0.819	0.533	0.338	0.052
0.500 L	1311.594	0.000	0.280	0.819	0.539	0.338	0.058
0.625 L	1322.36	17.19	0.29	0.819	0.533	0.338	0.052
0.750 L	1310.33	73.62	0.39	0.819	0.430	0.338	-0.050
0.867 L	1290.14	124.79	0.60	0.819	0.217	0.338	-0.263
0.931 L	1278.58	151.57	0.76	0.829	0.066	0.390	-0.374
1.000 L	1269.77	166.22	0.97	0.831	-0.143	0.436	-0.538
Dead end	1269.26	166.16	1.00	0.831	-0.169	0.436	-0.564

Prestress after Friction and Slip Loss: Cable 5

Cable No	Cable Profile	Sections	x (m)	y(m)	z (m)	θ _v (deg)	θ _H (deg)	θ _S (deg)	Cum. θ (deg)	L _{INC} (m)	X _{PRO} (m)	Friction coeff.	Force after Friction loss , p (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p' (kN)	p' _{hor} (kN)
Inclined	Node 1	Jacking End	0.000	1.300	0.000	8.887	0.000	8.887	0.000	0.000	0.000	1.000	1404.413	-	82.100		1240.212	1225.323
			0.000 L	0.200	1.269	8.887	0.000	8.887	0.000	0.202	0.202	1.000	1403.844	-	81.532	16.363	1240.780	1225.884
			Node 2	1.000	1.144	8.887	0.000	8.887	0.000	0.810	1.012	0.998	1401.572	-	79.260	64.317	1243.052	1228.129
Parabolic			0.069 L	1.850	1.017	8.059	0.000	8.059	0.828	0.859	1.872	0.994	1395.733	-	73.421	64.890	1248.891	1236.556
			0.133 L	3.350	0.824	6.590	0.000	6.590	2.297	1.512	3.384	0.987	1385.467	-	63.155	102.432	1259.157	1250.836
			0.250 L	6.138	0.570	3.839	0.000	3.839	5.048	2.799	6.183	0.973	1366.531	-	44.219	149.653	1278.093	1275.225
			0.375 L	9.106	0.447	0.890	0.000	0.890	7.997	2.971	9.154	0.959	1346.598	-	24.286	101.688	1298.026	1297.869
			Node 3	10.000	0.440	0.000	0.000	0.000	8.887	0.894	10.048	0.955	1340.650	-	18.338	19.048	1303.974	1303.974
Straight	Node 4	0.500 L	12.075	0.440	0.000	0.000	0.000	0.000	8.887	2.075	12.123	0.951	1335.098	-	12.786	32.291	1309.526	1309.526
			Node 5	14.150	0.440	0.000	0.000	0.000	8.887	2.075	14.198	0.947	1329.569	-	7.257	20.794	1315.055	1315.055
Parabolic			0.625 L	15.044	0.447	0.890	0.000	0.890	9.777	0.894	15.092	0.943	1323.696	1322.312	1.384	3.861	1320.928	1320.769
			0.750 L	18.013	0.570	3.839	0.000	3.839	12.726	2.971	18.063	0.929	1304.388	-	0.000	2.054	1304.388	1301.460
			0.867 L	20.800	0.824	6.590	0.000	6.590	15.477	2.799	20.862	0.916	1286.560	-	0.000	0.000	1286.560	1278.058
			0.931 L	22.300	1.017	8.059	0.000	8.059	16.946	1.512	22.375	0.909	1277.097	-	0.000	0.000	1277.097	1264.484
Inclined	Node 6		23.150	1.144	0.000	8.887	0.000	8.887	17.774	0.859	23.234	0.906	1271.776	-	0.000	0.000	1271.776	1256.509
			1.000 L	23.950	1.269	8.887	0.000	8.887	17.774	0.810	24.044	0.904	1269.718	-	0.000	0.000	1269.718	1254.475
			Node 7	24.150	1.300	0.000	8.887	0.000	8.887	17.774	0.202	0.904	1269.204	-	0.000	0.000	1269.204	1253.968

577.392

Total Length of Cable 5	=	24.246 m



Summary of Forces after Friction and Slip Losses: Cable 5

				Non-Composite Section		Composite Section	
Sections	p' _{hor} (kN)	p' _{ver} (kN)	c.g. of cable from soffit (m)	c.g. of Non-Composite Section from soffit	Eccentricity of cable	c.g. of Composite Section from soffit	Eccentricity of cable
Jacking End	1225.323	191.596	1.300	0.831	-0.469	0.436	-0.864
0.000 L	1225.884	191.684	1.269	0.831	-0.437	0.436	-0.833
0.069 L	1236.556	175.091	1.017	0.829	-0.188	0.390	-0.627

Calculation of Cable extension

for Cable No. 5

Average prestress force after friction loss = 1336.00 kN

Length of cable = 24.246 m

Cable extension = 168.30 mm

Calculation of Slip loss:

Assuming Slip travel upto 15.2565 m from anchorage end

Prestress force at null point = 1322.31 kN (After friction loss by Interpolation)

Assume Slip , s = 6 mm

Half shaded area $s \times E_s \times A / 2$ = 577.40 kN

area of force diagram above null point = 577.39 kN

equat Half shaded area - area of force diagram above null point to zero

= 0.00

0.133 L	1250.836	144.516	0.824	0.819	-0.005	0.338	-0.486
0.250 L	1275.225	85.575	0.570	0.819	0.249	0.338	-0.231
0.375 L	1297.869	20.153	0.447	0.819	0.372	0.338	-0.109
0.500 L	1309.526	0.000	0.440	0.819	0.379	0.338	-0.102
0.625 L	1320.769	20.509	0.447	0.819	0.372	0.338	-0.109
0.750 L	1301.460	87.336	0.570	0.819	0.249	0.338	-0.231
0.867 L	1278.058	147.661	0.824	0.819	-0.005	0.338	-0.486
0.931 L	1264.484	179.046	1.017	0.829	-0.188	0.390	-0.627
1.000 L	1254.475	196.154	1.269	0.831	-0.437	0.436	-0.833
Dead end	1253.968	196.075	1.300	0.831	-0.469	0.436	-0.864

COORDINATES OF PRESTRESSING CABLES

Cable Nodes	Cable 1		
	x	y	z
Node 1	0.000	0.300	0.150
Node 2	1.000	0.235	0.150
Node 3	4.500	0.120	0.150
Node 4	12.075	0.120	0.150
Node 5	19.650	0.120	0.150
Node 6	23.150	0.235	0.150
Node 7	24.150	0.300	0.150

Cable Nodes	Cable 2		
	x	y	z
Node 1	0.000	0.300	-0.150
Node 2	1.000	0.235	-0.150
Node 3	4.500	0.120	-0.150
Node 4	12.075	0.120	-0.150
Node 5	19.650	0.120	-0.150
Node 6	23.150	0.235	-0.150
Node 7	24.150	0.300	-0.150

Cable Nodes	Cable 3		
	x	y	z
Node 1	0.000	0.700	0.000
Node 2	1.000	0.595	0.000
Node 3	10.000	0.120	0.000
Node 4	12.075	0.120	0.000
Node 5	14.150	0.120	0.000
Node 6	23.150	0.595	0.000
Node 7	24.150	0.700	0.000

Cable Nodes	Cable 4		
	x	y	z
Node 1	0.000	1.000	0.000
Node 2	1.000	0.869	0.000
Node 3	10.000	0.280	0.000
Node 4	12.075	0.280	0.000
Node 5	14.150	0.280	0.000
Node 6	23.150	0.869	0.000
Node 7	24.150	1.000	0.000

Cable Nodes	Cable 5		
	x	y	z
Node 1	0.000	1.300	0.000
Node 2	1.000	1.144	0.000
Node 3	10.000	0.440	0.000
Node 4	12.075	0.440	0.000
Node 5	14.150	0.440	0.000
Node 6	23.150	1.144	0.000
Node 7	24.150	1.300	0.000

Summary of Prestressing Forces and Cable Eccentricity

After Friction and Slip Loss at Different Sections:

at Jacking End

Cable No	Stage	p_{hor} (kN)	p_{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1388.615	90.891	0.300	0.831	0.531	0.436	0.136
Cable 2	2	1388.614	90.891	0.300	0.831	0.531	0.436	0.136
Cable 3	1	1248.922	131.705	0.700	0.831	0.131	0.436	-0.264
Cable 4	2	1237.432	161.991	1.000	0.831	-0.169	0.436	-0.564
Cable 5	1	1225.323	191.596	1.300	0.831	-0.469	0.436	-0.864

Stage-I:

Total Horizontal Prestressing Force at section Jacking End = 2474.245 kN

Total Vertical Prestressing Force at section Jacking End = 323.300 kN

Non Composite Section

C.g. of cables at Jacking End from soffit = 0.997 m

Eccentricity of cables at Jacking End = -0.166 m

Composite Section

C.g. of cables at Jacking End from soffit = 0.997 m

Eccentricity of cables at Jacking End = -0.561 m

Stage-II:

Total Horizontal Prestressing Force at section Jacking End = 4014.661 kN

Total Vertical Prestressing Force at section Jacking End = 343.773 kN

Non Composite Section

C.g. of cables at 'Jacking End' from soffit = 0.516 m

Eccentricity of cables at 'Jacking End' = 0.315 m

Composite Section

C.g. of cables at 'Jacking End' from soffit = 0.516 m

Eccentricity of cables at 'Jacking End' = -0.080 m

at 0.000L

Cable No	Stage	p_{hor} (kN)	p_{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1389.232	90.931	0.287	0.831	0.544	0.436	0.149
Cable 2	2	1389.232	90.932	0.287	0.831	0.544	0.436	0.149
Cable 3	1	1249.484	131.764	0.679	0.831	0.152	0.436	-0.243
Cable 4	2	1237.993	162.065	0.974	0.831	-0.143	0.436	-0.538
Cable 5	1	1225.884	191.684	1.269	0.831	-0.437	0.436	-0.833

Stage-I:

Total Horizontal Prestressing Force at section 0 = 2475.368 kN

Total Vertical Prestressing Force at section 0 = 323.448 kN

Non Composite Section

C.g. of cables at 0 from soffit = 0.971 m

Eccentricity of cables at 0 = -0.140 m

Composite Section

C.g. of cables at 0 from soffit = 0.971 m

Eccentricity of cables at 0 = -0.535 m

Stage-II:

Total Horizontal Prestressing Force at section 0 = 4016.458 kN

Total Vertical Prestressing Force at section 0 = 343.927 kN

Non Composite Section

C.g. of cables at 0 from soffit	=	0.499	m
Eccentricity of cables at 0	=	0.333	m
Composite Section			
C.g. of cables at 0 from soffit	=	0.499	m
Eccentricity of cables at 0	=	-0.063	m

at 0.069 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1399.724	69.368	0.186	0.829	0.643	0.390	0.204
Cable 2	2	1399.724	69.368	0.186	0.829	0.643	0.390	0.204
Cable 3	1	1257.684	120.102	0.509	0.829	0.320	0.390	-0.120
Cable 4	2	1247.377	147.871	0.763	0.829	0.066	0.390	-0.374
Cable 5	1	1236.556	175.091	1.017	0.829	-0.188	0.390	-0.627

Stage-I:

Total Horizontal Prestressing Force at section 0.069L = 2494.240 kN

Total Vertical Prestressing Force at section 0.069L = 295.194 kN

Non Composite Section

C.g. of cables at 0.069L from soffit = 0.761 m

Eccentricity of cables at 0.069L = 0.068 m

Composite Section

C.g. of cables at 0.069L from soffit = 0.761 m

Eccentricity of cables at 0.069L = -0.371 m

Stage-II:

Total Horizontal Prestressing Force at section 0.069L = 4046.826 kN

Total Vertical Prestressing Force at section 0.069L = 286.607 kN

Non Composite Section

C.g. of cables at 0.069L from soffit = 0.364 m

Eccentricity of cables at 0.069L = 0.465 m

Composite Section

C.g. of cables at 0.069L from soffit = 0.364 m

Eccentricity of cables at 0.069L = 0.026 m

at 0.133 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1412.987	30.389	0.132	0.819	0.686	0.338	0.206
Cable 2	2	1412.987	30.388	0.132	0.819	0.686	0.338	0.206
Cable 3	1	1267.886	98.793	0.379	0.819	0.440	0.338	-0.041
Cable 4	2	1259.534	121.831	0.602	0.819	0.217	0.338	-0.263
Cable 5	1	1250.836	144.516	0.824	0.819	-0.005	0.338	-0.486

Stage-I:

Total Horizontal Prestressing Force at section 0.133L = 2518.722 kN

Total Vertical Prestressing Force at section 0.133L = 243.308 kN

Non Composite Section

C.g. of cables at 0.133L from soffit = 0.600 m

Eccentricity of cables at 0.133L = 0.219 m

Composite Section

C.g. of cables at 0.133L from soffit = 0.600 m

Eccentricity of cables at 0.133L = -0.262 m

Stage-II:

Total Horizontal Prestressing Force at section 0.133L = 4085.509 kN

Total Vertical Prestressing Force at section 0.133L = 182.608 kN

Non Composite Section

C.g. of cables at 0.133L from soffit = 0.277 m
 Eccentricity of cables at 0.133L = 0.542 m

Composite Section

C.g. of cables at 0.133L from soffit = 0.277 m
 Eccentricity of cables at 0.133L = 0.061 m

at 0.250 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1427.309	0.000	0.120	0.819	0.699	0.338	0.218
Cable 2	2	1427.309	0.000	0.120	0.819	0.699	0.338	0.218
Cable 3	1	1285.763	58.191	0.207	0.819	0.611	0.338	0.131
Cable 4	2	1280.545	71.943	0.389	0.819	0.430	0.338	-0.050
Cable 5	1	1275.225	85.575	0.570	0.819	0.249	0.338	-0.231

Stage-I:

Total Horizontal Prestressing Force at section 0.25L = 2560.988 kN
 Total Vertical Prestressing Force at section 0.25L = 143.766 kN

Non Composite Section

C.g. of cables at '0.25L from soffit = 0.388 m
 Eccentricity of cables at 0.25L = 0.431 m

Composite Section

C.g. of cables at 0.25L from soffit = 0.388 m
 Eccentricity of cables at 0.25L = -0.050 m

Stage-II:

Total Horizontal Prestressing Force at section 0.25L = 4135.163 kN
 Total Vertical Prestressing Force at section 0.25L = 71.943 kN

Non Composite Section

C.g. of cables at 0.25L from soffit = 0.203 m
 Eccentricity of cables at 0.25L = 0.616 m

Composite Section

C.g. of cables at 0.25L from soffit = 0.203 m
 Eccentricity of cables at 0.25L = 0.135 m

at 0.375 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2	1436.243	0.000	0.120	0.819	0.699	0.338	0.218
Cable 2	2	1436.243	0.000	0.120	0.819	0.699	0.338	0.218
Cable 3	1	1303.181	13.647	0.125	0.819	0.694	0.338	0.213
Cable 4	2	1300.515	16.907	0.286	0.819	0.533	0.338	0.052
Cable 5	1	1297.869	20.153	0.447	0.819	0.372	0.338	-0.109

Stage-I:

Total Horizontal Prestressing Force at section 0.375L = 2601.051 kN
 Total Vertical Prestressing Force at section 0.375L = 33.800 kN

Non Composite Section

C.g. of cables at 0.375L from soffit = 0.285 m
 Eccentricity of cables at 0.375L = 0.533 m

Composite Section

C.g. of cables at '0.375L' from soffit = 0.285 m
 Eccentricity of cables at '0.375L' = 0.053 m

Stage-II:

Total Horizontal Prestressing Force at section '0.375L'	=	4173.001	kN
Total Vertical Prestressing Force at section '0.375L'	=	16.907	kN
Non Composite Section			
C.g. of cables at '0.375L' from soffit	=	0.172	m
Eccentricity of cables at '0.375L'	=	0.647	m
Composite Section			
C.g. of cables at '0.375L' from soffit	=	0.172	m
Eccentricity of cables at '0.375L'	=	0.166	m

at 0.500 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1445.124	0.000	0.120	0.819	0.699	0.338	0.218
Cable 2	2.000	1445.124	0.000	0.120	0.819	0.699	0.338	0.218
Cable 3	1.000	1313.686	0.000	0.120	0.819	0.699	0.338	0.218
Cable 4	2.000	1311.594	0.000	0.280	0.819	0.539	0.338	0.058
Cable 5	1.000	1309.526	0.000	0.440	0.819	0.379	0.338	-0.102

Stage-I:

Total Horizontal Prestressing Force at section '0.5L'	=	2623.212	kN
Total Vertical Prestressing Force at section '0.5L'	=	0.000	kN

Non Composite Section

C.g. of cables at '0.5L' from soffit	=	0.280	m
Eccentricity of cables at '0.5L'	=	0.539	m

Composite Section

C.g. of cables at '0.5L' from soffit	=	0.280	m
Eccentricity of cables at '0.5L'	=	0.058	m

Stage-II:

Total Horizontal Prestressing Force at section '0.5L'	=	4201.841	kN
Total Vertical Prestressing Force at section '0.5L'	=	0.000	kN

Non Composite Section

C.g. of cables at '0.5L' from soffit	=	0.170	m
Eccentricity of cables at '0.5L'	=	0.649	m

Composite Section

C.g. of cables at '0.5L' from soffit	=	0.170	m
Eccentricity of cables at '0.5L'	=	0.168	m

at 0.625 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1453.952	0.000	0.120	0.819	0.699	0.338	0.218
Cable 2	2.000	1453.952	0.000	0.120	0.819	0.699	0.338	0.218
Cable 3	1.000	1323.965	13.865	0.125	0.819	0.694	0.338	0.213
Cable 4	2.000	1322.362	17.191	0.286	0.819	0.533	0.338	0.052
Cable 5	1.000	1320.769	20.509	0.447	0.819	0.372	0.338	-0.109

Stage-I:

Total Horizontal Prestressing Force at section '0.625L'	=	2644.735	kN
Total Vertical Prestressing Force at section '0.625L'	=	34.373	kN

Non Composite Section

C.g. of cables at '0.625L' from soffit	=	0.286	m
Eccentricity of cables at '0.625L'	=	0.533	m

Composite Section

C.g. of cables at '0.625L' from soffit	=	0.286	m
Eccentricity of cables at '0.625L'	=	0.053	m

Stage-II:

Total Horizontal Prestressing Force at section '0.625L' = 4230.266 kN

Total Vertical Prestressing Force at section '0.625L' = 17.191 kN

Non Composite Section

C.g. of cables at '0.625L' from soffit = 0.172 m

Eccentricity of cables at '0.625L' = 0.647 m

Composite Section

C.g. of cables at '0.625L' fromsoffit = 0.172 m

Eccentricity of cables at '0.625L' = 0.166 m

at 0.750 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1462.729	0.000	0.120	0.819	0.699	0.338	0.218
Cable 2	2.000	1462.728	0.000	0.120	0.819	0.699	0.338	0.218
Cable 3	1.000	1319.133	59.701	0.207	0.819	0.611	0.338	0.131
Cable 4	2.000	1310.328	73.617	0.389	0.819	0.430	0.338	-0.050
Cable 5	1.000	1301.460	87.336	0.570	0.819	0.249	0.338	-0.231

Stage-I:

Total Horizontal Prestressing Force at section '0.75L' = 2620.594 kN

Total Vertical Prestressing Force at section '0.75L' = 147.037 kN

Non Composite Section

C.g. of cables at '0.75L' from soffit = 0.387 m

Eccentricity of cables at '0.75L' = 0.431 m

Composite Section

C.g. of cables at '0.75L' from soffit = 0.387 m

Eccentricity of cables at '0.75L' = -0.049 m

Stage-II:

Total Horizontal Prestressing Force at section '0.75L' = 4235.785 kN

Total Vertical Prestressing Force at section '0.75L' = 73.617 kN

Non Composite Section

C.g. of cables at '0.75L' from soffit = 0.203 m

Eccentricity of cables at '0.75L' = 0.616 m

Composite Section

C.g. of cables at '0.75L' fromsoffit = 0.203 m

Eccentricity of cables at '0.75L' = 0.135 m

at 0.867 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1459.833	31.396	0.132	0.819	0.686	0.338	0.206
Cable 2	2.000	1459.833	31.396	0.132	0.819	0.686	0.338	0.206
Cable 3	1.000	1301.939	101.446	0.379	0.819	0.440	0.338	-0.041
Cable 4	2.000	1290.144	124.792	0.602	0.819	0.217	0.338	-0.263
Cable 5	1.000	1278.058	147.661	0.824	0.819	-0.005	0.338	-0.486

Stage-I:

Total Horizontal Prestressing Force at section '0.867L' = 2579.998 kN

Total Vertical Prestressing Force at section '0.867L' = 249.107 kN

Non Composite Section

C.g. of cables at '0.867L' from soffit = 0.600 m

Eccentricity of cables at '0.867L' = 0.219 m

Composite Section

C.g. of cables at '0.867L' from soffit = 0.600 m

Eccentricity of cables at '0.867L' = -0.261 m

Stage-II:

Total Horizontal Prestressing Force at section '0.867L' = 4209.810 kN

Total Vertical Prestressing Force at section '0.867L' = 187.584 kN

Non Composite Section

C.g. of cables at '0.867L' from soffit = 0.276 m

Eccentricity of cables at '0.867L' = 0.543 m

Composite Section

C.g. of cables at '0.867L' fromsoffit = 0.276 m

Eccentricity of cables at '0.867L' = 0.062 m

at 0.931 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1447.101	71.716	0.186	0.829	0.643	0.390	0.204
Cable 2	2.000	1447.101	71.716	0.186	0.829	0.643	0.390	0.204
Cable 3	1.000	1292.225	123.401	0.509	0.829	0.320	0.390	-0.120
Cable 4	2.000	1278.579	151.570	0.763	0.829	0.066	0.390	-0.374
Cable 5	1.000	1264.484	179.046	1.017	0.829	-0.188	0.390	-0.627

Stage-I:

Total Horizontal Prestressing Force at section '0.931L' = 2556.709 kN

Total Vertical Prestressing Force at section '0.931L' = 302.447 kN

Non Composite Section

C.g. of cables at '0.931L' from soffit = 0.760 m

Eccentricity of cables at '0.931L' = 0.069 m

Composite Section

C.g. of cables at '0.931L' from soffit = 0.760 m

Eccentricity of cables at '0.931L' = -0.371 m

Stage-II:

Total Horizontal Prestressing Force at section '0.931L' = 4172.780 kN

Total Vertical Prestressing Force at section '0.931L' = 295.002 kN

Non Composite Section

C.g. of cables at '0.931L' from soffit = 0.363 m

Eccentricity of cables at '0.931L' = 0.466 m

Composite Section

C.g. of cables at '0.931L' fromsoffit = 0.363 m

Eccentricity of cables at '0.931L' = 0.027 m

at 1.000 L

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1437.135	94.066	0.287	0.831	0.544	0.436	0.149
Cable 2	2.000	1437.135	94.067	0.287	0.831	0.544	0.436	0.149
Cable 3	1.000	1284.515	135.458	0.679	0.831	0.152	0.436	-0.243
Cable 4	2.000	1269.768	166.224	0.974	0.831	-0.143	0.436	-0.538
Cable 5	1.000	1254.475	196.154	1.269	0.831	-0.437	0.436	-0.833

Stage-I:

Total Horizontal Prestressing Force at section '1L' = 2538.991 kN

Total Vertical Prestressing Force at section '1L' = 331.612 kN

Non Composite Section

C.g. of cables at '1L' from soffit = 0.970 m

Eccentricity of cables at '1L' = -0.139 m

Composite Section

C.g. of cables at '1L' from soffit	=	0.970	m
Eccentricity of cables at '1L'	=	-0.535	m

Stage-II:

Total Horizontal Prestressing Force at section '1L'	=	4144.038	kN
Total Vertical Prestressing Force at section '1L'	=	354.358	kN

Non Composite Section

C.g. of cables at '1L' from soffit	=	0.497	m
Eccentricity of cables at '1L'	=	0.334	m

Composite Section

C.g. of cables at '1L' from soffit	=	0.497	m
Eccentricity of cables at '1L'	=	-0.062	m

at Dead end

Cable No	Stage	p _{hor} (kN)	p _{ver} (kN)	c.g. of cable from soffit (m)	Non Composite Section		Composite Section	
					C.G. of Non Composite Section from soffit	Eccentricity of cable (ec) (m)	C.G. of Composite Section from soffit	Eccentricity of cable (ec) (m)
Cable 1	2.000	1436.559	94.029	0.300	0.831	0.531	0.436	0.136
Cable 2	2.000	1436.559	94.029	0.300	0.831	0.531	0.436	0.136
Cable 3	1.000	1283.999	135.404	0.700	0.831	0.131	0.436	-0.264
Cable 4	2.000	1269.256	166.157	1.000	0.831	-0.169	0.436	-0.564
Cable 5	1.000	1253.968	196.075	1.300	0.831	-0.469	0.436	-0.864

Stage-I:

Total Horizontal Prestressing Force at section Dead end	=	2537.966	kN
Total Vertical Prestressing Force at section Dead end	=	331.478	kN

Non Composite Section

C.g. of cables at 'Dead end' from soffit	=	0.996	m
Eccentricity of cables at 'Dead end'	=	-0.165	m

Composite Section

C.g. of cables at 'Dead end' from soffit	=	0.996	m
Eccentricity of cables at 'Dead end'	=	-0.561	m

Stage-II:

Total Horizontal Prestressing Force at section 'Dead end'	=	4142.374	kN
Total Vertical Prestressing Force at section 'Dead end'	=	354.215	kN

Non Composite Section

C.g. of cables at 'Dead end' from soffit	=	0.514	m
Eccentricity of cables at 'Dead end'	=	0.317	m

Composite Section

C.g. of cables at 'Dead end' from soffit	=	0.514	m
Eccentricity of cables at 'Dead end'	=	-0.079	m

ANALYSIS OF OUTER LONG. GIRDER FOR TEMPERATURE

Material Properties:

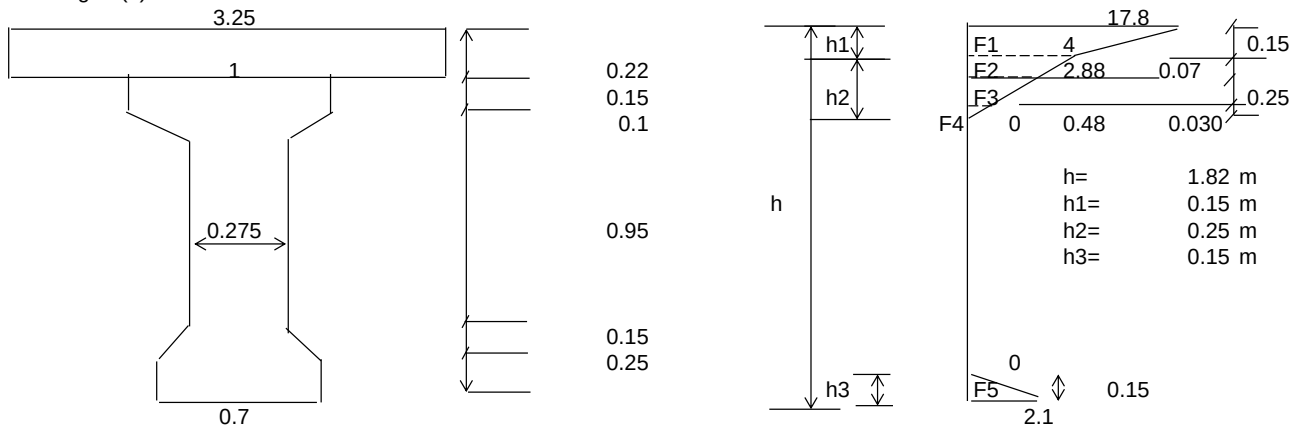
Concrete Grade for the superstructure	=	M 45
E_c for the concrete	=	3.40E+04 N/mm ²
α	=	Coefficient of Thermal expansion as per Clause 215.4 , IRC:6-2017
	=	1.20E-05 / °C

Sectional Properties of Composite Section

A	=	1.438 m ²	Z _{ts}	=	0.9141 m ³
y _{ts}	=	0.558 m	Z _{tg}	=	1.5089 m ³
y _{tg}	=	0.338 m	Z _{bg}	=	0.4044 m ³
y _{bg}	=	1.262 m			

Case 1 : Rising Temperature Gradient at MidSpan

Refer Fig 10(a) of IRC:6-2014



Zone	Δt (°C)	B (m)	D (m)	A_t (m ²)	$F_t = \text{Temp. Force (kN)}$	y (m) from top	y' = y _{ts} - y _t (m)	Mt = $F_t \times y'$ (kNm)
F1	10.900	3.25	0.15	0.488	2168.01	0.059	0.499	1081.794
F2	3.440	3.25	0.07	0.228	319.30	0.183	0.375	119.755
F3	1.680	1.00	0.15	0.150	102.82	0.277	0.281	28.892
F4	0.240	0.64	0.03	0.019	1.87	0.380	0.178	0.334
F5	1.050	0.70	0.15	0.105	44.98	1.770	-1.212	-54.511
SUM				0.989	2636.982			1176.264

Stress due to temperature gradient in restrained condition $= E\alpha T$

At the top of deck slab	σ_{ts}	=	7.26 N/mm ²
At the top of girder	σ_{tg}	=	1.18 N/mm ²
At the bottom of girder	σ_{bg}	=	0.86 N/mm ²

Direct tension over entire section

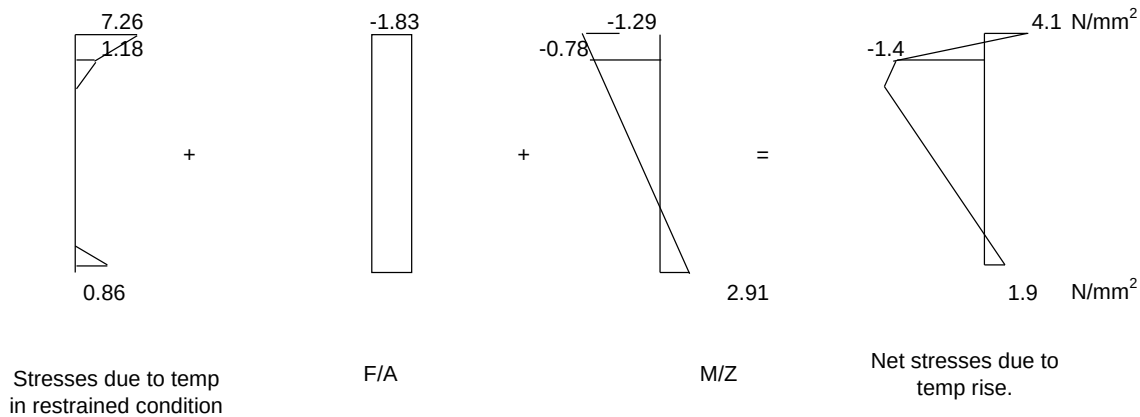
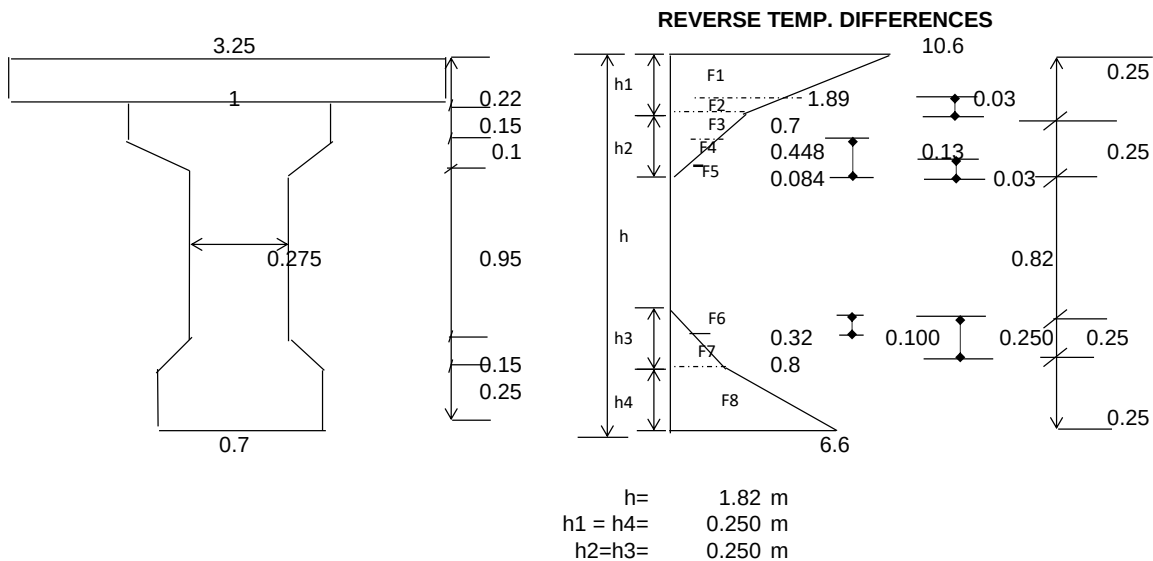
Force/Area	$(-F_t/A)$	=	-1.83 N/mm ²

Bending Stress Due to Direct Tension

At the top of deck slab	$(-M_t / Z_{ts})$	=	-1.29 N/mm ²
At the top of girder	$(-M_t / Z_{tg})$	=	-0.78 N/mm ²
At the bottom of girder	(M_t / Z_{bg})	=	2.91 N/mm ²

Final Temperature Stress:

At the top of deck slab	=	4.14 N/mm ²	σ_{ts}	(-Ft/At)	(- Mt / Zts)
At the Bottom of Deck Slab	=	-1.44 N/mm ²	σ_{tg}	(-Ft/At)	(- Mt / Ztg)
At the bottom of girder	=	1.93 N/mm ²	σ_{bg}	(-Ft/At)	+(Mt / Zbg)

**Case 2 : Falling Temperature Gradient at MidSpan**

Zone	Δt (°C)	B (m)	D (m)	At (m ²)	Ft = Temp. Force (kN)	y (m) from top	y' = yts - yt (m)	Mt = Ft x y' (kNm)
F1	6.244	3.25	0.22	0.715	1821.50	0.084	0.474	862.907
F2	1.294	3.25	0.03	0.098	51.48	0.233	0.325	16.753
F3	0.574	0.15	0.12	0.018	4.22	0.306	0.253	1.065
F4	0.266	0.64	0.22	0.140	15.22	0.455	0.103	1.571
F5	0.042	0.28	0.03	0.008	0.14	0.48	0.078	0.011
F6	0.160	0.28	0.100	0.028	1.80	1.387	-0.829	-1.487
F7	0.560	0.49	0.250	0.122	27.85	1.484	-0.926	-25.789
F8	3.700	0.70	0.25	0.175	264.18	1.728	-1.170	-308.959
SUM				1.303	2186.374			546.071

Stress due to temperature gradient in restrained condition = $E\alpha T$

At the top of deck slab	σ_{ts}	=	-4.32 N/mm ²
At the top of girder	σ_{tg}	=	-0.77 N/mm ²
At the bottom of girder	σ_{bg}	=	-2.69 N/mm ²

Direct tension over entire section

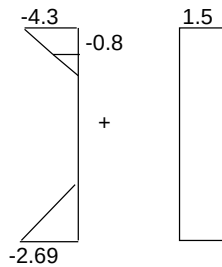
Force/Area	(Ft/A)	=	1.52 N/mm ²

Bending Stress Due to Direct Tension

At the top of deck slab	(Mt / Zts)	=	0.60 N/mm ²
At the top of girder	(Mt / Ztg)	=	0.36 N/mm ²
At the bottom of girder	(- Mt / Zbg)	=	-1.35 N/mm ²

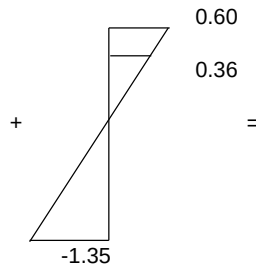
Final Temperature Stress:

At the top of deck slab	=	-2.21 N/mm ²	σ_{ts}	+ (Ft/At) + (Mt / Zts)
At the Bottom of Deck Slab	=	1.11 N/mm ²	σ_{tg}	+ (Ft/At) + (Mt / Ztg)
At the bottom of girder	=	-2.52 N/mm ²	σ_{bg}	+ (Ft/At) - (Mt / Zbg)

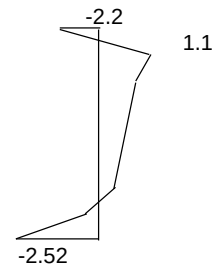


Stress due to temp in restrained condition

F/A



M/Z



Net stresses due to temp fall

Calculation of Stresses due to Differential Shrinkage

Age of precast girder at the time of casting of deck slab	=	35	days
Differential Shrinkage strain (ϵ_{diff}) at 35 days	=	6.08E-05	
Creep Coefficient ϕ (t^∞, t_0)	=	1.84	
Creep reduction factor (refer eq. 7.3 of IRC:112-2011) α	=	$\frac{1 - e^{-\phi}}{\phi}$	= 0.457
Effective area of Deck Slab A_{cf}	=	715000	mm ²
Modulus of elasticity of concrete in Deck slab E_{cf}	=	3.400E+04	N/mm ²

Section Properties of Composite Section

Total Depth of Section	=	D	=	1.820	m
Thickness of Deck Slab	=	Ds	=	0.220	m
Area of composite section	=	Ac	=	1.438	m ²
Distance of c.g. of section from top of slab	=	Yts	=	0.558	m
Distance of c.g. of section from top of girder	=	Ytg	=	0.338	m
Distance of c.g. of section from bottom of girder	=	Ybg	=	1.262	m
Sectional modulus at top of slab	=	Zts	=	0.914	m ³
sectional modulus at top of girder	=	Ztg	=	1.509	m ³
sectional modulus at bottom of girder	=	Zbg	=	0.404	m ³

Stresses in Outer Girder Due to Differential Shrinkage Effect:

Step (1) Stress due to Tensile Force in deck slab

$$\sigma_{\text{Tension}} = 3.400\text{E}+04 \times -6.08\text{E}-05 = -2.068\text{E}+00 \text{ N/mm}^2$$

Step (2) Calculate Restrained shrinkage stress

Restrained Force $F = \epsilon_{diff} \times E_{cf} \times A_{cf} \times \alpha$ acting at C.G. Of Deck Slab

$$F = 675.99 \text{ kN}$$

Restrained Moment @ c.g. of composite section

$$M = 675.99 \times (0.558 - 0.11) = 302.95 \text{ kNm}$$

$$\sigma_{\text{compression}} = \frac{F}{Ac} \pm \frac{M}{Z}$$

$\sigma_{\text{compression}}$ at top of deck slab	=	0.470	+	0.331	=	0.801	N/mm ²
$\sigma_{\text{compression}}$ at bottom of deck slab	=	0.470	+	0.201	=	0.671	N/mm ²
$\sigma_{\text{compression}}$ at top of girder	=	0.470	+	0.201	=	0.671	N/mm ²
$\sigma_{\text{compression}}$ at bottom of girder	=	0.470	+	-0.749	=	-0.279	N/mm ²

Step (3) : Net stresses at various levels of section

Step (1) + Step (2)

Total stress at top of deck slab	=	-2.068	+	0.801	=	-1.267	N/mm ²
Total stress at bottom of deck slab	=	-2.068	+	0.671	=	-1.398	N/mm ²
Total stress at top of girder	=	0.000	+	0.671	=	0.671	N/mm ²
Total stress at bottom of girder	=	0.000	+	-0.279	=	-0.279	N/mm ²

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter		-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

SECTION PROPERTIES

Properties of non-composite section: Precast Girder Only

Area of Precast Girder, A	m ²	1.171	1.171	0.936	0.723	0.723	0.723	0.723	0.723	0.723	0.723	0.936	1.171	1.171
Overall Depth of Girder , D	m	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600
Moment of Inertia, Iz	m ⁴	0.264	0.264	0.240	0.222	0.222	0.222	0.222	0.222	0.222	0.222	0.240	0.264	0.264
C.G. of section from top,Ytg	m	0.769	0.769	0.771	0.781	0.781	0.781	0.781	0.781	0.781	0.781	0.771	0.769	0.769
C.G. of section from bottom, Ybg	m	0.831	0.831	0.829	0.819	0.819	0.819	0.819	0.819	0.819	0.819	0.829	0.831	0.831
Sectional modulus at top, Ztg	m ³	0.343	0.343	0.311	0.284	0.284	0.284	0.284	0.284	0.284	0.284	0.311	0.343	0.343
Sectional modulus at bottom, Zbg	m ³	0.318	0.318	0.290	0.271	0.271	0.271	0.271	0.271	0.271	0.271	0.290	0.318	0.318
Perimeter of girder section,u	m	5.128	5.128	5.400	5.672	5.672	5.672	5.672	5.672	5.672	5.672	5.400	5.128	5.128
Notional size(mm) of the cross section , ho	mm	456.749	456.749	346.674	254.967	254.967	254.967	254.967	254.967	254.967	254.967	346.674	456.749	456.749
Ist Moment of area above and about N.A. of Comp. Section (Ay)p	m ³	0.086	0.086	0.069	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.069	0.086	0.086

Properties of Composite Section: Precast Girder + Deck Slab

Area of Composite Section, A	m ²	1.886	1.886	1.651	1.438	1.438	1.438	1.438	1.438	1.438	1.438	1.651	1.886	1.886
Overall Depth of Composite Section , D	m ²	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820
Moment of Inertia, Iz	m ⁴	0.610	0.610	0.558	0.510	0.510	0.510	0.510	0.510	0.510	0.510	0.558	0.610	0.610
C.G. of section from top,Yts	m	0.656	0.656	0.610	0.558	0.558	0.558	0.558	0.558	0.558	0.558	0.610	0.656	0.656
C.G. of section from top,Ytg	m	0.436	0.436	0.390	0.338	0.338	0.338	0.338	0.338	0.338	0.338	0.390	0.436	0.436
C.G. of section from bottom, Ybg	m	1.164	1.164	1.210	1.262	1.262	1.262	1.262	1.262	1.262	1.262	1.210	1.164	1.164
Sectional modulus at top, Zts	m ³	0.930	0.930	0.915	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.915	0.930	0.930
Sectional modulus at top, Ztg	m ³	1.400	1.400	1.431	1.509	1.509	1.509	1.509	1.509	1.509	1.509	1.431	1.400	1.400
Sectional modulus at bottom, Zbg	m ³	0.524	0.524	0.461	0.404	0.404	0.404	0.404	0.404	0.404	0.404	0.461	0.524	0.524
Perimeter of Composite girder section, u	m	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940
Notional size(mm) of the cross section , ho	mm	543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
Ist Moment of area above and about N.A. of Comp. Section (Ay)c	m ³	0.476	0.476	0.426	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.426	0.476	0.476

DETAILS OF PRESTRESSING CABLES AFTER FRICTION AND SLIP LOSS

STAGE -I CABLES

Nos of cables	nos	2	2	2	2	2	2	2	2	2	2	2	2	2
Area of Cables	m ²	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197	0.00197
c.g. of cables from soffit	m	0.997	0.971	0.761	0.600	0.388	0.285	0.280	0.286	0.387	0.600	0.760	0.970	0.996
Horizontal prestressing Force (P _{hor})	kN	2474.245	2475.368	2494.240	2518.722	2560.988	2601.051	2623.212	2644.735	2620.594	2579.998	2556.709	2538.991	2537.966
Vertical prestressing Force (P _{ver})	kN	323.300	323.448	295.194	243.308	143.766	33.800	0.000	34.373	147.037	249.107	302.447	331.612	331.478

Non-Composite Section

Ecc. of Cables from N.A.	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165

Composite Section

Ecc. of Cables from N.A.	m	-0.561	-0.535	-0.371	-0.262	-0.050	0.053	0.058	0.053	-0.049	-0.261	-0.371	-0.535	-0.561

STAGE -II CABLES

Nos of cables	nos	3	3	3	3	3	3	3	3	3	3	3	3	3
Area of Cables	m ²	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316	0.00316
c.g. of cables from soffit	m	0.516	0.499	0.364	0.277	0.203	0.172	0.170	0.172	0.203	0.276	0.363	0.497	0.514
Horizontal prestressing Force (P _{hor})	kN	4014.661	4016.458	4046.826	4085.509	4135.163	4173.001	4201.841	4230.266	4235.785	4209.810	4172.780	4144.038	4142.374
Vertical prestressing Force (P _{ver})	kN	343.773	343.927	286.607	182.608	71.943	16.907	0.000	17.191	73.617	187.584	295.002	354.358	354.215

Non-Composite Section

Ecc. of Cables from N.A.	m	0.315	0.333	0.465	0.542	0.616	0.647	0.649	0.647	0.616	0.543	0.466	0.334	0.317

Composite Section

Ecc. of Cables from N.A.	m	-0.080	-0.063	0.026	0.061	0.135	0.166	0.168	0.166	0.135	0.062	0.027	-0.062	-0.079

SHRINKAGE MODEL (micro strain)

(Refer Cl: 6.4.2.6 of IRC:112-2011)

$$\text{Total shrinkage strain } \epsilon_{cs} = \epsilon_{ca} + \epsilon_{cd}$$

where , ϵ_{ca} = Autogeneous shrinkage strain
 ϵ_{cd} = Drying shrinkage strain

Where

$$\epsilon_{cs}(t) = \beta_s(t, t_s) k_s \epsilon_{sh}$$

Eq. 6.14

where

$$\beta_s(t, t_s) = \frac{t - t_s}{t - t_s + 1}$$

$$k_s = \frac{f_{ck}}{f_{ck} + 1}$$

$$\begin{aligned} RH &= 80 \% \\ f_{ck} &= 45 \end{aligned}$$

$$\epsilon_{cd}(t) = \beta_{ds}(t) \epsilon_{cw}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \sqrt{h_o^3}}$$

Eq. 6.15

$$\text{where } \beta_{ds}(t) = 1 - \exp(-0.2 \sqrt{t})$$

Eq. 6.12

Eq. 6.13

ϵ_{cd}	=	252	from Table 6.8
ϵ_{cs}	=	65	from Table 6.6

ts = 3.000 days - age of concrete at the start of drying after curing

Items	Description	unit													
At age t (7 days)	notional size (mm) of the cross section, ho		456.749	456.749	346.674	254.967	254.967	254.967	254.967	254.967	254.967	254.967	346.674	456.749	456.749
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0101	0.0101	0.0153	0.0240	0.0240	0.0240	0.0240	0.0240	0.0240	0.0240	0.0153	0.0101	0.0101
	Coeff. Depends on notional size for shrinkage, kh		0.711	0.711	0.738	0.795	0.795	0.795	0.795	0.795	0.795	0.795	0.738	0.711	0.711
	Drying shrinkage strain, ϵ_{cd}		1.816	1.816	2.839	4.803	4.803	4.803	4.803	4.803	4.803	4.803	2.839	1.816	1.816
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411	0.411
	autogeneous shrinkage strain, ϵ_{as}		26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708	26.708
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	28.525	28.525	29.547	31.511	31.511	31.511	31.511	31.511	31.511	31.511	29.547	28.525	28.525
At age t (28 days)	notional size (mm) of the cross section, ho		543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0470	0.0470	0.0568	0.0690	0.0690	0.0690	0.0690	0.0690	0.0690	0.0690	0.0568	0.0470	0.0470
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.706	0.721	0.721	0.721	0.721	0.721	0.721	0.721	0.706	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		8.291	8.291	10.106	12.538	12.538	12.538	12.538	12.538	12.538	12.538	10.106	8.291	8.291
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653	0.653
	autogeneous shrinkage strain, ϵ_{as}		42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442	42.442
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	50.733	50.733	52.548	54.980	54.980	54.980	54.980	54.980	54.980	54.980	52.548	50.733	50.733
At age t (35 days)	notional size (mm) of the cross section, ho		543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0594	0.0594	0.0716	0.0866	0.0866	0.0866	0.0866	0.0866	0.0866	0.0866	0.0716	0.0594	0.0594
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.706	0.721	0.721	0.721	0.721	0.721	0.721	0.721	0.706	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		10.474	10.474	12.733	15.744	15.744	15.744	15.744	15.744	15.744	15.744	12.733	10.474	10.474
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694	0.694
	autogeneous shrinkage strain, ϵ_{as}		45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091	45.091
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	55.565	55.565	57.824	60.835	60.835	60.835	60.835	60.835	60.835	60.835	57.824	55.565	55.565
At age t (49 days)	notional size (mm) of the cross section, ho		543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0832	0.0832	0.0997	0.1200	0.1200	0.1200	0.1200	0.1200	0.1200	0.1200	0.0997	0.0832	0.0832
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.706	0.721	0.721	0.721	0.721	0.721	0.721	0.721	0.706	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		14.675	14.675	17.747	21.806	21.806	21.806	21.806	21.806	21.806	21.806	17.747	14.675	14.675
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753	0.753
	autogeneous shrinkage strain, ϵ_{as}		48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971	48.971
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	63.646	63.646	66.719	70.777	70.777	70.777	70.777	70.777	70.777	70.777	66.719	63.646	63.646
At age t (63 days)	notional size (mm) of the cross section, ho		543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.1058	0.1058	0.1263	0.1509	0.1509	0.1509	0.1509	0.1509	0.1509	0.1509	0.1263	0.1058	0.1058
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.706	0.721	0.721	0.721	0.721	0.721	0.721	0.721	0.706	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		18.669	18.669	22.467	27.441	27.441	27.441	27.441	27.441	27.441	27.441	22.467	18.669	18.669
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796	0.796
	autogeneous shrinkage strain, ϵ_{as}		51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711	51.711
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	70.380	70.380	74.178	79.152	79.152	79.152	79.152	79.152	79.152	79.152	74.178	70.380	70.380
At age t Infinity	notional size (mm) of the cross section, ho		543.575	543.575	475.816	414.445	414.445	414.445	414.445	414.445	414.445	414.445	475.816	543.575	543.575
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.706	0.721	0.721	0.721	0.721	0.721	0.721	0.721	0.706	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		176.400	176.400	177.924	181.790	181.790	181.790	181.790	181.790	181.790	181.790	177.924	176.400	176.400
	Coeff. For autogeneous shrinkage strain $\beta_{as}(t)$		1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
	autogeneous shrinkage strain, ϵ_{as}		65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000	65.000
	Total shrinkage strain, ϵ_{cs}	$\mu\epsilon$	241.400	241.400	242.924	246.790	246.790	246.790	246.790	246.790	246.790	246.790	242.924	241.400	241.400
Differential shrinkage strain															
	upto 7 days	$\mu\epsilon$	28.525	28.525	29.547	31.511	31.511	31.511	31.511	31.511	31.511	31.511	29.547	28.525	28.525
	from 7 days to 28 days	$\mu\epsilon$	22.208	22.208	23.001	23.468	23.468	23.468	23.468	23.468	23.468	23.468	23.001	22.208	22.208
	from 28 days to 35 days	$\mu\epsilon$	4.832	4.832	5.276	5.855	5.855	5.855	5.855	5.855	5.855	5.855	5.276	4.832	4.832
	from 35 days to 49 days	$\mu\epsilon$	8.081	8.081	8.895	9.942	9.942	9.942	9.942	9.942	9.942	9.942	8.895	8.081	8.081
	from 49 days to 63 days	$\mu\epsilon$	6.733	6.733	7.459	8.375	8.375	8.375	8.375	8.375	8.375	8.375	7.459	6.733	6.733
	from 63 days to Infinity	$\mu\epsilon$	171.020	171.020	168.746	167.638	167.638	167.638	167.638	167.638	167.638	167.638	168.746	171.020	171.020

CREEP MODEL (micro strain)

(Refer Cl: 6.4.2.7 of IRC:112-2011)

$$\Delta(t, t_s) = \beta(t, t_s) \cdot \Delta(t_s) \quad \text{where } \beta(t, t_s) = \left[\frac{t - t_s}{t_s} \right]^{0.3}, \Delta(t_s) = \phi_{sh} \cdot \Delta f_{cm} \cdot \Delta t_s$$

ϕ_{RH} = factor to allow for the effect of relative humidity on the notational creep coefficient
 t = age of concrete in days at the time considered
 t_s = age of concrete in days at the time of loading

$$\phi_{RH} = 1 + \frac{1 - RH/100}{0.1 \sqrt{h_o}} \text{ for } f_{cm} \leq 45 \text{ MPa} \quad \text{Eq. A2-16} \quad \beta(t_o) = \frac{1}{0.1 + t_o^{0.20}}$$

$$\phi_{RH} = \left[1 + \frac{1 - RH/100}{0.1 \sqrt{h_o}} \alpha_1 \right] \alpha_2 \text{ for } f_{cm} \leq 45 \text{ MPa} \quad \text{Eq. A2-17}$$

$$\beta_H = 1.5[1 + (0.012RH)^{0.8}] h_o + 250 \leq 1500 \text{ for } f_{cm} \leq 35 \quad \text{Eq. A2-22}$$

$$\beta_H = 1.5[1 + (0.012RH)^{0.8}] h_o + 250 \alpha_3 \leq 1500 \alpha_3 \text{ for } f_{cm} \geq 35 \quad \text{Eq. A2-23}$$

$$t_o = t_{o,T} \left(\frac{9}{2 + t_{o,T}^{1.2}} + 1 \right)^{0.5} \geq 0.5 \quad \text{Eq. A2-25}$$

t - to = actual duration of loading in days

fck = 45 mpa , fcm = 55 mpa

RH = 80 % a = 0.0000

a1 = 0.852 a2 = 0.955 a3 = 0.892

β(fcm) = 2.532

β(∞ to t_o) = factor to allow for the effect of concrete age at loading on the notational creep coefficient

$$t_T = \sum_{i=1}^n \Delta t_i \exp \left[13.65 - \frac{4000}{273 + T_{(\Delta t_i)} / t_o} \right] \quad \text{Eq. A2-13}$$

Items	Description													
from age t	ho	543.58	543.58	475.82	414.45	414.45	414.45	414.45	414.45	414.45	414.45	475.82	543.58	543.58
(7 days)	φ _{RH} , refer eq. A2-16 & A2-17	1.165	1.165	1.179	1.195	1.195	1.195	1.195	1.195	1.195	1.195	1.179	1.165	1.165
	β _H , Refer Eq. A2-22 & A2-23, where fcm ≤ 45	1337.824	1337.824	1279.000	1142.792	1142.792	1142.792	1142.792	1142.792	1142.792	1142.792	1279.000	1337.824	1337.824
to	Temperature adjusted conc. Age t _{0,T} , Refer Eq. A2-13	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785
Infinity	Modified age of loading, t _o , Refer Eq. A2-25	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785	8.785
	β(infinity, t _o), refer eq. A2-19	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608	0.608
	φ (t _∞ , t _o)	1.794	1.794	1.816	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.816	1.794	1.794
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.286	0.286	0.290	0.300	0.300	0.300	0.300	0.300	0.300	0.300	0.290	0.286	0.286
(28 days)	Creep Coefficient, φ(t,t _o)	0.513	0.513	0.527	0.552	0.552	0.552	0.552	0.552	0.552	0.552	0.527	0.513	0.513
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.312	0.312	0.316	0.326	0.326	0.326	0.326	0.326	0.326	0.326	0.316	0.312	0.312
(35 days)	Creep Coefficient, φ(t,t _o)	0.559	0.559	0.573	0.601	0.601	0.601	0.601	0.601	0.601	0.601	0.573	0.559	0.559
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.351	0.351	0.355	0.367	0.367	0.367	0.367	0.367	0.367	0.367	0.355	0.351	0.351
(49 days)	Creep Coefficient, φ(t,t _o)	0.629	0.629	0.645	0.676	0.676	0.676	0.676	0.676	0.676	0.676	0.645	0.629	0.629
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.381	0.381	0.386	0.399	0.399	0.399	0.399	0.399	0.399	0.399	0.386	0.381	0.381
(63 days)	Creep Coefficient, φ(t,t _o)	0.684	0.684	0.701	0.734	0.734	0.734	0.734	0.734	0.734	0.734	0.701	0.684	0.684
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Infinity	Creep Coefficient, φ(t,t _o)	1.793	1.793	1.815	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.815	1.793	1.793

Maximum Compressive strength of Concrete = 0.48 x fck for SLS - Rare Case Refer clause 12.2.1 of IRC:112-2011

= 0.48 x fck for SLS -Quasi-Permanent Case

Concrete Compressive Strength Development with Time is given by Eq. 6.3 of IRC:112-2011

$$f_{cm}(t) = \beta_{cc}(t) \times f_{cm} \quad \text{where} \quad f_{cm} = f_{ck} + 10 \quad \& \quad \beta_{cc}(t) = \exp \left\{ s \left[1 - (28/(t+1))^{1/2} \right] \right\} \quad \text{for Ordinary portland cement} \quad S = 0.25$$

Concrete Tensile Strength Development with Time is given by Eq. 6.7 of IRC:112-2011

$$f_{ctm}(t) = \beta_{cc}(t)^a \times f_{ctm} \quad \text{where} \quad a = 1 \text{ for } t < 28 \text{ days} \& \ 2/3 \text{ for } t \geq 28 \text{ days}, \quad \beta_{cc}(t) = \exp \left\{ s \left[1 - (28/(t+1))^{1/2} \right] \right\} \quad \& \quad f_{ctm} = 0.259 (f_{ck})^{2/3} \text{ for } f_{ck} \leq 60 \text{ mpa}, \quad f_{ctm} = 2.27 \ln[1 + (f_{cm}/12.5)] \text{ for } f_{ck} > 60 \text{ mpa}$$

Characteristic compressive strength with time, fck(t) = fcm(t) - 10

fck = 45.000 Mpa fcm = 55.000 Mpa

age of concrete (t) in days	$\beta_{cc}(t)$	$f_{cm}(t)$	$f_{ck}(t)$	f_{ctm}	$\beta_{cc}(t)^a$	$f_{ctm}(t)$
7	0.779	42.834	32.834	3.277	0.779	2.55
14	0.902	49.590	39.590		0.902	2.95
21	0.962	52.913	42.913		0.962	3.15
28	1.000	55.000	45.000		1.000	3.28
beyond 28 days	1.000	55.000	45.000		1.000	3.28

STAGE - 1 PRESTRESSING

DETAILS OF STAGE-I PRESTRESS BEFORE INSTANTANEOUS LOSS

(At 7 days)

Prestress after Friction & Slip Loss in Stage I Cables

P_{hor}	kN	2474.245	2475.368	2494.240	2518.722	2560.988	2601.051	2623.212	2644.735	2620.594	2579.998	2556.709	2538.991	2537.966
eccentricity of stage I cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-410.505	-346.005	169.517	550.630	1103.711	1387.001	1413.859	1409.940	1130.647	565.454	175.308	-353.177	-419.330
P_{hor} / A	N/mm ²	2.113	2.114	2.665	3.483	3.542	3.597	3.628	3.657	3.624	3.568	2.731	2.168	2.167
M_p / Z_{tg}	N/mm ²	-1.195	-1.007	0.545	1.940	3.889	4.887	4.981	4.967	3.983	1.992	0.563	-1.028	-1.221
M_p / Z_{bg}	N/mm ²	-1.292	-1.089	0.586	2.033	4.075	5.121	5.220	5.206	4.174	2.088	0.606	-1.112	-1.320

Stress due to Prestress after Friction & Slip loss in Stage I Cables

$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	3.308	3.121	2.120	1.543	-0.347	-1.290	-1.354	-1.310	-0.359	1.576	2.168	3.196	3.388
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.820	1.024	3.250	5.516	7.616	8.718	8.848	8.863	7.798	5.656	3.337	1.056	0.847

Stress Due to Self Weight of Girder

Moment due to Self weight of Girder (M_{DL})	kNm	0.00	-1.79	356.93	620.16	989.61	1228.60	1308.27	1228.60	989.61	620.16	356.93	-1.79	0.00
M_{DL} / Z_{tg}	N/mm ²	0.000	-0.005	1.147	2.185	3.487	4.329	4.609	4.329	3.487	2.185	1.147	-0.005	0.000
$-M_{DL} / Z_{bg}$	N/mm ²	0.000	0.006	-1.233	-2.290	-3.654	-4.536	-4.830	-4.536	-3.654	-2.290	-1.233	0.006	0.000

Net Stress due to Girder weight & Prestress after Friction & Slip loss in Stage I Cables

$P_{hor} / A - M_p / Z_{tg} + M_{DL} / Z_{tg}$	N/mm ²	3.308	3.116	3.267	3.728	3.140	3.039	3.256	3.018	3.127	3.761	3.315	3.191	3.388
$P_{hor} / A + M_p / Z_{bg} - M_{DL} / Z_{bg}$	N/mm ²	0.820	1.030	2.017	3.226	3.963	4.182	4.017	4.327	4.145	3.366	2.104	1.062	0.847

Stress in concrete at c.g. of Stage-I Cables	N/mm ²	2.371	2.296	2.611	3.415	3.763	3.978	3.884	4.093	3.898	3.514	2.679	2.353	2.429
Average stress in concrete at c.g. of Stage- I Cable(σ_{cq_avg})	=		3.176	N/mm ²										
Maximum stress in concrete at c.g. of Stage- I Cable(σ_{cq_max})	=		4.093	N/mm ²										
Area of Cables in stage I Prestressing (A_{cable})	=		1974	mm ²										
m = EPs / Ec.eff at 7 days	=		7.169											

LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING OF STAGE I CABLES

Loss due to Elastic Shortening	=	0.5 x σ_{cq_avg} x m x A_{cable}
	=	22.471 kN

Loss of Prestress Due to Elastic Shortening from Stage I Cables

% of Elastic Shortening	%	0.908	0.908	0.901	0.892	0.877	0.864	0.857	0.850	0.857	0.871	0.879	0.885	0.885
$P_{hor1} = \text{Loss due to Elastic Shortening}$	kN	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471
eccentricity of cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-3.728	-3.141	1.527	4.913	9.685	11.983	12.112	11.980	9.695	4.925	1.541	-3.126	-3.713
P_{hor} / A	N/mm ²	0.019	0.019	0.024	0.031	0.031	0.031	0.031	0.031	0.031	0.031	0.024	0.019	0.019
M_p / Z_{tg}	N/mm ²	-0.011	-0.009	0.005	0.017	0.034	0.042	0.043	0.042	0.034	0.017	0.005	-0.009	-0.011
M_p / Z_{bg}	N/mm ²	-0.012	-0.010	0.005	0.018	0.036	0.044	0.045	0.044	0.036	0.018	0.005	-0.010	-0.012

Stress due to Elastic Shortening Loss from Stage I Cables

$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.030	0.028	0.019	0.014	-0.003	-0.011	-0.012	-0.011	-0.003	0.014	0.019	0.028	0.030
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.007	0.009	0.029	0.049	0.067	0.075	0.076	0.075	0.067	0.049	0.029	0.009	0.007

DETAILS OF STAGE-I PRESTRESS AFTER INSTANTANEOUS LOSS

(At 7 days)

Net Stress Due to Girder Weight + Prestress after Friction + Slip Loss + Elastic Shortening from Stage I Cables

At Top of Precast Girder,ftg	N/mm ²	3.278	3.087	3.248	3.714	3.143	3.050	3.267	3.030	3.130	3.747	3.296	3.163	3.358
At Bottom of Precast Girder,fbg	N/mm ²	0.813	1.021	1.988	3.177	3.896	4.106	3.942	4.252	4.078	3.317	2.075	1.052	0.839
Stress in Concrete at c.g. of Stage I Cable	N/mm ²	2.349	2.275	2.587	3.379	3.713	3.918	3.824	4.033	3.848	3.478	2.655	2.332	2.408
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	3.138	N/mm ²											
Max stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	4.033	N/mm ²											
P_{hor} = Prestress after friction ,slip loss & Elastic Shortening	kN	2451.774	2452.897	2471.768	2496.251	2538.516	2578.579	2600.740	2622.263	2598.123	2557.526	2534.238	2516.519	2515.495
Prestress after friction + slip + Elastic shortening loss as a fraction of UTS		0.668 UTS	0.668 UTS	0.673 UTS	0.680 UTS	0.691 UTS	0.702 UTS	0.708UTS	0.714UTS	0.708UTS	0.697UTS	0.690UTS	0.685UTS	0.685UTS
Check for Maximum Prestress force as per clause 7.9.2 (3) of IRC:112-2011	< 0.74 UTS	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

(1)**LOSS OF PRESTRESS****From 7 days to 28 days FOR STAGE-I PRESTRESS****Loss due to Shrinkage In Stage - I Cables**

Es	=	1.95E+05	N/mm ²
Area of cables in stage -I Prestress, A _{cable}	=	1974.0000	mm ²

Differential shrinkage strain = ecs (upto 28days)	μc	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005	0.00005
shrinkage loss between (upto 28days)	%	0.797	0.796	0.818	0.848	0.834	0.821	0.814	0.807	0.815	0.827	0.798	0.776	0.776
Stress loss due to Shrinkage = ecs x Ep x Ac	kN	19.528	19.528	20.227	21.163	21.163	21.163	21.163	21.163	21.163	21.163	20.227	19.528	19.528

Loss due to Relaxation of steel In Stage - I Cables

Prestress after friction + slip + Elastic shortening loss as a fraction of UTS	=	0.668 UTS	0.668 UTS	0.673 UTS	0.680 UTS	0.691 UTS	0.702 UTS	0.708UTS	0.714UTS	0.708UTS	0.697UTS	0.690UTS	0.685UTS	0.685UTS

Initial prestress	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.6	0.6	0.6	0.6	0.6
Relaxation loss for low relaxation steel (%)	1.250	1.250	1.250	1.250	1.250	2.500	2.500	2.500	2.500	1.250	1.250	1.250	1.250	1.250
Initial prestress	0.7	0.7	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.7	0.7	0.7
Relaxation loss for low relaxation steel (%)	2.500	2.500	2.500	2.500	2.500	4.500	4.500	4.500	4.500	2.500	2.500	2.500	2.500	2.500

Relaxation loss at 1000 hours (Stage I Cables)	%	2.097	2.101	2.165	2.248	2.392	2.546	2.667	2.784	2.652	2.457	2.378	2.317	2.314

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 7days & 28days	=	21	days	500	90
Time in hours after transfer of prestressing	=	504	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	90.08	%	504	90.08

% Relaxation Loss	=	Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100
Relaxation Loss	=	% Relaxation Loss/100 x Prestress after F+S+ES Loss

% Relaxation Loss	%	1.889	1.892	1.950	2.025	2.155	2.293	2.402	2.508	2.389	2.213	2.142	2.087	2.084
Relaxation Loss	kN	46.312	46.418	48.205	50.557	54.703	59.134	62.470	65.757	62.074	56.604	54.279	52.532	52.431

Loss due to Creep of Concrete in Stage I Cables

Stress loss due to Creep	σ_{CS}	=	$\phi(t, t_0) \times E_p/E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of prestress	$\sigma_{cp,Avg.}$	=	3.138 N/mm ²	Modulus of elasticity of prestressing steel, E_{ps}		=	195000 N/mm ²
				Area of prestressing strand, A_{ps}		=	1974 mm ²

Creep loss	%	0.744	0.744	0.757	0.786	0.772	0.760	0.754	0.748	0.755	0.767	0.738	0.725	0.725
	kN	18.242	18.242	18.714	19.608	19.608	19.608	19.608	19.608	19.608	19.608	18.714	18.242	18.242

TOTAL PRESTRESS LOSS (From 7 days to 28 days)

Loss due to (SH + CR + RLEX)	%	3.429	3.432	3.526	3.659	3.761	3.874	3.970	4.062	3.958	3.807	3.678	3.588	3.586
	kN	84.082	84.188	87.147	91.329	95.475	99.906	103.242	106.528	102.845	97.376	93.220	90.302	90.201

Stress due to Loss of Prestress Due to SH+CR+RLEX Loss

		From 7 days to	28 days	FOR STAGE-I PRESTRESS										
Prestress after friction, slip loss & Shortening	kN	2451.774	2452.897	2471.768	2496.251	2538.516	2578.579	2600.740	2622.263	2598.123	2557.526	2534.238	2516.519	2515.495
Loss of Prestress due to SH+CR+RLEX	kN	84.082	84.188	87.147	91.329	95.475	99.906	103.242	106.528	102.845	97.376	93.220	90.302	90.201
eccentricity of cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-13.950	-11.768	5.923	19.966	41.147	53.274	55.645	56.792	44.372	21.342	6.392	-12.561	-14.903
P_{hor} / A	N/mm ²	0.072	0.072	0.093	0.126	0.132	0.138	0.143	0.147	0.142	0.135	0.100	0.077	0.077
M_p / Z_{tg}	N/mm ²	-0.041	-0.034	0.019	0.070	0.145	0.188	0.196	0.200	0.156	0.075	0.021	-0.037	-0.043
M_p / Z_{bg}	N/mm ²	-0.044	-0.037	0.020	0.074	0.152	0.197	0.205	0.210	0.164	0.079	0.022	-0.040	-0.047
$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.112	0.106	0.074	0.056	-0.013	-0.050	-0.053	-0.053	-0.014	0.059	0.079	0.114	0.120
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.028	0.035	0.114	0.200	0.284	0.335	0.348	0.357	0.306	0.213	0.122	0.038	0.030

NET STRESS IN GIRDER DUE TO SELF WEIGHT & PRESTRESS IN STAGE- I CABLES AFTER ALL LOSSES BEFORE CASTING OF DECK SLAB

(AT 28 days)

At Top of Deck Slab,fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.165	2.981	3.174	3.658	3.156	3.100	3.320	3.082	3.144	3.687	3.217	3.049	3.237
At Bottom of Precast Girder,fbg	N/mm ²	0.785	0.986	1.874	2.977	3.612	3.772	3.593	3.895	3.772	3.103	1.953	1.015	0.809
Stress in Concrete at c.g. of Stage I Cable	N/mm ²	2.268	2.197	2.492	3.233	3.501	3.652	3.546	3.750	3.620	3.322	2.554	2.248	2.322
Average stress in concrete at c.g. of Stage- I Cable($\sigma_{cg,avg}$)	=	2.977 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	1.552	1.608	2.170	3.095	3.554	3.699	3.564	3.807	3.692	3.204	2.239	1.647	1.590
Average stress in concrete at c.g. of Stage- II Cable($\sigma_{cg,avg}$)	=	2.725 N/mm ²												

STAGE - 2 PRESTRESSING**NET STRESS IN GIRDER**

AT 28 days

JUST AFTER PRESTRESSING OF STAGE-II CABLES & JUST BEFORE ES LOSS OF STAGE-II**NON-COMPOSITE SECTION****Stresses due to Stage-II Prestress. after Friction+Slip**

P_{hor}	kN	4014.661	4016.458	4046.826	4085.509	4135.163	4173.001	4201.841	4230.266	4235.785	4209.810	4172.780	4144.038	4142.374
eccentricity of stage II cable from N.A. (ec)	m	0.315	0.333	0.465	0.542	0.616	0.647	0.649	0.647	0.616	0.543	0.466	0.334	0.317
$M_p = P_{hor} \times ec$	kNm	1266.496	1335.843	1882.754	2213.093	2545.521	2700.149	2726.083	2736.540	2607.831	2284.044	1945.755	1383.461	1312.064
P_{hor} / A	N/mm ²	3.428	3.429	4.323	5.650	5.718	5.771	5.811	5.850	5.858	5.822	4.458	3.538	3.537
M_p / Z_{ts}	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
M_p / Z_{tg}	N/mm ²	3.687	3.889	6.050	7.797	8.968	9.513	9.604	9.641	9.188	8.047	6.253	4.028	3.820
M_p / Z_{bg}	N/mm ²	3.987	4.205	6.503	8.171	9.398	9.969	10.065	10.103	9.628	8.433	6.721	4.355	4.130
Stress due to Prestress after Friction & Slip loss in Stage II Cables														
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	-0.260	-0.460	-1.727	-2.147	-3.250	-3.742	-3.794	-3.791	-3.330	-2.225	-1.795	-0.490	-0.283

$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	7.415	7.635	10.827	13.821	15.117	15.740	15.875	15.953	15.486	14.254	11.179	7.894	7.667

Net Stress in Girder just before Elastic Shortening Loss of Stage-II Prestress

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.906	2.521	1.447	1.511	-0.094	-0.643	-0.473	-0.709	-0.186	1.462	1.422	2.559	2.954
At Bottom of Precast Girder,fbg	N/mm ²	8.200	8.620	12.701	16.798	18.729	19.511	19.469	19.848	19.258	17.358	13.132	8.908	8.477

Stress at c.g of Stage I Cables	N/mm ²	4.900	4.919	7.349	11.064	14.167	15.915	15.982	16.178	14.551	11.401	7.567	5.058	5.037

Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})

$$= 10.315 \text{ N/mm}^2$$

Stress at c.g of Stage II Cables	N/mm ²	6.493	6.720	10.143	14.151	16.339	17.349	17.351	17.640	16.790	14.614	10.478	6.935	6.701

Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})

$$= 12.439 \text{ N/mm}^2$$

Increase in Average Stress at cg. of Stage-I Cables

$$= 7.337 \text{ N/mm}^2$$

Increase in Average Stress at cg. of Stage-II Cables

$$= 9.714 \text{ N/mm}^2$$

ADDN. LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING FROM STAGE-I CABLES

$$m = Eps / E_{c,eff} \text{ at 28 days} = 5.735$$

$$E_{c,eff} = E_{cm} / (1 + \phi) = 34000 \text{ N/mm}^2$$

$$Eps = 195000 \text{ N/mm}^2$$

$$A_{cable} = 1974 \text{ mm}^2$$

$$\text{Elastic Shortening Loss due to Stage-I Cables} = 1.0 \times \sigma_{cg\_avg} \times m \times A_{cable \text{ of stage I}} = 83.070 \text{ kN}$$

Elastic Shortening Loss	%	3.357	3.356	3.330	3.298	3.244	3.194	3.167	3.141	3.170	3.220	3.249	3.272	3.273

Stress due to Stage-I Pres. after Additional Elastic Shortening from Stage-II

Prestress due to Elastic shortening loss(P_{hor})	kN	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070
eccentricity of stage I cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-13.782	-11.611	5.646	18.160	35.801	44.297	44.773	44.286	35.840	18.206	5.696	-11.555	-13.725
P_{hor} / A	N/mm ²	0.071	0.071	0.089	0.115	0.115	0.115	0.115	0.115	0.115	0.115	0.089	0.071	0.071
M_p / Z_{ts}	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
M_p / Z_{tg}	N/mm ²	-0.040	-0.034	0.018	0.064	0.126	0.156	0.158	0.156	0.126	0.064	0.018	-0.034	-0.040
M_p / Z_{bg}	N/mm ²	-0.043	-0.037	0.020	0.067	0.132	0.164	0.165	0.164	0.132	0.067	0.020	-0.036	-0.043
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.111	0.105	0.071	0.051	-0.011	-0.041	-0.043	-0.041	-0.011	0.051	0.070	0.105	0.111
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.028	0.034	0.108	0.182	0.247	0.278	0.280	0.278	0.247	0.182	0.108	0.035	0.028

LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING FROM STAGE-II CABLES

$$m = Eps / E_{c,eff} \text{ at 28 days} = 5.735$$

$$A_{cable \text{ of stage II}} = 3158.4 \text{ mm}^2$$

$$\sigma_{cg\_avg} = 9.714 \text{ N/mm}^2$$

$$\text{Elastic Shortening Loss due to Stage-II Cables} = 0.5 \times \sigma_{cg\_avg} \times m \times A_{cable \text{ of stage II}} = 87.980 \text{ kN}$$

Elastic Shortening Loss	%	2.191	2.190	2.174	2.153	2.128	2.108	2.094	2.080	2.077	2.090	2.108	2.123	2.124

Stress due to Stage-II Pres. after Elastic Shortening from Stage-II

Prestress due to Elastic shortening loss(P_{hor})	kN	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980
eccentricity of stage II cable from N.A. (ec)	m	0.315	0.333	0.465	0.542	0.616	0.647	0.649	0.647	0.616	0.543	0.466	0.334	0.317
$M_p = P_{hor} \times ec$	kNm	27.755	29.261	40.932	47.658	54.159	56.928	57.080	56.914	54.166	47.734	41.025	29.372	27.867
P_{hor} / A	N/mm ²	0.075	0.075	0.094	0.122	0.122	0.122	0.122	0.122	0.122	0.122	0.094	0.075	0.075
M_p / Z_{ts}	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
M_p / Z_{tg}	N/mm ²	0.081	0.085	0.132	0.168	0.191	0.201	0.201	0.201	0.191	0.168	0.132	0.086	0.081
M_p / Z_{bg}	N/mm ²	0.087	0.092	0.141	0.176	0.200	0.210	0.211	0.210	0.200	0.176	0.142	0.092	0.088

$P_{nor} / A - M_p / Z_{ts}$	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$P_{nor} / A - M_p / Z_{tb}$	N/mm ²	-0.006	-0.010	-0.038	-0.046	-0.069	-0.079	-0.079	-0.079	-0.069	-0.047	-0.038	-0.010	-0.006
$P_{nor} / A + M_p / Z_{bg}$	N/mm ²	0.162	0.167	0.235	0.298	0.322	0.332	0.332	0.332	0.322	0.298	0.236	0.168	0.163

NET STRESS IN GIRDER**AT 28 days****JUST AFTER PRESTRESSING OF STAGE-II CABLES & JUST AFTER ES LOSS OF STAGE-II****Net Stress in Girder after Losses due to Friction + Slip + Elastic Shortening of Stage-II Cables**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder, ffg	N/mm ²	2.800	2.427	1.414	1.507	-0.014	-0.523	-0.351	-0.589	-0.105	1.458	1.389	2.465	2.849
At Bottom of Precast Girder, fbg	N/mm ²	8.010	8.419	12.357	16.318	18.160	18.901	18.856	19.238	18.689	16.878	12.788	8.706	8.286

Stress at c.g of Stage I Cables	N/mm ²	4.763	4.782	7.153	10.763	13.756	15.435	15.498	15.699	14.140	11.099	7.371	4.921	4.900

Average stress in concrete at c.g. of Stage- I

=

10.022

N/mm²Cable($\sigma_{cq \text{ avg}}$)

Stress at c.g of Stage II Cables

N/mm²

6.330

6.551

9.870

13.754

15.852

16.817

16.816

17.108

16.304

14.216

10.205

6.766

6.538

Average stress in concrete at c.g. of Stage- II

=

12.087

N/mm²Cable($\sigma_{cq \text{ avg}}$)

Prestress after friction + slip + ES loss (Stage II Cable)	kN	3926.681	3928.478	3958.846	3997.529	4047.183	4085.021	4113.861	4142.286	4147.805	4121.830	4084.800	4056.058	4054.394
Prestress after friction + slip + ES loss as a fraction of UTS (Stage II Cable)		0.668UTS	0.669UTS	0.674UTS	0.680UTS	0.689UTS	0.695UTS	0.700UTS	0.705UTS	0.706UTS	0.702UTS	0.695UTS	0.690UTS	0.690UTS

Initial prestress	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.6	0.6	0.6
Relaxation loss for low relaxation steel (%)	1.250	1.250	1.250	1.250	1.250	1.250	1.250	2.500	2.500	2.500	2.500	1.250	1.250	1.250

Initial prestress	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.7
Relaxation loss for low relaxation steel (%)	2.500	2.500	2.500	2.500	2.500	2.500	2.500	4.500	4.500	4.500	4.500	2.500	2.500	2.500

Relaxation loss at 1000 hours (Stage II Cable)	%	2.105	2.109	2.174	2.256	2.362	2.442	2.505	2.602	2.621	2.533	2.442	2.380	2.377

(2) LOSS OF PRESTRESS**From 28 days to 35 days****FOR STAGE-I & STAGE-II PRESTRESS****Non-composite Section****Loss due to Shrinkage****Shrinkage Loss for Stage I Cables**

E_p	=	1.95E+05	N/mm ²
Area of cables in stage -I Prestress, A_{cable}	=	1974.000	mm ²

Differential shrinkage strain = ecs	μC	4.832	4.832	5.276	5.855	5.855	5.855	5.855	5.855	5.855	5.855	5.276	4.832	4.832
shrinkage loss between (28 days to 35days)	%	0.076	0.076	0.082	0.090	0.089	0.087	0.087	0.086	0.087	0.088	0.080	0.074	0.074
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	1.860	1.860	2.031	2.254	2.254	2.254	2.254	2.254	2.254	2.254	2.031	1.860	1.860

Shrinkage Loss for Stage II Cables

E_p	=	1.95E+05	N/mm ²
Area of cables in stage -II Prestress, A_{cable}	=	3158.400	mm ²

Differential shrinkage strain = ecs	μC	4.832	4.832	5.276	5.855	5.855	5.855	5.855	5.855	5.855	5.855	5.276	4.832	4.832
shrinkage loss between (28 days to 35days)	%	0.076	0.076	0.082	0.090	0.089	0.088	0.088	0.087	0.087	0.087	0.080	0.073	0.073
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	2.976	2.976	3.249	3.606	3.606	3.606	3.606	3.606	3.606	3.606	3.249	2.976	2.976

Loss due to Relaxation of steel**FOR STAGE-I CABLES:**

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 7days & 35days	=	28	days	500	90
Time in hours after transfer of prestressing	=	672	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	93.44	%	672	93.44

Relaxation Loss between 7days & 28days	=	90.08	%

Relaxation Loss between 28days & 35days	=	93.44	-	90.08
	=	3.36	% of loss at 1000hrs	

% Relaxation Loss = Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100
 Relaxation Loss = % Relaxation Loss/100 x Prestress after F+S+ES Loss

% Relaxation Loss For Stage I Cables	%	0.070	0.071	0.073	0.076	0.080	0.086	0.090	0.094	0.089	0.083	0.080	0.078	0.078
Relaxation Loss For Stage I Cables	kN	1.727	1.731	1.798	1.886	2.040	2.206	2.330	2.453	2.315	2.111	2.025	1.959	1.956

FOR STAGE-II CABLES:

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 28days & 35days = 7 days 100 72
 Time in hours after transfer of prestressing = 168 hrs 200 79
 Relaxation loss as % of loss at 1000 hrs = 76.76 % 168 76.76

% Relaxation Loss = Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100
 Relaxation Loss = % Relaxation Loss/100 x Prestress after F+S+ES Loss

Relaxation Loss between 28days & 35days

% Relaxation Loss For Stage II Cables	%	1.616	1.619	1.668	1.732	1.813	1.874	1.923	1.997	2.012	1.944	1.874	1.827	1.824
Relaxation Loss For Stage II Cables	kN	63.451	63.595	66.050	69.221	73.363	76.574	79.116	82.740	83.449	80.128	76.555	74.112	73.971

Loss due to Creep of Concrete

Stress loss due to Creep σ_{cs} = $\{\phi(t_2, t_0) - \phi(t_1, t_0)\} \times E_p / E_{c,eff} \times \sigma_{cp} \times A_p$ Modulus of elasticity of concrete $E_{c,eff} = E_{cm}$ = 34000 N/mm²
 Average stress at c.g. of stage-I Cables $\sigma_{cp,Avg,-I}$ = 10.022 N/mm² Modulus of elasticity of prestressing steel, E_p = 195000 N/mm²
 Average stress at c.g. of stage-II Cables $\sigma_{cp,Avg,-II}$ = 12.087 N/mm² Area of prestressing strand of Stage -I Prestress, A_p = 1974 mm²
 Creep coeff for age between from 7 to 28 = $\phi(t_1, t_0)$ Area of prestressing strand of Stage -II Prestress, A_p = 3158.4 mm²
 Creep coeff for age between from 7 to 35 = $\phi(t_2, t_0)$

$\phi(t_2, t_0)$		0.559	0.559	0.573	0.601	0.601	0.601	0.601	0.601	0.601	0.601	0.573	0.559	0.559
$\phi(t_1, t_0)$		0.513	0.513	0.527	0.552	0.552	0.552	0.552	0.552	0.552	0.552	0.527	0.513	0.513
Creep loss for Stage I Cables	%	0.210	0.210	0.214	0.221	0.217	0.214	0.212	0.211	0.213	0.216	0.208	0.205	0.205
	kN	5.153	5.153	5.282	5.521	5.521	5.521	5.521	5.521	5.521	5.521	5.282	5.153	5.153

$\phi(t_2, t_0)$		0.559	0.559	0.573	0.601	0.601	0.601	0.601	0.601	0.601	0.601	0.573	0.559	0.559
$\phi(t_1, t_0)$		0.513	0.513	0.527	0.552	0.552	0.552	0.552	0.552	0.552	0.552	0.527	0.513	0.513
Creep loss for Stage II Cables	%	0.253	0.253	0.257	0.267	0.263	0.261	0.259	0.257	0.257	0.258	0.250	0.245	0.245
	kN	9.943	9.943	10.192	10.654	10.654	10.654	10.654	10.654	10.654	10.654	10.192	9.943	9.943

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 28 days to 35 days**

Total Prestress Loss for Stage-I Cables	%	0.356	0.356	0.369	0.387	0.387	0.387	0.389	0.390	0.388	0.387	0.368	0.357	0.357
	kN	8.740	8.744	9.111	9.661	9.816	9.981	10.105	10.228	10.091	9.886	9.337	8.972	8.968
Total Prestress Loss for Stage-II Cables	%	1.945	1.948	2.008	2.088	2.165	2.224	2.270	2.342	2.356	2.290	2.203	2.146	2.143
	kN	76.370	76.514	79.492	83.482	87.624	90.834	93.377	97.001	97.709	94.389	89.996	87.031	86.891

Stress due to Loss of Prestress Due to SH+CR+RLEX Loss**From 28 days to 35 days****FOR STAGE-I & STAGE-II PRESTRESS****NON-COMPOSITE SECTION****Stage I Cables**

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	8.740	8.744	9.111	9.661	9.816	9.981	10.105	10.228	10.091	9.886	9.337	8.972	8.968
Eccentricity of Stage I cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-1.450	-1.222	0.619	2.112	4.230	5.322	5.447	5.453	4.354	2.167	0.640	-1.248	-1.482

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	76.370	76.514	79.492	83.482	87.624	90.834	93.377	97.001	97.709	94.389	89.996	87.031	86.891
Eccentricity of Stage II cable from N.A. (ec)	m	0.315	0.333	0.465	0.542	0.616	0.647	0.649	0.647	0.616	0.543	0.466	0.334	0.317
$M_p = P_{hor} \times ec$	kNm	24.092	25.448	36.983	45.222	53.939	58.775	60.581	62.749	60.156	51.211	41.965	29.055	27.522

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	85.110	85.259	88.602	93.143	97.440	100.815	103.482	107.229	107.800	104.275	99.333	96.004	95.859
M_p (Stage I & Stage II Cables)	kNm	22.642	24.226	37.602	47.334	58.170	64.097	66.028	68.202	64.510	53.378	42.605	27.807	26.040
P_{hor} / A	N/mm ²	0.073	0.073	0.095	0.129	0.135	0.139	0.143	0.148	0.149	0.144	0.106	0.082	0.082
M_p / Z_{is}	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
M_p / Z_{tg}	N/mm ²	0.066	0.071	0.121	0.167	0.205	0.226	0.233	0.240	0.227	0.188	0.137	0.081	0.076
M_p / Z_{bg}	N/mm ²	0.071	0.076	0.130	0.175	0.215	0.237	0.244	0.252	0.238	0.197	0.147	0.088	0.082
$P_{hor} / A - M_p / Z_{is}$	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.007	0.002	-0.026	-0.038	-0.070	-0.086	-0.090	-0.092	-0.078	-0.044	-0.031	0.001	0.006
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.144	0.149	0.225	0.304	0.350	0.376	0.387	0.400	0.387	0.341	0.253	0.170	0.164

Stress Due to Weight of Shuttering on Girder**At 35 days**

Moment due to shuttering	kNm	0.00	-0.03	8.18	14.57	23.77	29.72	31.70	29.72	23.77	14.57	8.18	-0.03	0.00
M_{constr} / Z_{tg}	N/mm ²	0.000	0.000	0.026	0.051	0.084	0.105	0.112	0.105	0.084	0.051	0.026	0.000	0.000
$-M_{constr} / Z_{bg}$	N/mm ²	0.000	0.000	-0.028	-0.054	-0.088	-0.110	-0.117	-0.110	-0.088	-0.054	-0.028	0.000	0.000

NET STRESS IN GIRDER**(At 35 days)****JUST BEFORE CASTING OF DECK SLAB**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.794	2.424	1.466	1.596	0.140	-0.331	-0.150	-0.392	0.057	1.553	1.446	2.464	2.843
At Bottom of Precast Girder,fbg	N/mm ²	7.866	8.270	12.105	15.961	17.723	18.415	18.352	18.728	18.214	16.482	12.506	8.537	8.122

Stress at c.g of Stage I Cables	N/mm ²	4.705	4.722	7.045	10.573	13.462	15.070	15.117	15.315	13.819	10.888	7.251	4.854	4.835
Average stress in concrete at c.g. of Stage- I Cable (σ_{ca_avg})	=	9.820	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	6.231	6.448	9.687	13.474	15.490	16.404	16.387	16.675	15.909	13.906	10.000	6.649	6.425
Average stress in concrete at c.g. of Stage- II Cable (σ_{ca_avg})	=	11.822	N/mm ²											

Stress in Girder Due to Green Weight of Deck slab**(At 35 days)**

Moment due to Green Weight + Temp. LL	kNm	0.00	-2.22	657.45	1171.66	1911.05	2389.36	2548.80	2389.36	1911.05	1171.66	657.45	-2.22	0.00
M_{DL1} / Z_{tg}	N/mm ²	0.000	-0.006	2.113	4.128	6.733	8.418	8.980	8.418	6.733	4.128	2.113	-0.006	0.000
$-M_{DL1} / Z_{bg}$	N/mm ²	0.000	0.007	-2.271	-4.326	-7.056	-8.822	-9.410	-8.822	-7.056	-4.326	-2.271	0.007	0.000

NET STRESS IN GIRDER**At 35 days****JUST AFTER CASTING OF DECK SLAB**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.794	2.418	3.579	5.724	6.873	8.087	8.830	8.026	6.790	5.681	3.559	2.457	2.843
At Bottom of Precast Girder,fbg	N/mm ²	7.866	8.277	9.834	11.635	10.667	9.594	8.942	9.906	11.158	12.157	10.235	8.544	8.122

Stress at c.g of Stage I Cables	N/mm ²	4.705	4.721	6.859	9.418	9.748	9.325	8.923	9.571	10.101	9.730	7.063	4.853	4.835

Average stress in concrete at c.g. of **Stage- I****Cable**($\sigma_{cq \text{ avg}}$) = 7.681 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	6.231	6.451	8.412	10.612	10.185	9.432	8.930	9.704	10.604	11.039	8.722	6.652	6.425

Average stress in concrete at c.g. of **Stage- II****Cable**($\sigma_{cq \text{ avg}}$) = 8.723 N/mm²**(3) LOSS OF PRESTRESS From 35 days to 49 days FOR STAGE-I & STAGE-II PRESTRESS Non-composite Section****Loss due to Shrinkage****Shrinkage Loss for Stage I Cables** E_p = 1.95E+05 N/mm²Area of cables in stage -I Prestress, A_{cable} = 1974.000 mm²

Differential shrinkage strain = ecs	μc	8.081	8.081	8.895	9.942	9.942	9.942	9.942	9.942	9.942	9.942	8.895	8.081	8.081
shrinkage loss between (35 days to 49days)	%	0.127	0.127	0.139	0.153	0.151	0.148	0.147	0.146	0.147	0.150	0.135	0.124	0.124
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	3.111	3.111	3.424	3.827	3.827	3.827	3.827	3.827	3.827	3.827	3.424	3.111	3.111

Shrinkage Loss for Stage II Cables E_s = 1.95E+05 N/mm²Area of cables in stage -II Prestress, A_{cable} = 3158.400 mm²

Differential shrinkage strain = ecs	μc	8.081	8.081	8.895	9.942	9.942	9.942	9.942	9.942	9.942	9.942	8.895	8.081	8.081
shrinkage loss between (35 days to 49days)	%	0.127	0.127	0.138	0.153	0.151	0.150	0.149	0.148	0.148	0.149	0.134	0.123	0.123
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	4.977	4.977	5.478	6.123	6.123	6.123	6.123	6.123	6.123	6.123	5.478	4.977	4.977

Loss due to Relaxation of steel**FOR STAGE-I CABLES:**

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 7days & 49days	=	42	days	500	90
Time in hours after transfer of prestressing	=	1008	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	100.00	%	1008	100.00

Relaxation Loss between 7days & 35days = 93.44 %

Relaxation Loss between 35days & 49days = 100.00 - 93.44

= 6.56 % of loss at 1000hrs

% Relaxation Loss = Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100

Relaxation Loss = % Relaxation Loss/100 x Prestress after F+S+ES Loss

% Relaxation Loss For Stage I Cables	%	0.138	0.138	0.142	0.147	0.157	0.167	0.175	0.183	0.174	0.161	0.156	0.152	0.152
Relaxation Loss For Stage I Cables	kN	3.373	3.380	3.511	3.682	3.984	4.306	4.549	4.789	4.520	4.122	3.953	3.826	3.818

FOR STAGE-II CABLES:

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 28days & 49days	=	21	days	500	90
Time in hours after transfer of prestressing	=	504	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	90.08	%	504	90.08

Relaxation Loss between 28days & 35days = 76.76 %

Relaxation Loss between 35days & 49days = 90.08 - 76.76

= 13.32 % of loss at 1000hrs

% Relaxation Loss = Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100

Relaxation Loss = % Relaxation Loss/100 x Prestress after F+S+ES Loss

% Relaxation Loss For Stage II Cables	%	0.280	0.281	0.290	0.300	0.315	0.325	0.334	0.347	0.349	0.337	0.325	0.317	0.317
Relaxation Loss For Stage II Cables	kN	11.010	11.036	11.462	12.012	12.731	13.288	13.729	14.358	14.481	13.904	13.284	12.861	12.836

Loss due to Creep of Concrete

Stress loss due to Creep	σ_{cs}	=	$\{\phi(t_2, t_0) - \phi(t_1, t_0)\} \times E_p / E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of stage-I Cables	$\sigma_{cp,Avg,-I}$	=	7.681 N/mm ²	Modulus of elasticity of prestressing steel , E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg,-II}$	=	8.723 N/mm ²	Area of prestressing strand of Stage -I Prestress, A_{ps}		=	1974 mm ²
Creep coeff for age between from 7 to 49		=	$\phi(t_1, t_0)$	Area of prestressing strand of Stage -II Prestress, A_{ps}		=	3158.40 mm ²
Creep coeff for age between from 7 to 35		=	$\phi(t_2, t_0)$				

$\phi(t_2, t_0)$		0.629	0.629	0.645	0.676	0.676	0.676	0.676	0.676	0.676	0.676	0.645	0.629	0.629
$\phi(t_1, t_0)$		0.559	0.559	0.573	0.601	0.601	0.601	0.601	0.601	0.601	0.601	0.573	0.559	0.559
Creep loss for Stage I Cables	%	0.250	0.249	0.254	0.262	0.258	0.254	0.252	0.250	0.252	0.256	0.247	0.243	0.243
	kN	6.118	6.118	6.268	6.545	6.545	6.545	6.545	6.545	6.545	6.545	6.268	6.118	6.118

$\phi(t_2, t_0)$		0.629	0.629	0.645	0.676	0.676	0.676	0.676	0.676	0.676	0.676	0.645	0.629	0.629
$\phi(t_1, t_0)$		0.559	0.559	0.573	0.601	0.601	0.601	0.601	0.601	0.601	0.601	0.573	0.559	0.559
Creep loss for Stage II Cables	%	0.283	0.283	0.288	0.297	0.294	0.291	0.289	0.287	0.287	0.289	0.279	0.274	0.274
	kN	11.116	11.116	11.390	11.892	11.892	11.892	11.892	11.892	11.892	11.892	11.390	11.116	11.116

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 35 days to 49 days**

Total Prestress Loss for Stage-I Cables	%	0.514	0.514	0.534	0.563	0.565	0.569	0.574	0.578	0.573	0.567	0.538	0.519	0.519
	kN	12.601	12.609	13.202	14.053	14.355	14.678	14.921	15.160	14.892	14.494	13.645	13.054	13.047
Total Prestress Loss for Stage-II Cables	%	0.690	0.691	0.716	0.751	0.760	0.766	0.772	0.782	0.783	0.774	0.738	0.714	0.714
	kN	27.104	27.129	28.330	30.027	30.746	31.303	31.744	32.373	32.496	31.920	30.152	28.954	28.930

Stress after Loss of Prestress Due to SH+CR+RLEX Loss

From 35 days to 49 days

FOR STAGE-I & STAGE-II PRESTRESS

NON-COMPOSITE SECTION

Stage I Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	12.601	12.609	13.202	14.053	14.355	14.678	14.921	15.160	14.892	14.494	13.645	13.054	13.047
Eccentricity of Stage I cable from N.A. (ec)	m	-0.166	-0.140	0.068	0.219	0.431	0.533	0.539	0.533	0.431	0.219	0.069	-0.139	-0.165
$M_p = P_{hor} \times ec$	kNm	-2.091	-1.762	0.897	3.072	6.187	7.827	8.042	8.082	6.425	3.177	0.936	-1.816	-2.156

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	27.104	27.129	28.330	30.027	30.746	31.303	31.744	32.373	32.496	31.920	30.152	28.954	28.930
Eccentricity of Stage II cable from N.A. (ec)	m	0.315	0.333	0.465	0.542	0.616	0.647	0.649	0.647	0.616	0.543	0.466	0.334	0.317
$M_p = P_{hor} \times ec$	kNm	8.550	9.023	13.180	16.266	18.927	20.255	20.595	20.942	20.007	17.318	14.060	9.666	9.163

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	39.705	39.738	41.532	44.080	45.101	45.981	46.665	47.533	47.388	46.413	43.797	42.008	41.976
M_p (Stage I & Stage II Cables)	kNm	6.460	7.260	14.077	19.338	25.113	28.082	28.637	29.024	26.432	20.495	14.996	7.850	7.008
P_{hor} / A	N/mm ²	0.034	0.034	0.044	0.061	0.062	0.064	0.065	0.066	0.066	0.064	0.047	0.036	0.036
M_p / Z_{ts}	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
M_p / Z_{tg}	N/mm ²	0.019	0.021	0.045	0.068	0.088	0.099	0.101	0.102	0.093	0.072	0.048	0.023	0.020
M_p / Z_{bg}	N/mm ²	0.020	0.023	0.049	0.071	0.093	0.104	0.106	0.107	0.098	0.076	0.052	0.025	0.022
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.015	0.013	-0.001	-0.007	-0.026	-0.035	-0.036	-0.037	-0.028	-0.008	-0.001	0.013	0.015
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.054	0.057	0.093	0.132	0.155	0.167	0.170	0.173	0.163	0.140	0.099	0.061	0.058

NET STRESS IN GIRDER**(AT 49 days)****JUST BEFORE COMPOSITE ACTION**

At Top of Deck Slab,fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.778	2.405	3.580	5.731	6.899	8.122	8.866	8.062	6.817	5.689	3.560	2.444	2.828
At Bottom of Precast Girder,fbg	N/mm ²	7.812	8.220	9.741	11.503	10.512	9.427	8.772	9.733	10.995	12.017	10.136	8.483	8.064

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.675	4.691	6.811	9.338	9.636	9.194	8.788	9.435	9.984	9.646	7.012	4.821	4.803
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	7.603 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	6.189	6.408	8.340	10.503	10.053	9.287	8.782	9.554	10.465	10.925	8.646	6.606	6.381
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	8.626 N/mm ²												

STRESS IN COMPOSITE GIRDER DUE TO REMOVAL OF SHUTTERING**(At 49 days)**

Moment due to Self weight of Girder (M_{DL})	kNm	0.00	0.03	-8.18	-14.57	-23.77	-29.72	-31.70	-29.72	-23.77	-14.57	-8.18	0.03	0.00
M/Z_{cg}	N/mm ²	0.000	0.000	-0.006	-0.010	-0.016	-0.020	-0.021	-0.020	-0.016	-0.010	-0.006	0.000	0.000
$-M/Z_{bg}$	N/mm ²	0.000	0.000	0.018	0.036	0.059	0.073	0.078	0.073	0.059	0.036	0.018	0.000	0.000

NET STRESS IN GIRDER**At 49 days****AFTER COMPOSITE ACTION AND REMOVAL OF SHUTTERING**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.778	2.405	3.574	5.721	6.883	8.102	8.845	8.043	6.801	5.679	3.555	2.444	2.828
At Bottom of Precast Girder,fbg	N/mm ²	7.812	8.220	9.758	11.539	10.571	9.500	8.850	9.807	11.054	12.053	10.154	8.483	8.064

Stress at c.g of Stage I Cables	N/mm ²	4.675	4.691	6.817	9.357	9.677	9.251	8.849	9.492	10.024	9.665	7.018	4.821	4.803
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	7.626 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	6.189	6.408	8.353	10.531	10.103	9.350	8.850	9.617	10.514	10.953	8.659	6.606	6.381
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	8.655 N/mm ²												

(4)**LOSS OF PRESTRESS****From 49 days to 63 days****FOR STAGE-I & STAGE-II PRESTRESS****COMPOSITE SECTION****Loss due to Shrinkage**

Shrinkage Loss for Stage I Cables

E_p	=	1.95E+05	N/mm ²
Area of cables in stage -I Prestress, A_{cable}	=	1974.00	mm ²

Differential shrinkage strain = ecs	$\mu\epsilon$	6.733	6.733	7.459	8.375	8.375	8.375	8.375	8.375	8.375	8.375	7.459	6.733	6.733
shrinkage loss between (49 days to 63days)	%	0.106	0.106	0.116	0.129	0.127	0.125	0.124	0.123	0.124	0.126	0.113	0.103	0.103
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	2.592	2.592	2.871	3.224	3.224	3.224	3.224	3.224	3.224	3.224	2.871	2.592	2.592

Shrinkage Loss for Stage II Cables

E_s	=	1.95E+05	N/mm ²
Area of cables in stage -II Prestress, A_{cable}	=	3158.40000	mm ²

Differential shrinkage strain = ecs	$\mu\epsilon$	6.733	6.733	7.459	8.375	8.375	8.375	8.375	8.375	8.375	8.375	7.459	6.733	6.733
shrinkage loss between (49 days to 63days)	%	0.106	0.106	0.116	0.129	0.127	0.126	0.125	0.125	0.124	0.125	0.112	0.102	0.102
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	4.147	4.147	4.594	5.158	5.158	5.158	5.158	5.158	5.158	5.158	4.594	4.147	4.147

Loss due to Relaxation of steel**FOR STAGE-I CABLES:**

As per Table 6.3 of IRC:112-2011

Age of Concrete Between 7days & 63days	=	56	days	500	90
Time in hours after transfer of prestressing	=	1344	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	100.00	%	1344	100.00

Relaxation Loss between 7days & 49days

Relaxation Loss between 49days & 63days

% Relaxation Loss	=	0.00	% of loss at 1000hrs	100.00
Relaxation Loss	=	Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100		
	=	% Relaxation Loss/100 x Prestress after F+S+ES Loss		

% Relaxation Loss For Stage I Cables	%	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Relaxation Loss For Stage I Cables	kN	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

FOR STAGE-II CABLES:

As per Table 6.3 of IRC:112-2011.

Age of Concrete Between 28days & 63days	=	35	days	500	90
Time in hours after transfer of prestressing	=	840	hrs	1000	100
Relaxation loss as % of loss at 1000 hrs	=	96.80	%	840	96.80

Relaxation Loss between 28days & 49days = 90.08 %

Relaxation Loss between 49days & 63days = 96.80 - 90.08

= 6.72 % of loss at 1000hrs

% Relaxation Loss = Relaxation loss at 1000 hrs x (Relaxation loss as % of loss at 1000 hrs)/100

Relaxation Loss = % Relaxation Loss/100 x Prestress after F+S+ES Loss

Relaxation Loss between 49days & 63days

% Relaxation Loss For Stage II Cables	%	0.141	0.142	0.146	0.152	0.159	0.164	0.168	0.175	0.176	0.170	0.164	0.160	0.160
Relaxation Loss For Stage II Cables	kN	5.555	5.567	5.782	6.060	6.423	6.704	6.926	7.244	7.306	7.015	6.702	6.488	6.476

Loss due to Creep of Concrete

Stress loss due to Creep	σ_{CS}	=	$\{\phi(t_2, to)-\phi(t1, to)\} \times E_p/E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of stage-I Cables	$\sigma_{cp,Avg,-I}$	=	7.626 N/mm ²	Modulus of elasticity of prestressing steel , E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg,-II}$	=	8.655 N/mm ²	Area of prestressing strand of Stage -I Prestress, A_{ps}		=	1974.00 mm ²
Creep coeff for age between from 7 to 63		=	$\phi(t1, to)$	Area of prestressing strand of Stage -II Prestress, A_{ps}		=	3158.40 mm ²
Creep coeff for age between from 7 to 49		=	$\phi(t2, to)$				

$\phi(t_2, t_0)$		0.684	0.684	0.701	0.734	0.734	0.734	0.734	0.734	0.734	0.734	0.701	0.684	0.684
$\phi(t_1, t_0)$		0.629	0.629	0.645	0.676	0.676	0.676	0.676	0.676	0.676	0.676	0.645	0.629	0.629
Creep loss for Stage I Cables	%	0.192	0.192	0.195	0.202	0.198	0.195	0.194	0.192	0.194	0.197	0.191	0.187	0.188
	kN	4.717	4.717	4.830	5.036	5.036	5.036	5.036	5.036	5.036	5.036	4.830	4.717	4.717

$\phi(t_2, t_0)$		0.684	0.684	0.701	0.734	0.734	0.734	0.734	0.734	0.734	0.734	0.701	0.684	0.684
$\phi(t_1, t_0)$		0.629	0.629	0.645	0.676	0.676	0.676	0.676	0.676	0.676	0.676	0.645	0.629	0.629
Creep loss for Stage II Cables	%	0.218	0.218	0.222	0.229	0.226	0.224	0.222	0.221	0.220	0.222	0.215	0.211	0.211
	kN	8.566	8.566	8.771	9.144	9.144	9.144	9.144	9.144	9.144	9.144	8.771	8.566	8.566

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 49 days to 63 days**

Total Prestress Loss for Stage-I Cables	%	0.298	0.298	0.312	0.331	0.325	0.320	0.318	0.315	0.318	0.323	0.304	0.290	0.291
	kN	7.309	7.309	7.702	8.259	8.259	8.259	8.259	8.259	8.259	8.259	7.702	7.309	7.309
Total Prestress Loss for Stage-II Cables	%	0.465	0.465	0.484	0.509	0.512	0.514	0.516	0.520	0.521	0.517	0.491	0.473	0.473
	kN	18.268	18.280	19.148	20.362	20.724	21.005	21.228	21.545	21.607	21.316	20.067	19.201	19.189

Stress after Loss of Prestress Due to SH+CR+RELEX Loss

From 49 days to 63 days

FOR STAGE-I & STAGE-II PRESTRESS**COMPOSITE SECTION****Stage I Cables**

Prestress After SH+CR+RELEX Loss(P_{hor})	kN	7.309	7.309	7.702	8.259	8.259	8.259	8.259	8.259	8.259	8.259	7.702	7.309	7.309
Eccentricity of Stage I cable from N.A. (ec)	m	-0.561	-0.535	-0.371	-0.262	-0.050	0.053	0.058	0.053	-0.049	-0.261	-0.371	-0.535	-0.561
$M_p = P_{hor} \times ec$	kNm	-4.104	-3.913	-2.860	-2.164	-0.410	0.435	0.482	0.434	-0.406	-2.159	-2.856	-3.908	-4.099

Stage II Cables

Prestress After SH+CR+RELEX Loss(P_{hor})	kN	18.268	18.280	19.148	20.362	20.724	21.005	21.228	21.545	21.607	21.316	20.067	19.201	19.189
Eccentricity of Stage II cable from N.A. (ec)	m	-0.080	-0.063	0.026	0.061	0.135	0.166	0.168	0.166	0.135	0.062	0.027	-0.062	-0.079
$M_p = P_{hor} \times ec$	kNm	-1.463	-1.151	0.496	1.245	2.798	3.497	3.571	3.583	2.919	1.321	0.541	-1.185	-1.513

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	25.577	25.590	26.849	28.621	28.983	29.264	29.487	29.804	29.866	29.576	27.769	26.510	26.498
M_p (Stage I & Stage II Cables)	kNm	-5.567	-5.064	-2.364	-0.919	2.388	3.932	4.053	4.017	2.513	-0.838	-2.315	-5.093	-5.612
P_{dor} / A	N/mm ²	0.014	0.014	0.016	0.020	0.020	0.020	0.021	0.021	0.021	0.021	0.017	0.014	0.014
M_p / Z_{ts}	N/mm ²	-0.006	-0.005	-0.003	-0.001	0.003	0.004	0.004	0.004	0.003	-0.001	-0.003	-0.005	-0.006
M_p / Z_{tg}	N/mm ²	-0.004	-0.004	-0.002	-0.001	0.002	0.003	0.003	0.003	0.002	-0.001	-0.002	-0.004	-0.004
M_p / Z_{bg}	N/mm ²	-0.011	-0.010	-0.005	-0.002	0.006	0.010	0.010	0.010	0.006	-0.002	-0.005	-0.010	-0.011
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.020	0.019	0.019	0.021	0.018	0.016	0.016	0.016	0.016	0.018	0.021	0.019	0.020

$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.018	0.017	0.018	0.021	0.019	0.018	0.018	0.018	0.019	0.021	0.018	0.018	0.018
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.003	0.004	0.011	0.018	0.026	0.030	0.031	0.031	0.027	0.018	0.012	0.004	0.003

NET STRESS IN GIRDER**(AT 63 days)****JUST BEFORE SIDL PLACEMENT**

At Top of Deck Slab,fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	2.761	2.388	3.556	5.701	6.865	8.085	8.828	8.025	6.782	5.658	3.536	2.427	2.810
At Bottom of Precast Girder,fbg	N/mm ²	7.809	8.216	9.747	11.521	10.545	9.470	8.820	9.776	11.027	12.034	10.142	8.479	8.061

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.663	4.679	6.803	9.338	9.653	9.223	8.821	9.464	9.999	9.645	7.003	4.808	4.791
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	7.607	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	6.181	6.400	8.340	10.513	10.077	9.321	8.821	9.588	10.488	10.934	8.645	6.597	6.372
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	8.637	N/mm ²											

STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT**(At 63 days)**

Moment due to SIDL1	kNm	0.00	22.62	126.63	245.19	414.50	523.51	560.00	523.50	414.54	245.19	126.63	22.62	0.00
M/Z_{ts}	N/mm ²	0.000	0.024	0.138	0.268	0.453	0.573	0.613	0.573	0.453	0.268	0.138	0.024	0.000
M/Z_{tg}	N/mm ²	0.000	0.016	0.088	0.162	0.275	0.347	0.371	0.347	0.275	0.163	0.088	0.016	0.000
$-M/Z_{bg}$	N/mm ²	0.000	-0.043	-0.275	-0.606	-1.025	-1.295	-1.385	-1.295	-1.025	-0.606	-0.275	-0.043	0.000

STRESS IN COMPOSITE GIRDER DUE TO SIDL2 DUE TO WEARING COAT**(At 63 days)**

Moment due to SIDL2	kNm	0.00	12.24	61.59	101.98	156.51	191.61	203.16	191.60	156.52	101.98	61.59	12.24	0.00
M/Z_{ts}	N/mm ²	0.000	0.013	0.067	0.112	0.171	0.210	0.222	0.210	0.171	0.112	0.067	0.013	0.000
M/Z_{tg}	N/mm ²	0.000	0.009	0.043	0.068	0.104	0.127	0.135	0.127	0.104	0.068	0.043	0.009	0.000
$-M/Z_{bg}$	N/mm ²	0.000	-0.023	-0.134	-0.252	-0.387	-0.474	-0.502	-0.474	-0.387	-0.252	-0.134	-0.023	0.000

NET STRESS IN GIRDER**(AT 63 days)****JUST AFTER SIDL PLACEMENT**

At Top of Deck Slab,fts	N/mm ²	0.000	0.037	0.206	0.380	0.625	0.782	0.835	0.782	0.625	0.380	0.206	0.037	0.000
At Top of Precast Girder,ftg	N/mm ²	2.761	2.413	3.687	5.931	7.243	8.558	9.333	8.499	7.161	5.888	3.668	2.452	2.810
At Bottom of Precast Girder,fbg	N/mm ²	7.809	8.150	9.339	10.663	9.133	7.701	6.932	8.008	9.614	11.176	9.734	8.412	8.061

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.663	4.668	6.651	8.888	8.675	7.854	7.352	8.095	9.021	9.194	6.851	4.797	4.791
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	7.039	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	6.181	6.362	8.054	9.843	8.893	7.793	7.187	8.060	9.303	10.263	8.359	6.559	6.372
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	7.941	N/mm ²											

(5)**LOSS OF PRESTRESS****From 63 days to Infinity****FOR STAGE-I & STAGE-II PRESTRESS****COMPOSITE SECTION****Loss due to Shrinkage**

Shrinkage Loss for Stage I Cables

E_p	=	1.95E+05	N/mm ²
Area of cables in stage -I Prestress, A_{cable}	=	1974.000	mm ²

Differential shrinkage strain = ecs	μc	171.020	171.020	168.746	167.638	167.638	167.638	167.638	167.638	167.638	167.638	168.746	171.020	171.020
shrinkage loss between (63 days to Infinity)	%	2.685	2.684	2.628	2.585	2.542	2.503	2.481	2.461	2.484	2.523	2.563	2.616	2.617
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	65.831	65.831	64.955	64.529	64.529	64.529	64.529	64.529	64.529	64.529	64.955	65.831	65.831

Shrinkage Loss for Stage II Cables

E_s	=	1.95E+05	N/mm ²
Area of cables in stage -II Prestress, A_{cable}	=	3158.400	mm ²

Differential shrinkage strain = ecs	μc	171.020	171.020	168.746	167.638	167.638	167.638	167.638	167.638	167.638	167.638	168.746	171.020	171.020
shrinkage loss between (63 days to Infinity)	%	2.682	2.681	2.625	2.583	2.551	2.527	2.510	2.492	2.489	2.505	2.544	2.597	2.598
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	105.329	105.329	103.928	103.246	103.246	103.246	103.246	103.246	103.246	103.246	103.928	105.329	105.329

Loss due to Relaxation of steel**FOR STAGE-I CABLES:**

As per Table 6.3 of IRC:112-2011

$$\text{Relaxation loss as \% of loss at 1000 hrs} = 3.00 \times 100 \% = 300 \%$$

$$\text{Relaxation Loss between 7days \& 63days} = 100.00 \%$$

$$\text{Relaxation Loss between 63days \& Infinitydays} = 300.00 - 100.00$$

$$= 200.00 \%$$

$$\% \text{ Relaxation Loss} = \text{Relaxation loss at 1000 hrs} \times (\text{Relaxation loss as \% of loss at 1000 hrs})/100$$

$$\text{Relaxation Loss} = \% \text{ Relaxation Loss}/100 \times \text{Prestress after F+S+ES Loss}$$

% Relaxation Loss For Stage I Cables	%	4.194	4.202	4.330	4.497	4.784	5.092	5.333	5.568	5.305	4.914	4.755	4.635	4.628
Relaxation Loss For Stage I Cables	kN	102.824	103.059	107.028	112.249	121.455	131.292	138.699	145.996	137.819	125.675	120.512	116.634	116.411

FOR STAGE-II CABLES:

As per Table 6.3 of IRC:112-2011

$$\text{Relaxation loss as \% of loss at 1000 hrs} = 3.00 \times 100 \% = 300 \% \text{ at Infinity}$$

$$\text{Relaxation Loss between 28days \& 63days} = 96.80 \%$$

$$\text{Relaxation Loss between 63days \& Infinitydays} = 300.00 - 96.80$$

$$= 203.20 \%$$

$$\% \text{ Relaxation Loss} = \text{Relaxation loss at 1000 hrs} \times (\text{Relaxation loss as \% of loss at 1000 hrs})/100$$

$$\text{Relaxation Loss} = \% \text{ Relaxation Loss}/100 \times \text{Prestress after F+S+ES Loss}$$

Relaxation Loss between 63days & Infinitydays

% Relaxation Loss For Stage II Cables	%	4.278	4.285	4.417	4.584	4.799	4.962	5.091	5.288	5.326	5.146	4.961	4.837	4.830
Relaxation Loss For Stage II Cables	kN	167.967	168.349	174.849	183.243	194.208	202.707	209.438	219.030	220.906	212.116	202.657	196.190	195.818

Loss due to Creep of Concrete

Stress loss due to Creep	σ_{cs}	=	$\{\phi(t_2, t_0) - \phi(t_1, t_0)\} \times E_p/E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of stage-I Cables	$\sigma_{cp,Avg,-I}$	=	7.039 N/mm ²	Modulus of elasticity of prestressing steel , E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg,-II}$	=	7.941 N/mm ²	Area of prestressing strand of Stage -I Prestress, A_{ps}		=	1974 mm ²
Creep coeff for age between from 7 to Infinity		=	$\phi(t_1, t_0)$	Area of prestressing strand of Stage -II Prestress, A_{ps}		=	3158.40 mm ²
Creep coeff for age between from 7 to 63		=	$\phi(t_2, t_0)$				

$\phi(t_2, t_0)$		1.793	1.793	1.815	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.815	1.793	1.793
$\phi(t_1, t_0)$		0.684	0.684	0.701	0.734	0.734	0.734	0.734	0.734	0.734	0.734	0.701	0.684	0.684
Creep loss for Stage I Cables	%	3.605	3.603	3.591	3.530	3.471	3.417	3.388	3.360	3.391	3.445	3.502	3.512	3.513
	kN	88.381	88.381	88.759	88.111	88.111	88.111	88.111	88.111	88.111	88.111	88.759	88.381	88.381

$\phi(t_2, t_0)$		1.793	1.793	1.815	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.815	1.793	1.793
$\phi(t_1, t_0)$		0.684	0.684	0.701	0.734	0.734	0.734	0.734	0.734	0.734	0.734	0.701	0.684	0.684
Creep loss for Stage II Cables	%	4.063	4.061	4.047	3.979	3.930	3.894	3.866	3.840	3.835	3.859	3.922	3.933	3.935
	kN	159.539	159.539	160.222	159.053	159.053	159.053	159.053	159.053	159.053	159.053	160.222	159.539	159.539

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 63 days to Infinity**

Total Prestress Loss for Stage-I Cables	%	10.484	10.488	10.549	10.611	10.797	11.011	11.202	11.388	11.180	10.882	10.821	10.763	10.758
	kN	257.036	257.271	260.742	264.890	274.096	283.933	291.340	298.636	290.459	278.316	274.227	270.845	270.622
Total Prestress Loss for Stage-II Cables	%	11.023	11.028	11.089	11.145	11.280	11.383	11.467	11.620	11.650	11.510	11.428	11.367	11.363
	kN	432.836	433.218	438.999	445.543	456.507	465.006	471.737	481.330	483.205	474.415	466.808	461.059	460.687

Stress after Loss of Prestress Due to SH+CR+RLEX Loss

From 63 days to Infinity

FOR STAGE-I & STAGE-II PRESTRESS

COMPOSITE SECTION

Stage I Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	257.036	257.271	260.742	264.890	274.096	283.933	291.340	298.636	290.459	278.316	274.227	270.845	270.622
Eccentricity of Stage I cable from N.A. (ec)	m	-0.561	-0.535	-0.371	-0.262	-0.050	0.053	0.058	0.053	-0.049	-0.261	-0.371	-0.535	-0.561
$M_p = P_{hor} \times ec$	kNm	-144.321	-137.729	-96.833	-69.390	-13.596	14.956	17.017	15.690	-14.269	-72.753	-101.675	-144.813	-151.763

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	432.836	433.218	438.999	445.543	456.507	465.006	471.737	481.330	483.205	474.415	466.808	461.059	460.687
Eccentricity of Stage II cable from N.A. (ec)	m	-0.080	-0.063	0.026	0.061	0.135	0.166	0.168	0.166	0.135	0.062	0.027	-0.062	-0.079
$M_p = P_{hor} \times ec$	kNm	-34.671	-27.283	11.373	27.232	61.632	77.414	79.351	80.056	65.278	29.404	12.585	-28.459	-36.315

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	689.872	690.489	699.742	710.432	730.603	748.938	763.077	779.966	773.664	752.730	741.034	731.904	731.309
M_p (Stage I & Stage II Cables)	kNm	-178.991	-165.012	-85.460	-42.158	48.036	92.370	96.368	95.747	51.009	-43.348	-89.089	-173.272	-188.077
P_{hor} / A	N/mm ²	0.366	0.366	0.424	0.494	0.508	0.521	0.531	0.542	0.538	0.523	0.449	0.388	0.388
M_p / Z_{ts}	N/mm ²	-0.192	-0.177	-0.093	-0.046	0.053	0.101	0.105	0.105	0.056	-0.047	-0.097	-0.186	-0.202
M_p / Z_{ta}	N/mm ²	-0.128	-0.118	-0.060	-0.028	0.032	0.061	0.064	0.063	0.034	-0.029	-0.062	-0.124	-0.134
M_p / Z_{bg}	N/mm ²	-0.342	-0.315	-0.186	-0.104	0.119	0.228	0.238	0.237	0.126	-0.107	-0.193	-0.331	-0.359
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.558	0.544	0.517	0.540	0.455	0.420	0.425	0.438	0.482	0.571	0.546	0.574	0.590
$P_{hor} / A - M_p / Z_{ta}$	N/mm ²	0.494	0.484	0.484	0.522	0.476	0.460	0.467	0.479	0.504	0.552	0.511	0.512	0.522
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.024	0.051	0.238	0.390	0.627	0.749	0.769	0.779	0.664	0.416	0.255	0.057	0.029

NET STRESS IN GIRDER

(At Infinity

At Top of Deck Slab,fts	N/mm ²	-0.558	-0.506	-0.311	-0.160	0.169	0.363	0.410	0.345	0.143	-0.191	-0.340	-0.537	-0.590
At Top of Precast Girder,ftg	N/mm ²	2.267	1.929	3.204	5.409	6.767	8.099	8.867	8.020	6.657	5.336	3.157	1.940	2.288
At Bottom of Precast Girder,fbg	N/mm ²	7.785	8.099	9.100	10.273	8.506	6.952	6.163	7.229	8.950	10.760	9.478	8.355	8.032

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.346	4.354	6.296	8.448	8.084	7.157	6.636	7.370	8.395	8.727	6.474	4.465	4.455

Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg}) = 6.554 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	6.006	6.176	7.760	9.431	8.285	7.075	6.451	7.314	8.659	9.823	8.046	6.361	6.185

Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg}) = 7.506 N/mm²**Stress in Composite Girder due to FPLL**

(At Infinity

Bending Moment due to FPLL	kNm	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Stress in Composite Girder due to CWLL (WITH IMPACT)**(At Infinity)**

Bending Moment due to CWLL	kNm	19.05	432.20	822.57	847.39	1601.44	2323.61	2612.56	2335.63	1621.17	809.77	410.92	413.64	19.52
At Top of Deck Slab, fts	N/mm ²	0.020	0.465	0.899	0.927	1.752	2.542	2.858	2.555	1.773	0.886	0.449	0.445	0.021
At Top of Precast Girder,ftg	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.296	0.014
At Bottom of Precast Girder,fbg	N/mm ²	-0.036	-0.825	-1.786	-2.096	-3.960	-5.746	-6.461	-5.776	-4.009	-2.003	-0.892	-0.790	-0.037

Stress in Composite Girder due to CWLL + FPLL**(At Infinity)**

At Top of Deck Slab, fts	N/mm ²	0.020	0.465	0.899	0.927	1.752	2.542	2.858	2.555	1.773	0.886	0.449	0.445	0.021
At Top of Precast Girder,ftg	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.296	0.014
At Bottom of Precast Girder,fbg	N/mm ²	-0.036	-0.825	-1.786	-2.096	-3.960	-5.746	-6.461	-5.776	-4.009	-2.003	-0.892	-0.790	-0.037

NET STRESS IN GIRDER**(At Infinity)****JUST AFTER CWLL+FPLL**

At Top of Deck Slab, fts	N/mm ²	-0.538	-0.041	0.588	0.767	1.921	2.904	3.268	2.900	1.916	0.695	0.109	-0.092	-0.569
At Top of Precast Girder,ftg	N/mm ²	2.281	2.238	3.779	5.970	7.828	9.639	10.598	9.568	7.731	5.873	3.444	2.235	2.302
At Bottom of Precast Girder,fbg	N/mm ²	7.748	7.273	7.315	8.177	4.545	1.206	-0.298	1.452	4.941	8.757	8.586	7.565	7.995

Stress at c.g of Stage I Cables	N/mm ²	4.341	4.217	5.633	7.349	5.341	2.710	1.607	2.901	5.616	7.676	6.143	4.333	4.449
Average stress in concrete at c.g. of Stage- I Cable($\sigma_{ca\ avg}$)	=	4.794	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	5.986	5.704	6.511	7.795	4.962	2.111	0.860	2.324	5.295	8.259	7.421	5.908	6.164
Average stress in concrete at c.g. of Stage- II Cable($\sigma_{ca\ avg}$)	=	5.331	N/mm ²											

FINAL SUMMARY OF PRESTRESS LOSSES**FOR STAGE-I PRESTRESS**

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15

LOSS OF PRESTRESS DUE TO FRICTION AND SLIP**Before Transfer of Prestress**

Prestress Force applied at Jacks	kN	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825	2808.825
Prestress Force at Sec. after (fric.+slip)	kN	2474.245	2475.368	2494.240	2518.722	2560.988	2601.051	2623.212	2644.735	2620.594	2579.998	2556.709	2538.991	2537.966
Prestress Loss due to (Friction+Slip)	%	11.912	11.872	11.200	10.328	8.824	7.397	6.608	5.842	6.701	8.147	8.976	9.607	9.643
	kN	334.580	333.457	314.585	290.103	247.837	207.774	185.613	164.090	188.231	228.827	252.116	269.834	270.859

LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING**(At 7 days)**

Elastic Shortening Loss	%	0.908	0.908	0.901	0.892	0.877	0.864	0.857	0.850	0.857	0.871	0.879	0.885	0.885
	kN	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471	22.471

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 7 days to 28 days**

Shrinkage Loss	%	0.797	0.796	0.818	0.848	0.834	0.821	0.814	0.807	0.815	0.827	0.798	0.776	0.776
	kN	19.528	19.528	20.227	21.163	21.163	21.163	21.163	21.163	21.163	21.163	20.227	19.528	19.528
Creep Loss	%	0.744	0.744	0.757	0.786	0.772	0.760	0.754	0.748	0.755	0.767	0.738	0.725	0.725
	kN	18.242	18.242	18.714	19.608	19.608	19.608	19.608	19.608	19.608	19.608	18.714	18.242	18.242
Relaxation Loss	%	1.889	1.892	1.950	2.025	2.155	2.293	2.402	2.508	2.389	2.213	2.142	2.087	2.084
	kN	46.312	46.418	48.205	50.557	54.703	59.134	62.470	65.757	62.074	56.604	54.279	52.532	52.431
TOTAL PRESTRESS LOSS (S + C + R)	%	3.429	3.432	3.526	3.659	3.761	3.874	3.970	4.062	3.958	3.807	3.678	3.588	3.586
	kN	84.082	84.188	87.147	91.329	95.475	99.906	103.242	106.528	102.845	97.376	93.220	90.302	90.201

LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING**AT 28 days**

Elastic Shortening Loss	%	3.357	3.356	3.330	3.298	3.244	3.194	3.167	3.141	3.170	3.220	3.249	3.272	3.273
	kN	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070	83.070

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 28 days to 35 days**

Shrinkage Loss	%	0.076	0.076	0.082	0.090	0.089	0.087	0.087	0.086	0.087	0.088	0.080	0.074	0.074
	kN	1.860	1.860	2.031	2.254	2.254	2.254	2.254	2.254	2.254	2.254	2.031	1.860	1.860
Creep Loss	%	0.210	0.210	0.214	0.221	0.217	0.214	0.212	0.211	0.213	0.216	0.208	0.205	0.205
	kN	5.153	5.153	5.282	5.521	5.521	5.521	5.521	5.521	5.521	5.521	5.282	5.153	5.153
Relaxation Loss	%	0.070	0.071	0.073	0.076	0.080	0.086	0.090	0.094	0.089	0.083	0.080	0.078	0.078
	kN	1.727	1.731	1.798	1.886	2.040	2.206	2.330	2.453	2.315	2.111	2.025	1.959	1.956
TOTAL PRESTRESS LOSS (S + C + R)	%	0.356	0.356	0.369	0.387	0.387	0.387	0.389	0.390	0.388	0.387	0.368	0.357	0.357
	kN	8.740	8.744	9.111	9.661	9.816	9.981	10.105	10.228	10.091	9.886	9.337	8.972	8.968

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 35 days to 49 days**

Shrinkage Loss	%	0.127	0.127	0.139	0.153	0.151	0.148	0.147	0.146	0.147	0.150	0.135	0.124	0.124
	kN	3.111	3.111	3.424	3.827	3.827	3.827	3.827	3.827	3.827	3.827	3.424	3.111	3.111
Creep Loss	%	0.250	0.249	0.254	0.262	0.258	0.254	0.252	0.250	0.252	0.256	0.247	0.243	0.243
	kN	6.118	6.118	6.268	6.545	6.545	6.545	6.545	6.545	6.545	6.545	6.268	6.118	6.118
Relaxation Loss	%	0.138	0.138	0.142	0.147	0.157	0.167	0.175	0.183	0.174	0.161	0.156	0.152	0.152
	kN	3.373	3.380	3.511	3.682	3.984	4.306	4.549	4.789	4.520	4.122	3.953	3.826	3.818
TOTAL PRESTRESS LOSS (S + C + R)	%	0.514	0.514	0.534	0.563	0.565	0.569	0.574	0.578	0.573	0.567	0.538	0.519	0.519
	kN	12.601	12.609	13.202	14.053	14.355	14.678	14.921	15.160	14.892	14.494	13.645	13.054	13.047

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 49 days to 63 days**

Shrinkage Loss	%	0.106	0.106	0.116	0.129	0.127	0.125	0.124	0.123	0.124	0.126	0.113	0.103	0.103
	kN	2.592	2.592	2.871	3.224	3.224	3.224	3.224	3.224	3.224	3.224	2.871	2.592	2.592
Creep Loss	%	0.192	0.192	0.195	0.202	0.198	0.195	0.194	0.192	0.194	0.197	0.191	0.187	0.188
	kN	4.717	4.717	4.830	5.036	5.036	5.036	5.036	5.036	5.036	5.036	4.830	4.717	4.717
Relaxation Loss	%	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	kN	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL PRESTRESS LOSS (S + C + R)	%	0.298	0.298	0.312	0.331	0.325	0.320	0.318	0.315	0.318	0.323	0.304	0.290	0.291
	kN	7.309	7.309	7.702	8.259	8.259	8.259	8.259	8.259	8.259	8.259	7.702	7.309	7.309

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 63 days to Infinity**

Shrinkage Loss	%	2.685	2.684	2.628	2.585	2.542	2.503	2.481	2.461	2.484	2.523	2.563	2.616	2.617
	kN	65.831	65.831	64.955	64.529	64.529	64.529	64.529	64.529	64.529	64.529	64.955	65.831	65.831
Creep Loss	%	3.605	3.603	3.591	3.530	3.471	3.417	3.388	3.360	3.391	3.445	3.502	3.512	3.513
	kN	88.381	88.381	88.759	88.111	88.111	88.111	88.111	88.111	88.111	88.111	88.759	88.381	88.381
Relaxation Loss	%	4.194	4.202	4.330	4.497	4.784	5.092	5.333	5.568	5.305	4.914	4.755	4.635	4.628
	kN	102.824	103.059	107.028	112.249	121.455	131.292	138.699	145.996	137.819	125.675	120.512	116.634	116.411
TOTAL PRESTRESS LOSS (S + C + R)	%	10.484	10.488	10.549	10.611	10.797	11.011	11.202	11.388	11.180	10.882	10.821	10.763	10.758
	kN	257.036	257.271	260.742	264.890	274.096	283.933	291.340	298.636	290.459	278.316	274.227	270.845	270.622

TOTAL PRESTRESS LOSS**From 7 days to Infinity**

STAGE I CABLES	%	20.775	20.736	20.171	19.458	17.983	16.606	15.881	15.178	15.967	17.321	17.993	18.518	18.553
	kN	552.854	551.848	537.288	518.946	481.284	446.140	427.682	409.807	429.860	464.384	481.561	495.013	495.926

FOR STAGE-II PRESTRESS

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15

LOSS OF PRESTRESS DUE TO FRICTION AND SLIP**AT 28 days**

Prestress Force applied at Jacks	kN	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120
Prestress Force at Sec. after (fric.+slip)	kN	4014.661	4016.458	4046.826	4085.509	4135.163	4173.001	4201.841	4230.266	4235.785	4209.810	4172.780	4144.038	4142.374
Prestress Loss due to (Friction+Slip)	%	10.669	10.629	9.953	9.092	7.987	7.145	6.504	5.871	5.748	6.326	7.150	7.790	7.827
	kN	479.459	477.662	447.294	408.611	358.957	321.119	292.279	263.854	258.335	284.310	321.340	350.082	351.746

LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING**AT 28 days**

Elastic Shortening Loss	%	2.191	2.190	2.174	2.153	2.128	2.108	2.094	2.080	2.077	2.090	2.108	2.123	2.124
	kN	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980	87.980

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 28 days to 35 days**

Shrinkage Loss	%	0.076	0.076	0.082	0.090	0.089	0.088	0.088	0.087	0.087	0.087	0.080	0.073	0.073
	kN	2.976	2.976	3.249	3.606	3.606	3.606	3.606	3.606	3.606	3.606	3.249	2.976	2.976
Creep Loss	%	0.253	0.253	0.257	0.267	0.263	0.261	0.259	0.257	0.257	0.258	0.250	0.245	0.245
	kN	9.943	9.943	10.192	10.654	10.654	10.654	10.654	10.654	10.654	10.654	10.192	9.943	9.943
Relaxation Loss	%	1.616	1.619	1.668	1.732	1.813	1.874	1.923	1.997	2.012	1.944	1.874	1.827	1.824
	kN	63.451	63.595	66.050	69.221	73.363	76.574	79.116	82.740	83.449	80.128	76.555	74.112	73.971
TOTAL PRESTRESS LOSS (S + C + R)	%	1.945	1.948	2.008	2.088	2.165	2.224	2.270	2.342	2.356	2.290	2.203	2.146	2.143
	kN	76.370	76.514	79.492	83.482	87.624	90.834	93.377	97.001	97.709	94.389	89.996	87.031	86.891

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 35 days to 49 days**

Shrinkage Loss	%	0.127	0.127	0.138	0.153	0.151	0.150	0.149	0.148	0.148	0.149	0.134	0.123	0.123
	kN	4.977	4.977	5.478	6.123	6.123	6.123	6.123	6.123	6.123	6.123	5.478	4.977	4.977
Creep Loss	%	0.283	0.283	0.288	0.297	0.294	0.291	0.289	0.287	0.287	0.289	0.279	0.274	0.274
	kN	11.116	11.116	11.390	11.892	11.892	11.892	11.892	11.892	11.892	11.892	11.390	11.116	11.116
Relaxation Loss	%	0.280	0.281	0.290	0.300	0.315	0.325	0.334	0.347	0.349	0.337	0.325	0.317	0.317
	kN	11.010	11.036	11.462	12.012	12.731	13.288	13.729	14.358	14.481	13.904	13.284	12.861	12.836
TOTAL PRESTRESS LOSS (S + C + R)	%	0.690	0.691	0.716	0.751	0.760	0.766	0.772	0.782	0.783	0.774	0.738	0.714	0.714
	kN	27.104	27.129	28.330	30.027	30.746	31.303	31.744	32.373	32.496	31.920	30.152	28.954	28.930

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 49 days to 63 days**

Shrinkage Loss	%	0.106	0.106	0.116	0.129	0.127	0.126	0.125	0.125	0.124	0.125	0.112	0.102	0.102
	kN	4.147	4.147	4.594	5.158	5.158	5.158	5.158	5.158	5.158	5.158	4.594	4.147	4.147
Creep Loss	%	0.218	0.218	0.222	0.229	0.226	0.224	0.222	0.221	0.220	0.222	0.215	0.211	0.211
	kN	8.566	8.566	8.771	9.144	9.144	9.144	9.144	9.144	9.144	9.144	8.771	8.566	8.566
Relaxation Loss	%	0.141	0.142	0.146	0.152	0.159	0.164	0.168	0.175	0.176	0.170	0.164	0.160	0.160
	kN	5.555	5.567	5.782	6.060	6.423	6.704	6.926	7.244	7.306	7.015	6.702	6.488	6.476
TOTAL PRESTRESS LOSS (S + C + R)	%	0.465	0.465	0.484	0.509	0.512	0.514	0.516	0.520	0.521	0.517	0.491	0.473	0.473
	kN	18.268	18.280	19.148	20.362	20.724	21.005	21.228	21.545	21.607	21.316	20.067	19.201	19.189

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 63 days to Infinity**

Shrinkage Loss	%	2.682	2.681	2.625	2.583	2.551	2.527	2.510	2.492	2.489	2.505	2.544	2.597	2.598
	kN	105.329	105.329	103.928	103.246	103.246	103.246	103.246	103.246	103.246	103.246	103.928	105.329	105.329
Creep Loss	%	4.063	4.061	4.047	3.979	3.930	3.894	3.866	3.840	3.835	3.859	3.922	3.933	3.935
	kN	159.539	159.539	160.222	159.053	159.053	159.053	159.053	159.053	159.053	159.053	160.222	159.539	159.539
Relaxation Loss	%	4.278	4.285	4.417	4.584	4.799	4.962	5.091	5.288	5.326	5.146	4.961	4.837	4.830
	kN	167.967	168.349	174.849	183.243	194.208	202.707	209.438	219.030	220.906	212.116	202.657	196.190	195.818
TOTAL PRESTRESS LOSS (S + C + R)	%	11.023	11.028	11.089	11.145	11.280	11.383	11.467	11.620	11.650	11.510	11.428	11.367	11.363
	kN	432.836	433.218	438.999	445.543	456.507	465.006	471.737	481.330	483.205	474.415	466.808	461.059	460.687

TOTAL PRESTRESS LOSS**From 28 days to Infinity**

STAGE II CABLES	%	15.960	15.923	15.334	14.594	13.552	12.758	12.155	11.594	11.485	11.998	12.691	13.246	13.281
	kN	689.181	687.566	662.243	630.461	586.031	552.242	526.608	502.753	498.127	519.915	549.536	573.249	574.736

Classification of Characteristic Actions used for Superstructure Design

Permanent Action (G)		Variable Action (Q)		Accidental Action (A) / Seismic Action	
Symbol	Action Designation & Description	Symbol	Action Designation & Description	Symbol	Action Designation & Description
G1	DL : Dead load	Q1	Qs : Pedestrian LL	A1	VC : Vehicular collision on element of bridge structure
G2	SIDL1 : SIDL excluding surfacing	Q2	QL : CWLL	A2	Barge impact or impact due to floating bodies in water current
G3	SIDL2: Wearing coat weight	Q3	Fb : Braking effect of the vehicle	A3	Vehicular impact on crash barrier
G4	GS : snow load	Q4	Fcf : Centrifugal force of vehicle	A4	AE : Seismic action
G5	SE: Settlement effect	Q5	Temperature effect due to restraint to free structural deformation		
G6	CR: Creep effect				
G7	SH: Shrinkage effect				
G8	P : Prestressing Force				

G9 G10	FS: Secondary effect of prestress DSH: Differential Shrinkage				

LOAD COMBINATION :

SLS : Serviceability Limit State

ULS : Ultimate Limit State

Description of Combination		Stage	Details of Force in Combination	Combination With Load Factor	Purpose of Checking
RARE	SLS 1	Non-Composite	Prestress (Stage 1) +DL _{gir}	1.0 G8 + 1.0 G1	(7 days) Maximum Stresses
	SLS 2	Non-Composite	Prestress (Stage 1 & 2) +DL _{gir}	1.0 G8 + 1.0 G1	(28 days) Maximum Stresses
	SLS 3	Non-Composite	Prestress (Stage 1 & 2) +DL _{gir} + DL _{slab} (green wt.)	1.0 G8 + 1.0 G1	(35 days) Maximum Stresses
	SLS 4	Composite	Prestress (Stage 1 & 2) +DL _{gir} + DL _{slab} + DSH	1.0 G8 + 1.0 G1 +1.0 G10	(49 days) Maximum Stresses
	SLS 5	Composite	Prestress (Stage 1 & 2) +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ DSH+ Q	1.0 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10 + 1.0 (Q1+Q2) + 0.6 Q5	(63 days) Maximum Stresses
	SLS 6	Composite	Final Prestress +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ DSH+ Q	1.1 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10 + 1.0 (Q1+Q2) + 0.6 Q5	(Long Term) Maximum stresses with γ_{sup}
FREQUENT	SLS 7	Composite	Final Prestress +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ DSH+ Q	0.9 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10 + 1.0 (Q1+Q2) + 0.6 Q5	(Long Term) Maximum stresses with γ_{inf}
	SLS 8	Composite	Final Prestress +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ DSH+ Q	1.0 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 0.75 G10 + 0.75 (Q1+Q2) + 0.5 Q5	(Long Term) Decompression , Crack width
QUASI-PERMANENT	SLS 9	Composite	Final Prestress +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ DSH+ Q	1.0 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 0.75 G10 + 0.5 Q5	(Long Term) Deflection
ULTIMATE (BASIC)	USL 1	Composite	Final Prestress +DL _{gir} + DL _{slab} + SIDL1+SIDL 2+ Q	1.0 G8 + 1.35 G1 + 1.35 G2 + 1.75 G3 + 1.5 (Q1+Q2)	(Long Term) Strength check in ULS

SUMMARY OF STRESS DUE TO PRESTRESS

(G8)

1 Stress due to Prestress after Friction & Slip loss in Stage I Cables

Before Transfer of Prestress

(At 7 days)

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.308	3.121	2.120	1.543	-0.347	-1.290	-1.354	-1.310	-0.359	1.576	2.168	3.196	3.388
At Bottom of Precast Girder,fbg	N/mm ²	0.820	1.024	3.250	5.516	7.616	8.718	8.848	8.863	7.798	5.656	3.337	1.056	0.847

2 Stress due to Elastic Shortening Loss from Stage I Cables

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.030	0.028	0.019	0.014	-0.003	-0.011	-0.012	-0.011	-0.003	0.014	0.019	0.028	0.030
At Bottom of Precast Girder,fbg	N/mm ²	0.007	0.009	0.029	0.049	0.067	0.075	0.076	0.075	0.067	0.049	0.029	0.009	0.007

3 Stress due to Loss of Prestress Due to SH+CR+RLEX Loss

FOR STAGE-I PRESTRESS

From 7 days to 28 days

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.112	0.106	0.074	0.056	-0.013	-0.050	-0.053	-0.053	-0.014	0.059	0.079	0.114	0.120
At Bottom of Precast Girder,fbg	N/mm ²	0.028	0.035	0.114	0.200	0.284	0.335	0.348	0.357	0.306	0.213	0.122	0.038	0.030

4 Stress due to Prestress after Friction & Slip loss in Stage II Cables

AT 28 days

NON-COMPOSITE SECTION

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	-0.260	-0.460	-1.727	-2.147	-3.250	-3.742	-3.794	-3.791	-3.330	-2.225	-1.795	-0.490	-0.283
At Bottom of Precast Girder,fbg	N/mm ²	7.415	7.635	10.827	13.821	15.117	15.740	15.875	15.953	15.486	14.254	11.179	7.894	7.667

5 Stress due to Stage-I Pres. after Additional Elastic Shortening from Stage-II

AT 28 days

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.111	0.105	0.071	0.051	-0.011	-0.041	-0.043	-0.041	-0.011	0.051	0.070	0.105	0.111
At Bottom of Precast Girder,fbg	N/mm ²	0.028	0.034	0.108	0.182	0.247	0.278	0.280	0.278	0.247	0.182	0.108	0.035	0.028

6 Stress due to Stage-II Pres. after Elastic Shortening from Stage-II

AT 28 days

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

	At Top of Precast Girder,ftg	N/mm ²	-0.006	-0.010	-0.038	-0.046	-0.069	-0.079	-0.079	-0.079	-0.069	-0.047	-0.038	-0.010	-0.006
	At Bottom of Precast Girder,fbg	N/mm ²	0.162	0.167	0.235	0.298	0.322	0.332	0.332	0.332	0.322	0.298	0.236	0.168	0.163
7	Stress due to Loss of Prestress Due to SH+CR+RLEX Loss		FOR STAGE-I & STAGE-II PRESTRESS				From 28 days to 35 days			NON-COMPOSITE SECTION					
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.007	0.002	-0.026	-0.038	-0.070	-0.086	-0.090	-0.092	-0.078	-0.044	-0.031	0.001	0.006
	At Bottom of Precast Girder,fbg	N/mm ²	0.144	0.149	0.225	0.304	0.350	0.376	0.387	0.400	0.387	0.341	0.253	0.170	0.164
8	Stress after Loss of Prestress Due to SH+CR+RLEX Loss		FOR STAGE-I & STAGE-II PRESTRESS				From 35 days to 49 days			NON-COMPOSITE SECTION					
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.015	0.013	-0.001	-0.007	-0.026	-0.035	-0.036	-0.037	-0.028	-0.008	-0.001	0.013	0.015
	At Bottom of Precast Girder,fbg	N/mm ²	0.054	0.057	0.093	0.132	0.155	0.167	0.170	0.173	0.163	0.140	0.099	0.061	0.058
9	Stress after Loss of Prestress Due to SH+CR+RLEX Loss		FOR STAGE-I & STAGE-II PRESTRESS				From 49 days to 63 days			COMPOSITE SECTION					
	At Top of Deck Slab, fts	N/mm ²	0.020	0.019	0.019	0.021	0.018	0.016	0.016	0.016	0.018	0.021	0.019	0.020	0.020
	At Bottom of Deck Slab, fbs	N/mm ²	0.018	0.017	0.018	0.021	0.019	0.018	0.018	0.018	0.019	0.021	0.018	0.018	0.018
	At Top of Precast Girder,ftg	N/mm ²	0.018	0.017	0.018	0.021	0.019	0.018	0.018	0.018	0.019	0.021	0.018	0.018	0.018
	At Bottom of Precast Girder,fbg	N/mm ²	0.003	0.004	0.011	0.018	0.026	0.030	0.031	0.031	0.027	0.018	0.012	0.004	0.003
10	Stress after Loss of Prestress Due to SH+CR+RLEX Loss		FOR STAGE-I & STAGE-II PRESTRESS				From 63 days to Infinity			COMPOSITE SECTION					
	At Top of Deck Slab, fts	N/mm ²	0.558	0.544	0.517	0.540	0.455	0.420	0.425	0.438	0.482	0.571	0.546	0.574	0.590
	At Bottom of Deck Slab, fbs	N/mm ²	0.494	0.484	0.484	0.522	0.476	0.460	0.467	0.479	0.504	0.552	0.511	0.512	0.522
	At Top of Precast Girder,ftg	N/mm ²	0.494	0.484	0.484	0.522	0.476	0.460	0.467	0.479	0.504	0.552	0.511	0.512	0.522
	At Bottom of Precast Girder,fbg	N/mm ²	0.024	0.051	0.238	0.390	0.627	0.749	0.769	0.779	0.664	0.416	0.255	0.057	0.029

SUMMARY OF STRESS IN CONCRETE (EXCLUDING PRESTRESS EFFECT)

1	Stress Due to Self Weight of Girder (G1,1)		(At 7 days)												
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.000	-0.005	1.147	2.185	3.487	4.329	4.609	4.329	3.487	2.185	1.147	-0.005	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.006	-1.233	-2.290	-3.654	-4.536	-4.830	-4.536	-3.654	-2.290	-1.233	0.006	0.000
2	Stress Due to Weight of Shuttering on Girder (G1,2)		(At 35 days)												
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.000	0.000	0.026	0.051	0.084	0.105	0.112	0.105	0.084	0.051	0.026	0.000	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	-0.028	-0.054	-0.088	-0.110	-0.117	-0.110	-0.088	-0.054	-0.028	0.000	0.000
3	Stress in Girder Due to Green Weight of Deck slab (G1,3)		(At 35 days)												
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.000	-0.006	2.113	4.128	6.733	8.418	8.980	8.418	6.733	4.128	2.113	-0.006	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.007	-2.271	-4.326	-7.056	-8.822	-9.410	-8.822	-7.056	-4.326	-2.271	0.007	0.000
4	STRESS IN COMPOSITE GIRDER DUE TO REMOVAL OF SHUTTERING (G1a)		(At 49 days)												
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.000	0.000	-0.006	-0.010	-0.016	-0.020	-0.021	-0.020	-0.016	-0.010	-0.006	0.000	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.018	0.036	0.059	0.073	0.078	0.073	0.059	0.036	0.018	0.000	0.000
5	Stress due to Differential Shrinkage (G10)		(At 49 days)												
	At Top of Deck Slab, fts	N/mm ²	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27	-1.27
	At Bottom of Deck Slab, fbs	N/mm ²	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40	-1.40
	At Top of Precast Girder,ftg	N/mm ²	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
	At Bottom of Precast Girder,fbg	N/mm ²	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28	-0.28
6	STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT (G2)		(At 63 days)												
	At Top of Deck Slab, fts	N/mm ²	0.000	0.024	0.138	0.268	0.453	0.573	0.613	0.573	0.453	0.268	0.138	0.024	0.000

At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.016	0.088	0.162	0.275	0.347	0.371	0.347	0.275	0.163	0.088	0.016	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.016	0.088	0.162	0.275	0.347	0.371	0.347	0.275	0.163	0.088	0.016	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.043	-0.275	-0.606	-1.025	-1.295	-1.385	-1.295	-1.025	-0.606	-0.275	-0.043	0.000

7	STRESS IN COMPOSITE GIRDER DUE TO SIDL2 DUE TO WEARING COAT (G3) (At 63 days)													
	At Top of Deck Slab, fts	N/mm ²	0.000	0.013	0.067	0.112	0.171	0.210	0.222	0.210	0.171	0.112	0.067	0.013
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.009	0.043	0.068	0.104	0.127	0.135	0.127	0.104	0.068	0.043	0.009
	At Top of Precast Girder,ftg	N/mm ²	0.000	0.009	0.043	0.068	0.104	0.127	0.135	0.127	0.104	0.068	0.043	0.009
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.023	-0.134	-0.252	-0.387	-0.474	-0.502	-0.474	-0.387	-0.252	-0.134	-0.023

8	Stress in Composite Girder due to FPLL (Q1) (At Infinity)													
	At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Top of Precast Girder,ftg	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

9	Stress in Composite Girder due to CWLL (WITH IMPACT) (Q2) (At Infinity)													
	At Top of Deck Slab, fts	N/mm ²	0.020	0.465	0.899	0.927	1.752	2.542	2.858	2.555	1.773	0.886	0.449	0.021
	At Bottom of Deck Slab, fbs	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.014
	At Top of Precast Girder,ftg	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.014
	At Bottom of Precast Girder,fbg	N/mm ²	-0.036	-0.825	-1.786	-2.096	-3.960	-5.746	-6.461	-5.776	-4.009	-2.003	-0.892	-0.037

10	Stress due to Positive Temperature Diff. (Q5,1)													
	At Top of Deck Slab, fts	N/mm ²	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.14
	At Bottom of Deck Slab, fbs	N/mm ²	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44
	At Top of Precast Girder,ftg	N/mm ²	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44	-1.44
	At Bottom of Precast Girder,fbg	N/mm ²	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93

11	Stress due to Reverse Temperature Diff. (Q5,2)													
	At Top of Deck Slab, fts	N/mm ²	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21	-2.21
	At Bottom of Deck Slab, fbs	N/mm ²	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
	At Top of Precast Girder,ftg	N/mm ²	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
	At Bottom of Precast Girder,fbg	N/mm ²	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52	-2.52

STRESS LIMIT CHECK (SLS CASE)

STRESS LIMIT CHECK AT SLS -1			AT	(7 days)	1.0 G8 + 1.0 G1					Load Factor	G8 = 1 G1 = 1				
Effective stress due to prestress			(G8)												
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Top of Precast Girder,ftg	N/mm ²	3.338	3.149	2.139	1.557	-0.350	-1.301	-1.365	-1.321	-0.363	1.589	2.187	3.224	3.418	
At Bottom of Precast Girder,fbg	N/mm ²	0.828	1.034	3.279	5.565	7.683	8.793	8.923	8.938	7.865	5.705	3.366	1.065	0.854	
Stress Due to Self Weight of Girder			(G1,1)												
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Top of Precast Girder,ftg	N/mm ²	0.000	-0.005	1.147	2.185	3.487	4.329	4.609	4.329	3.487	2.185	1.147	-0.005	0.000	
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.006	-1.233	-2.290	-3.654	-4.536	-4.830	-4.536	-3.654	-2.290	-1.233	0.006	0.000	
Resultant stress in Girder			1.0 G8 + 1.0 G1												
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
At Top of Precast Girder,ftg	N/mm ²	3.338	3.144	3.286	3.742	3.137	3.028	3.244	3.007	3.124	3.774	3.334	3.219	3.418	
At Bottom of Precast Girder,fbg	N/mm ²	0.828	1.039	2.046	3.276	4.030	4.257	4.093	4.402	4.212	3.415	2.133	1.071	0.854	

Maximum Compressive Stress in Girder	=	4.402	N/mm ²	<	15.760	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	0.828	N/mm ²	>	-2.552	N/mm ²	(Allowable tensile stress)	SAFE

STRESS LIMIT CHECK AT SLS -2			AT	(28 days)	1.0 G8 + 1.0 G1					Load Factor	G8 = 1 G1 = 1				
Effective stress due to prestress			(G8)	AT	(28 days)										
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	

At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.296	2.890	0.519	-0.530	-3.693	-5.213	-5.334	-5.285	-3.787	-0.572	0.504	2.943	3.360
At Bottom of Precast Girder,fbg	N/mm ²	8.460	8.905	14.563	20.065	23.652	25.478	25.760	25.859	24.226	20.653	15.011	9.199	8.742

Stress Due to Self Weight of Girder (G1,1)

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	-0.005	1.147	2.185	3.487	4.329	4.609	4.329	3.487	2.185	1.147	-0.005	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.006	-1.233	-2.290	-3.654	-4.536	-4.830	-4.536	-3.654	-2.290	-1.233	0.006	0.000

Resultant stress in Girder 1.0 G8 + 1.0 G1

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.296	2.885	1.666	1.655	-0.207	-0.884	-0.725	-0.957	-0.301	1.613	1.651	2.937	3.360
At Bottom of Precast Girder,fbg	N/mm ²	8.460	8.910	13.330	17.776	19.999	20.942	20.929	21.323	20.572	18.363	13.778	9.204	8.742

Maximum Compressive Stress in Girder = 21.323 N/mm² < **21.600** N/mm² (Allowable compressive stress) **SAFE**

Minimum Compressive Stress in Girder = -0.957 N/mm² > **-3.3** N/mm² (Allowable tensile stress) **SAFE**

STRESS LIMIT CHECK AT SLS -3**AT (35 days)****1.0 G8 + 1.0 G1**

Load Factor

G8 =	1
G1 =	1

Effective stress due to prestress**(G8) AT (35 days)**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.303	2.892	0.493	-0.568	-3.763	-5.299	-5.424	-5.377	-3.865	-0.616	0.473	2.944	3.366
At Bottom of Precast Girder,fbg	N/mm ²	8.604	9.054	14.788	20.369	24.002	25.854	26.146	26.259	24.613	20.994	15.264	9.368	8.906

Stress Due to Self Weight of Girder (G1,1) + Stress Due to Weight of Shuttering on Girder (G1,2) + Stress in Girder Due to Green Weight of Deck slab (G1,3)

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	-0.012	3.286	6.364	10.303	12.851	13.701	12.851	10.303	6.364	3.286	-0.012	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.013	-3.532	-6.669	-10.797	-13.467	-14.357	-13.467	-10.797	-6.669	-3.532	0.013	0.000

Resultant stress in Girder 1.0 G8 + 1.0 G1

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.303	2.881	3.779	5.797	6.540	7.552	8.277	7.474	6.438	5.748	3.759	2.932	3.366
At Bottom of Precast Girder,fbg	N/mm ²	8.604	9.066	11.256	13.700	13.205	12.387	11.789	12.791	13.816	14.325	11.732	9.381	8.906

Maximum Compressive Stress in Girder = 14.325 N/mm² < **21.600** N/mm² (Allowable compressive stress) **SAFE**

Minimum Compressive Stress in Girder = 2.881 N/mm² > **-3.3** N/mm² (Allowable tensile stress) **SAFE**

STRESS LIMIT CHECK AT SLS -4**AT (49 days)****1.0 G8 + 1.0 G1 +1.0 G10**

Load Factor

G8 =	1	G10 =	1
G1 =	1		

Effective stress due to prestress**(G8) AT (49 days)**

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	3.318	2.905	0.492	-0.575	-3.789	-5.334	-5.460	-5.414	-3.893	-0.624	0.472	2.957	3.381
At Bottom of Precast Girder,fbg	N/mm ²	8.659	9.111	14.881	20.501	24.157	26.021	26.317	26.432	24.776	21.134	15.363	9.429	8.964

X1 = (G1,1)+(G1,2)+(G1,3)

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	-0.012	3.286	6.364	10.303	12.851	13.701	12.851	10.303	6.364	3.286	-0.012	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.013	-3.532	-6.669	-10.797	-13.467	-14.357	-13.467	-10.797	-6.669	-3.532	0.013	0.000

STRESS IN COMPOSITE GIRDER DUE TO REMOVAL OF SHUTTERING (G1a)

At Top of Deck Slab, fts	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.000	-0.006	-0.010	-0.016	-0.020	-0.021	-0.020	-0.016	-0.010	-0.006	0.000	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.018	0.036	0.059	0.073	0.078	0.073	0.059	0.036	0.018	0.000	0.000

Stress due to Differential Shrinkage (G10)

At Top of Deck Slab, fts	N/mm ²	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267
At Bottom of Deck Slab, fbs	N/mm ²	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398
At Top of Precast Girder,ftg	N/mm ²	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671
At Bottom of Precast Girder,fbg	N/mm ²	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279	-0.279

Resultant stress in Girder 1.0 G8 + 1.0 G1 +1.0 G10

At Top of Deck Slab, fts	N/mm ²	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267
At Top of Deck Slab, fts	N/mm ²	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398
At Top of Precast Girder,ftg	N/mm ²	3.989	3.564	4.443	6.451	7.169	8.168	8.890	8.089	7.065	6.401	4.423	3.616	4.052
At Bottom of Precast Girder,fbg	N/mm ²	8.379	8.844	11.087	13.589	13.140	12.348	11.759	12.759	13.759	14.221	11.569	9.162	8.685

Maximum Compressive Stress in Girder	=	14.221	N/mm ²	<	21.600 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.564	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.398	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

STRESS LIMIT CHECK AT SLS -5**AT****(63 days)****1.0 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10
+ 1.0 (Q1+Q2) + 0.6 Q5****Load Factor**

G8 =	1	G3 =	1	Q5 =	0.6
G1 =	1	G10 =	1		
G2 =	1	Q1+Q2 =	1		

Effective stress due to prestress	(G8)	AT (63 days)												
At Top of Deck Slab, fts	N/mm ²	0.020	0.019	0.019	0.021	0.018	0.016	0.016	0.016	0.018	0.021	0.019	0.020	0.020
At Bottom of Deck Slab, fbs	N/mm ²	0.018	0.017	0.018	0.021	0.019	0.018	0.018	0.018	0.019	0.021	0.018	0.018	0.018
At Top of Precast Girder,ftg	N/mm ²	3.335	2.922	0.510	-0.554	-3.771	-5.317	-5.442	-5.396	-3.874	-0.603	0.490	2.974	3.399
At Bottom of Precast Girder,fbg	N/mm ²	8.662	9.114	14.892	20.519	24.183	26.051	26.347	26.462	24.803	21.152	15.374	9.433	8.967

X2	=	X1	+	(G1a)	+	(G10)								
At Top of Deck Slab, fts	N/mm ²	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267	-1.267
At Bottom of Deck Slab, fbs	N/mm ²	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398	-1.398
At Top of Precast Girder,ftg	N/mm ²	0.671	0.659	3.951	7.025	10.958	13.502	14.350	13.502	10.958	7.025	3.951	0.659	0.671
At Bottom of Precast Girder,fbg	N/mm ²	-0.279	-0.267	-3.794	-6.912	-11.017	-13.673	-14.558	-13.673	-11.017	-6.912	-3.794	-0.267	-0.279

STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT (G2)

At Top of Deck Slab, fts	N/mm ²	0.000	0.024	0.138	0.268	0.453	0.573	0.613	0.573	0.453	0.268	0.138	0.024	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.016	0.088	0.162	0.275	0.347	0.371	0.347	0.275	0.163	0.088	0.016	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.016	0.088	0.162	0.275	0.347	0.371	0.347	0.275	0.163	0.088	0.016	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.043	-0.275	-0.606	-1.025	-1.295	-1.385	-1.295	-1.025	-0.606	-0.275	-0.043	0.000

STRESS IN COMPOSITE GIRDER DUE TO SIDL2 DUE TO WEARING COAT (G3)

At Top of Deck Slab, fts	N/mm ²	0.000	0.013	0.067	0.112	0.171	0.210	0.222	0.210	0.171	0.112	0.067	0.013	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.009	0.043	0.068	0.104	0.127	0.135	0.127	0.104	0.068	0.043	0.009	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.009	0.043	0.068	0.104	0.127	0.135	0.127	0.104	0.068	0.043	0.009	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.023	-0.134	-0.252	-0.387	-0.474	-0.502	-0.474	-0.387	-0.252	-0.134	-0.023	0.000

Stress due to Positive Temperature Diff. (Q5,1)

At Top of Deck Slab, fts	N/mm ²	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142	4.142
At Bottom of Deck Slab, fbs	N/mm ²	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438
At Top of Precast Girder,ftg	N/mm ²	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438	-1.438
At Bottom of Precast Girder,fbg	N/mm ²	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932	1.932

Resultant stress in Girder Case 1 : WITHOUT CWLL PLACEMENT & TEMPERATURE RISE CASE

At Top of Deck Slab, fts	N/mm ²	1.238	1.275	1.443	1.619	1.860	2.017	2.069	2.017	1.861	1.620	1.443	1.275	1.238
At Bottom of Deck Slab, fbs	N/mm ²	-2.243	-2.218	-2.111	-2.010	-1.863	-1.769	-1.737	-1.768	-1.863	-2.009	-2.111	-2.218	-2.242
At Top of Precast Girder,ftg	N/mm ²	3.143	2.743	3.730	5.838	6.703	7.797	8.551	7.718	6.600	5.790	3.710	2.795	3.207
At Bottom of Precast Girder,fbg	N/mm ²	9.542	9.941	11.849	13.907	12.913	11.769	11.061	12.180	13.533	14.541	12.331	10.259	9.848

Maximum Compressive Stress in Girder	=	14.541	N/mm ²	<	21.600	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	2.743	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.243	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE

Stress due to Reverse Temperature Diff. (Q5,2)

At Top of Deck Slab, fts	N/mm ²	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207	-2.207
At Bottom of Deck Slab, fbs	N/mm ²	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112
At Top of Precast Girder,ftg	N/mm ²	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112	1.112
At Bottom of Precast Girder,fbg	N/mm ²	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523	-2.523

Resultant stress in Girder Case 2 : WITHOUT CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-2.572	-2.535	-2.367	-2.191	-1.949	-1.793	-1.740	-1.793	-1.949	-2.190	-2.366	-2.534	-2.571
At Bottom of Deck Slab, fbs	N/mm ²	-0.713	-0.688	-0.581	-0.480	-0.333	-0.239	-0.207	-0.238	-0.333	-0.479	-0.580	-0.688	-0.712
At Top of Precast Girder,ftg	N/mm ²	4.673	4.273	5.260	7.368	8.233	9.327	10.081	9.248	8.130	7.320	5.240	4.325	4.737
At Bottom of Precast Girder,fbg	N/mm ²	6.869	7.268	9.176	11.234	10.240	9.096	8.388	9.507	10.860	11.867	9.658	7.586	7.174

Maximum Compressive Stress in Girder	=	11.867	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	4.273	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.572	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE

Stress in Composite Girder due to CWLL (WITH IMPACT) (Q1)+(Q2)

At Top of Deck Slab, fts	N/mm ²	0.020	0.465	0.899	0.927	1.752	2.542	2.858	2.555	1.773	0.886	0.449	0.445	0.021
At Bottom of Deck Slab, fbs	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.296	0.014
At Top of Precast Girder,ftg	N/mm ²	0.014	0.309	0.575	0.562	1.061	1.540	1.731	1.548	1.074	0.537	0.287	0.296	0.014
At Bottom of Precast Girder,fbg	N/mm ²	-0.036	-0.825	-1.786	-2.096	-3.960	-5.746	-6.461	-5.776	-4.009	-2.003	-0.892	-0.790	-0.037

Resultant stress in Girder Case 3 : WITH CWLL PLACEMENT & TEMPERATURE RISING CASE

At Top of Deck Slab, fts	N/mm ²	1.258	1.739	2.342	2.546	3.612	4.558	4.927	4.572	3.634	2.505	1.893	1.720	1.259
At Bottom of Deck Slab, fbs	N/mm ²	-2.229	-1.910	-1.536	-1.448	-0.802	-0.229	-0.005	-0.221	-0.788	-1.473	-1.823	-1.922	-2.228
At Top of Precast Girder,ftg	N/mm ²	3.157	3.052	4.304	6.400	7.764	9.337	10.282	9.266	7.674	6.326	3.997	3.091	3.221
At Bottom of Precast Girder,fbg	N/mm ²	9.505	9.115	10.063	11.812	8.952	6.023	4.600	6.404	9.524	12.538	11.439	9.469	9.810

Maximum Compressive Stress in Girder	=	12.538	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.052	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.229	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 4 : WITH CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-2.551	-2.070	-1.467	-1.264	-0.197	0.749	1.118	0.762	-0.175	-1.304	-1.917	-2.089	-2.550
At Bottom of Deck Slab, fbs	N/mm ²	-0.699	-0.380	-0.006	0.082	0.728	1.301	1.525	1.310	0.742	0.057	-0.293	-0.392	-0.698
At Top of Precast Girder,ftg	N/mm ²	4.687	4.582	5.834	7.930	9.294	10.867	11.812	10.796	9.204	7.856	5.527	4.621	4.751
At Bottom of Precast Girder,fbg	N/mm ²	6.832	6.442	7.390	9.139	6.279	3.350	1.927	3.731	6.850	9.865	8.766	6.796	7.137

Maximum Compressive Stress in Girder	=	11.812	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	1.927	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.551	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

STRESS LIMIT CHECK AT SLS -6

(Long Term)
i.e. AT Infinity

1.1 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10
+ 1.0 (Q1+Q2) + 0.6 Q5

with Superior value of prestress, γ_{sup}

G8 =	1.1	G3 =	1	Q5 =	0.6
G1 =	1	G10 =	1		
G2 =	1	Q1+Q2 =	1		

Effective stress due to prestress (G8) AT (Long Term)

At Top of Deck Slab, fts	N/mm ²	0.578	0.563	0.536	0.561	0.473	0.436	0.441	0.454	0.500	0.592	0.566	0.594	0.610
At Bottom of Deck Slab, fbs	N/mm ²	0.511	0.501	0.501	0.542	0.495	0.477	0.485	0.497	0.523	0.573	0.529	0.530	0.540
At Top of Precast Girder,ftg	N/mm ²	3.829	3.406	0.993	-0.032	-3.295	-4.857	-4.976	-4.917	-3.370	-0.051	1.001	3.486	3.921
At Bottom of Precast Girder,fbg	N/mm ²	8.686	9.165	15.130	20.909	24.810	26.801	27.116	27.241	25.467	21.568	15.630	9.490	8.996

Resultant stress in Girder Case 1 : WITHOUT CWLL PLACEMENT & TEMPERATURE RISE CASE

At Top of Deck Slab, fts	N/mm ²	1.854	1.875	2.014	2.215	2.363	2.480	2.538	2.500	2.393	2.250	2.046	1.909	1.889
At Bottom of Deck Slab, fbs	N/mm ²	-1.698	-1.684	-1.577	-1.434	-1.338	-1.261	-1.222	-1.240	-1.306	-1.400	-1.547	-1.653	-1.666
At Top of Precast Girder,ftg	N/mm ²	4.020	3.568	4.312	6.357	6.850	7.771	8.520	7.705	6.767	6.337	4.321	3.656	4.122
At Bottom of Precast Girder,fbg	N/mm ²	10.434	10.908	13.600	16.388	16.021	15.198	14.542	15.683	16.744	17.114	14.150	11.265	10.776

Maximum Compressive Stress in Girder	=	17.114	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.568	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.698	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 2 : WITHOUT CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-1.956	-1.935	-1.796	-1.594	-1.446	-1.330	-1.271	-1.310	-1.416	-1.560	-1.763	-1.900	-1.920
At Bottom of Deck Slab, fbs	N/mm ²	-0.168	-0.154	-0.047	0.096	0.192	0.269	0.308	0.290	0.224	0.130	-0.016	-0.123	-0.136
At Top of Precast Girder,ftg	N/mm ²	5.550	5.098	5.842	7.887	8.380	9.301	10.050	9.235	8.297	7.867	5.851	5.186	5.652
At Bottom of Precast Girder,fbg	N/mm ²	7.761	8.235	10.927	13.715	13.348	12.525	11.869	13.010	14.071	14.441	11.477	8.592	8.103

Maximum Compressive Stress in Girder	=	14.441	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	5.098	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.956	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 3 : WITH CWLL PLACEMENT & TEMPERATURE RISING CASE

At Top of Deck Slab, fts	N/mm ²	1.874	2.339	2.913	3.142	4.115	5.022	5.396	5.055	4.167	3.135	2.495	2.354	1.910
At Bottom of Deck Slab, fbs	N/mm ²	-1.685	-1.376	-1.003	-0.872	-0.276	0.278	0.510	0.308	-0.232	-0.863	-1.259	-1.358	-1.652
At Top of Precast Girder,ftg	N/mm ²	4.033	3.877	4.887	6.919	7.911	9.311	10.251	9.253	7.841	6.873	4.608	3.951	4.135

At Bottom of Precast Girder,fbg	N/mm ²	10.398	10.083	11.814	14.292	12.060	9.452	8.081	9.907	12.734	15.111	13.258	10.476	10.739

Maximum Compressive Stress in Girder	=	15.111	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.877	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.685	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 4 : WITH CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-1.935	-1.470	-0.897	-0.667	0.306	1.212	1.587	1.245	0.357	-0.674	-1.314	-1.456	-1.899
At Bottom of Deck Slab, fbs	N/mm ²	-0.155	0.155	0.527	0.658	1.254	1.809	2.040	1.838	1.298	0.667	0.271	0.172	-0.122
At Top of Precast Girder,ftg	N/mm ²	5.563	5.407	6.417	8.449	9.441	10.841	11.781	10.783	9.371	8.403	6.138	5.481	5.665
At Bottom of Precast Girder,fbg	N/mm ²	7.725	7.410	9.141	11.619	9.387	6.779	5.407	7.234	10.061	12.438	10.585	7.802	8.065

Maximum Compressive Stress in Girder	=	12.438	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	5.407	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.935	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

STRESS LIMIT CHECK AT SLS -7

i.e. AT (Long Term) Infinity 0.9 G8 + 1.0 G1 + 1.0 G2 + 1.0 G3 + 1.0 G10
+ 1.0 (Q1+Q2) + 0.6 Q5

load factor G8 = 0.9
with inferior value of prestress, γ_{inf}

Resultant stress in Girder Case 1 : WITHOUT CWLL PLACEMENT & TEMPERATURE RISE CASE

At Top of Deck Slab, fts	N/mm ²	1.738	1.762	1.907	2.103	2.269	2.393	2.450	2.409	2.293	2.131	1.933	1.790	1.767
At Bottom of Deck Slab, fbs	N/mm ²	-1.800	-1.785	-1.678	-1.542	-1.437	-1.357	-1.319	-1.339	-1.411	-1.514	-1.652	-1.759	-1.774
At Top of Precast Girder,ftg	N/mm ²	3.254	2.887	4.114	6.363	7.509	8.742	9.515	8.688	7.441	6.347	4.121	2.959	3.337
At Bottom of Precast Girder,fbg	N/mm ²	8.697	9.075	10.574	12.206	11.059	9.838	9.118	10.235	11.650	12.800	11.024	9.367	8.977

Maximum Compressive Stress in Girder	=	12.800	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	2.887	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.800	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 2 : WITHOUT CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-2.013	-1.991	-1.849	-1.650	-1.494	-1.373	-1.315	-1.355	-1.466	-1.619	-1.820	-1.960	-1.981
At Bottom of Deck Slab, fbs	N/mm ²	-0.219	-0.204	-0.097	0.042	0.143	0.221	0.260	0.240	0.171	0.073	-0.069	-0.176	-0.190
At Top of Precast Girder,ftg	N/mm ²	5.167	4.757	5.743	7.890	8.709	9.786	10.548	9.727	8.634	7.872	5.751	4.837	5.259
At Bottom of Precast Girder,fbg	N/mm ²	6.893	7.319	9.414	11.624	10.867	9.845	9.157	10.286	11.524	12.284	9.914	7.643	7.203

Maximum Compressive Stress in Girder	=	12.284	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	4.757	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.013	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 3 : WITH CWLL PLACEMENT & TEMPERATURE RISING CASE

At Top of Deck Slab, fts	N/mm ²	1.759	2.227	2.806	3.030	4.020	4.935	5.308	4.964	4.067	3.017	2.382	2.235	1.788
At Bottom of Deck Slab, fbs	N/mm ²	-1.787	-1.476	-1.103	-0.981	-0.375	0.183	0.413	0.209	-0.337	-0.978	-1.365	-1.463	-1.760
At Top of Precast Girder,ftg	N/mm ²	3.268	3.195	4.688	6.925	8.570	10.282	11.247	10.236	8.515	6.884	4.408	3.254	3.351
At Bottom of Precast Girder,fbg	N/mm ²	8.661	8.250	8.788	10.110	7.098	4.092	2.657	4.459	7.641	10.797	10.132	8.578	8.939

Maximum Compressive Stress in Girder	=	11.247	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	2.657	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.787	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Resultant stress in Girder Case 4 : WITH CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-2.051	-1.583	-1.004	-0.780	0.211	1.125	1.499	1.155	0.257	-0.793	-1.427	-1.575	-2.021
At Bottom of Deck Slab, fbs	N/mm ²	-0.257	0.054	0.427	0.549	1.155	1.713	1.943	1.739	1.193	0.552	0.165	0.067	-0.230
At Top of Precast Girder,ftg	N/mm ²	4.798	4.726	6.218	8.455	10.100	11.812	12.777	11.766	10.045	8.414	5.938	4.784	4.881
At Bottom of Precast Girder,fbg	N/mm ²	5.988	5.577	6.115	7.437	4.425	1.419	-0.016	1.786	4.968	8.124	7.459	5.904	6.266

Maximum Compressive Stress in Girder	=	12.777	N/mm ²	<	21.6 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	-0.016	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE

Design Calculation				RODIC		Final Stress
Minimum stress in Deck slab	=	-2.051	N/mm ²	>	-3.3 N/mm ² (Allowable tensile stress)	SAFE

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

ULTIMATE BENDING MOMENT AND SHEAR FORCE

a) Maximum Bending Moment & corresponding Shear Force

Bending Moment (kNm)	1.35 x DL _{girder}	0.0	-2.4	481.9	837.2	1336.0	1658.6	1766.2	1658.6	1336.0	837.2	481.9	-2.4	0.0
	1.35 x DL _{slab}	0.0	-3.0	887.6	1581.7	2579.9	3225.6	3440.9	3225.6	2579.9	1581.7	887.6	-3.0	0.0
	1.35 x SIDL1	0.0	30.5	171.0	331.0	559.6	706.7	756.0	706.7	559.6	331.0	171.0	30.5	0.0
	1.75 x SIDL2	0.0	21.4	107.8	178.5	273.9	335.3	355.5	335.3	273.9	178.5	107.8	21.4	0.0
	1.5 LL	28.6	648.3	1233.8	1271.1	2402.2	3485.4	3918.8	3503.4	2431.8	1214.7	616.4	620.5	29.3
	TOTAL M_u	28.6	694.8	2882.0	4199.5	7151.5	9411.7	10237.4	9429.7	7181.2	4143.1	2264.5	667.0	29.3
Shear Force (kN) corresponding to Max Bending Moment	1.35 x DL _{girder}	326.1	326.1	260.9	212.9	144.9	72.5	0.0	72.5	144.9	212.9	260.9	326.1	326.1
	1.35 x DL _{slab}	580.0	580.0	499.4	426.2	290.0	145.0	0.0	145.0	290.0	426.2	499.4	580.0	580.0
	1.35 x SIDL1	123.9	123.9	105.0	82.9	50.2	16.6	16.6	16.6	50.2	82.9	105.0	123.9	123.9
	1.75 x SIDL2	51.4	51.4	44.7	33.2	20.2	6.8	6.8	6.8	20.2	33.2	44.7	51.4	51.4
	1.5 LL	401.0	368.9	403.2	403.2	405.8	220.5	42.5	236.6	329.8	379.5	396.7	393.8	393.8
	TOTAL V_u	1482.4	1450.3	1313.2	1158.4	911.2	461.4	65.9	477.5	835.2	1134.8	1306.8	1475.2	1475.2

Torsional Moment (kNm)	1.35 x DL _{girder}	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	1.35 x DL _{slab}	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	1.35 x SIDL1	48.3	48.3	41.2	26.8	15.2	5.0	5.0	5.0	15.2	26.7	41.2	48.3	48.3
	1.75 x SIDL2	37.3	37.3	31.2	19.8	11.0	3.6	3.6	3.6	11.0	19.8	31.2	37.3	37.3
	1.5 LL	72.8	72.8	72.8	30.8	18.7	10.0	12.6	11.7	16.0	9.1	9.8	14.0	14.0
	TOTAL M_u	158.5	158.5	145.2	77.3	44.9	18.6	21.2	20.3	42.2	55.6	82.1	99.7	99.7

b) Maximum Shear force and Corressponding Bending Moment

SHEAR FORCE (kN)	1.35 x DL _{girder}	326.1	326.1	260.9	212.9	144.9	72.5	0.0	72.5	144.9	212.9	260.9	326.1	326.1
	1.35 x DL _{slab}	580.0	580.0	499.4	426.2	290.0	145.0	0.0	145.0	290.0	426.2	499.4	580.0	580.0
	1.35 x SIDL1	123.9	123.9	105.0	82.9	50.2	16.6	16.6	16.6	50.2	82.9	105.0	123.9	123.9
	1.75 x SIDL2	51.4	51.4	44.7	33.2	20.2	6.8	6.8	6.8	20.2	33.2	44.7	51.4	51.4
	1.5 LL	671.0	671.0	554.6	348.0	303.0	154.2	187.5	224.1	229.7	315.4	418.1	498.0	483.2
	TOTAL V_u	1752.4	1752.4	1464.7	1103.2	808.4	395.0	210.9	465.0	735.0	1070.7	1328.2	1579.5	1564.7
BENDING MOMENTS (kNm) CORRESPONDING TO MAX SHEAR	1.35 x DL _{girder}	0.0	-2.4	481.9	837.2	1336.0	1658.6	1766.2	1658.6	1336.0	837.2	481.9	-2.4	0.0
	1.35 x DL _{slab}	0.0	-3.0	887.6	1581.7	2579.9	3225.6	3440.9	3225.6	2579.9	1581.7	887.6	-3.0	0.0
	1.35 x SIDL1	0.0	30.5	171.0	331.0	559.6	706.7	756.0	706.7	559.6	331.0	171.0	30.5	0.0
	1.75 x SIDL2	0.0	21.4	107.8	178.5	273.9	335.3	355.5	335.3	273.9	178.5	107.8	21.4	0.0
	1.5 LL	18.5	352.8	896.6	1521.5	2358.6	2703.2	2513.9	2019.0	1350.1	775.8	807.6	339.6	12.2

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
TOTAL Mu	18.5	399.3	2544.8	4450.0	7108.0	8629.5	8832.4	7945.2	6099.5	3704.3	2455.7	386.1	12.2

COMPOSITE SECTION PROPERTIES

Sectional area, A	m ²	1.886	1.886	1.651	1.438	1.438	1.438	1.438	1.438	1.438	1.438	1.651	1.886	1.886
Moment of Inertia, Iz	m ⁴	0.610	0.610	0.558	0.510	0.510	0.510	0.510	0.510	0.510	0.510	0.558	0.610	0.610
C.G. of section from top, Yt	m	0.656	0.656	0.610	0.558	0.558	0.558	0.558	0.558	0.558	0.558	0.610	0.656	0.656
C.G. of section from bottom, Yb	m	1.164	1.164	1.210	1.262	1.262	1.262	1.262	1.262	1.262	1.262	1.210	1.164	1.164
Sectional modulus at top, Zt	m ³	0.930	0.930	0.915	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.915	0.930	0.930
Sectional modulus at bottom, Zb	m ³	0.524	0.524	0.461	0.404	0.404	0.404	0.404	0.404	0.404	0.404	0.461	0.524	0.524
Effective width of Deck slab	m	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250
Width of the web, b	m	0.700	0.700	0.488	0.275	0.275	0.275	0.275	0.275	0.275	0.275	0.488	0.700	0.700
Overall Depth of Section, D	m	1.82												

CHECK FOR ULTIMATE MOMENT

For equilibrium section, Compression force in concrete to Tension force in steel

C = T

Assume Depth of Neutral Axis, X = 136.772 mm

Depth of Rectangular portion = 3 / 7 X	= 58.617 mm
Depth of Parabolic Portion = 4 / 7 X	= 78.156 mm

Material Safety Factor:

ym for steel = 1.15

ym for concrete = 1.5

Ep = 1.95E+05 N/mm²

Es = 2.00E+05 N/mm²

Ecm = 3.40E+04 N/mm²

Material Stress

fck = 45 N/mm²

fyk = 500 N/mm²

εcu2 = 0.0035

εc2 = 0.002

C = 7222.01 kN

T = 7222.01 kN

C - T = 0.00

fcd = 20.07 N/mm²

fyd = 434.783 N/mm²

Width of deck slab bs = 3250 mm

Thickness of Deck slab ds = 220 mm

width of top flange of girder w1 = 1000 mm

As per clause 7.9.5 of IRC:112-2011

Partial safety factor for prestressing force γp = 1.00

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end

STAGE - I CABLES

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter		-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
Prestress force applied at jack	kN	2809	2809	2809	2809	2809	2809	2809	2809	2809	2809	2809	2809	2809
Total prestress loss	kN	553	552	537	519	481	446	428	410	430	464	482	495	496
Final prestressing force = $P_1 \cdot \gamma_p$	kN	2255.97	2256.98	2271.54	2289.88	2327.54	2362.69	2381.14	2399.02	2378.97	2344.44	2327.26	2313.81	2312.90
Area of Cables , A_{cable}	m ²	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020
c.g. of cables from soffit	m	1.00	0.97	0.76	0.60	0.39	0.29	0.28	0.29	0.39	0.60	0.76	0.97	1.00
STAGE -II CABLES														
Prestress force applied at jack	kN	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120	4494.120
Total prestress loss	kN	689.181	687.566	662.243	630.461	586.031	552.242	526.608	502.753	498.127	519.915	549.536	573.249	574.736
Final prestressing force = $P_2 \cdot \gamma_p$	kN	3804.939	3806.554	3831.877	3863.659	3908.089	3941.878	3967.512	3991.367	3995.993	3974.205	3944.584	3920.871	3919.384
Area of Cables , A_{cable}	m ²	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032	0.0032
c.g. of cables from soffit	m	0.516	0.499	0.364	0.277	0.203	0.172	0.170	0.172	0.203	0.276	0.363	0.497	0.514
Final Pretressing Force, Pd	Stage I & Stage II													
$P_d = P_1 \cdot \gamma_p + P_2 \cdot \gamma_p$	kN	6060.910	6063.531	6103.414	6153.537	6235.630	6304.563	6348.655	6390.385	6374.959	6318.646	6271.848	6234.683	6232.284
eff. C.g. of cables from soffit (e_{ef})	m	0.695	0.674	0.512	0.397	0.272	0.214	0.211	0.215	0.272	0.396	0.510	0.673	0.693
$d = D - e_{ef}$	m	1.125	1.146	1.308	1.423	1.548	1.606	1.609	1.605	1.548	1.424	1.310	1.147	1.127

Check for Depth of Neutral Axis:**Assume N.A Lies in Deck Slab**

Strain at bottom of deck slab = $0.0035 \cdot (X - d_s) / X = -0.0021$

Concrete stress at bottom of deck slab = $0.446 f_{ck} [1 - (1 - \epsilon_c / 0.002)^2] = 0.00 \text{ N/mm}^2$

Compression Force due to rectangular portion C1 = $20.07 \times 3250.00 \times 58.62 \times 0.001 = 3823.416 \text{ kN}$

Comp. Force due to upper part of Parabolic portion C2 = $(0.00 \times 3250.000 \times 78.156 + 0.667 \times 20.1 \times 3250.000 \times 78.156) \times 0.001 = 3398.59 \text{ kN}$

Comp. Force due to lower part of Parabolic portion C3 = $0.667 \times 0.00 \times 1000.0 \times 0.000 \times 0.001 = 0 \text{ kN}$

Total Compression Force $C = C_1 + C_2 + C_3 = 7222.01 \text{ kN}$

Design Prestressing stress in tendons = $f_{pd} = 1407.1 \text{ N/mm}^2$ as Tendons are yielded, hence full design prestressing stress will develop

Total Tensile Force , T = $f_{pd} \times A_{cables} = 1407.1 \times 5132.400 \times 0.001 = 7222.009 \text{ kN}$

For Balance section , $C = T = 7222.009 - 7222.009 = \boxed{0.00}$

thus $C = T$,Therefore section balances and Assumed Depth of Neutral Axis is Correct

Ultimate stress in Cable , f_{uk}	N/mm ²	1860	1860	1860	1860	1860	1860	1860	1860	1860	1860	1860	1860	1860
$f_{p0.1k} = 0.87 \times f_{uk}$	N/mm ²	1618	1618	1618	1618	1618	1618	1618	1618	1618	1618	1618	1618	1618
Design Stress = $f_{pd} = f_{p0.1k} / \gamma_s$	N/mm ²	1407	1407	1407	1407	1407	1407	1407	1407	1407	1407	1407	1407	1407
Design Strain = $\epsilon_d = f_{pd} / E_p$		0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072	0.0072
Prestrain in Stage-I Cable = $(P_1 / A_{cable}) / E_p$		0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
Prestrain in Stage-II Cable = $(P_2 / A_{cable}) / E_p$		0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
Total pretstrain = ϵ_1		0.012	0.012	0.012	0.012	0.012	0.013	0.013	0.013	0.013	0.013	0.012	0.012	0.012

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter		-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
Concrete strain at c.g. of cables = $\epsilon_2 = 0.0035 (d-X) / X$		0.025	0.026	0.030	0.033	0.036	0.038	0.038	0.038	0.036	0.033	0.030	0.026	0.025
Minimum strain in tendons = $\epsilon_1 + \epsilon_2$		0.037	0.038	0.042	0.045	0.049	0.050	0.050	0.050	0.049	0.045	0.042	0.038	0.038
Check for Yielding of Tendons		> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007	> 0.007
		Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields	Tendon yields
ULTIMATE MOMENT CAPACITY														
$M_{ults} = T \cdot (d - 0.416 X)$	kNm	7714.3	7862.2	9039.1	9864.1	10768.4	11185.3	11208.4	11183.7	10770.2	9872.1	9048.7	7873.4	7725.8
M_u		28.6	694.8	2882.0	4199.5	7151.5	9411.7	10237.4	9429.7	7181.2	4143.1	2264.5	667.0	29.3
Moment Capacity		O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

Design of Longitudinal Girder For Torsion

Ultimate Torsion Moment , T_{Ed}	kNm	158.490	158.490	145.172	77.321	44.894	18.608	21.230	20.273	42.194	55.619	82.106	99.698	99.698
Area of composite section , A_k	m ²	1.886	1.886	1.651	1.438	1.438	1.438	1.438	1.438	1.438	1.438	1.651	1.886	1.886
perimeter of composite section , u_k	m	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940	6.940
$t_{ef,i}$	m	0.272	0.272	0.238	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.238	0.272	0.272
Shear stress due to pure Torsion , $\tau_{t,i} = T_{Ed} / (2 \cdot A_k \cdot t_{ef,i})$	N/mm ²	0.155	0.155	0.185	0.130	0.075	0.031	0.036	0.034	0.071	0.093	0.105	0.097	0.097
Width of web , Z_i	m	0.700	0.700	0.488	0.275	0.275	0.275	0.275	0.275	0.275	0.275	0.488	0.700	0.700
Shear force due to torsion = $V_{Ed,i} = \tau_{t,i} \cdot t_{ef} \cdot Z_i$	kN	29.41	29.41	21.43	7.39	4.29	1.78	2.03	1.94	4.03	5.32	12.12	18.50	18.50
Mean compr. stress at c.g. sec. $\sigma_{cp} = N_{Ed} / A_c \leq 0.2 f_{cd}$	N/mm ²	3.213	3.215	3.697	4.014	4.014	4.014	4.014	4.014	4.014	4.014	3.799	3.305	3.304
Coefficient α_{cw} (eq.10.9 of IRC:112-2011)		1.160	1.160	1.184	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.189	1.165	1.165
strength reduction factor for section cracked in shear = $\nu = 0.6 [1 - f_{ck}/310]$		0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51
cot θ		2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22	22	22	22	22	22	22	22
Maximum Torsional Moment $T_{Rd,max} = 2 \cdot \nu \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t_{ef} \cdot \sin \theta \cdot \cos \theta$	kNm	4222.12	4222.37	3302.27	2538.81	2538.81	2538.81	2538.81	2538.81	2538.81	2538.81	3316.45	4238.83	4238.60
Maximum shear capacity $V_{Rd,max}$	kN	3601.92	3671.18	2538.70	1361.17	1485.95	1543.48	1546.67	1543.26	1486.20	1362.27	2552.30	3690.77	3621.35
$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max} \leq 1.0$		0.046	0.046	0.052	0.036	0.021	0.008	0.010	0.009	0.019	0.026	0.030	0.029	0.029
Check		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Torsional Long. Reinf. $A_{sl} = T_{Ed} \cdot \cot \theta \cdot u_k / (2 \cdot A_k \cdot f_{yd})$	mm ²	1676.53	1676.53	1754.33	1072.75	622.86	258.17	294.54	281.27	585.40	771.65	992.21	1054.61	1054.61

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

CHECK FOR ULTIMATE SHEAR AND TORSION

$$f_{ck} = 45 \text{ N/mm}^2 \quad f_{cd} = 0.446 \times f_{ck} = 20.07 \text{ N/mm}^2$$

	Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
Ultimate Design Shear Force due to Flexure and Torsion, $V_{Ed} = V_u + V_{Ed,t}$	kN	1781.8	1781.8	1486.1	1110.6	812.7	396.8	212.9	466.9	739.1	1076.0	1340.3	1598.0	1583.2
Coexisting ULS Moment, $M_{Ed} = M_u$	kNm	18.5	399.3	2544.8	4450.0	7108.0	8629.5	8832.4	7945.2	6099.5	3704.3	2455.7	386.1	12.2
Final Prestressing Force, $N_{Ed} = P_d$	kN	6061	6064	6103	6154	6236	6305	6349	6390	6375	6319	6272	6235	6232
Smallest Width of C/S, b_w	m	0.700	0.700	0.488	0.275	0.275	0.275	0.275	0.275	0.275	0.275	0.488	0.700	0.700
Effective depth, d	m	1.125	1.146	1.308	1.423	1.548	1.606	1.609	1.605	1.548	1.424	1.310	1.147	1.127

Shear Capacity without Shear Reinforcement

Refer Clause 10.3.2 of IRC:112-2011

$$V_{Rd,c} > V_{Ed}$$

Shear Resistance without shear Reinforcement, $V_{Rd,c} = [0.12 K (80 \cdot \rho_1 f_{ck})^{0.33} + 0.15 \sigma_{cp}] \cdot b_w \cdot d$

Long. Untension Reinf. $A_{sl} =$

6 - 16 dia

Subjected minimum $V_{Rd,c} = (v_{min} + 0.15 \sigma_{cp}) \cdot b_w \cdot d$ where $v_{min} = 0.031 K^{3/2} \cdot f_{ck}^{1/2}$

$K = 1 + \sqrt{200/d} \leq 2.0$		1.422	1.418	1.391	1.375	1.359	1.353	1.353	1.353	1.359	1.375	1.391	1.418	1.421
Long. untension Reinf. A_{sl}	m ²	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012	0.0012
$\rho_1 = A_{sl} / (b_w \cdot d) \leq 0.02$		0.002	0.002	0.002	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.002	0.002	0.002
steel ratio limited to 0.02		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
$V_{Rd,c}$	kN	615.60	624.91	554.22	378.42	405.76	418.29	418.99	418.24	405.81	378.66	564.46	636.54	627.06
v_{min}	N/mm ²	0.35	0.35	0.34	0.34	0.33	0.33	0.33	0.33	0.33	0.34	0.34	0.35	0.35
min $V_{Rd,c}$	kN	657.19	668.19	571.32	366.75	396.62	410.36	411.13	410.31	396.68	367.01	581.63	679.94	668.78
Max ($V_{Rd,c}$, min $V_{Rd,c}$)	kN	657.19	668.19	571.32	378.42	405.76	418.29	418.99	418.24	405.81	378.66	581.63	679.94	668.78
Check for Requirement of Shear Reinf.	$V_{Rd,c}$	< VEd	< VEd	< VEd	< VEd	< VEd	> VEd	> VEd	< VEd	< VEd	< VEd	< VEd	< VEd	< VEd
	Hence	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	NO REQ.	NO REQ.	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED

Shear Capacity with Design Shear Reinforcement

Refer Clause 10.3.3 of IRC:112-2011

Refer eq. 10.14 of IRC:112-2011, Width of web = $b_{w,nom} = b_w - 0.5 \Sigma \phi$ for $\phi > b_w/8$, & $b_{w,nom} = b_w$ for $\phi < b_w/8$ where ϕ = outer diameter of duct

Diameter of duct, $\phi = 0.075 \text{ m}$

Characteristic yield stress of steel, f_{yk}

$$= 500 \text{ N/mm}^2$$

Design yield strength of shear reinforcement, $f_{ywd} = 0.8 \cdot f_{yk}$

$$= 400 \text{ N/mm}^2$$

Strength reduction factor, v_1

$$= 0.6$$

Refer clause 10.3.3.2 of IRC:112-2011

(Refer clause 10.3.3.2 of IRC:112-2011)

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot \theta$$

Eq. 10.7

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

Eq. 10.8

Dia. of shear stirrups, ϕ	mm	12	12	12	12	12	12	12	12	12	12	12	12	12

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
Nos. of legged		2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg	2 leg
Spacing of stirrups, S	mm	190 c/c	190 c/c	200 c/c	200 c/c	200 c/c	200 c/c	200 c/c	200 c/c	200 c/c	200 c/c	190 c/c	190 c/c
Width of web , bw.nom	m	0.700	0.700	0.413	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.413	0.700
Vertical component of Prestress force after losses, $V_{pd} = \gamma_p \cdot P_{vert}$ (stage I & II)	kN	545.04	545.54	478.31	351.92	180.11	42.94	0.00	44.35	188.72	371.04	505.59	577.15
Net Shear Force = $V_{Rd,s} = V_{Ed} - V_{pd}$	kN	1236.8	1236.3	1007.8	758.7	632.6	353.9	212.9	422.6	550.3	704.9	834.7	1020.4
Area of shear Reinf. A_{sw}	mm ²	226	226	226	226	226	226	226	226	226	226	226	226
Coefficient , α_{cw}		1.160	1.160	1.184	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.189	1.165
Lever arm , $Z = d - 0.416 X$	m	1.068	1.089	1.252	1.366	1.491	1.549	1.552	1.549	1.491	1.367	1.253	1.090
$1 \geq \cot \theta \geq 2.5$		2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22	22	22	22	22	22	22	22	22	22	22	22
$V_{Rd,max}$	kN	3601.92	3671.18	2538.70	1361.17	1485.95	1543.48	1546.67	1543.26	1486.20	1362.27	2552.30	3690.77
Section check for shear capacity , $V_{Ed} - V_{pd} \leq V_{Rd,max}$		SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE
Reqd spacing of shear Reinf. S	mm	195	199	281	407	533	990	1649	829	613	439	340	242
Check for Provided Spacing		SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE

Minimum Shear Reinforcement

Min Shear Reif. ratio , $p_{min} = 0.072 f_{ck}^{0.5} / f_{yk}$	Refer Eq. 10.20 of IRC:112-2011	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010
Shear Reinf. ratio provided , $p_w = A_{sw} / (S \cdot bw \cdot \sin \alpha)$	Refer Eq. 16.4 of IRC:112-2011	0.0017	0.0017	0.0027	0.0057	0.0057	0.0057	0.0057	0.0057	0.0057	0.0057	0.0027	0.0017	0.0017
Check for Shear Reinf. Ratio		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Maximum Spacing $S_{l,max} = 0.75 d (1 + \cos \alpha)$, where $\alpha = 90^\circ$, refer eq. 16.6	mm	844	859	981	1067	1161	1204	1207	1204	1161	1068	982	860	845
Check for Provided Spacing		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

Check For Web Shear Cracking under uncracked Section (Refer Clause 12.3.5 of IRC:112-2011)**step (1) : Check for Tension crack in section**

width of web bw.nom	m	0.700	0.700	0.413	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.413	0.700	0.700
Depth of precast girder only, Dgir	m	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600
Depth of composite section, D	m	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820	1.820
C.G. of comp. section from soffit, y_b	m	1.164	1.164	1.210	1.262	1.262	1.262	1.262	1.262	1.262	1.262	1.210	1.164	1.164
M.I. of Girder alone , I_p	m ⁴	0.264	0.264	0.240	0.222	0.222	0.222	0.222	0.222	0.222	0.222	0.240	0.264	0.264
M.I. of comp. section , I_c	m ⁴	0.610	0.610	0.558	0.510	0.510	0.510	0.510	0.510	0.510	0.510	0.558	0.610	0.610
First moment of area above and about N.A. of comp. Sect. ($A_y p$)	m ³	0.086	0.086	0.069	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.069	0.086	0.086

Section	Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter	0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter	-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
First moment of area above and about N.A. of comp. Sect. (Ay)c	m ³	0.476	0.476	0.426	0.376	0.376	0.376	0.376	0.376	0.376	0.426	0.476	0.476
Characteristic Tensile strength 5% fractile, $f_{ctk,0.05} = 0.7 \cdot (0.259 \cdot f_{ck}^{2/3})$, refer Eq. A2-3 of IRC:112	N/mm ²	2.294	2.294	2.294	2.294	2.294	2.294	2.294	2.294	2.294	2.294	2.294	2.294
Long. Term vertical component of prestress, V_{pd}	kN	545.04	545.54	478.31	351.92	180.11	42.94	0.00	44.35	188.72	371.04	505.59	577.15
Design shear on precast sec. Before composite, V_{Ed1}	kN	906.1	906.1	760.3	639.1	434.9	217.5	0.0	217.5	434.9	639.1	760.3	906.1
Design shear on precast sec. after composite, V_{Ed2}	kN	846.3	846.3	704.4	464.1	373.5	177.6	210.9	247.5	300.1	431.6	567.9	658.5
Design Moment on precast sec. Before composite, M_{Ed1}	kNm	0.0	-5.4	1369.4	2419.0	3915.9	4884.3	5207.0	4884.3	3915.9	2419.0	1369.4	-5.4
Design Moment on precast sec. after composite, M_{Ed2}	kNm	28.6	700.2	1512.6	1780.5	3235.6	4527.5	5030.4	4545.5	3265.3	1724.1	895.1	672.4

Stress due to prestress after all losses

at top of precast girder ft.gir	N/mm ²	3.34	2.92	0.51	-0.55	-3.77	-5.32	-5.44	-5.40	-3.87	-0.60	0.49	2.97	3.40
at bottom of precast girder fb.gir	N/mm ²	8.66	9.11	14.89	20.52	24.18	26.05	26.35	26.46	24.80	21.15	15.37	9.43	8.97
Stress due to prestress at centroidal axis, fcp	N/mm ²	4.79	4.61	4.01	3.90	2.14	1.31	1.28	1.34	2.19	3.99	4.11	4.73	4.92

Stress due to Moment M_{Ed1}

at top of precast girder ft.gir	N/mm ²	0.000	-0.016	4.401	8.522	13.796	17.208	18.345	17.208	13.796	8.522	4.401	-0.016	0.000
at bottom of precast girder fb.gir	N/mm ²	0.000	0.017	-4.730	-8.931	-14.457	-18.033	-19.224	-18.033	-14.457	-8.931	-4.730	0.017	0.000
Stress due to prestress at centroidal axis, fcp	N/mm ²	0.000	-0.007	2.177	4.834	7.825	9.760	10.405	9.760	7.825	4.834	2.177	-0.007	0.000

Stress due to Moment M_{Ed2}

at top of precast girder ft.gir	N/mm ²	0.020	0.500	1.057	1.180	2.144	3.001	3.334	3.013	2.164	1.143	0.625	0.480	0.021
at bottom of precast girder fb.gir	N/mm ²	-0.055	-1.337	-3.284	-4.403	-8.002	-11.197	-12.441	-11.241	-8.075	-4.264	-1.943	-1.284	-0.056
Stress due to prestress at centroidal axis, fcp	N/mm ²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Stress in precast girder due to prestress + M_{Ed}

at top of precast girder ft.gir	N/mm ²	3.36	3.41	5.97	9.15	12.17	14.89	16.24	14.82	12.09	9.06	5.52	3.44	3.42
at bottom of precast girder fb.gir	N/mm ²	8.61	7.79	6.88	7.18	1.72	-3.18	-5.32	-2.81	2.27	7.96	8.70	8.17	8.91
Stress due to prestress at centroidal axis, fcp	N/mm ²	4.79	4.60	6.19	8.73	9.96	11.07	11.68	11.10	10.01	8.83	6.29	4.73	4.92

Check for Tension crack in the section

Tensile stress in the section	N/mm ²	3.36	3.41	5.97	7.18	1.72	-3.18	-5.32	-2.81	2.27	7.96	5.52	3.44	3.42

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter		-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95
Design value of conc. Tensile strength $f_{ctk,0.05}/\gamma_m = f_{ctd}$	N/mm ²	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53	-1.53
Section Type (Cracked/Uncracked)		Uncracked	Uncracked	Uncracked	Uncracked	Uncracked	Cracked	Cracked	Cracked	Uncracked	Uncracked	Uncracked	Uncracked	Uncracked

check at composite section centroid

Direct stress at centroid σ_{c1}	N/mm ²	4.79	4.60	6.19	8.73	9.96	11.07	11.68	11.10	10.01	8.83	6.29	4.73	4.92
Shear stress due to VEd1 , $\tau_{Ed1} = V_{Ed1}/(I_p b) \cdot (Ay)p$	N/mm ²	0.42	0.42	0.53	0.72	0.49	0.24	0.00	0.24	0.49	0.72	0.53	0.42	0.42
Shear stress due to VEd2 , $\tau_{Ed2} = V_{Ed2}/(I_c b) \cdot (Ay)c$	N/mm ²	0.94	0.94	1.31	1.71	1.38	0.65	0.78	0.91	1.11	1.59	1.05	0.75	0.73
Total shear stress $\tau_{Ed} = \tau_{Ed1} + \tau_{Ed2}$	N/mm ²	1.37	1.37	1.84	2.43	1.87	0.90	0.78	1.16	1.60	2.31	1.58	1.17	1.16
From Mohrs Circle Centre $\sigma_{co} = \sigma_{c1}/2$	N/mm ²	2.39	2.30	3.09	4.37	4.98	5.54	5.84	5.55	5.01	4.41	3.15	2.36	2.46
sqrt ($\tau_{Ed}^2 + (\sigma_{c1} - \sigma_{co})^2$) radius σ_r	N/mm ²	2.76	2.68	3.60	5.00	5.32	5.61	5.89	5.67	5.25	4.98	3.52	2.64	2.72
Larger Principle stress $\sigma_3 = \min$ of (0.6 fck , $\sigma_{co} + \sigma_r$)	N/mm ²	5.15	4.98	6.69	9.36	10.30	11.15	11.73	11.22	10.26	9.40	6.67	5.00	5.17
smaller principle stress $\sigma_1 = \sigma_{co} - \sigma_r$	N/mm ²	-0.36	-0.37	-0.50	-0.63	-0.34	-0.07	-0.05	-0.12	-0.25	-0.57	-0.38	-0.28	-0.26
Tensile stress in web prior to cracking $f_{ctb} = (1 - 0.8 \sigma_3/f_{ck}) \cdot f_{ctk,0.05}$	N/mm ²	2.08	2.09	2.02	1.91	1.87	1.84	1.82	1.84	1.88	1.91	2.02	2.09	2.08
Check for minimum Long. Reinf. In Girder	($\sigma_1 < f_{ctb}$, Not reqd.) & ($\sigma_1 > f_{ctb}$, crack width need to be controlled)	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd	Not Req'd

Section		Jacking End	0.000 L	0.069 L	0.133 L	0.250 L	0.375 L	0.500 L	0.625 L	0.750 L	0.867 L	0.931 L	1.000 L	Dead end
At Distance from Jack End in meter		0.00	0.20	1.85	3.35	6.14	9.11	12.08	15.04	18.01	20.80	22.30	23.95	24.15
At Distance from C.L. of Left Bearing in meter		-0.20	0.00	1.65	3.15	5.94	8.91	11.88	14.84	17.81	20.60	22.10	23.75	23.95

DESIGN OF SHEAR CONNECTOR (Refer Clause No. 10.3.4 of IRC:112-2011)

Shear Connectors between cast-in-situ deck slab and precast girder are designed to restrain the separation at the interface of two different concrete. The size and spacing of **Shear Connectors** are evaluated for long. shear per unit length at the interface of precast girder and slab.

V_{Ed} = Design shear force resulting from external loading and that due to prestressing tendons (V_{pd})

Effective Shear Force V_{Ed}	kN	1236.77	1236.26	1007.80	758.69	632.57	353.88	212.91	422.57	550.33	704.94	834.71	1020.35	1006.02
Effective width of flange at interface, b_f	m	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250	3.250
Thickness of slab at interface, h_f	m	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220

Design value of shear stress at interface = $v_{Edi} = \beta V_{Ed} / (z \cdot b_i)$

Width of girder at interface, b_i	m	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Lever arm of composite section = z	m	1.068	1.089	1.252	1.366	1.491	1.549	1.552	1.549	1.491	1.367	1.253	1.090	1.070
Proportion of longitudinal tensile force in top slab, $\beta = F_1 / (F_1 + F_2)$		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
where, Compressive force in top slab, F_1	kN	7222	7222	7222	7222	7222	7222	7222	7222	7222	7222	7222	7222	7222
where, Comp. force in precast Girder, F_2	kN	0	0	0	0	0	0	0	0	0	0	0	0	0
Design value of shear stress at interface, v_{Edi}	N/mm²	1.16	1.14	0.81	0.56	0.42	0.23	0.14	0.27	0.37	0.52	0.67	0.94	0.94

Design Shear Resistance at Interface = $v_{Rdi} = \mu \sigma_n + \rho f_{yd} [\mu \sin \alpha + \cos \alpha] \leq 0.5 v_{fcd}$

Area of the interface, A_i	mm ² /m	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06	1.E+06
Angle of reinforcement to interface, α	deg	90	90	90	90	90	90	90	90	90	90	90	90	90
Minimum coexisting normal stress = $\sigma_n < 0.6 f_{cd}$	N/mm ²	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Factors depends on interface roughness, Assuming smooth surface μ		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
strength reduction factor for section cracked in shear = $\nu = 0.6 [1 - f_{ck}/310]$		0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51	0.51
maximum value of interface shear = $0.5 v_{fcd} = v_{Rdmax}$	N/mm²	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15	5.15
Check for interface shear stress		< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}	< v_{Edi}
		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

Equating the $v_{Edi} = v_{Rdi}$ to get interface reinf. ρ_{reqd}		0.004	0.004	0.003	0.002	0.002	0.001	0.001	0.001	0.001	0.002	0.003	0.004	0.004
$\rho_{min} = 0.15\%$ of interface area		0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
Governing reinforcement ratio. ρ		0.004	0.004	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.004	0.004
Spacing required, S_{reqd}		181	185	261	378	278	302	302	302	302	407	315	224	223
Dia. of shear connector, ϕ	mm	16	16	16	16	12	12	12	12	12	16	16	16	16
Nos. of legged		4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg	4 leg
Spacing of connectors, S	mm	180 c/c	180 c/c	260 c/c	370 c/c	270 c/c	300 c/c	300 c/c	300 c/c	270 c/c	370 c/c	260 c/c	180 c/c	180 c/c
Area of shear connectors, A_s	mm ²	4468	4468	3093	2174	1676	1508	1508	1508	1676	2174	3093	4468	4468
$\rho_{provided} = A_s / A_i$		0.004	0.004	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.004	0.004

Check for Shear in flange portion (Refer Clause No. 10.3.5 of IRC:112-2011)

Flexural compression and tension is carried mainly by the flanges and the variation of flexure leads to shear at the junction of flanges with web. The flanges also will be subjected to shear flow in its own plane

Long. Shear stress at junction between Flange and web $v_{Edf} = \Delta F_d / (h_f \cdot \Delta x)$

Thickness of flange at junc. of flange and web, h_f	mm	411	411	441	470	470	470	470	470	470	470	441	411	411
Ultimate bending moment	kNm	694.8	694.8	2882.0	4199.5	7151.5	9411.7	10237.4	9429.7	7181.2	4143.1	2264.5	667.0	667.0
Lever arm of composite section, z	m	1.1	1.1	1.3	1.4	1.5	1.5	1.6	1.5	1.5	1.4	1.3	1.1	1.1
Design force along height, F_d	kN	650.5	638.3	2302.6	3074.7	4796.3	6076.9	6596.4	6089.4	4815.4	3030.9	1807.4	611.8	623.5
Change in Force along length Δx , ΔF_d	kN	-12.2	1664.4	772.0	1721.6	1280.6	519.5	519.5	507.0	1274.0	1784.5	1223.5	1195.6	-11.7
Δx	m	0.20	1.65	1.50	2.79	2.97	2.97	2.97	2.97	2.97	2.79	1.50	1.65	0.20
Long. Shear stress at interface, v_{Edf}	N/mm ²	-0.15	2.45	1.17	1.31	0.92	0.37	0.37	0.36	0.91	1.36	1.85	1.76	-0.14
$\cot \theta_f$		2	2	2	2	2	2	2	2	2	2	2	2	2
θ_f	rad	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46
$v_{fcd} \sin \theta_f \cdot \cos \theta_f$	N/mm ²	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12	4.12
Check for crushing of junction		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
$f_{ctd} = f_{ctk} \cdot 0.05/\gamma_m$	N/mm ²	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53	1.53
$0.4 f_{ctd}$	N/mm ²	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Check for reqd. Transverse Reinf. Other Than Shear Reinf.	=	NOT REQD.	REQD	REQD	REQD	REQD	NOT REQD.	NOT REQD.	NOT REQD.	REQD	REQD	REQD	REQD	NOT REQD.

Transverse Reinf. Asf /sf = vED . hf / fyd .														
cot θ_f		0	1513	772	926	647	262	262	256	644	960	1224	1087	0
Dia. of Reinf.	mm	12	12	12	12	12	12	12	12	12	12	12	12	12
Legs		2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg	2 Leg
Asf provided	mm ²	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2	226.2
Spacing reqd.	mm	0.0	149.5	293.0	244.2	349.6	861.7	861.7	883.0	351.4	235.6	184.9	208.1	0.0
Spacing Provided	mm	0	140	290	240	340	860	860	880	350	230	180	200	0

DESIGN OF END BLOCK

End block shall be designed as per clause 13.5 of IRC :112-2011

Due to Bursting Force

Ultimate Tensile Force of one cable = 2203.0 kN

Size of Plate = $2 Y_{P0}$ = 240 mm

Size of End block = $2 Y_0$ = 350 mm

Cable	Prism Dimension $2Y_0$		Y_{P0}/Y_0		F_{bst}/P_k	
	Vertically	Laterally	Vertically	Laterally	Vertically	Laterally
1	300	350	0.80	0.69	0.12	0.12
2	300	350	0.80	0.69	0.12	0.12
3	300	700	0.80	0.34	0.12	0.247
4	300	700	0.80	0.34	0.12	0.247
5	300	700	0.80	0.34	0.12	0.247
Max					0.120	0.196
Fbst					264.4	432.4

To avoid crack width check , permissible stress kept in steel = **250.00** N/mm²

Area of steel Required as per Clause 13.5.3 = $\frac{432.42}{250.00} \times 1000$

= 1729.67 mm²

Above steel is to be distributed within a range from 0.2 Y_0 to $2Y_0$ from the loaded face of end block

i.e. 35.00 mm to 350.00 mm from loaded face. I.e 315.00

BURSTING REINFORCEMENT

Diameter of Helical Reinf. = 16 mm

Nos. of turns reqd. = $\frac{1729.67}{201.06} \times 315.00$

= 8.603

Provide	9	nos. of turns of dia	16 mm (providing reinf. =	1809.5574 mm ²

Anti Spalling Reinforcement

In addition to the bursting reinforcement , provide antispalling reinforcement between the concrete face and far end of the anchor in the form of hairpin meshes. One hairpin mesh made of 12 diameter bars, will be provided nearest to the concrete face of end block and other at the end of diameter 16mm bar .

Force = 0.04 P_k = 0.04 x 2203.00

= 88.12 kN

Area of steel Required as per Clause 13.5.3 = $\frac{88.12}{250} \times 1000$

= 352.48 mm²

Adopt **16** mm dia bar for spalling Reinforcement

Provide	2	Nos.	16 mm dia bar , Provided Ast =	402.12 mm ²	OK

Hence 2 Nos. - 16 dia Horizontally as well as Vertically

Provision of Untensioned Reinforcement in Girder

Refer Clause 12.3.3. (Note -4) of IRC:112-2011

In Prestressed members , no minimum Reinf. is required in sections where the concrete is in compression under Rare Combination of loads and the characteristic value of prestress, However minimum reinforcement for the other consideration such as, early thermal and shrinkage cracking , prior to application of prestressing , shall be provided as per section 16

Surface Reinforcement As per Clause No. 16.5.4 of IRC:112-2011

Area of surface Reinf.	As. Sur	=	0.01 x Act,ext	
where , Act, ext = area of cover portion outside the stirrups / Links				
Perimeter of cross section of the Girder alone (running section)		=	5672.3	mm
Perimeter of cross section of the Girder alone (support section)		=	5128.45	mm
Clear cover to Reinforcement		=	40	mm
Maximum spacing of Surface reinf. (cl. 16.5.4.2 of IRC:112)		=	200	mm
Area of surface reinforcement = $0.01 \times 5672.3 \times 40$		=	2268.92	mm ²
Torsional Reinforcement required		=	1754.33	mm ²
Governing Reinforcement (max of above)		=	2268.92	mm ²
Adopt diameter of surface reinf.		=	12	mm
Nos. of Bars reqd.		=	20.0616	mm
		say	21	
Provide	21 Nos. -	12 mm dia	, Ast = 2375.04	mm² OK

DESIGN OF END DIAPHRAGM NEAR EXPANSION JOINT

Nos. of Long. Girder	=	4			
c/c distance of Long. Girder	=	3 m			
Depth of Girder excluding slab	=	1.6 m			
Depth of Girder with slab	=	1.82 m			
Nos. of bearing under long. Girder	=	4			
Spacing between brg.	=	3 m			
Nos. of jack	=	3 Nos			
Spacing between jacks	=	3 m			
Thickness of end cross girder	=	0.4 m			
Depth of cross girder	=	1.57 m			
Length of End cross girder	=	3 m			
Weight of end diaphragm	=	0.40 x	1.57 x	3.00 x	25
	=	47.1 KN	=		15.7 kN/m

Girder Reaction :

Support reaction under each girder	Unit	OG1	IG1	IG2	OG2
Self-Weight of girder	KN	251.81	251.81	251.81	251.81
Deck slab weight	KN	442.31	408.28	408.28	442.31
Total DL	KN	694.12	660.10	660.10	694.12
from staad pro					
SIDL 1 (except surfacing)	KN	145.02	-45.23	-45.23	145.02
SIDL2 (weight of surfacing)	KN	76.22	81.58	81.58	76.22
Foothpath	KN	0.00	0.00	0.00	0.00
CWLL	1-70R + 1-CLASS A	KN	286.98	440.37	397.19
					-9.32

STAAD-PRO RESULTS :

Jacking Up Condition

	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (KNm)	Maximum Shear Force - Cantilever portion (KN)
Total DL	308	1073	726
SIDL 1 (except surfacing)	71.6	218	145
SIDL2 (weight of surfacing)	37.4	114	76
CWLL	0	0	0

BEARING REACTION

		Bearing 1	Bearing 2	Bearing 3	Bearing 4
Total DL	KN	694.12	660.10	660.10	694.12
SIDL 1 (except surfacing)	KN	145.02	-45.23	-45.23	145.02
SIDL2 (weight of surfacing)	KN	76.22	81.58	81.58	76.22
Foothpath	KN	0.00	0.00	0.00	0.00
CWLL	KN	286.98	440.37	397.19	-9.32

1202.34 1136.82 1093.63 906.03

JACK REACTION

		JACK 1	JACK 2	JACK 3
Total DL	KN	1381	379	1105
SIDL 1 (except surfacing)	KN	218	-194	175
SIDL2 (weight of surfacing)	KN	147	49	119
Total		1746	234	1399

there fore , Capacity of each jack = 2095.2 KN

SLS MOMENT - RARE COMBINATION FOR STRESS CHECK

Jacking Up Condition

	Load Factor	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (KNm)	Maximum Shear Force (KN)
Total DL	1.00	308	1073	726
SIDL 1 (except surfacing)	1.00	71.6	218	145
SIDL2 (weight of surfacing)	1.20	44.88	136.8	91.2
CWLL	1.00	0	0	0
Total SLS Moment		424.48	1427.8	962.2

SLS MOMENT - FREQUENT COMBINATION

Jacking Up Condition

	Load Factor	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (KNm)	Maximum Shear Force (KN)
Total DL	1.00	308	1073	726
SIDL 1 (except surfacing)	1.00	71.6	218	145
SIDL2 (weight of surfacing)	1.20	44.88	136.8	91.2
CWLL	0.75	0	0	0
Total SLS Moment		424.48	1427.8	962.2

SLS MOMENT - QUASI-PERMANENT COMBINATION FOR CRACK CHECK **Jacking Up Condition**

	Load Factor	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (kNm)	Maximum Shear Force (KN)
Total DL	1.00	308	1073	726
SIDL 1 (except surfacing)	1.00	71.6	218	145
SIDL2 (weight of surfacing)	1.20	44.88	136.8	91.2
CWLL	0.00	0	0	0
Total SLS Moment		424.48	1427.8	962.2

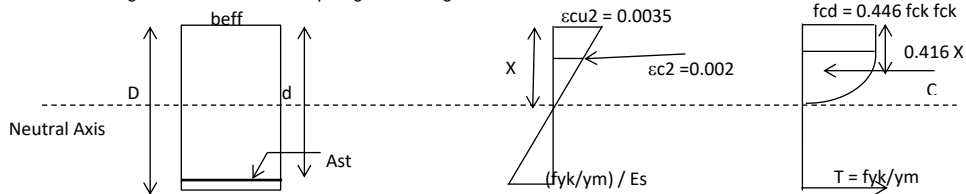
ULS MOMENT- BASIC CASE**Jacking Up Condition**

	Load Factor	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (kNm)	Maximum Shear Force (KN)
Total DL	1.35	415.8	1448.55	980.1
SIDL 1 (except surfacing)	1.35	96.66	294.3	195.75
SIDL2 (weight of surfacing)	1.75	65.45	199.5	133
CWLL	1.5	0	0	0
Total ULS Moment		577.91	1942.35	1308.85

ULS DESIGN OF SECTION OF END DIAPHRAGM

section will be designed for ULS Case and check for Servicability condition

Considered rectangular section of End Diaphragm for design



beff	=	400 mm
D	=	1570.00 mm
Clear cover, c	=	40.00 mm

Material Safety Factor:

γm for steel	=	1.15
γm for concrete	=	1.5

Concrete Strain

εcu2	=	0.0035
εc2	=	0.002

Material Stress

fck	=	45 N/mm ²	fcd	=	20.07 N/mm ²
fyk	=	500 N/mm ²	fyd	=	434.78 N/mm ²
fctm	=	3.3 N/mm ²			

DESIGN OF SECTION FOR HOGGING MOMENT

Assume	Depth of Neutral Axis , X	=	295.561 mm

Compression Force due to rectangular portion C1	=	0.446 fck . 3/7X . beff	=	1016.90 KN
Comp. Force due to upper part of Parabolic portion C2	=	2/3 . 0.446 fck . 4/7X . beff	=	903.91 KN
Total Compressive Force = C = C1+ C2	=	1920.811 KN	eq 1	
Maximum Spacing between jack , L	=	3.00 m		
L/D	=	1.91	<	2.5
Provide Reinforcement in	=	3 layers		

DEEP BEAM

Provide	3 Nos of	25 mm dia bars	(= 1472.622 mm ²)	in 1st Layer
Provide	3 Nos of	25 mm dia bars	(= 1472.622 mm ²)	in 2nd Layer
Provide	3 Nos of	25 mm dia bars	(= 1472.622 mm ²)	in 3rd Layer

Total Area of steel provided , Ast	=	4417.865 mm ²	OK
Minimum area of Long. Steel reqd. , Ast.min	=	1131.122 mm ²	
Percentage of steel provided	=	0.703 %	
Spacing between reinf. c/c	=	90.3 mm	

Effective cover to bottom reinf.	=	114.5 mm
Effective depth provided	=	1455.50 mm

Total Tensile Force	T	=	fyk/γm x Ast	=	1920.811 KN	eq. 2

Force section , C = T	=	0.000	Hence assume Neutral axis is correct

Check for Effective depth of cross girder

Mult	=	0.167 fck . b . d ²	=	1942.35 kNm
effective depth reqd.	dreq	=	803.839 mm	
Provided effective depth	dpro	=	1455.500 mm	OK

Check that steel will yield

conc. Strain at level of tensile steel $0.0035 / x \cdot (d-x) = 0.0137$
 Yield strain of tensile reinf. $= 0.002 < 0.0137$ **Steel Yield**

Moment Capacity of section $M_R = T \cdot (d - 0.412 X) = 2561.840 \text{ kNm} > 1942.35 \text{ SAFE}$

Minimum Reinforcement Required

a) Min area of Reinf. Req'd. For crack control $A_{s,min} = k_c \cdot K \cdot f_{ct,eff} \cdot A_{ct} / \sigma_{ct}$
 (Refer CL: 12.3.3 (2) of IRC:112-2011 where
 Coeff. $K = 0.930$
 $f_{ct,eff} = 3.3 \text{ N/mm}^2$
 coeff. $K_c = 0.400$ for Flexural Beam, axial compressive stress will be zero
 area of concrete within tensile zone, $A_{ct} = 463975.476 \text{ mm}^2$
 $\sigma_{ct} = f_{yk} = 500 \text{ N/mm}^2$

$A_{s,min} = \text{min. Area of reinf. Within tensile zone} = 1131.122 \text{ mm}^2$

b) Min Long. Reinf. Req'd. $A_{s,min} = 0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d$ Refer eq. 16.5.1.1 & 16.6.1.1 of IRC:112
 (Refer CL: 12.3.3 (2) of IRC:112-2011 OR $= 0.0013 \cdot b \cdot d$ Refer clause 16.9 of IRC:112-2011
 $A_{st,min} = 1070.051 \text{ mm}^2$
 Governing min Long. Reinf. $A_{st,min} = 1131.122 \text{ mm}^2$

DESIGN OF SECTION FOR SAGGING (SPAN) MOMENT

Assume **Depth of Neutral Axis, X** $= 98.520 \text{ mm}$

Compression Force due to rectangular portion C1 $= 0.446 f_{ck} \cdot 3/7 X \cdot b \cdot e_{eff} = 338.967 \text{ KN}$
 Comp. Force due to upper part of Parabolic portion C2 $= 2/3 \cdot 0.446 f_{cd} \cdot 4/7 X \cdot b \cdot e_{eff} = 301.304 \text{ KN}$
 Total Compressive Force $= C = C1 + C2 = 640.270 \text{ KN}$ eq 1

Provide Reinforcement in $= 1$ layers

Provide	3 Nos of	25 mm dia bars	(= 1472.62 mm²)	in 1st Layer
Provide	0 Nos of	25 mm dia bars	(= 0.00 mm²)	in 2nd Layer
Provide	0 Nos of	20 mm dia bars	(= 0.00 mm²)	in 3rd Layer

Total Area of steel provided, $A_{st} = 1472.62 \text{ mm}^2$ **OK**
 Minimum area of Long. Steel req'd., $A_{st,min} = 1131.12 \text{ mm}^2$
 Percentage of steel provided $P_t = 0.234 \%$
 Spacing between reinf. $c/c = 117.0 \text{ mm}$

Effective cover to bottom reinf. $= 89.5 \text{ mm}$
 Effective depth provided $d_{pro} = 1480.50 \text{ mm}$

Total Tensile Force $T = f_{yk} / y_m \times A_{st} = 640.270 \text{ KN}$ eq. 2

Force section, C = T $= 0.000$ Hence assume Neutral axis is correct

Check for Effective depth of cross girder

Ultimate Sagging Moment = Mult $= 0.167 f_{ck} \cdot b \cdot d^2 = 577.91 \text{ kNm}$
 effective depth req'd. $d_{req} = 438.47 \text{ mm}$
 Provided effective depth $d_{pro} = 1480.50 \text{ mm}$ **OK**

Check that steel will yield

conc. Strain at level of tensile steel $0.0035 / x \cdot (d-x) = 0.0491$
 Yield strain of tensile reinf. $= 0.002 < 0.0491$ **Steel Yield**

Moment Capacity of section $M_R = T \cdot (d - 0.412 X) = 921.931 \text{ kNm} > 577.91 \text{ SAFE}$

ULS SHEAR CHECK OF CANTILEVER PORTION

Shear Capacity without Shear Reinforcement

$V_{Rd,c} > V_{Ed}$ -- Refer Clause 10.3.2 of IRC:112-2011

Shear Resistance without shear Reinforcement, $V_{Rd,c} = [0.12 K (80 \cdot \rho_1 f_{ck})^{0.33} + 0.15 \sigma_{cp}] \cdot b_w \cdot d$

Subjected minimum $V_{Rd,c} = (v_{min} + 0.15 \sigma_{cp}) \cdot b_w \cdot d$ where $v_{min} = 0.031 K^{3/2} \cdot f_{ck}^{1/2}$

Ultimate Design Shear Force	V_{Ed}	KN	1308.850
Width of section	b	mm	400
Effective depth	d	mm	1455.50
f_{ck}		N/mm ²	45.00
$K = 1 + \text{SQRT} (200 / d) \leq 2.0$			1.371
Mean compr. stress at c.g. sec. $\sigma_{cp} = N_{Ed} / A_c \leq 0.2 f_{cd}$			0.000
Long. tension Reinf. A_{st}		mm ²	4418
$\rho_1 = A_{sl} / (b_w \cdot d) \leq 0.02$			0.008
steel ratio limited to 0.02			OK
$V_{Rd,c}$		KN	285.25
v_{min}		N/mm ²	0.33

min $V_{Rd,c}$	KN	194.29
Max ($V_{Rd,c}$, min $V_{Rd,c}$)	KN	285.25
Check for Requirement of Shear Reinf.	$V_{Rd,c}$	< V_{Ed}
	Hence	REQUIRED

Shear Capacity with Design Shear Reinforcement**Refer Clause 10.3.3 of IRC:112-2011**

Characteristic yield stress of steel , f_{yk} = 500 N/mm²
 Design yield strength of shear reinforcement, $f_{ywd} = 0.8 \cdot f_{yk}$ = 400 N/mm² Refer clause 10.3.3.2 of IRC:112-2011
 Strength reduction factor , v_1 = 0.6 Refer clause 10.3.3.2 of IRC:112-2011

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot \theta$$

Eq. 10.7

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

Eq. 10.8

Dia. of shear stirrups , ϕ	mm	12
Nos. of legged		2 leg
Spacing of stirrups, S	mm	200 c/c
Width of web , b_w	mm	400.00
$V_{Rd,s} = V_{Ed}$	KN	1308.9
Area of shear Reinf. Provided A_{sw}	mm ²	226
Coefficient , α_{cw} for $\sigma_{cp} = 0$		1.00
Lever arm , $Z = 0.9 d$	mm	1309.95
$1 \geq \cot \theta \geq 2.5$		2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22
$V_{Rd,max}$	KN	2175.78
Section check for shear capacity , $V_{Ed} \leq V_{Rd,max}$		SAFE
Reqd spacing of shear Reinf. S	mm	226
Check for Provided Spacing		SAFE

Minimum Shear Reinforcement

Min Shear Reif. ratio , $\rho_{min} = 0.072 f_{ck}^{0.5}/f_{yk}$		0.0010
Shear Reinf. ratio provided , $\rho_w = A_{sw}/(S \cdot b_w \cdot \sin \alpha)$		0.0028
Check for Shear Reinf. Ratio		OK
Maximum Spacing $S_{l,max} = 0.75 d (1 + \cos \alpha)$, where $\alpha = 90^\circ$, refer eq. 16.6	mm	1092
Check for Provided Spacing		OK

Refer Eq. 10.20 of IRC:112-2011

Refer Eq. 16.4 of IRC:112-2011

SLS CHECK OF END DIAPHRAGM

1. EARLY AGE CHECK (BEFORE CREEP HAS OCCURED)
2. LONG TERM CHECK (AFTER CREEP HAS TAKEN PLACE)

Maximum compressive stress in concrete under rare combination	$0.48 \cdot f_{ck}$	=	21.6	N/mm ²
Maximum tensile stress in steel under rare combination		=	300	N/mm ²
Maximum Tensile stress in concrete	f_{ctm}	=	3.277	N/mm ²
Permissible crack width	mm - For Moderate Exposure Condition	=	0.3	mm

Section Properties of Uncracked Section:

Width of Section , b	=	400.00	mm
Depth of Section , D	=	1570.00	mm
Gross Cross sectional area , A_g	=	628000	mm ²
Gross Moment of Inertia I_{gxx}	=	1.29E+11	mm ⁴
Gross Moment of Inertia I_{gyy}	=	8.37E+09	mm ⁴

Modular Ratio : for Early Age Check

Modulus of elasticity of concrete, $E_{ceff} = E_{cm}$	=	34000	N/mm ²
Modulus of elasticity of steel, E_s	=	200000	N/mm ²
Modular Ratio (m) = E_s / E_{ceff}	=	5.88	

Modular Ratio: for Long Term Check

Perimeter of section (u)	=	3940	mm
$2 A_c/u$	=	318.782	mm
Modulus of elasticity of concrete, E_{cm}	=	34000	N/mm ²
Modulus of elasticity of steel, E_s	=	200000	N/mm ²
For Dry atmosperic condition			

Creep coeff. $\emptyset = 2.31$

$$E_{eff} = \frac{E_{cm}}{(1 + \emptyset)} = 1.03E+04$$

Modular Ratio (m) $= E_s / E_{eff} = 19.49$

STRESS CHECK IN CRACKED SECTION

Depth of neutral axis =

$$d_c = \frac{-A_s E_s + \sqrt{(A_s E_s)^2 + 2bA_s E_s E_{c,eff} d}}{bE_{c,eff}}$$

Cracked Second Moment of Area $I_{NA} = A_s (d - d_c)^2 + \frac{1}{3} \frac{E_{c,eff}}{E_s} b d_c^3$
(in steel units)

	unit	Rare Case				Quasi-Permanent Case		Remarks
		For Early Age		For Long Term Check		Hogging Moment	Sagging Moment	
		Hogging Moment	Sagging Moment	Hogging Moment	Sagging Moment			
SLS Moment	kNm	1427.800	424.480	1427.800	424.480	1427.800	424.480	
Width of Section, b	mm	400.00	400.00	400.00	400.00	400.000	400.000	
Depth of section, D	mm	1570.00	1570.00	1570.00	1570.00	1570.00	1570.00	
Effective cover, C_{eff}	mm	100.50	75.50	100.50	75.50	100.50	75.50	
Effective depth, d	mm	1469.50	1494.50	1469.50	1494.50	1469.50	1494.50	
E_{eff}	N/mm ²	34000.00	34000.00	10264.26	10264.26	34000.00	34000.00	
E_s	N/mm ²	200000.00	200000.00	200000.00	200000.00	200000.00	200000.00	
Ast Provided, A_s	mm ²	4417.86	1472.62	4417.86	1472.62	4417.86	1472.62	
dc		376.80	233.69	608.69	396.84	376.80	233.69	
Cracked Second Moment of area, I_{NA}	mm ⁴	6.49E+09	2.63E+09	4.82E+09	2.20E+09	6.49E+09	2.63E+09	
section modulus, $Z_t = I_{NA} / d_c$	mm ³	1.72E+07	1.13E+07	7.91E+06	5.55E+06	1.72E+07	1.13E+07	
section modulus, $Z_b = I_{NA} / (d - d_c)$	mm ³	5.94E+06	2.09E+06	5.60E+06	2.01E+06	5.94E+06	2.09E+06	
Maximum compressive stress in concrete = $M/Z_t \times E_{eff} / E_s$	N/mm ²	14.10	6.41	9.26	3.93	14.10	6.41	< 21.6 SAFE
Maximum Tensile stress in concrete = $M/Z_b \times E_{eff} / E_s$	N/mm ²	40.88	34.59	13.10	10.86	40.88	34.59	
Maximum Tensile stress in steel = M/Z_b	N/mm ²	240.49	203.48	255.16	211.60	240.49	203.48	< 300 SAFE

CRACK WIDTH CHECK

Refer Clause 12.3.4 of IRC:112-2011

$$\text{Crack Width} = W_k = S_{r \max} (\epsilon_{sm} - \epsilon_{cm})$$

Where, $S_{r \max}$ = maximum crack spacing
 ϵ_{sm} = mean strain in the reinforcement
 ϵ_{cm} = mean strain in concrete between cracks

Spacing between reinf. = $5^*(c + \phi/2)$	mm	502.5	377.5
Spacing provided	mm	90.3	117.0
Check for spacing criteria		OK	OK
$S_{r \max} =$	3.4 c	+	$0.425 k_1 k_2 \phi$
Clear Cover, c	mm	40.00	$\rho_{p,eff}$ 40.00
Diameter of Main Bar, ϕ	mm	25.00	25.00
Coefficient, k_1		0.8	0.80
Coefficient, k_2		0.5	0.50
Width of section, b	mm	400.00	400.00
Depth of section, D	mm	1570.00	1570.00
Effective Depth of Section, d	mm	1469.50	1494.50
Depth of Neutral axis, $y_t = d_c$	mm	376.80	233.69
$h_{c,eff} = \text{Min of } 2.5 (D - d), D - d_c/3, D/2$	mm	251.25	188.75
$A_{c,eff} = b * h_{c,eff}$	mm ²	100500	75500
$\rho_{p,eff} = A_s / A_{c,eff}$		0.0440	0.0195
$S_{r \max}$	mm	232.68	353.89
$(\epsilon_{sm} - \epsilon_{cm})$	$= \frac{\sigma_{sc} - k_1 f_{t,eff} (1 + \alpha_e \rho_{p,eff})}{E_s}$		
Stress in tension steel, σ_{sc} (in Quasi-Permanent Case)	N/mm²	240.49	203.48
Kt		0.5	0.5
Tensile strength of concrete = $f_{ct,eff} = f_{ctm}$	N/mm ²	3.28	3.28
$\alpha_e = E_s / E_{cm}$		5.882	5.882
$(\epsilon_{sm} - \epsilon_{cm})$		0.0010	0.0005
Crack Width, W_k	mm	0.23	0.19
Limited Crack width	mm	0.30	0.30
Check for Crack width		< 0.3 mm SAFE	< 0.3 mm SAFE

DESIGN OF DIAPHRAGM-DEEP BEAM DESIGN**(Jacking condition)**

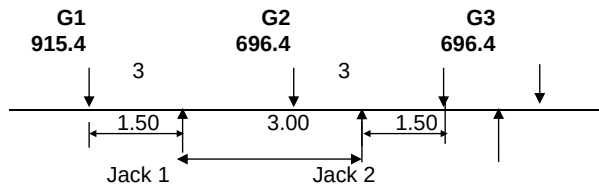
All The Girder are supported on bearing and the reaction transferred to the bearing directly.Hence End Diaphragm is critical in Jacking up condition.The reactions below the each girder due to SIDL & DL has been calculated with the help of STAAD. The end diaphragm is considered as a beam supported on six jacks.The reactions of the girder has been applied on the beam.Then the analysis has been done by the help of STAAD.

$$\text{Skew Angle} = 0 \text{ Degree} = 0 \text{ Radians}$$

$$\cos 0 = 1$$

Girder No.

Load (DL+SIDL)



$$\begin{aligned} \text{No of Jacks} &= 3 \\ \text{Overall Depth of Diaphragm} &= 1.57 \text{ m} \\ \text{Width of diaphragm} &= 0.4 \text{ m} \end{aligned}$$

SUMMARY OF GIRDER REACTIONS FROM STAAD OUTPUT

SUPPORT NO	DL (kN)	SIDL (kN)	TOTAL REACTION IN KN	LOAD APPLIED IN STAAD (Say) IN KN
G1	694.12	221.24	915.4	915.4
G2	660.10	36.35	696.4	696.4
G3	660.10	36.35	696.4	696.4
G4	694.12	221.24	696.4	696.4

from STAAD output

$$\text{Maximum Sagging Moments} = 356.95 \text{ KN-M} \quad \text{TAKEN 25\% OF HOGGING MOMENT}$$

$$\text{Maximum Hogging Moments} = 1427.8 \text{ KN-M}$$

$$\text{Maximum Shear} = 962.2 \text{ KN}$$

DESIGN

$$L/D = 1.911 < 2.5 \text{ Design as deep beam as per IS : 456 -2000}$$

$$\begin{aligned} \text{Liver arm} &= 0.2 (l + 1.5D) \quad \text{As per clause 29.2 of IS : 456 -2000} \\ &= 0.2 (3.000 + 1.5 \times 1.57) = 1.071 \text{ m} \end{aligned}$$

$$\text{Maximum sagging B M} = 356.95 \text{ kN-m}$$

$$\begin{aligned} \text{Ast required} &= \frac{356.95 \times 1000000}{240 \times 1071.000} \\ &= 1389 \text{ mm}^2 \\ \text{Minimum steel} &= 0.2 b d \\ &= 0.2 / 100 \times 400 \times 1501.5 = 1201.2 \text{ mm}^2 \\ &= \frac{d}{1501.5} = \frac{1.57}{1501.5} = 0.04 - 0.0125 - 0.016 \end{aligned}$$

Provide	3 Nos of	25 mm dia bars	(= 1474.69 mm ²)	in 1st Layer
Provide	2 Nos of	20 mm dia bars	(= 629.20 mm ²)	in 2nd Layer

This is to be placed within a zone of depth equal to 0.25 D - 0.05 L i.e. 0.243 m from the tension face (soffit) of the girder ,

$$\text{Design hogging B M} = 1427.80 / 1 = 1427.80 \text{ kN-m}$$

$$\begin{aligned} I / D &= 3.000 / 1.57 = 1.911 > 1.0 \\ \text{Lever arm} &= 0.2 (l + 1.5D) = 1.07 \quad \text{As per cl: 29.2 of IS:456-1978} \\ \text{Net liver arm} &= 1.071 \end{aligned}$$

$$\begin{aligned} \text{Ast required} &= \frac{1427.80 \times 1000000}{240 \times 1071} \\ &= 5554.8 \text{ mm}^2 \end{aligned}$$

Distribution of negative steel

As per cl: 29.3.2(b) of IS:456-2000

(I) A zone of depth 0.2 D adjacent to the tension (top) face of cross girder , which shall contain a proportion of the tension steel given by $0.5 (I / D - 0.5) = 0.71$

$$\text{Ast (Required) for this zone} = 0.71 \times 5555 = 3918 \text{ mm}^2$$

Provide	3 Nos of	25 mm dia bars	(= 1475 mm ²)	in 1st layers
Provide	3 Nos of	25 mm dia bars	(= 1475 mm ²)	in 2st layers
Provide	3 Nos of	25 mm dia bars	(= 1475 mm ²)	in 3rd layers

(ii) A zone of measuring 0.3 D on either sides of the mid of the cross girder
 containing the remainder of the total steel i.e.
$$= 5555 - \frac{4424 \text{ in}}{0.471 \text{ m}} \times \frac{314 \text{ mm}}{1000} = 1636 \text{ mm}^2$$

Provide	16 Nos of	12 mm dia bars	(=	1812 mm²)	@ Equal Spacing

As per cl: 28.1 (b) of IS:456, there is no need to check for shear .

However provide	2 L egged	16 mm @	150 mm c/c	for binding of longitudinal steel

DESIGN OF LIFTING HOOK

Vertical Load on hook	=	679.889 kN	
Diameter of the cable		12.7	
Cable provided	=	5-12.7mm HTS strands 7 Ply class II	
Area of Bar	=	493.5 mm ²	IRC: 112 -2011 Table 18.4
Ultimate capacity of Bar	=	1861.196 N/mm ²	
Allowable direct tension stress	=	0.87 fy 1619.24 N/mm ²	
Actual Direct tension	=	1377.69 N/mm ²	OK

CHECK FOR BONDING

Bonding stress	=	3.00 N/mm ²	(IRC: 112-2011 Table 15.3)
Ld	$\frac{\phi \sigma_s}{4\tau_{bd}}$	=	1458.05 mm (IRC: 112-2011 Cl 15.2.3.3)

DESIGN OF ABUTMENT(A2) WITH
OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
17+785

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 0 m
Design speed of vehicle = 0 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 17.700m	17.700m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side) =	1.125m	1.125m
Slab projection Beyond C/L of Bearing (Span Side) =	1.125m	1.125m
Span -c/c of E.J.	= 20.000m	20.00m
Type of Superstructure	= Precast RCC Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 11.500m	
Projection beyond crash barrier	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 0.000m	
Railing/kerb on footpath edge	= 0.000m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 4
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 320 kN/m² (as per geotechnical report)
SBC of soil-Normal Case = 346 kN/m²
SBC of soil-Seismic Case = 432 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1.5 Normal Case
= 1.25 Seismic Case

Permissible FOS against Overturning = 2 Normal Case
= 1.5 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 2.644m	
Height of Tapering portion	= 0.000m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 2.780m	2.780m
Width of Abutmentcap at bottom of Tapering Portion	= 1.000m	1.000m
Projection of Abutment Cap (Span Side)	= 0.750m	0.750m
Projection of Abutment Cap Back Side	= 1.030m	1.030m
Abutmentcap thickness (Uniform portion)	= 0.300m	

Abutment cap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	1.000m	1.000m
Thickness of Abutment shaft at Bottom	=	1.000m	1.000m

Solid Return Wall

Length of Return wall	=	2.700m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	5.500m
abutment pedestal width	=	1.000m
abutment pedestal Height	=	0.700m
Width of Toe Slab	=	1.800m
Width of Heel Slab	=	2.700m
Thickness of Toe slab at tip	=	0.300m
Thickness of Toe slab near shaft	=	0.700m
Thickness of heel slab at tip	=	0.300m
Thickness of heel slab near shaft	=	0.700m
Width of backfill on heel slab	=	2.700m
Thickness of heel slab at back fill edge	=	0.700m
Height of back fill at bottom edge of heel slab	=	5.485m
Height of back fill at back fill edge of heel slab	=	5.085m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1458.036m	Cross Slope (Bi-directional)	=	7.000%	
Deck level at Outer Edge =	1457.634m	Height of Superstructure	=	1.820m	
Deck level at center line =	1458.036m	Min. Height of Footpath Side Pedestal (1)	=	0.400m	
Soffit Level at center of bridge =	1456.151m	Height of Pedestal (2)	=	0.610m	
Abutment cap top level =	1455.191m	Height of Pedestal (3)	=	0.820m	
Abutment cap bottom lvl (uniform portion ends)	1454.891m	Height of Pedestal (4)	=	1.030m	
Abutment cap bottom lvl (corbel portion ends)	1454.591m	Height of Pedestal (5)	=	0.000m	
Abutment shaft top level =	1454.591m	Height of Pedestal (6)	=	0.000m	
Ground level/LBL =	1454.617m	Distance of nearest girder to c.l. of deck	=	1.500m	
Abutment shaft bottom level =	1452.750m	Height (Avg.) of Dirt Wall	=	2.644m	
Foundation level =	1452.050m	Abutment shaft Above G.L	=	0.000m	
HFL	=	1455.617m	Abutment Shaft below G.L	=	1.867m
			Height of abutment shaft	=	1.841m
			MSL	=	1454.617m
			Wedge over girder flange	=	0.0700m

Material Specification

Concrete Grade	=	M 35
Characteristic compressive strength of concrete, f _{ck}	=	35.00 Mpa at 28 days
Design Compressive strength of Concrete, f _{cd}	=	15.63 Mpa at 28 (0.67/1.5 * f _{ck})
Tensile strength of concrete , f _{ctm}	=	2.77 MPa

Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E _{cm}	=	32308.250	N/mm ²
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f _y or f _{yk}	=	500	Mpa
Design Yield Strength of Reinforcement, f _{yd}	=	434.78	Mpa (1/1.15 * f _y)
Modulus of Elasticity of Steel (E _s)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48	f _{ck}
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm
<u>Seismic Data:</u>			
Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction
<u>Properties of backfill material :</u>			
c	=	0	
φ	=	35	
θ	=	90	
β	=	0	
δ	=	22.5	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	-0.055m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

		eT
DL Reaction on Bearing No.-1	=	431.21 4.50
DL Reaction on Bearing No.-2	=	413.49 1.50
DL Reaction on Bearing No.-3	=	413.49 -1.50
DL Reaction on Bearing No.-4	=	431.21 -4.50
DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		1689.39 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	135.917 4.50
SIDL Reaction on Bearing No.-2	=	-55.917 1.50
SIDL Reaction on Bearing No.-3	=	-55.917 -1.50
SIDL Reaction on Bearing No.-4	=	135.917 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		160.00 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	57.778 4.50
SIDL Reaction on Bearing No.-2	=	63.030 1.50
SIDL Reaction on Bearing No.-3	=	63.030 -1.50
SIDL Reaction on Bearing No.-4	=	57.778 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		241.62 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 2-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	1000.02	KN	Corr. Transv	=	950.021 kNm	ecc.	0.95
min	=	349.98	KN	Corr. Transv	=	332.479 kNm	ecc.	0.95

SV Loading

max	=	2020.42	KN	Corr. Transv	=	606.127 kNm	ecc.	0.30
min	=	1399.58	KN	Corr. Transv	=	419.873 kNm	ecc.	0.30

**MAXIMUM TRASVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	696.67	KN	Corr. Transv	=	2197.998 kNm	ecc.	3.16
min	=	203.33	KN	Corr. Transv	=	641.502 kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.095
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.095
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.400 x	0.800 x	0.800 =	0.2560 m ³	4.500
Pedestal 2	=	0.610 x	0.800 x	0.800 =	0.3904 m ³	1.500
Pedestal 3	=	0.820 x	0.800 x	0.800 =	0.5248 m ³	-1.500
Pedestal 4	=	1.030 x	0.800 x	0.800 =	0.6592 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					1.8304 m ³	-1.101

VOLUME CALCULATION

C.G. Of Footing	=	2.750 m
C.G. Of shaft from toe tip	=	2.300 m
Distance between c.g. of shaft and footing	=	0.450 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	2.644	9.914	-1.380	-0.930	-3.680	0.000
-Trapering portion	1	12.50	0.300	0.000	0.000	-1.380	-0.930	-3.680	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.680	-1.230	-3.980	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-1.630	-1.180	-3.930	0.000
Cap (uniform portion)	1	12.50	2.780	0.300	10.425	-0.140	0.310	-2.440	0.000
Cap (Corbel Portion)	1	12.50	2.780	0.300	6.809	-0.140	0.310	-2.440	0.000
		12.50	1.000						
Shaft above HFL	1	12.50	1.000	0.000	0.000	0.000	0.450	-2.300	0.000
Shaft below HFL	1	12.50	1.000	1.841	23.013	0.000	0.450	-2.300	0.000
Solid Return Wall	1	2.70	0.500	5.486	7.406	-1.850	-1.400	-4.150	6.000
Cantilever Return wall(Rectangular portion)	1	0.00	0.500	0.000	0.000	-0.500	-0.050	-2.800	6.000
Cantilever Return wall(Traingular portion)	1	0.00	0.500	0.000	0.000	-0.500	-0.050	-2.800	6.000
Footing									
Heel Slab	1	12.50	2.700	0.500	16.875		-1.220	-3.970	0.000
Toe Slab	1	12.50	1.800	0.500	11.250		1.730	-1.020	0.000
Portion between Heel and Toe	1	12.50	1.000	0.700	8.750		0.450	-2.300	0.000
Back filling above HFL over Heel Slab	1	12.50	2.700	2.419	81.641		-1.400	-4.150	0.000
Back filling below HFL over Heel Slab	1	12.50	2.700	3.067	103.511		-1.429	-4.179	0.000
Backfill above Heel slab	1	12.50	2.700	5.285	178.360		-1.417	-4.167	0.000
Front Filling over Toe Slab	1	12.50	1.800	2.067	46.508		1.821	-0.929	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	1.800	2.067	46.508		1.371	-1.379	0.000
Side filling between heel and toe	1	0.00	1.000	2.067	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-1.680	-1.230	-3.980	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.000	2.419	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.000	3.067	0.000		0.000	0.000	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barr	2	8 kN/m	1.750	28.00kN		-1.380	-0.930	-3.680

SECTIONAL PROPERTIES

Width of Footing (B)	=	5.500 m
Length of Footing (L)	=	12.500 m

A	=	5.500	x	12.500	=	68.750	m ²
ZL	=	12.500	x	5.042	=	63.021	m ³
ZT	=	IT1	+	IT2			
		distance of extreme point from centre					
IT1	=	5.500	x	162.760	=	895.18	m ⁴
IT2 (moment of inertia of triangle)	=	5.500	x	0.000	+	0.500 x 5.500 x	0.000 x 39.063
from centre of footing	=	0.000	m ⁴				
Moment of inertia of two triangle	=	0.000	m ⁴				
Total moment of inertia	=	895.18	m ⁴				
Distance of extreme point from centre of footing	=	6.250	+	0.000	=	6.250	m
Total Section modulus (ZT)	=	143.229	m ³				

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.000	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Serviceability Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.00	1.00	1.00
wearing course	1.20	1.20	1.20
Back Filling Weight	1.00	1.00	1.00
Shrinkage Creep Effect	1.00	1.00	1.00
Earth Pressure due to back filling	1.00	1.00	1.00
CWLL and Associate load and FPLL	1.00	0.75	0.00
Live Load Surcharge	0.80	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

Horizontal Seismic Force For Zone

4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.)
or horizontal vibrations.

$$= 2.0 \frac{D^{1/2}}{1000F}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 3.541 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 5.438 m

Abutment Cap Top Level - Foundation Level

= 3.141 m

Dimensions of Abutment Shaft

Length = 12.50 m

Width = 1.00 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 0.781 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 162.760 m^4

E_{cm} = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 2051.640 KN
 D = 2091.01 KN
 T = 0.0638 sec

Transverse Direction

Force = 242787.252 KN
 D = 2361.010 KN
 T = 0.0062 sec

For Hard or Rocky Strata, with $N > 30$

$\frac{S_a}{g}$ = 1.958

$\frac{S_a}{g}$ = 1.094

For Medium Strata, with $N > 10 \& N < 30$

$\frac{S_a}{g}$ = 1.958

$\frac{S_a}{g}$ = 1.094

For Soft Soil Strata, with $N < 10$

$\frac{S_a}{g}$ = 1.957742413

$\frac{S_a}{g}$ = 1.094

Medium Soil Strata

S_a/g = 1.957742413

S_a/g = 1.094

Seismic Coeff. In Longitudinal Direction = 0.0939716

Seismic Coeff. In Transverse Direction = 0.157

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.094	In Longitudinal direction
Ah	=	0.157	In Transverse direction
Av	=	0.105	In Vertical direction

For Design of Foundation

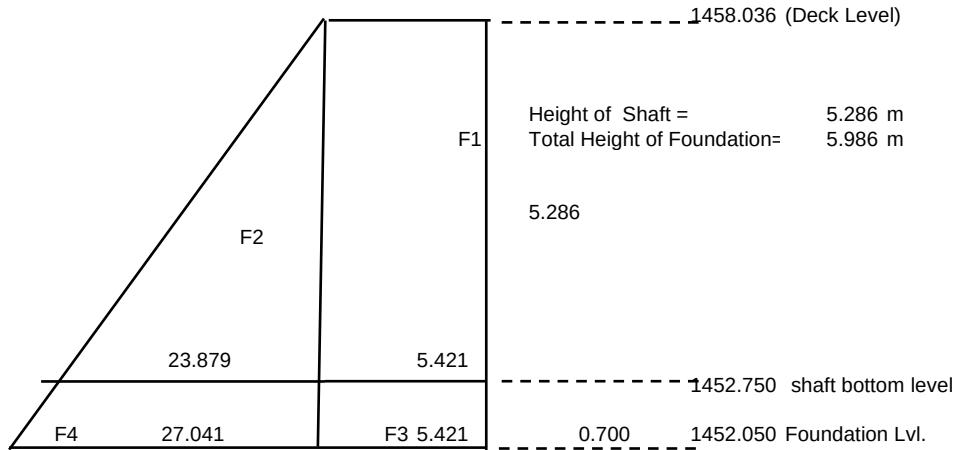
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.127	In Longitudinal direction
Ah	=	0.157	In Transverse direction
Av	=	0.105	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = **1.2 m**
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1452.750 m (at Shaft Base)			
@ RL					1452.050 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	5.286	x	12.500	=	358.180 kN
M1	=	358.18	x	2.64	=	946.669 kN.m		at Shaft Bottom
F3	=	5.421	x	5.986	x	12.500	=	405.612 kN
M3	=	405.612	x	2.993	=	1213.996 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	5.286	= 23.879 kN/m ²
F2	=	0.5	x	23.88	x	5.286	x 12.50
	=	788.891 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	788.89	x	2.22	=	1751.433 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	5.986	= 27.041 kN/m ²
F4	=	0.5	x	27.04	x	5.986	x 12.50
	=	1011.664 kN					
M4	=	1011.66	x	2.51	=	2543.444 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	5.286	=	25.373 kN/m ²	
F	=	0.5	x	25.373	x	5.286	x 12.500
	=	838.254 kN					

$$M = 838.25 \times 1.762 = 1477.003 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 5.986 = 28.73 \text{ kN/m}^2$$

$$F = 0.5 \times 28.733 \times 5.99 \times 12.500 = 1074.966 \text{ kN}$$

$$M = 1074.97 \times 1.995 = 2144.915 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

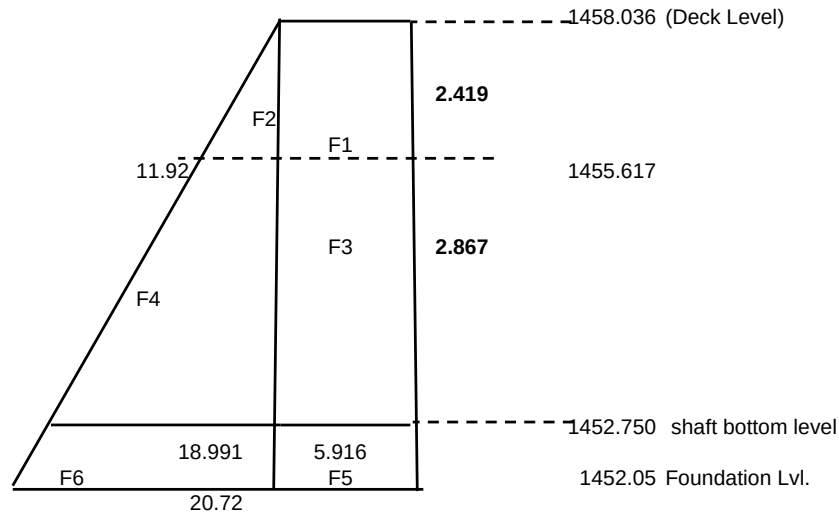
$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	1751.433	2543.444	788.891	1011.664
Due to Minimum Fluid Pressure	1477.003	2144.915	838.254	1074.966
Governing of Two	1751.433	2543.444	838.254	1074.966
Due to Live Load Surcharge	946.669	1213.996	358.180	405.612
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	11.3 degree	0.196 radians
	Ka'	=	0.246 active component	
	Kp'	=	5.249 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	
Assuming				

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1452.8 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m ²
F1	=	5.916	x	5.286	x	12.500	=	390.868 kN
M1	=	390.87	x	2.64	=	1033.065 kN.m		at Shaft Bottom
F3	=	5.916	x	5.986	x	12.500	=	442.629 kN
M3	=	442.63	x	2.99	=	1324.789 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	2.419	=	11.925 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	2.867	=	7.067 kN/m ²
F2	=	0.5	x	11.92	x	2.419	x	12.50
	=	180.287 kN						
F4	=	$\left(\frac{11.92 + 18.99}{2} \right)$	x	2.87	x	12.50		
	=	553.976 kN						
Total Force	=	734.263 kN						
M2	=	180.29	x	3.88	=	700.050 kN.m		
M4	=	553.98	x	1.32	=	733.620 kN.m		
Total Moment	=	1433.67 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 2.419 = 11.925 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 3.567 = 8.792 \text{ kN/m}^2$$

$$F2 = 0.5 \times 11.92 \times 2.419 \times 12.50 = 180.287 \text{ kN}$$

$$F6 = \left(\frac{11.92 + 20.72}{2} \right) \times 3.57 \times 12.50 = 727.699 \text{ kN}$$

$$\text{Total Force} = 907.986 \text{ kN}$$

$$M2 = 180.29 \times 4.58 = 826.251 \text{ kN.m}$$

$$M6 = 727.70 \times 1.62 = 1181.325 \text{ kN.m}$$

$$\text{Total Moment} = 2007.58 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

$$= 5.249 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2$$

$$\begin{aligned} \text{Force due to passive @ Foundation, F} &= 0.5 \times 0.000 \times 12.50 \\ &= 0.000 \text{ kN} \end{aligned}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	1433.670	2007.576	734.263	907.986
Due to Minimum Fluid Pressure	1477.003	2144.915	838.254	1074.966
Governing of Two	1477.003	2144.915	838.254	1074.966
Due to Live Load Surcharge	1033.065	1324.789	390.868	442.629
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

$$\begin{aligned} \alpha h &= 0.094 \\ \alpha v &= 0.105 \\ \phi &= 35.00 & 0.611 \\ \delta &= 22.50 & 0.393 \\ \alpha &= 0.00 & 0.000 \\ \beta &= 0.00 & 0.000 \end{aligned}$$

$$\begin{aligned} \alpha h &= \text{HORIZONTAL SEISMIC COEFFICIENT} \\ \alpha v &= \text{VERTICAL SEISMIC COEFFICIENT} \\ \phi &= \text{ANGLE OF INTERNAL FRICTION OF SOIL} \\ \delta &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL} \\ \alpha &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,} \\ \beta &= \text{SLOPE OF EARTH FILL} \end{aligned}$$

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \frac{0.085}{0.105}$$

$$C_a = \frac{1}{0.302} \frac{2}{0.256}$$

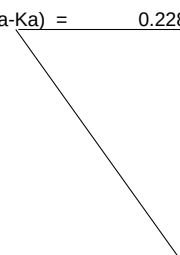
$$\begin{aligned} C_a &= 0.302 \\ K_a &= 0.226 \\ \text{Dynamic Increment} &= 0.302 \quad -0.226 \quad 0.076 \end{aligned}$$

3 Earth Pressure : DRY CASE (Seismic case)

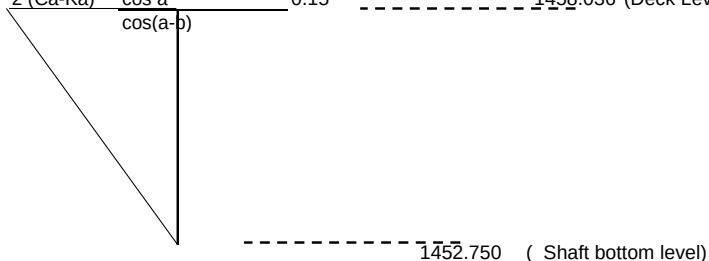
$$\begin{aligned} \text{Equivalent Live Load Surcharge height} &= 1.2 \text{ m} \\ \text{Assuming } \gamma_{\text{dry}} &= 20 \text{ kN/m}^3 \\ \gamma_{\text{water}} &= 10.00 \text{ kN/m}^3 \end{aligned}$$

Active Earth Pressure

$$3 (C_a - K_a) = 0.228$$

**Surcharge Earth Pressure**

$$2 (C_a - K_a) \frac{\cos a}{\cos(a-b)} = 0.15 \quad \text{--- 1458.036 (Deck Level)}$$

**Earth Pressure Diagram for Dynamic Increment****Due to Dynamic Live Load Surcharge**

$$= 0.152 \times 20.00 \times 1.2 = 3.650 \text{ kN/m}^2$$

at Shaft Bottom Level

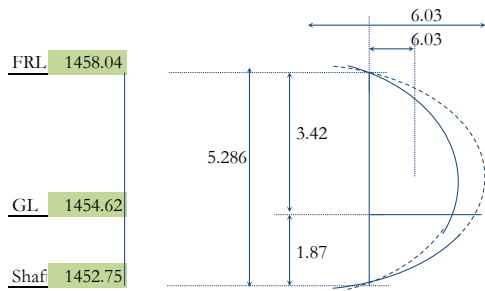
$$\begin{aligned} E1 &= 0.50 \times 3.650 \times 5.286 = 12.500 \\ &= 120.573 \text{ kN} \end{aligned}$$

$$M1 = 120.573 \times 3.542 = 427.023 \text{ kN.m}$$

at Foundation Bottom Level

$$\begin{aligned} E2 &= 0.50 \times 3.650 \times 5.286 = 12.500 \\ &= 120.573 \text{ kN} \end{aligned}$$

$$M2 = 120.573 \times 4.242 = 511.424 \text{ kN.m}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p _{mid_height}	=	6.03 t/m ²
h	=	5.286 m
y	=	0.776 m
L	=	12.500 m

Parabola below Water Level

p _{mid_height}	=	6.03 t/m ²
h	=	5.286 m
y	=	-0.776 m
L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	189.6	3.2
Parabola below Water Level	76.0	1.2
Total Dynamic Earth Pressure	265.6	2.6

$$E3 = 265.562 \text{ kN}$$

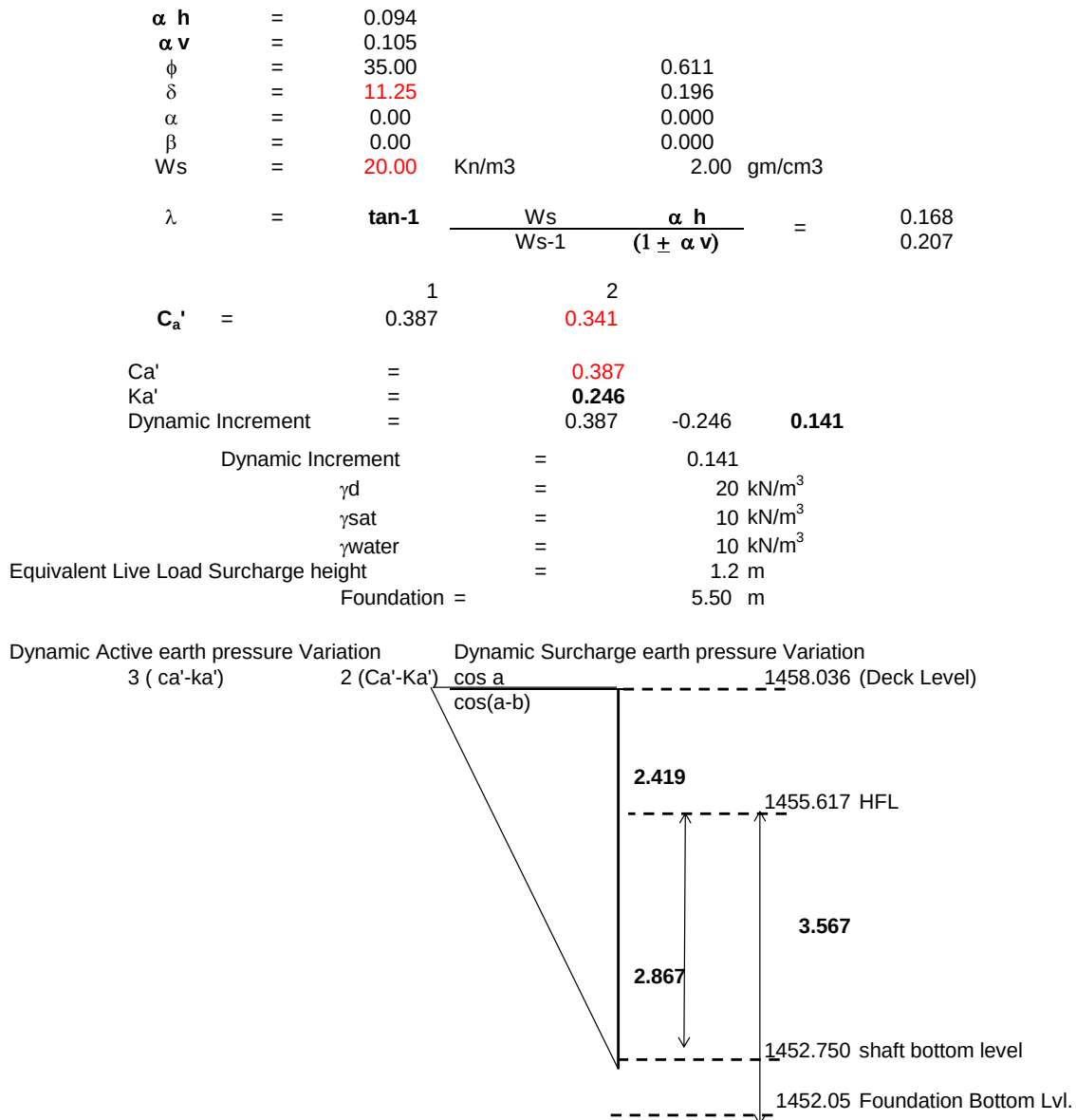
$$E4 = 265.562 \text{ kN}$$

$$M3 = 265.56 \times 2.64 = 701.880 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 265.56 \times 3.34 = 887.773 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	1751.433	2543.444	788.891	1011.664
Due to active Earth Pressure (dynamic Increment)	701.880	887.773	265.562	265.562
Total Earth Pressure	2453.313	3431.217	1054.453	1277.225
Due to Minimum Fluid Pressure	1477.003	2144.915	838.254	1074.966
Governing of Two	1751.433	3431.217	1054.453	1277.225
Due to Live Load Surcharge (Static)	946.669	1213.996	358.180	405.612
Due to Live Load Surcharge(Dynamic)	427.023	511.424	120.573	120.573
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case**Earth Pressure Diagram**

h'	=	2.867
h	=	5.286

Dynamic active earth pressure

3 (ca'-ka')	=	0.228 at deck level
3 (ca'-ka')	=	0.422 at deck level
3 (ca'-ka')h'/h	=	0.229 at HFL level

Dynamic Surcharge earth pressure

2 (Ca'-Ka') $\frac{\cos a}{\cos(a-b)}$	=	0.282 at deck level	Pressure	=	6.76 kN/m ²
	=	0.153 at HFL level	Pressure	=	3.67 kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	3.650 kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	3.67 kN/m ²

at Shaft Bottom Level

F1	=	3.657	x	2.419	x	12.500	=	110.591 kN
M1	=	110.59	x	4.08	=	450.825 kN.m		
F2	=	0.50	x	3.665	x	2.867	x	12.500
	=	65.677 kN						

$$M2 = 65.68 \times 1.92 = 126.158 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 176.268 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 576.983 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

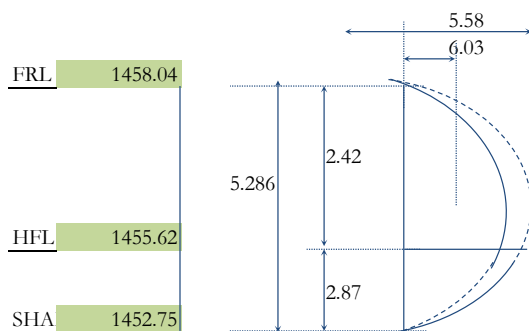
$$F3 = 3.657 \times 2.419 \times 12.500 = 110.591 \text{ kN}$$

$$M3 = 110.59 \times 4.78 = 528.238 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 3.665 \times 2.867 \times 12.500 \\ &= 65.677 \text{ kN} \end{aligned}$$

$$M4 = 65.68 \times 2.62 = 172.132 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 176.268 \text{ kN} \\ \text{Total Moment at Founding level} &= 700.370 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 6.03 \text{ t/m}^2 \\ h &= 5.286 \text{ m} \\ y &= -0.224 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 5.58 \text{ t/m}^2 \\ h &= 5.286 \text{ m} \\ y &= 0.224 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	115.9	3.8
Parabola below Water Level	138.5	1.8
Total Dynamic Earth Pressure	254.5	2.7

Total Force (F1 + F2)	=	254.464 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	254.464 kN	at Foundation Bottom Level

$$M1 = 254.46 \times 2.68 = 682.171 \text{ kN.m}$$

$$\text{Total Mome} = 682.171 \text{ kN.m at Shaft Bottom}$$

$$M1 = 254.46 \times 3.38 = 860.296 \text{ kN.m}$$

$$\text{Total Mome} = 860.296 \text{ kN.m at Foundation Bottom Level}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Bottom	At Shaft Bottom Lvl	At Foundation Bottom Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure(Static)	1433.670	2007.576	734.263	907.986
Due to active Earth Pressure (Dynamic Increment)	682.171	860.296	254.464	254.464
Total Earth Pressure	2115.840	2867.872	988.728	1162.450
Due to Minimum Fluid Pressure	1477.003	2144.915	838.254	1074.966
Governing of Two	2115.840	2867.872	988.728	1162.450
Due to Live Load Surcharge(Static)	1033.065	1324.789	390.868	442.629
Due to Live Load Surcharge (Dynamic Increment)	576.983	700.370	176.268	176.268
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - SPHERICAL BEARING

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1689.39	1.35	1.35	2280.68	2280.68	
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00	
WC	=	241.62	1.75	1.75	422.83	422.83	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	1000.02	1.5	0.20	1500.03	200.00	(1-70RW + 2-CLASS A)
CWLLmax-Reaction case	=	2020.42	1	0.20	2020.42	404.08	SV Loading
CWLLmin	=	349.98	1.5	0.20	524.97	70.00	(1-70RW + 2-CLASS A)
CWLLmin	=	1399.58	1	0.20	1399.58	279.92	SV Loading
CWLLmax-Transv. Moment Case		696.67	1.5	0.20	1045.01	139.33	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 1108 = 255.4 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2919.51	0	145.975	145.975	-145.975	145.975	
DL+SIDL+LL-Max Reaction case	3444.48	383.1	172.224	363.774	210.876	363.774	(1-70RW + 2-CLASS A)
	4844.05	0	242.203	242.203	-242.203	242.203	SV Loading
DL+SIDL+LL-Min Reaction case	4419.54	383.1	220.977	412.527	162.123	412.527	(1-70RW + 2-CLASS A)
	4939.93	0	246.997	246.997	-246.997	246.997	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3964.52	383.1	198.226	389.776	184.874	389.776	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2919.51	589.49	145.975	440.719	443.511	589.487	
DL+SIDL+LL-Max Reaction case		2989.50	640.57	149.475	469.759	491.092	640.567	Dry Case
DL+SIDL+LL-Min Reaction case		3119.51	640.57	155.976	476.259	484.591	640.567	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3058.84	640.57	152.942	473.226	487.625	640.567	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2919.51	176.846	145.975	234.398	30.871	234.398	
DL+SIDL+LL-Max Reaction case		2989.50	227.926	149.475	263.438	78.451	263.438	Dry Case
DL+SIDL+LL-Min Reaction case		3119.51	227.926	155.976	269.939	71.950	269.939	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3058.84	227.926	152.942	266.905	74.984	266.905

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.35	1.35	2280.68	2280.68
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.62	1.75	1.75	422.83	422.83
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	1000.02	1.5	0.75	1500.03	750.02
CWLLmax-Transv. Moment Case	=	696.67	1.5	0.75	1045.01	522.50
CWLLmin	=	349.98	1.5	0.75	524.97	262.48

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2919.51	0.000	145.975	145.975	-145.975	145.975	
DL+SIDL+LL-Max Reaction case	3444.48	383.100	172.224	363.774	210.876	363.774	(1-70RW + 2-CLASS A)
	4844.05	0.000	242.203	242.203	-242.203	242.203	SV Loading
DL+SIDL+LL-Min Reaction case	4419.54	383.100	220.977	412.527	162.123	412.527	(1-70RW + 2-CLASS A)
	4939.93	0.000	246.997	246.997	-246.997	246.997	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3964.52	383.100	198.226	389.776	184.874	389.776	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2919.51	795.81	145.975	543.879	649.832	795.807	
DL+SIDL+LL-Max Reaction case		3181.99	846.89	159.100	582.543	687.788	846.887	Dry Case
DL+SIDL+LL-Min Reaction case		3669.53	846.89	183.476	606.920	663.411	846.887	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3442.01	846.89	172.101	595.544	674.787	846.887	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2919.51	238.742	145.975	265.347	92.767	265.347	
DL+SIDL+LL-Max Reaction case		3181.99	289.822	159.100	304.011	130.723	304.011	Dry Case
DL+SIDL+LL-Min Reaction case		3669.53	289.822	183.476	328.387	106.346	328.387	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3442.01	289.822	172.101	317.012	117.722	317.012	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1	1.00	1689.39	1689.39
SIDL except wc	=	160.00	1	1.00	160.00	160.00
WC	=	241.62	1	1.00	241.62	241.62
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	1000.02	1	0.20	1000.02	200.00
CWLLmax- Transv. Moment Case		696.67	1	0.20	696.67	139.33
CWLLmin	=	349.98	1	0.20	349.98	70.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2091.01	0.000	104.550	104.550	-104.550	104.550	
DL+SIDL+LL-Max Reaction case	2440.99	255.400	122.049	249.749	133.351	249.749	(1-70RW + 2-CLASS A)
	3490.59	0.000	174.529	174.529	-174.529	174.529	SV Loading
DL+SIDL+LL-Min Reaction case	3091.03	255.400	154.552	282.252	100.848	282.252	(1-70RW + 2-CLASS A)
	4111.43	0.000	205.572	205.572	-205.572	205.572	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2787.68	255.400	139.384	267.084	116.016	267.084	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2091.01	530.538	104.550	369.820	425.988	530.538	
DL+SIDL+LL-Max Reaction case	2161.01	581.618	108.050	398.859	473.568	581.618	Dry Case
DL+SIDL+LL-Min Reaction case	2291.01	581.618	114.551	405.360	467.067	581.618	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2230.34	581.618	111.517	402.326	470.101	581.618	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2091.01	159.161	104.550	184.131	54.611	184.131	
DL+SIDL+LL-Max Reaction case	2161.01	210.241	108.050	213.171	102.191	213.171	Dry Case
DL+SIDL+LL-Min Reaction case	2291.01	210.241	114.551	219.671	95.691	219.671	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2230.34	210.241	111.517	216.638	98.724	216.638	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.1	1.10	1858.33	1858.33
SIDL except wc	=	160.00	1.1	1.10	176.00	176.00
WC	=	241.62	1.35	1.35	326.18	326.18
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	1000.02	1.5	0.00	1500.03	0.00
CWLLmax-Transv. Moment Case		696.67	1.5	0.00	1045.01	0.00
CWLLmin	=	349.98	1.5	0.00	524.97	0.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2360.51	0.000	118.026	118.026	-118.026	118.026	
DL+SIDL+LL-Max Reaction case	2885.48	383.100	144.274	335.824	238.826	335.824	(1-70RW + 2-CLASS A)
	3760.09	0.000	188.005	188.005	-188.005	188.005	SV Loading
DL+SIDL+LL-Min Reaction case	3860.55	383.100	193.027	384.577	190.073	384.577	(1-70RW + 2-CLASS A)
	4380.94	0.000	219.047	219.047	-219.047	219.047	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3405.52	383.100	170.276	361.826	212.824	361.826	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2360.51	795.81	118.026	515.929	677.781	795.807	
DL+SIDL+LL-Max Reaction case		2360.51	795.81	118.026	515.929	677.781	795.807	Dry Case
DL+SIDL+LL-Min Reaction case		2360.51	795.81	118.026	515.929	677.781	795.807	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2360.51	795.81	118.026	515.929	677.781	795.807	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	$F_h/2 + \mu R$	$F_h - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2091.01	2360.51	238.742	118.026	237.397	120.716	238.742	
DL+SIDL+LL-Max Reaction case		2360.51	238.742	118.026	237.397	120.716	238.742	Dry Case
DL+SIDL+LL-Min Reaction case		2360.51	238.742	118.026	237.397	120.716	238.742	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2360.51	238.742	118.026	237.397	120.716	238.742	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1689.39	1	1	1	1689.39	1689.39	1689.39
SIDL except wc	=	160.00	1	1	1	160.00	160.00	160.00
WC	=	241.62	1.20	1.20	1.20	289.94	289.94	289.94
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	1000.02	1	0.75	0	1000.02	750.02	0.00
CWLLmax-Transv. Moment Case	=	696.67	1	0.75	0	696.67	522.50	0.00
CWLLmin	=	349.98	1	0.75	0	349.98	262.48	0.00

Braking Force = 255.4 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.33	0.000	106.967	106.967	-106.967	106.967	
DL+SIDL+LL-Max Reaction case	2489.31	255.400	124.466	252.166	130.934	252.166	(1-70RW + 2-CLASS A)
	3538.91	0.000	176.945	176.945	-176.945	176.945	SV Loading
DL+SIDL+LL-Min Reaction case	3139.35	255.400	156.968	284.668	98.432	284.668	(1-70RW + 2-CLASS A)
	4159.76	0.000	207.988	207.988	-207.988	207.988	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2836.00	255.400	141.800	269.500	113.600	269.500	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.33	0.000	106.967	106.967	-106.967	106.967	
DL+SIDL+LL-Max Reaction case	2401.82	191.550	120.091	215.866	71.459	215.866	Dry Case
DL+SIDL+LL-Min Reaction case	2889.35	191.550	144.467	240.242	47.083	240.242	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2661.84	191.550	133.092	228.867	58.458	228.867	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2139.33	0.000	106.967	106.967	-106.967	106.967	
DL+SIDL+LL-Max Reaction case	2139.33	0.000	106.967	106.967	-106.967	106.967	Dry Case
DL+SIDL+LL-Min Reaction case	2139.33	0.000	106.967	106.967	-106.967	106.967	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2139.33	0.000	106.967	106.967	-106.967	106.967	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	0.00	kmph	0.00	kmph
Live Load	W	=	1000.02	kN	1000.02	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

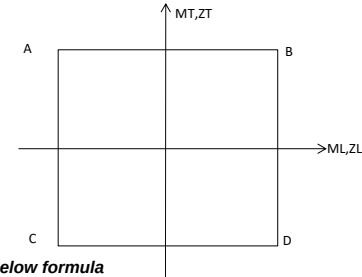
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1452.050 m

Properties of Footing Base:

A	=	68.750	m ²
ZL	=	63.021	m ³
ZT	=	143.229	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1689.395	0.395	667.311	0.000	0.000
SIDL except Wearing Course	1			160.000	0.395	63.200	0.000	0.000
Wearing Course	1			241.615	0.395	95.438	0.000	0.000
Bearing Pedestal	1	25	1.830	45.760	0.395	18.075	-1.101	-50.400
				2136.770		844.024		-50.400
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	9.914	247.852	-0.930	-230.502	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.000	0.000	-0.930	0.000	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	-1.230	-34.594	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	-1.180	-16.594	0.000	0.000
Cap - (uniform portion)	1	25	10.425	260.625	0.310	80.794	0.000	0.000
Cap - (corbel portion)	1	25	6.809	170.229	0.310	52.771	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	-0.050	0.000	6.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	-0.050	0.000	6.000	0.000
RCC Railing or Crash Barrier	1			28.000	-0.930	-26.040	0.000	0.000
Approach Slab	1	25	6.563	164.063	-1.230	-201.797	0.000	0.000
				912.956		-375.962		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	7.406	185.153	-1.400	-259.214	6.000	1110.915
Abutment Shaft	1	25	23.013	575.313	0.450	258.891	0.000	0.000
Back filling over heel slab	1	20	178.360	3567.206	-1.417	-5054.839	0.000	0.000
Front Filling over toe slab	1	20	46.508	930.150	1.821	1693.778	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	16.875	421.875	-1.220	-514.688	0.000	0.000
Toe slab	1	25	11.250	281.250	1.730	486.563	0.000	0.000
portion between heel & toe	1	25	8.750	218.750	0.450	98.438	0.000	0.000
Vertical Components of active earth pressure	1			445.265	-2.750	-1224.480	0.000	0.000
				6624.962		-4515.552		1110.915
Total				9674.687		-4047.489		1060.515

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.900	1520.455	-2.355	-3580.672
0.900	144.000	-2.355	-339.120
1.000	241.615	-2.355	-569.003
0.900	41.184	-2.355	-96.988
	1947.254		-4585.784
0.900	223.066	-3.680	-820.884
0.900	0.000	-3.680	0.000
0.900	25.313	-3.980	-100.744
0.900	12.656	-3.930	-49.739
0.900	234.563	-2.440	-572.333
0.900	153.206	-2.440	-373.823
0.900	0.000	-2.800	0.000
0.900	0.000	-2.800	0.000
0.900	25.200	-3.680	-92.736
0.900	147.656	-3.980	-587.672
	821.660		-2597.931
0.900	166.637	-4.150	-691.545
0.900	517.781	-2.300	-1190.897
0.900	3210.486	-4.167	-13378.190
0.900	837.135	-0.929	-777.722
0.900	0.000	0.000	0.000
0.900	379.688	-3.970	-1507.359
0.900	253.125	-1.020	-258.188
0.900	196.875	-2.300	-452.813
0.900	400.739	-5.500	-2204.064
	5962.466		-20460.777
	8731.380		-27644.491

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	104.550	1456.151	428.762
due to Earth pressure	1	1074.966		2543.444

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		118.026	1456.151	484.024
due to Earth pressure	1.5	1612.449		3815.166
		1730.475		4299.189

Summary of Forces For SBC

P	9674.687	kN
ML	-1075.284	kNm
MT	1060.515	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		9674.687		-4047.489		1060.515
CWLL-Max. Reaction case	1	1000.022	0.395	395.009	0.950	950.021
Vertical Components of LL Surcharge	1	168.010	-2.750	-462.027	0.000	0.000
Total		10842.719		-4114.508		2010.536

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	249.749	1456.151	1024.222
due to Earth pressure	1	1074.966		2543.444
due to Live load surcharge	1	405.612		1213.996

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		335.824	1456.151	1377.215
due to Earth pressure	1.5	1612.449		3815.166
due to Live load surcharge	1.2	486.734		1456.796
		2435.007		6649.176

Summary of Forces For SBC

P	10842.719	kN
ML	667.155	kNm
MT	2010.536	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
104.55	428.76	0.00	0.00
1074.97	2543.44	0.00	0.00
1179.516	2972.205	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
118.03	484.02	0.00	0.00
1612.45	3815.17	0.00	0.00
1730.475	4299.189	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	8731.379992		-27644.49
0.000		-2.355	0.00
0.900		-5.500	-831.65
	8731.380		-28476.14

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
249.75	1024.22	0.00	0.00
1074.97	2543.44	0.00	0.00
405.612	1213.996	0.000	0.000
1730.33	4781.66	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
335.82	1377.21	0.00	0.00
1612.45	3815.17	0.00	0.00
486.73	1456.80	0.00	0.00
2435.007	6649.176	0.000	0.000

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				2136.770		844.024		-50.400
Substructure & Foundation -Portion 1				912.956		-375.962		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	7.406	185.153	-1.400	-259.214	6.000	1110.915
Shaft above HFL	1	25	0.000	0.000	0.450	0.000	0.000	0.000
Shaft below HFL	1	15	23.013	345.188	0.450	155.334	0.000	0.000
Back filling above HFL over heel slab	1	20	81.641	1632.825	-1.400	-2285.955	0.000	0.000
Back filling below HFL over heel slab	1	10	103.511	1035.113	-1.429	-1479.533	0.000	0.000
Front Filling over toe slab	1	10	46.508	465.075	1.371	637.605	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	16.875	253.125	-1.220	-308.813	0.000	0.000
Toe slab	1	15	11.250	168.750	1.730	291.938	0.000	0.000
Portion between Heel & Toe	1	15	8.750	131.250	0.450	59.063	0.000	0.000
Vertical Components of active earth pressure	1			445.265	-2.750	-1224.480	0.000	0.000
				4661.743		-4414.054		1110.915
Total				7711.468		-3945.992		1060.515

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	1947.254		-4585.784
	821.660		-2597.931
0.900	166.637	-4.150	-691.545
0.900	0.000	-2.300	0.000
0.900	310.669	-2.300	-714.538
0.900	1469.543	-4.150	-6098.601
0.900	931.601	-4.179	-3893.483
0.900	418.568	-1.379	-577.216
0.900	0.000	0.000	0.000
0.900	227.813	-3.970	-904.416
0.900	151.875	-1.020	-154.913
0.900	118.125	-2.300	-271.688
0.900	400.739	-5.500	-2204.064
	4195.569		-15510.462
	6964.483		-22694.177

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	104.550	1456.151	428.762
due to Earth pressure	1	1074.966		2144.915

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
104.55	428.76	0.00	0.00
1074.97	2144.92	0.00	0.00
1179.52	2573.68	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		118.026	1456.151	484.024
due to Earth pressure	1.5	1612.449		3217.373
		1730.475		3701.396

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
118.03	484.02	0.00	0.00
1612.45	3217.37	0.00	0.00
1730.47	3701.40	0.00	0.00

Summary of Forces For SBC

P	7711.468	kN
ML	-1372.315	kNm
MT	1060.515	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		7711.468		-3945.992		1060.515
CWLL-Min. Reaction case	1	349.978	0.395	138.241	0.950	332.479
Vertical Components of LL Surcharge	1	183.343	-2.750	-504.193	0.000	0.000
Total		8244.789		-4311.944		1392.994

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	6964.483		-22694.177
0.000	0.000	-2.355	0
0.900	165.009	-5.500	-907.548
	7129.492		-23601.725

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	282.252	1456.151	1157.514
due to Earth pressure	1	1074.966		2144.915
due to Live load surcharge	1	442.629		1324.789

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
282.25	1157.51	0.00	0.00
1074.97	2144.92	0.00	0.00
442.629	1324.789	0.000	0.000
1799.85	4627.22	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		384.577	1456.151	1577.152
due to Earth pressure	1.5	1612.449		3217.373
due to live load surcharge	1.2	531.155		1589.747
		2528.181		6384.272

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
384.58	1577.15	0.00	0.00
1612.45	3217.37	0.00	0.00
531.155	1589.747	0.000	0.000
2528.18	6384.27	0.00	0.00

Summary of Forces For SBC

P	8244.789	kN
ML	315.275	kNm
MT	1392.994	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1

ah= 0.127 In Longitudinal direction
ah= 0.157 In Transverse direction
av= 0.105 In Vertical direction

Weight of shaft below Ground level

= 583.437 KN

Weight of back fill below Ground level

= 1260.225 KN

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1689.395		79.810	53.206	0.395	667.311	21.017	1457.364		0.000	0.000	424.134
SIDL except Wearing Course	1			160.000		7.559	5.039	0.395	63.200	1.990	1458.485		0.000	0.000	48.639
Wearing Course	1			241.615		11.414	7.610	0.395	95.438	3.006	1458.036		0.000	0.000	68.326
Bearing Pedestal	1	25	1.830	45.760		2.162	1.441	0.395	18.075	0.569			-1.101	-50.400	
				2136.770		100.944	67.296		844.024	26.582				-50.400	541.099
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	9.914	247.852	31.443	11.709	7.806	-0.930	-230.502	-7.260	1456.714	146.653	0.000	0.000	54.612
Dirt Wall-Tapered portion	1	25	0.000	0.000	0.000	0.000	0.000	-0.930	0.000	0.000	1455.392	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125				-1.230	-34.594						
Bracket - Tapered portion	1	25	0.563	14.063				-1.180	-16.594						
Cap - (uniform portion)	1	25	10.425	260.625	33.063	12.312	8.208	0.310	80.794	2.545	1455.041	98.892	0.000	0.000	36.826
Cap - (corbel portion)	1	25	6.809	170.229	21.596	8.042	5.361	0.310	52.771	1.662	1454.741	58.114	0.000	0.000	21.641
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	-0.050	0.000	0.000	1458.036	0.000	6.000	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	-0.050	0.000	0.000	1458.036	0.000	6.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				-0.930	-26.040				0.000	0.000	
Approach Slab	1	25	6.563	164.063				-1.230	-201.797				0.000	0.000	

				912.956	86.102	32.063	21.375		-375.962	-3.053		303.660		0.000	113.078
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	7.406	185.153	23.489	8.747	5.831	-1.400	-259.214	-8.164	1455.394	78.538	6.000	1110.915	29.246
Abutment Shaft	1	25	23.013	575.313	-1.031	-0.384	-0.256	0.450	258.891	-0.115	1454.604	-2.633	0.000	0.000	-0.980
Back filling over heel slab	1	20	178.360	3567.206	0.000	0.000	0.000	-1.417	-5054.839	0.000	1455.394	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	46.508	930.150				1.821	1693.778				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	16.875	421.875				-1.220	-514.688				0.000	0.000	
Toe slab	1	25	11.250	281.250				1.730	486.563				0.000	0.000	
portion between heel & toe	1	25	8.750	218.750				0.450	98.438				0.000	0.000	
Vertical component of active earth pressure	1			419.045				-2.750	-1152.373				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			109.999				-2.750	-302.498				0.000	0.000	
				6708.740	22.458	8.363	5.575		-4745.943	-8.279		75.905		1110.915	28.266
Total =				9758.466	108.560	141.370	94.247		-4277.880	15.250		379.565		1060.515	682.444
								-94.247		-15.250					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Pxel2	MLs due to Fv
Superstructure								
Dead Load	0.9			1520.455	47.886	-2.355	-3580.672	-112.771
SIDL except Wearing Course	0.9			144.000	4.535	-2.355	-339.120	-10.680
Wearing Course	1.00			241.615	7.610	-2.355	-569.003	-17.920
Bearing Pedestal	0.9	25	1.8304	41.184	-2.355	-2.355	-96.988	
				1947.254	60.030		-4585.784	-141.372
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	9.914	223.066	7.025	-3.680	-820.884	-25.853
Dirt Wall-Tapered portion	0.9	25	0.000	0.000	0.000	-3.680	0.000	0.000
Bracket - Uniform portion	0.9	25	1.125	25.313				
Bracket - Tapered portion	0.9	25	0.563	12.656				
Cap - (uniform portion)	0.9	25	10.425	234.563	7.387	-2.440	-572.333	-18.025
Cap - (corbel portion)	0.9	25	6.809	153.206	4.825	-2.440	-373.823	-11.773
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-2.800	0.000	0.000
Cantilever Return Wall-Traingle portion	0.9	25	0.000	0.000	0.000	-2.800	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-3.680	-92.736	
Approach Slab	0.9	25	6.563	147.656		-3.980	-587.672	
				821.660	19.238		-2447.448	-55.652
Substructure & Foundation -Portion 2								
Abutment Shaft	0.9	25	23.013	517.781	-2.068	-2.300	-1190.897	4.756
Solid Return wall	0.9	25	7.406	166.637	5.248	-4.150	-691.545	-21.780
Back filling over heel slab	0.9	20	178.360	3210.486	0.000	-4.167	-13378.2	0.000
Front Filling over toe	0.9	20	46.508	837.135		-0.929	-777.722	
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000	
Heel slab	0.9	25	16.875	379.688		-3.970	-1507.359	
Toe slab	0.9	25	11.250	253.125		-1.020	-258.188	
portion between heel & toe	0.9	25	8.750	196.875		-2.300	-452.813	
Vertical component of active earth pressure	0.9			400.739		-5.500	-2204.064	
Vertical component of dynamic increment of earth pressure	0.9			98.999		-5.500	-544.496	
				6061.465	3.180		-21005.27	-17.024
Total =				8830.379	82.449		-28038.50	-214.047
						-82.449		214.047

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	31.443	1456.714	146.653
1.0	0.000	1455.392	0.000
1.0	33.063	1455.041	98.892
1.0	21.596	1454.741	58.114
1.0	0.000	1458.036	0.000
1.0	0.000	1458.036	0.000
	86.102		303.660
1.0	-9.255	1454.604	-23.637
1.0	23.489	1455.394	78.538
1.0	0.000	1455.394	0.000
	14.234		54.901
	100.336		358.561

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	530.538	100.944	1456.151	2175.737	541.099
due to Substructure	1	108.560	40.426		379.565	141.344
due to Earth pressure	1	1277.225			3431.217	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		795.807	1456.151	3263.61
due to Substructure	1.5	150.504		537.84
due to Active Earth pressure	1	1011.664		2543.44
due to dynamic Earth pressure	1.5	398.343		1331.66
		2356.317		7676.550

Summary of Forces For SBC

	Downward	Upward	
P	9852.713	9664.219	kN
ML	1723.888	1693.388	kNm
MT	1742.959	1742.959	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
530.54	2175.74	0.00	0.00	0.00	0.00	100.94	541.10
108.56	379.56	0.00	0.00	0.00	0.00	40.43	141.34
1277.23	3431.22						
1916.32	5986.52	0.00	0.00	0.00	0.00	141.37	682.44

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
795.81	3263.61	0.00	0.00
150.50	537.84	0.00	0.00
1011.664	2543.444	0.000	0.000
398.343	1331.659	0.000	0.000
2356.32	7676.55	0.00	0.00

Summary of Restoring Forces

Vertical Load	8747.931	kN
Moment	-28252.552	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				2136.770		100.944	67.296		844.024	26.582				-50.400	541.099
Forces from Substructure				7621.696	108.560	40.426	26.951		-5121.904	-11.332		379.565		1110.915	141.344
CWLL-Max. Reaction case	0.20			200.00		9.449	6.299	0.395	79.002	2.488	1459.236		0.950	190.004	67.897
Vertical component of LL Surcharge	0.20			33.60				-2.750	-92.405				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			9.99				-2.750	-27.469				0.000	0.000	
Total =				10002.061	108.560	150.819	100.546		-4318.753	17.738		379.565		1250.519	750.341
							-100.546			-17.738					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Forces from Superstructure				1947.254	60.030	0.000	-4585.78	-141.372
Forces from Substructure				6883.125	22.418	0.000	-23452.72	-72.676
CWLL-Max. Reaction case	0.00			0.00	0.000	-2.355	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-5.500	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-5.500	0.00	
Total =				8830.379	82.449		-28038.50	-214.047
					-82.449			214.047

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	581.618	110.393	1456.151	2385.216	608.996
due to Substructure	1	108.560	40.426		379.565	141.344
due to Earth pressure	1	1277.225			3431.217	
due to Live load surcharge	0.20	105.237			345.084	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		795.807	1456.151	3263.605
due to Substructure	1.5	150.504		537.841
due to Active Earth pressure	1	1011.664		2543.444
due to dynamic Earth pressure	1.5	398.343		1331.65945
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		2356.317		7676.550

Summary of Forces For SBC

	Downward	Upward	
P	10102.607	9901.515	kN
ML	2240.067	2204.591	kNm
MT	2000.860	2000.860	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2136.770		100.944	67.296		844.024	26.582				-50.400	541.099
Substructure & Foundation -Portion 1				912.956	86.102	32.063	21.375		-375.962	-3.053		303.660		0.000	113.078
Substructure & Foundation -Portion 2															
Solid Return wall	1		7.41	185.15	23.489	8.747	5.831	-1.40	-259.214	-8.164	1455.39	78.538	6.00	1110.915	29.246
Shaft above HFL	1	25	0.000	0.000	0.000	0.000	0.000	0.450	0.000	0.000	1455.104	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	23.013	345.188	-1.031	-0.384	-0.256	0.450	155.334	-0.115	1455.117	-3.161	0.000	0.000	-1.177
Back filling above HFL over heel slab	1	20	81.641	1632.825	0.000	0.000	0.000	-1.400	-2285.955	0.000	1456.827	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	103.511	1035.113	0.000	0.000	0.000	-1.429	-1479.533	0.000	1454.009	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	46.508	465.075				1.371	637.605				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	16.875	253.125				-1.220	-308.813				0.000	0.000	
Toe slab	1	15	11.250	168.750				1.730	291.938				0.000	0.000	
portion between heel & toe	1	15	8.750	131.250				0.450	59.063				0.000	0.000	
Vertical component of active earth pressure	1			376.100				-2.750	-1034.275				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			105.403				-2.750	-289.857				0.000	0.000	
				4697.980	22.458	8.363	5.575		-4513.706	-8.279		75.376		1110.915	28.069
Total =				7747.706	108.560	141.370	94.247		-4045.644	15.250		379.036		1060.515	682.247

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
581.62	2385.22	0.00	0.00	0.00	0.00	110.39	609.00
108.56	379.56	0.00	0.00	0.00	0.00	40.43	141.34
1277.23	3431.22						
105.24	345.08						
2072.64	6541.08	0.00	0.00	0.00	0.00	150.82	750.34

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
795.81	3263.61	0.00	0.00
150.50	537.84	0.00	0.00
1011.664	2543.444		
398.343	1331.659		
0.000	0.000		
0.000	0.000		
2356.32	7676.55	0.00	0.00

Summary of Restoring Forces

Vertical Load	8747.931	kN
Moment	-28252.552	kNm

-94.247

-15.250

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				1947.254	60.030		-4585.784	-141.372
Substructure & Foundation -Portion 1				821.660	19.238		-2447.448	-55.652
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	7.4061	166.637	5.248	-4.15	-691.545	-21.780
Shaft above HFL	0.9	25	0.000	0.000	0.000	-2.300	0.000	0.000
Shaft below HFL	0.9	15	23.013	310.669	-2.068	-2.300	-714.538	4.756
Back filling above HFL over heel slab	0.9	20	81.641	1469.543	0.000	-4.150	-6098.601	0.000
Back filling below HFL over heel slab	0.9	10	103.511	931.601	0.000	-4.179	-3893.483	0.000
Front Filling ovetoe	0.9	10	46.508	418.568		-1.379	-577.216	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	16.875	227.813		-3.970	-904.416	
Toe slab	0.9	15	11.250	151.875		-1.020	-154.913	
portion between heel & toe	0.9	15	8.750	118.125		-2.300	-271.688	
Vertical component of active earth pressure	0.9			338.490		-5.500	-1861.695	
Vertical component of dynamic increment of earth pressure	0.9			94.862		-5.500	-521.743	
				4228.182	3.180		-15689.84	-17.024
Total =				6997.097	82.449		-22723.07	-214.047

-82.449

214.047

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
	86.102		303.660
1.0	23.489	1455.394	78.538
1.0	0.000	1455.104	0.000
1.0	-9.255	1455.117	-28.384
1.0	0.000	1456.827	0.000
1.0	0.000	1454.009	0.000
	14.234		50.153
	100.336		353.813

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	530.538	100.944	1456.151	2175.737	541.099
due to Substructure	1	108.560	40.426		379.036	141.148
due to Earth pressure	1	1162.450			2867.872	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		795.807	1456.151	3263.605
due to Substructure	1.5	150.504		530.719
due to Active Earth pressure	1	907.986		2007.576
due to dynamic Earth pressure	1.5	381.697		1290.444
		2235.993		7092.345

Summary of Forces For SBC

	Downward	Upward	
P	7841.952	7653.459	kN
ML	1392.251	1361.751	kNm
MT	1742.762	1742.762	kNm

Forces along Long. Axis

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
530.54	2175.74	0.00	0.00	0.00	0.00	100.94	541.10
108.56	379.04	0.00	0.00	0.00	0.00	40.43	141.15
1162.45	2867.87						
1801.55	5422.64	0.00	0.00	0.00	0.00	141.37	682.25

Forces along Long. Axis

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
795.81	3263.61	0.00	0.00
150.50	530.72	0.00	0.00
907.986	2007.576		
381.697	1290.444		
2235.99	7092.34	0.00	0.00

Summary of Restoring Forces

Vertical Load	6914.648	kN
Moment	-22937.115	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha h \times P$ (kN)	FT = $0.3 \times \alpha h \times P$ (kN)	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = $P \times eL1$	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = $P \times eT$	MTs due to FT
Forces from Superstructure				2136.770		100.944	67.296		844.024	26.582				-50.400	541.099
Forces from Substructure				5610.936	108.560	40.426	26.951		-4889.668	-11.332		379.036		1110.915	141.148
CWLL-Max. Reaction case	0.20			70.00		3.307	2.204	0.395	27.648	0.871	1459.236		0.950	66.496	23.762
Vertical component of LL Surcharge	0.20			36.67				-2.750	-100.839				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			14.60				-2.750	-40.157				0.000	0.000	
Total =				7868.972	108.560	144.677	96.451		-4158.991	16.121		379.036		1127.011	706.009
								-96.451		-16.121					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = $P \times eL2$	MLs due to Fv
Forces from Superstructure				1947.254	60.030		-4585.784	-141.372
Forces from Substructure				5049.842	22.418		-18137.28	-72.676
CWLL-Max. Reaction case	0.00			0.00	0.00		-2.36	0.00
Vertical component of LL Surcharge	0.00			0.00		-2.355	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-5.500	0.000	0.000
Total =				6997.097	82.449		-22725.42	-214.047
					-82.449			214.047

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	581.618	104.251	1456.151	2385.216	564.861
due to Substructure	1	108.560	40.426		379.036	141.148
due to Earth pressure	1	1162.450			2867.872	
due to Live load surcharge	0.20	123.779			405.032	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
581.62	2385.22	0.00	0.00	0.00	0.00	104.25	564.86
108.56	379.04	0.00	0.00	0.00	0.00	40.43	141.15
1162.45	2867.87						
123.78	405.03						
1976.41	6037.16	0.00	0.00	0.00	0.00	144.68	706.01

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		795.807	1456.151	3263.605
due to Substructure	1.5	150.504		530.719361
due to Active Earth pressure	1	907.986		2007.576
due to dynamic Earth pressure	1.5	381.697		1290.44361
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		2235.993		7092.345

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
795.81	3263.61	0.00	0.00
150.50	530.72	0.00	0.00
907.986	2007.576		
381.697	1290.444		
0.000	0.000		
0.000	0.000		
2235.99	7092.34	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	7965.424	7772.521	kN
ML	1894.286	1862.044	kNm

Summary of Restoring Forces

Vertical Load	6914.648	kN
Moment	-22939.470	kNm

MT	1833.020	1833.020	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x α_h x P (kN)	FT = α_h x P (kN)	Fv = 0.3 x α_v x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				2136.770		336.481	67.296		844.024	26.582				-50.400	1803.664
Substructure & Foundation -Portion 1				912.956	25.831	106.877	21.375		-375.962	-3.053		91.098		0.000	376.928
Substructure & Foundation -Portion 2				6708.740	6.737	27.877	5.575		-4745.943	-8.279		22.772		1110.915	94.220
Total =				9758.466	32.568	471.234	94.247		-4277.880	15.250		113.869		1060.515	2274.812
										-94.247					
										-15.250					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x α_v x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				1947.254	60.030		-4585.78	-141.372
Substructure & Foundation -Portion 1				821.660	19.238		-2447.45	-55.652
Substructure & Foundation -Portion 2				6061.465	3.180		-21005.27	-17.024
Total =				8830.379	82.449		-28038.50	-214.047
								-82.449
								214.047

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x α_h x P (kN)	C.g. of Force (m)	MLs due to FL
	86.102		303.660
	14.234		54.901
	100.336		358.561

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	184.131	336.481	1456.151	755.122	1803.664
due to Substructure	1	32.568	134.754		113.869	471.148
due to Earth pressure	1	1277.225			3431.217	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
184.13	755.12	0.00	0.00	0.00	0.00	336.48	1803.66
32.57	113.87	0.00	0.00	0.00	0.00	134.75	471.15
1277.23	3431.22						
1493.92	4300.21	0.00	0.00	0.00	0.00	471.23	2274.81

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		238.742	1456.151	979.082
due to Substructure	1.5	150.504		537.840882
due to Active Earth pressure	1	1011.664		2543.44375
due to dynamic Earth pressure	1.5	119.503		399.497835
		1520.412		4459.864

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
238.74	979.08	0.00	0.00
150.50	537.84	0.00	0.00
1011.664	2543.444		
119.503	399.498		
1520.41	4459.86	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	9852.713	9664.219	kN
ML	37.578	7.078	kNm
MT	3335.327	3335.327	kNm

Summary of Restoring Forces

Vertical Load	8747.931	kN
Moment	-28252.552	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Total =	10002.061	100.546	-4318.75	17.738	1250.519
		-100.546		-17.738	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	213.171	367.976	1456.151	874.214	2029.988
due to Substructure	1	32.568	134.754		113.869	471.148
due to Earth pressure	1	1277.225			3431.217	
due to Live load surcharge	0.2	105.237			345.084	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		238.74	1456.151	979.082
due to Substructure	1.5	45.15		161.352
due to Active Earth pressure	1	1011.66		2543.444
due to dynamic Earth pressure	1.5	119.50		399.498
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		1415.06		4083.375

Summary of Forces For SBC

	Downward	Upward	
P	10102.607	9901.515	kN
ML	463.370	427.894	kNm
MT	3751.655	3751.655	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L1}$	MLs due to F_v	MT = $P \times e_T$
Superstructure	2136.770	67.296	844.024	26.582	-50.400
Substructure & Foundation -Portion 1	912.956	21.375	-375.962	-3.053	0.000
Substructure & Foundation -Portion 2	4697.980	5.575	-4513.706	-8.279	1110.915
Total =	7747.706	94.247	-4045.644	15.250	1060.515
		-94.247		-15.250	

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	ML = $P \times e_{L2}$	MLs due to F_v
Total =	8830.379	82.449	-28038.50	-214.047
		-82.449		214.047

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
213.17	874.21	0.00	0.00	0.00	0.00	367.98	2029.99
32.57	113.87	0.00	0.00	0.00	0.00	134.75	471.15
1277.23	3431.22						
105.24	345.08						
1628.20	4764.38	0.00	0.00	0.00	0.00	502.73	2501.14

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
238.74	979.08	0.00	0.00
45.15	161.35	0.00	0.00
1011.664	2543.444		
119.503	399.498		
0.000	0.000		
0.000	0.000		
1415.06	4083.38	0.00	0.00

Summary of Restoring Forces

Vertical Load	8747.931	kN
Moment	-28252.552	kNm

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \text{av} \times P$ (kN)	Long. Ecc. (e_{L2}) @ Toe (m)	ML = $P \times e_{L2}$	MLs due to F_v
Superstructure	1947.254	60.030		-4585.784	-141.372
Substructure & Foundation	821.660	19.238		-2447.448	-55.652
Substructure & Foundation	4228.182	3.180		-15689.836	-17.024
Total =	6997.097	82.449		-22723.068	-214.047
		-82.449			214.047

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	184.13	336.48	1456.15	755.12	1803.66
due to Substructure	1	32.57	134.75		113.71	470.49
due to Earth pressure	1	1162.45			2867.87	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		238.742	1456.151	979.08
due to Substructure	1.5	45.151		159.22
due to Active Earth pressure	1	907.986		2007.58
due to dynamic Earth pressure	1.5	114.509		387.13
		1306.388		3533.01

Summary of Forces For SBC

	Downward	Upward	
P	7841.952	7653.459	kN
ML	-293.689	-324.189	kNm
MT	3334.671	3334.671	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	$ML = P \times eL1$	MLs due to F_v	$MT = P \times eT$
Total =	7868.972	96.451	-4158.99	16.121	1127.011
		-96.451		-16.121	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	219.671	347.50291	1456.151	900.873	1882.8707
due to Substructure	1	32.568	134.75369		113.711	470.491785
due to Earth pressure	1	1162.450			2867.872	
due to Live load surcharge	0.2	123.779			405.032	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		238.74	1456.151	979.08
due to Substructure	1.5	45.15		159.22
due to Active Earth pressure	1	907.99		2007.58
due to dynamic Earth pressure	1.5	114.51		387.13
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		1306.39		3533.01

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
184.13	755.12	0.00	0.00	0.00	0.00	336.48	1803.66
32.57	113.71	0.00	0.00	0.00	0.00	134.75	470.49
1162.45	2867.87						
1379.15	3736.70	0.00	0.00	0.00	0.00	471.23	2274.16

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
238.74	979.08	0.00	0.00
45.15	159.22	0.00	0.00
907.986	2007.576		
114.509	387.133		
1306.39	3533.01	0.00	0.00

Summary of Restoring Forces

Vertical Load	6914.648	kN
Moment	-22937.115	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	$ML = P \times eL2$	MLs due to F_v
Total =	6997.097	82.449		-22725.42	-214.047
		-82.449			214.047

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
219.67	900.87	0.00	0.00	0.00	0.00	347.50	1882.87
32.57	113.71	0.00	0.00	0.00	0.00	134.75	470.49
1162.45	2867.87						
123.78	405.03						
1538.47	4287.49	0.00	0.00	0.00	0.00	482.26	2353.36

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
238.74	979.08	0.00	0.00
45.15	159.22	0.00	0.00
907.986	2007.576		
114.509	387.133		
0.000	0.000		
0.000	0.000		
1306.39	3533.01	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	7965.424	7772.521	kN
ML	144.617	112.375	kNm
MT	3480.373	3480.373	kNm

Summary of Restoring Forces

Vertical Load	6914.648	kN
Moment	-22939.470	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1459.236 -	1452.050)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1459.236 -	1452.050)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	9674.687	-1075.284	1060.515	165.189	131.065	150.381	116.256	165.189	116.256	1730.475	6111.966	3.53	4299.189	27644.49	6.43
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	10842.719	667.155	2010.536	161.163	182.336	133.089	154.261	182.336	133.089	2435.007	6111.966	2.51	6649.176	28476.14	4.28
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	7711.468	-1372.315	1060.515	141.347	97.796	126.538	82.987	141.347	82.987	1730.475	4875.138	2.82	3701.396	22694.18	6.13
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	8244.789	315.275	1392.994	124.647	134.653	105.196	115.201	134.653	105.196	2528.181	4990.644	1.97	6384.272	23601.73	3.70
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	9852.713	1723.888	1742.959	128.127	182.835	103.789	158.497	182.835	103.789	2356.317	6123.552	2.60	7676.550	28252.55	3.68
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	10102.607	2240.067	2000.860	125.372	196.462	97.433	168.522	196.462	97.433	2356.317	6123.552	2.60	7676.550	28252.55	3.68
								SAFE	SAFE			SAFE			SAFE

Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	7841.952	1392.251	1742.762	104.140	148.324	79.805	123.989	148.324	79.805	2235.993	4840.254	2.16	7092.345	22937.12	3.23
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	7965.424	1894.286	1833.020	98.600	158.717	73.005	133.121	158.717	73.005	2235.993	4840.254	2.16	7092.345	22939.47	3.23
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	9852.713	37.578	3335.327	166.003	167.195	119.429	120.622	167.195	119.429	1520.412	6123.552	4.03	4459.864	28252.55	6.33
Case 10 : DL+SIDL+LL- (Maximum Reaction Case)-Trans. Seismic Dry Case	10102.607	463.370	3751.655	165.788	180.493	113.401	128.106	180.493	113.401	1415.060	6123.552	4.33	4083.375	28252.55	6.92
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	7841.952	-293.689	3334.671	142.007	132.687	95.443	86.123	142.007	86.123	1306.388	4840.254	3.71	3533.007	22937.12	6.49
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	7965.424	144.617	3480.373	137.865	142.455	89.267	93.856	142.455	89.267	1306.388	4840.254	3.71	3533.007	22939.47	6.49
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

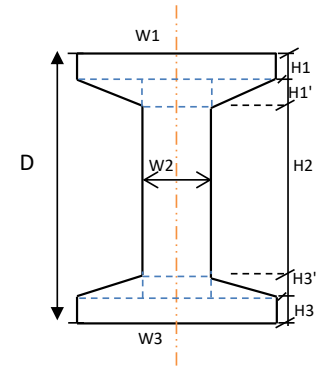
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	325
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	675	100	33750
1' (Rect.)	325	100	32500
2 (Rect.)	325	950	308750
3' (Tri.)	300	150	22500
3' (Rect.)	325	150	48750
3 (Rect.)	625	250	156250
			752500



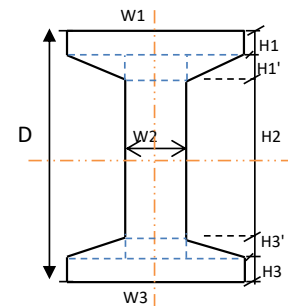
Total Area of Section A = 0.753 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	78
H2	=	1122
H3'	=	0
H3	=	250
W1	=	1000
W2	=	475
W3	=	475

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	525	78	20417
1' (Rect.)	475	78	36944
2 (Rect.)	475	1122	533056
3' (Tri.)	0	0	0
3' (Rect.)	475	0	0
3 (Rect.)	475	250	118750
			859167



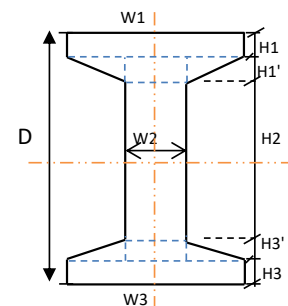
Total Area of Section A = 0.859 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

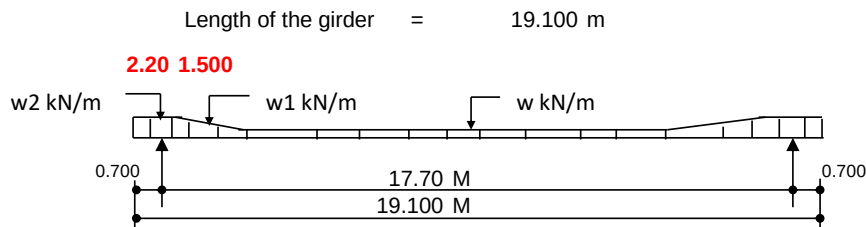
D	=	1600
H1	=	150
H1'	=	56
H2	=	1394
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	56	10417
1' (Rect.)	625	56	34722
2 (Rect.)	625	1394	871528
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066667



Total Area of Section A = 1.067 m²

1.0 Self weight of Inner Girder & Outer Girder



UDL , w	=	18.81 kN/m
Avg. , w ₁	=	21.48 kN/m
UDL , w ₂	=	26.67 kN/m
Supp. Reaction , R	=	200.94 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	

UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder

Supp. Reaction , R	=	170.706 kN	For Outer Girder
	=	157.575 kN	For Inner Girder

weight of deck slab has been Increase by 5 %

Supp. Reaction , R	=	179.242 kN	For Outer Girder
	=	165.454 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

4.0 Weight due to wearing coat

Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	

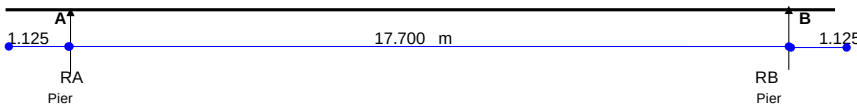
UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder

Supp. Reaction , R	=	57.778 kN	For Outer Girder
	=	63.030 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)

Maximum Live Load Reaction Case**Due to Class A 1-Lane , 2-Lanes, 3-Lanes & 4-Lanes**Considering **114.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **17.70 m**
 Projection beyond c/L of bearing at Abut. end = **1.125 m**
 Projection beyond c/L of bearing at far end = **1.125 m**
 Total length including Overhangs = **19.95 m**

Nos. of Lane for design purpose = **3 LANE**

Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

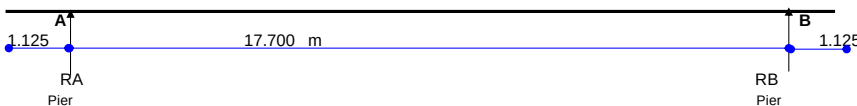
1st Train	Total Load	500.0kN				
					370.379	129.621
114.000	3.200	0.000	-1.125	-128.250		
114.000	1.200	1.200	0.075	8.550		
68.000	4.300	5.500	4.375	297.500		
68.000	3.000	8.500	7.375	501.500		
68.000	3.000	11.500	10.375	705.500		
68.000	3.000	14.500	13.375	909.500		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	370.379	129.621

Due to Class 70R WheeledConsidering **170.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **17.70 m**
 Projection beyond c/L of bearing at Abut. end = **1.125 m**
 Projection beyond c/L of bearing at far end = **1.125 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

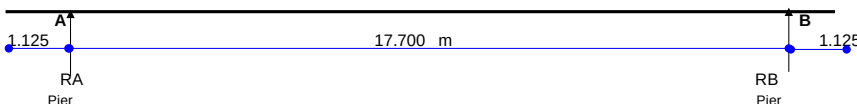
1st Train	Total Load	1000.0kN				
					774.079	225.921
170.000		0.000	-1.125	-191.250		
170.000	1.370	1.370	0.245	41.650		
170.000	3.050	4.420	3.295	560.150		
170.000	1.370	5.790	4.665	793.050		
120.000	2.130	7.920	6.795	815.400		
120.000	1.520	9.440	8.315	997.800		
80.000	3.960	13.400	12.275	982.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	774.079	225.921

Due to Class A 1-Lane/2-Lane (along with Class 70R)Considering **68.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **17.70 m**
 Projection beyond c/L of bearing at Abut. end = **1.125 m**
 Projection beyond c/L of bearing at far end = **1.125 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

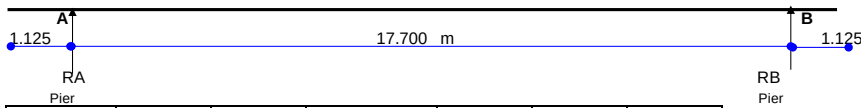
1st Train	Total Load	554.0kN				
					285.331	268.669
68.000		0.000	-1.125	-76.500		
68.000	3.000	3.000	1.875	127.500		
68.000	3.000	6.000	4.875	331.500		
68.000	3.000	9.000	7.875	535.500		
114.000	4.300	13.300	12.175	1387.950		
114.000	1.200	14.500	13.375	1524.750		
27.000	3.200	17.700	16.575	447.525		
27.000	1.100	18.800	17.675	477.225		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	285.331	268.669

Due to Class SV LoadingConsidering **180.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = 17.70 m
 Projection beyond c.L. of bearing at Abut. end = 1.125 m
 Projection beyond c.L. of bearing at far end = 1.125 m
 Total length including Overhangs = 19.95 m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	3420.0kN				
180.000	1.500	0.000	-1.125	-202.500	2020.424	1399.576
180.000	1.500	1.500	0.375	67.500		
180.000	1.500	3.000	1.875	337.500		
180.000	1.500	4.500	3.375	607.500		
180.000	1.500	6.000	4.875	877.500		
180.000	1.500	7.500	6.375	1147.500		
180.000	1.500	9.000	7.875	1417.500		
180.000	1.500	10.500	9.375	1687.500		
180.000	1.500	12.000	10.875	1957.500		
180.000	1.500	13.500	12.375	2227.500		
180.000	1.500	15.000	13.875	2497.500		
180.000	1.500	16.500	15.375	2767.500		
180.000	1.500	18.000	16.875	3037.500		
180.000	1.500	19.500	18.375	3307.500		
0.000	1.500	0.000	-1.125	0.000		
180.000	1.500	1.500	0.375	67.500		
180.000	1.500	3.000	1.875	337.500		
180.000	1.500	4.500	3.375	607.500		
180.000	1.500	6.000	4.875	877.500		
180.000	1.500	7.500	6.375	1147.500		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	2020.424	1399.576

Deck Width = 12.500 m

transverse eccentricity of superstructure about c.g of abutment and foundation = 0 m

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	2020.424 KN	Corr. MT	= 606.127 kNm
Minimum Reaction (RB)	=	1399.576 KN	Corr. MT	= 419.873 kNm

1-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-0.9	0	0	=	4.45 m
Maximum Reaction (RA)	=	370.379	KN	Corr. MT	=	1648.184	kNm		
Minimum Reaction (RB)	=	129.621	KN	Corr. MT	=	576.816	kNm		

2-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-0.85	0	=	2.70 m
Maximum Reaction (RA)	=	740.757 KN	Corr. MT	=	2000.044 kNm				
Minimum Reaction (RB)	=	259.243 KN	Corr. MT	=	699.956 kNm				

3-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	0.95 m
Maximum Reaction (RA)	=	1111.136 KN	Corr. MT	=	1055.579 kNm				
Minimum Reaction (RB)	=	388.864 KN	Corr. MT	=	369.421 kNm				

4-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-0.8 m
Maximum Reaction (RA)	=	1481.514 KN		Corr. MT	=	-1185.211 kNm				
Minimum Reaction (RB)	=	518.486 KN		Corr. MT	=	-414.789 kNm				

1- 70RW

Transverse Ecc.	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m
Maximum Reaction (RA)	=	774.079 KN	Corr. MT	=	2442			
Minimum Reaction (RB)	=	225.921 KN	Corr. MT	=	713			

1-CLASS A + 1-70RW

Transverse Ecc.	for Class A	=	6.25	-0.5	-0.4	-0.9	=	4.45 m			
	for 70RW	=	6.25	-0.5	-0.4	-1.8	-0.25	-1.8	-1.395	=	0.105
Maximum Reaction (RA)	=	1059.410 KN	Corr. MT	=	285.33 x	4.45 +	774.08 x	0.105	=	1351	
Minimum Reaction (RB)	=	494.590 KN	Corr. MT	=	268.669	4.45 +	225.921	0.105	=	1219	

2-CLASS A + 1-70RW

<u>CORNER AT 70RW</u>												
Transverse Ecc.	=	for 1st Class A	=	4.45 m								
	=	for 2nd Class A	=	4.45	-0.90	-0.85	0	=	2.7 m			
	=	for 70RW	=	2.7	-0.85		-1.8	-0.25	-1.2	-1.395	=	-2.795
Maximum Reaction (RA)	=	1344.740 KN		Corr. MT	=	285.33 x	4.45 +	285.33 x	2.700	=	-123	
							+	774.08 x	-2.795			
Minimum Reaction (RB)	=	763.260 KN		Corr. MT	=	268.669	4.45 +	268.67 x	2.700	=	1290	
							+	225.92 x	-2.795			

1- 70RW + 1-CLASS A

70RW - CLASS A		SEAL									
Transverse Ecc.	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155	m		
	for Class A	=	6.25	-0.5	-7.25	-0.25	=	-0.9	=	-2.650	m
Maximum Reaction (RA)	=	1059.410 KN	Corr. MT	=	774.08 x	3.155 +	285.33 x	-2.650	=	1686	
Minimum Reaction (RB)	=	494.590 KN	Corr. MT	=	225.921	3.155 +	268.669	-2.650	=	1	

1- 70RW + 2-CLASS A

Transverse Ecc.	=	3.155 m								
for 70RW	=	-2.650 m								
for 1st Class	=	-2.650		-0.9	-0.85	=		-4.400 m		
for 2nd Clas:	=									
Maximum Reaction (RA)	=	1344.740 KN	Corr. MT	=	774.08 x	3.155 +	285.33 x	-2.650	=	431
					+		285.33 x	-4.400		
Minimum Reaction (RB)	=	763.260 KN	Corr. MT	=	225.921	3.155 +	268.669	-2.650	=	-1181
					+		268.67 x	-4.400		

2- 70RW

Transverse Ecc.	for 1-70RW for 2-70RW	= =	3.16 m 3.155	-1.395	-1.2	-1.40	=	-0.835	m	
Maximum Reaction (RA)	=	1548.158 KN	Corr. MT	=	774.08 x	3.155 +	774.08 x	-0.835	=	1796
Minimum Reaction (RB)	=	451.842 KN	Corr. MT	=	225.921	3.155 +	225.921	-0.835	=	524

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	370.379	1648.184
2-CLASS A	740.757	2000.044
3-CLASS A	1111.136	1055.579
4-CLASS A	0.000	0.000
1- 70RW	774.079	2442.220
1-CLASS A + 1-70RW	1059.410	1350.999
1- 70RW + 1-CLASS A	1059.410	1686.094
2-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 2-CLASS A	0.000	0.000
2- 70RW	0.000	0.000

Reaction RB, , (kN)	Corr. MT (kNm)
129.621	576.816
259.243	699.956
388.864	369.421
0.000	0.000
225.921	712.780
494.590	1219.301
494.590	0.806
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 1111.136 KN
 Reaction RB = 388.864 KN

& Corresponding moment = 1055.579 kNm
 & Corresponding moment = 369.421 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 774.079 KN
 Reaction RB = 225.921 KN

& Max moment = 2442.220 kNm
 & Max moment = 712.780 kNm

Stability Check Summary

Description	P _{max} (kN/m ²)	P _{min} (kN/m ²)	Sliding	Overturnin g	Shear (Heel slab)	Shear (Toe slab)
Normal Dry Case						
Case 1 : DL+SIDL-Normal Dry Case	165.19	116.26	3.53	6.43	0.232	0.559
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	182.34	133.09	2.51	4.28		
Permissible	346	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Normal HFLCase						
Case 3 : DL+SIDL-Normal HFL Case	141.35	82.99	2.82	6.13		
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	134.65	105.20	1.97	3.70		
Permissible	346	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic Dry Case						
Case 5 : DL+SIDL-Long. Seismic Dry Case	182.84	103.79	2.60	3.68		
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	196.46	97.43	2.60	3.68		
Permissible	432	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic HFL Case						
Case 7 : DL+SIDL-Long. Seismic HFL Case	148.324	79.805	2.16	3.23		
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	158.717	73.005	2.16	3.23		
Permissible	432	0	1.25	1.50		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic Dry Case						
Case 9 : DL+SIDL-Trans. Seismic Dry Case	167.195	119.429	4.03	6.33		
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	180.493	113.401	4.33	6.92		
Permissible	432	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic HFL Case						
Case 11 : DL+SIDL-Trans. Seismic HFL Case	142.007	86.123	3.71	6.49		
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	142.455	89.267	3.71	6.49		
Permissible	432	0	1.25	1.50	4.192	4.192
Remarks	O.K	O.K	O.K	O.K	O.K	O.K

Reinforcement summary

Type of reinforcement	Area of steel required	Area of steel provided						
Heel Slab								
Main steel at bottom face	889	16	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
		12	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	314	12	mm bar @	200	mm c/c (i.e.)	565	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 20 Legged bar @	200	mm c/c (i.e.)			
Toe Slab								
Main steel at bottom face	1819	16	mm bar @	200	mm c/c (i.e.)	2011	mm ²	(O.K)
		16	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	402	12	mm bar @	200	mm c/c (i.e.)	565	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 22 Legged bar @	200	mm c/c (i.e.)			

Abutment Shaft								
Vertical steel at earth face	1200	16	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
		12	mm bar @	200	mm c/c (i.e.)			
Vertical steel at non-earth face	1200	20	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
Distribution steel	1000	12	mm 2 Legged bar @	150	mm c/c (i.e.)	1508	mm ²	(O.K)
Abutment Cap								
Top Longitudinal Reinf.	1576	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Bottom Longitudinal Reinf.	1576	12	mm dia bar	15	Nos.	1696	mm ²	(O.K)
Side Face Longitudinal Reinf.		12	mm dia bar	200	mm c/c (i.e.)			
Transverse Reinf.	3.13E+06	12	mm 2 Legged bar @	150	mm c/c (i.e.)	4.02E+06	mm ³ /m	(O.K)
Corbel Reinforcement								
Primary tensile reinf.	1554	20	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
Horizontal Stirrups	698.78	12	mm 2 Legged bar @	in 4 Vertical Layers		905	mm ² /m	(O.K)
Vertical Stirrups	388.47	10	mm 2 Legged bar @	200	mm c/c (i.e.)	785	mm ³ /m	(O.K)

DESIGN OF PIER P1 WITH OPEN
FOUNDATION

FOR MINOR BRIDGE AT
CHAINAGE- 17+785

Input**Span Arrangement****Left Span****Right Span**

c/c distance between Bearing - Left Span = 17.700 m
 projection beyond CL of brg.-Left support = 1.125 m
 projection beyond CL of brg.-Right support = 1.125 m
 Total Length of Left Span = 19.950 m
 Expansion Gap = 50.000 mm
 c/c distance between Expansion Gap - Left Span = 20.000 m
 Coeff. Of friction for bearing for Left Side = 0.05

c/c distance between Bearing -Right Span = 17.700 m
 projection beyond CL of brg.-Left support = 1.125 m
 projection beyond CL of brg.-Right support = 1.125 m
 Total Length of Right Span = 19.950 m
 Expansion Gap = 50.000 mm
 c/c distance bet. Expansion Gap - Right Span = 20.000 m
 Coeff. Of friction for bearing for Right Side = 0.05

Type of Superstructure = RCC GIRDER
 Radius of Curvature = 0.000 m

Type of Superstructure = RCC Girder
 Radius of Curvature = 0.000 m

Angle of Skew = 0 Degrees

Cos 0 = 1.000 Sin 0 = 0.000 tan 0 = 0.000

Calculation of Impact Factor

AS per IRC 6:2010, Clause 208.2

Impact Factor - 70 RW	=	1.190 Superstructure	Impact Factor - 70 RW	=	1.288 Superstructure
Impact Factor -Class A {1+4.5/(6+L)}	=	1.190 Superstructure	Impact Factor -Class A {1+9/(13.5+L)}	=	1.288 Superstructure
Impact Factor - 70 RW	=	1.190 Bearing	Impact Factor - 70 RW	=	1.288 Bearing
Impact Factor -Class A	=	1.190 Bearing	Impact Factor -Class A	=	1.288 Bearing
Impact Factor - 70 RW	=	1.095 Pier Cap	Impact Factor - 70 RW	=	1.144 Pier Cap
Impact Factor -Class A	=	1.095 Pier Cap	Impact Factor -Class A	=	1.144 Pier Cap

Bearings

Type of Bearing = SPHERICAL BEARING
 No of Bearings/Pedestral = 4
 C/C Distance of Bearing in Transverse Direction = 3.000m (Skew)

Type of Bearing = SPHERICAL BEARING
 No of Bearings/Pedestral = 4
 C/C Distance of Bearing in Transvers = 3.000m (Skew)

Details of Superstructures and Salient Reduced Levels's (RL's)**LEFT SIDE SPAN**

Deck level at Median edge = 1458.446 m
 Deck level at outer edge = 1458.044 m
 Deck level at center line of bridge = 1458.446 m
 Soffit Level = 1456.561 m

 Pier Cap Top Level = 1455.923 m
 C/C of span = 17.700 m

RIGHT SIDE SPAN

Deck level at Median Edge = 1458.446 m
 Deck level at outer edge = 1458.044 m
 Deck level at center line of bridge = 1458.446 m
 Soffit Level = 1456.561 m

 Pier Cap Top Level = 1455.741 m
 C/C of span = 17.700 m

Max Depth of Superstructure	=	1.820 m	Max Depth of Superstructure	=	1.820 m
Depth of Bearing+Pedestal 1	=	0.400 m	Depth of Bearing+Pedestal 1	=	0.400 m
Depth of Bearing+Pedestal 2	=	0.610 m	Depth of Bearing+Pedestal 2	=	0.610 m
Depth of Bearing+Pedestal 3	=	0.820 m	Depth of Bearing+Pedestal 3	=	0.820 m
Depth of Bearing+Pedestal 4	=	1.030 m	Depth of Bearing+Pedestal 4	=	1.030 m
Depth of Bearing+Pedestal 5	=	0.000 m	Depth of Bearing+Pedestal 5	=	0.000 m
Depth of Bearing+Pedestal 6	=	0.000 m	Depth of Bearing+Pedestal 6	=	0.000 m
Distance of nearest girder from c.l. of bridge	=	3.000 m	Distance of nearest girder from c.l. of bridge	=	0.000 m

Common For Both Span

Pier Cap Bottom Level	=	1454.323 m	Thickness of Wearing Coat	=	0.065 m
Pier Shaft Top Level	=	1454.323 m	Camber slope	=	7.00 %
Pier Shaft Bottom Level	=	1451.650 m	Width of Carriageway	=	11.5 m
GL	=	1452.531 m	Total Width of Superstructure	=	12.5 m
Foundation level	=	1450.650 m	HFL	=	1453.531 m
			MSL/Rock Level	=	1452.531 m
			Wedge over girde	=	0.056 m

Hydraulic data :

Velocity of flow	=	2.100

SBC :

NBC of soil -Normal Case	=	330 kN/m ²	(as per geotechnical report)
SBC of soil-Normal Case	=	349 kN/m ²	
SBC of soil-Wind Case	=	436 kN/m ²	(increase by 25%)
SBC of soil-Seismic Case	=	436 kN/m ²	(increase by 25%)
Type of Soil	=	1 Medium	

Dimensions of Substructure & Foundation**(i) Pier Cap****Along L-L Axis**

Width of Uniform Portion	=	4.000 m Right
Width of Uniform Portion	=	4.000 m Skew
Width of Cap at Bottom	=	4.000 m Right
Height of Uniform depth	=	0.800 m
Height of triangular portion	=	0.800 m

Along T-T Axis

Length of Uniform Portion	=	11.000 m Right
Length of Uniform Portion	=	11.000 m Skew
Length of Varying Portion at bottom	=	4.785 m Skew

		SPAN-LEFT		SPAN-Right		Expansion Gap	
C/C distance of Bearing along L-L	=	1.125	+	1.125	+	0.050	= 2.300 m
C/C distance of Bearing along T-T	=	3.000 m	(Straight)	3.000m	(Skew)		

(ii) Pier Shaft

Width of pier shaft	=	1.00 m		Pier Shaft above HFL	=	0.792 m
Length of Pier	=	4.00 m		Pier Shaft below HFL	=	1.881 m
cut water portion(semi-circular)	=	0.50 m	0.3927			
Effective Length of Pier	=	4.79 m				
Height of pier shaft	=	2.67 m				

(iii) Pier Footing**along Traffic Direction**

width of footing at bottom	=	7.000 m
width of footing at top	=	1.000 m
Thickness of footing at tip	=	0.5 m

across Traffic direction

Length of footing at bottom	=	10.000 m
Length of footing at top	=	4.785 m
Thickness of footing at root	=	1.000 m

Design Data & Material Specification

Conc Grade	=	M 35	
Characteristic Compressive Strength of Concrete, fck	=	35.00 Mpa at 28 days	
Design Compressive strength of Concrete, fcd	=	15.63 Mpa at 28 days (0.67/1.5 * fck)	
Tensile strength of concrete , fctm	=	2.77 MPa	
Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
Ecm	=	32308.250 N/mm ²	
Steel Grade	=	Fe 500	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500 Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78 Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00 Mpa	
Dry weight of Concrete	=	25 kN/m ³	
Dry unit weight of soil	=	20 kN/m ³	
Permissible Crack Width	=	0.3 mm - For Severe Exposure Condition	
Maximum compressive stress in concrete under rare	=	0.48 fck	
	=	16.8 N/mm ²	
maximum tensile stress in steel under rare combination	=	300 N/mm ²	
Seismic Data:			
Seismic Zone	=	4	
Z =Zone factor	=	0.24	
I =Importance factor	=	1.2	
R =Response Reduction factor	=	3	in Longitudinal direction
	=	3	In Transverse direction

CALCULATION OF DEAD LOAD , SIDL , CWLL**Dead Load Reaction from Superstructure Analysis**

			Trans. Ecce. (m)			Trans. Ecce. (m)
		LHS			RHS (kN)	
DL reaction on Bearing1	=	431.21 kN	4.500	DL reaction on Bearing1	: 431.21	4.500
DL reaction on Bearing2	=	413.49 kN	1.500	DL reaction on Bearing2	: 413.49	1.500
DL reaction on Bearing3	=	413.49 kN	-4.500	DL reaction on Bearing3	: 413.49	-4.500
DL reaction on Bearing4	=	431.21 kN	-1.500	DL reaction on Bearing4	: 431.21	-1.500
DL reaction on Bearing5	=	kN	0.000	DL reaction on Bearing5	=	0.000
DL reaction on Bearing6	=	kN	0.000	DL reaction on Bearing6	=	0.000
Total DL Reaction	=	1689.395 kN	0.031	Total DL Reaction	=	1689.395 0.031

SIDL Reaction from Superstructure Analysis (without Wearing Course)

			Trans. Ecce. (m)			Trans. Ecce. (m)
		LHS			RHS (kN)	
SIDL reaction on Bearing1	=	135.917 kN	4.500	SIDL reaction on Bearing1	: 135.917	4.500
SIDL reaction on Bearing2	=	-55.917 kN	1.500	SIDL reaction on Bearing2	: -55.917	1.500
SIDL reaction on Bearing3	=	-55.917 kN	-4.500	SIDL reaction on Bearing3	: -55.917	-4.500
SIDL reaction on Bearing4	=	135.917 kN	-1.500	SIDL reaction on Bearing4	: 135.917	-1.500
SIDL reaction on Bearing5	=	kN	0.000	SIDL reaction on Bearing5	=	0.000
SIDL reaction on Bearing6	=	kN	0.000	SIDL reaction on Bearing6	=	0.000
Total SIDL Reaction	=	160.000 kN	3.597	Total SIDL Reaction	=	160.000 3.597

REACTION DUE TO WEARING COUSRE ONLY

			Trans. Ecce. (m)			Trans. Ecce. (m)
		LHS			RHS (kN)	
WC reaction on Bearing1	=	57.778 kN	4.500	WC reaction on Bearing1	= 57.778	4.500
WC reaction on Bearing2	=	63.030 kN	1.500	WC reaction on Bearing2	= 63.030	1.500
WC reaction on Bearing3	=	63.030 kN	-4.500	WC reaction on Bearing3	= 63.030	-4.500
WC reaction on Bearing4	=	57.778 kN	-1.500	WC reaction on Bearing4	= 57.778	-1.500
WC reaction on Bearing5	=	kN	0.000	WC reaction on Bearing5	=	0.000
WC reaction on Bearing6	=	kN	0.000	WC reaction on Bearing6	=	0.000
TOTAL REACTION (WC)	=	241.615 kN	-0.065	TOTAL REACTION (WC)	=	241.615 -0.065

FPLL reaction on Bearing1	=	0.0 kN		FPLL reaction on Bearing 1	=	0.000
FPLL reaction on Bearing2	=	0.0 kN		FPLL reaction on Bearing 2	=	0.000
FPLL reaction on Bearing3	=	0.0 kN		FPLL reaction on Bearing 3	=	0.000
FPLL reaction on Bearing4	=	0.0 kN		FPLL reaction on Bearing 4	=	0.000
FPLL reaction on Bearing5	=	0.0 kN		FPLL reaction on Bearing 5	=	0.000
FPLL reaction on Bearing6	=	0.0 kN		FPLL reaction on Bearing 6	=	0.000
Total FPLL reaction	=	0.000 kN		Total FPLL	=	0.000

Carriageway Live Load Reactions

Cogestion factor	=	1.20	(As per Table 3 of IRC :112-2014)			
Reduction Factor	=	0.9	(For a 3 Lane carriageway)			
Reduction Factor	=	1.0	(For a 2 Lane carriageway)			
Case 1: Maximum Reaction Case and Corresponding moment						
1-70RW + 2-Class A	max	=	1000.022 kN	Corr. Transverse Moment	=	950.021 kNm
1-70RW + 2-Class A	min	=	349.978 kN	Corr. Transverse Moment	=	332.479 kNm
SV Load	max	=	2020.424 kN	Corr. Transverse Moment	=	606.127 kNm
SV Load	min	=	1399.576 kN	Corr. Transverse Moment	=	419.873 kNm
Case 2: Maximum Transverse Moment and Corresponding Reaction						
1-70RW + 2-Class A	max	=	696.671 kN	Corr. Transverse Moment	=	2197.998 kNm
1-70RW + 2-Class A	min	=	203.329 kN	Corr. Transverse Moment	=	641.502 kNm

Computation of volumes of Subsoil Components**i) Bearing Pedestal**

						Trans. Ecc
Left Span :						
Pedestal 1	=	0.400 x	0.800 x	0.800	=	0.2560 m ³ 4.500
Pedestal 2	=	0.610 x	0.800 x	0.800	=	0.3904 m ³ 1.500
Pedestal 3	=	0.820 x	0.800 x	0.800	=	0.5248 m ³ -4.500
Pedestal 4	=	1.030 x	0.800 x	0.800	=	0.6592 m ³ -1.500
Pedestal 5	=	0.000 x	0.800 x	0.800	=	0.0000 m ³ 0.000
Pedestal 6	=	0.000 x	0.800 x	0.800	=	0.0000 m ³ 0.000
						1.8304 m ³ -0.881

Right Span :

Pedestal 1	=	0.400 x	0.800 x	0.800	=	0.2560 m ³	4.500
Pedestal 2	=	0.610 x	0.800 x	0.800	=	0.3904 m ³	1.500
Pedestal 3	=	0.820 x	0.800 x	0.800	=	0.5248 m ³	-4.500
Pedestal 4	=	1.030 x	0.800 x	0.800	=	0.6592 m ³	-1.500
Pedestal 5	=	0.000 x	0.800 x	0.800	=	0.0000 m ³	0.000
Pedestal 5	=	0.000 x	0.800 x	0.800	=	0.0000 m ³	0.000
						<u>1.8304 m³</u>	<u>-0.881</u>

ii) Seismic Arrestor

Left Span:

Long. Arrestor	=				=	0.000 m ³	Nos. 0
Transverse arrestor	=	0.500 x	0.800 x	1.130	=	0.452 m ³	2

Right Span:

Long. Arrestor	=	0.500 x	1.000 x	1.100	=	0.550 m ³	0
Transverse Arrestor	=	0.500 x	0.800 x	1.130	=	0.452 m ³	2

(iii) Pier Cap

Volume	=	11.000	x	4.000	x	0.800	=	60.457	m ³
	+	4.000	x (	11.000	+	4.785)	x 0.80		
									2.000

Volume 2 due to additional height of cap = 0.182x 0.000 x 11.000 = 0.000 m³

Long. Eccentricity due to additional weight of cap = 2.000 - 0.000 = 0.000 m

(iv) Pier Shaft

Volumn of Pier Shaft	=	1.000	x	4.785	x	2.673	=	12.791	m ³
Total Volume of Pier Shaft	=	12.791	m ³						
Volume of Pier shaft above HFL	=	3.790	m ³						
Volume of Pier shaft below HFL	=	9.001	m ³						

(v) Footing

	=	{ A1	+	A2	+	SQRT(A1*A2) }	x H/3		
Trapezoidal portion	=	(70.000	+	4.785	+	18.30)	x 0.17	=	15.515 m ³
Uniform portion	=	7.000	x	10.000	x	0.500	=	35.000	m ³
Total volume of footing	=	15.515	+	35.000			=	50.515	m ³

(vi) Volume of Overburden Earth Over Footing

Total volume	=	7.000	x	10.000	x	1.881	=	131.670	m ³
Volum eof pier shaft below GL	=	1.000	x	4.79	x	0.881	=	4.216	m ³
Volume of earthfill over footing	=	(131.67	-	50.515	-	4.22)	=	76.939	m ³

Load Factors (As per IRC:6-2014)

Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

Loads	Normal Case	Seismic Case
Dead Load+SIDL , Back Fill except wearing course	1.350	1.35
SIDL (Wearing Course only)	1.750	1.75
CWLL (Service)	1.500	0.20
Wind Load	0.900	-
Seismic Effect (during service)		1.50
Seismic Effect (during construction)		0.75
Water Current Forces	1.00	1.00
Hydrodynamic effect	-	1.00

Partial Safety Factor For Verification of Serviceability Limit State:

-Refer Table 3.3 of IRC:6-2014

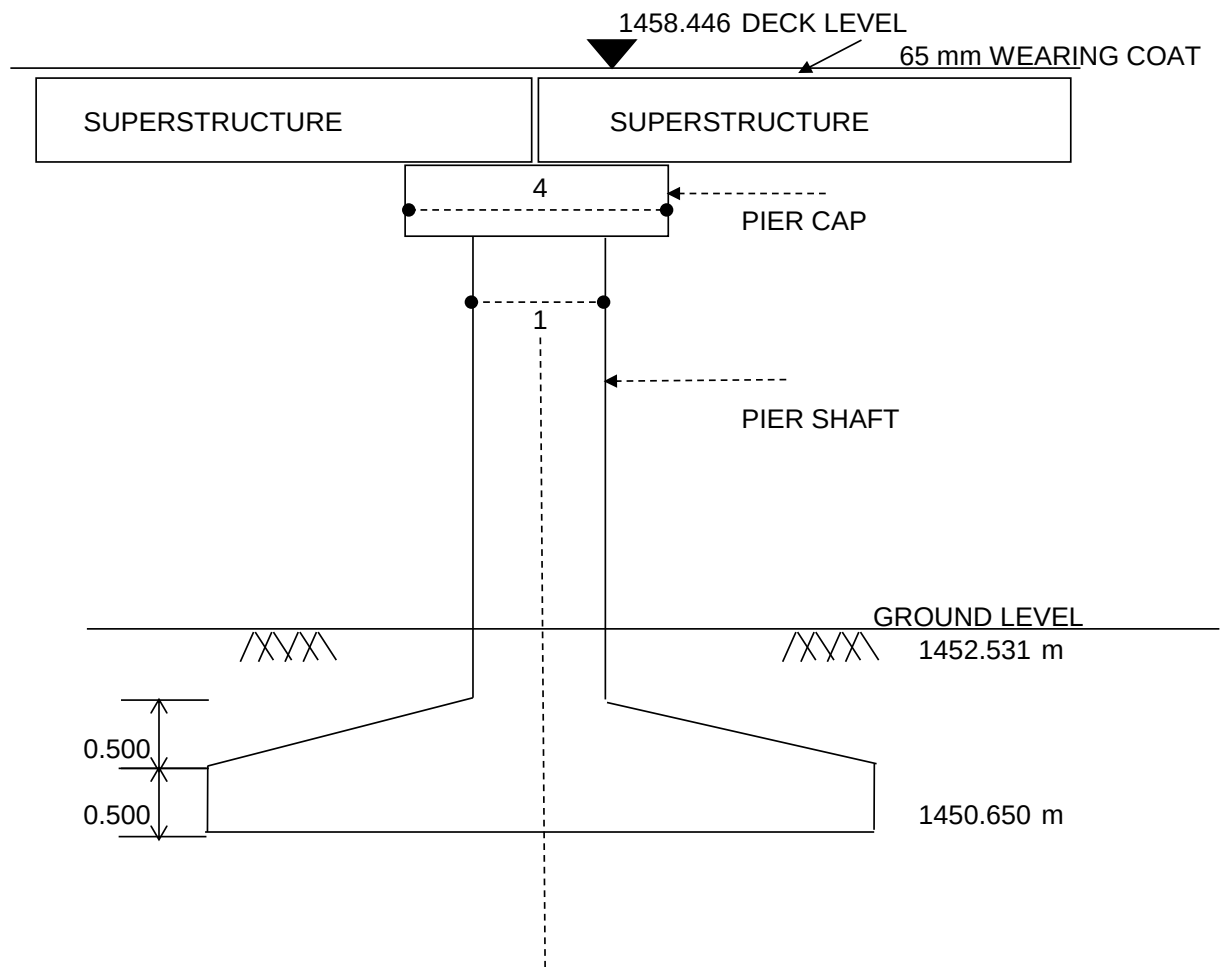
Loads	Rare Case	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL	1.00	1.00	1.00
Wearing Coat	1.20	1.20	1.20
CWLL	1.00	0.75	0.00
Wind Load	0.60	0.50	0.00

Partial Safety Factor For Design of Foundation**-Refer Table 3.4 of IRC:6-2014**

Loads	Normal Case	Seismic Case
Dead Load+SIDL except wearing course	1.350	1.35
SIDL (Wearing Course only)	1.750	1.75
Back Filling Weight	1.350	1.35
CWLL (Service)	1.500	0.20
Wind Load	0.900	-
Seismic Effect (during service)		1.50
Seismic Effect (during construction)		0.75
Water Current Forces	1.00	1.00
Hydrodynamic Effect	-	1.00

Load Combinations

1	COMB 1: DL+SIDL-Normal Dry Case	NORMAL CASE
2	COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case	
3	COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL Case	
4	COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry Case	
5	COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL Case	
6	COMB 6 : DL+SIDL -one Span Dislodege Normal case	
7	COMB 7 : COMB 1 + WL -Normal Dry Case	WIND CASE
8	COMB 8: COMB 2 + WL -Wind Dry Case	
9	COMB 9 : DL+SIDL+WL -one Span Dislodege Wind case	
10	COMB 10: COMB 1 + EQ-L -Seismic Dry Case	SEISMIC CASE
11	COMB 11 : COMB 1 + EQ-T -Seismic Dry Case	
12	COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case	
13	COMB 13: COMB 3 (Min Reaction) + EQ-L -Seismic HFL Case	
14	COMB 14: COMB 4 (Max Transverse Moment)+ EQ-L -Seismic Dry Case	
15	COMB 15: COMB 5 (Max Transverse Moment)+ EQ-L -Seismic HFL Case	
16	COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case	
17	COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case	
18	COMB 18: COMB 4 (Max Transverse Moment) + EQ-T -Seismic Dry Case	
19	COMB 19: COMB 5 (Max Transverse Moment) + EQ-T -Seismic HFL Case	
20	COMB 20 : DL+SIDL+EQ-L -one Span Dislodege Seismic case	
21	COMB 21 : DL+SIDL+EQ-T -one Span Dislodege Seismic case	

FIGURE SHOWING SALIENT FEATURE OF PIER AND FOUNDATION

Seismic coefficient Calculation

(As Per IRC:6-2014 , Clause 219.5,Annexure D)

Horizontal Seismic Force

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\left(\frac{Z}{2} \right) \left(\frac{S_a}{g} \right)}{\left(\frac{R}{I} \right)}$$

Z = Zone factor

I = Importance factor

T = Fundamental period of the bridge member (in sec.) or horizontal vibrations.

$$= \frac{2.0}{1000F} D^{1/2}$$

D = appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in

R = Response reduction factor =

Dimensions of Pier Shaft

Width = 4.79 m

Thickness = 1.00 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 0.30 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 6.85 m^4

E_{cm} = 32308.25 N/mm^2

AS B/D= 4.79 Pier wil behave as Rectangular Wall

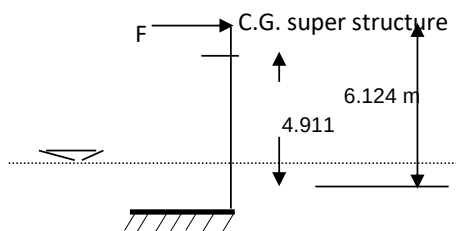
For medium soil sites

$\frac{S_a}{g}$ = 2.5 $0.0 < T < 0.55$

g = 1.36 / T $0.55 < T < 4.0$

$$A_h = \frac{Z}{2} \frac{S_a}{g} \frac{I}{R}$$

$$\delta = \frac{F L^3}{3 E I}$$



For Lonitudinal Seismic case

F (Long.)=		178.57	kN
D	=	4182	kN
T	=	0.306	sec
Z	=	0.24	
I	=	1.2	
R	=	3	
Soil type	=	1	
sa/g	=	2.500	
Ah	=	0.120	

For Tranverse Seismic case (20% live load)

F (Trans.)=		2889.96	kN
D	=	4382	kN
T	=	0.078	sec
Z	=	0.24	
I	=	1.2	
R	=	3	
Soil type	=	1	
sa/g	=	2.168	
Ah	=	0.104	

Summary of Horizontal and Vertical Sismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.104	In Transverse direction
Av	=	0.080	In Vertical direction

For Design of Foundation

Ah	=	0.162	In Longitudinal direction
Ah	=	0.140	In Transverse direction
Av	=	0.108	In Vertical direction

During Construction ,

Importance factor = **1.00****For One Span Dislodged case**

F (Long.)=		178.57	kN	F (Trans.)=		2889.96	kN
D	=	2091	kN	D	=	2091	kN
T	=	0.216	sec	T	=	0.054	sec
Z	=	0.24		Z	=	0.24	
I	=	1.00		I	=	1.00	
R	=	3		R	=	3	
Soil type	=	1		Soil type	=	1	
sa/g	=	2.500		sa/g	=	1.807	
Ah	=	0.100		Ah	=	0.072	

Summary of Horizontal and Vertical Sismic Coeff. In Dislodge Case

For Design of Substructure

Ah	=	0.100	In Longitudinal direction
Ah	=	0.072	In Transverse direction
Av	=	0.067	In Vertical direction

For Design of Foundation

Ah	=	0.135	In Longitudinal direction
Ah	=	0.098	In Transverse direction
Av	=	0.065	In Vertical direction

Horizontal Force (HL) AT Bearings in Ultimate Limit State

(Refer Clause 211.5.1.1 of IRC-6-2014)

Type of bearing - SPHERICAL BEARING

Left Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.35	1.35	2280.68	2280.68
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.62	1.75	1.75	422.83	422.83
FPLL	=	0.00	1.50	0.20	0.00	0.00
CWLLmax	=	0.00	1.50	0.20	0.00	0.00
CWLLmin	=	0.00	1.50	0.20	0.00	0.00

Right Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.35	1.35	2280.68	2280.68
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.62	1.75	1.75	422.83	422.83
FPLL	=	0.00	1.5	0.20	0.00	0.00
CWLLmax-Max Reaction	=	1000.02	1.5	0.20	1500.03	200.00
CWLLmin-Max Reaction	=	349.98	1.5	0.20	524.97	70.00
CWLLmax-Max Transv. Moment	=	696.67	1.5	0.20	1045.01	139.33
CWLLmin-Max Transv. Moment	=	203.33	1.5	0.20	304.99	40.67

1-70RW +
2-Class A1-70RW +
2-Class A**COMB 1: DL+SIDL-Normal Dry Case**

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \\ &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 145.98 \quad \text{kN} \end{aligned}$$

From Right Span

$$F_h = 0 \quad \text{kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \\ &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 145.98 \quad \text{kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	145.98	0.00
145.98	-145.98	0.00

Maximum Net Horizontal Force = 0.00 kN**COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case**

From Left Span

$$\mu R = 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL})$$

$$= 0.05 \times 2919.51$$

$$= 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 0.2 \times (1500.00) + 0.05 \times (831.00)$$

$$= 341.55 \text{ kN}$$

$$\mu R = 0.05 \times (2280.68 + 216.00 + 422.83 + 524.97)$$

$$= 0.05 \times 3444.48$$

$$= 172.22 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	343.00	197.02
145.98	169.33	315.30

Maximum Net Horizontal Force = 315.30 kN

COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55$$

$$\mu R = 0.05 \times (2280.68 + 216.00 + 422.83 + 1500.03)$$

$$= 0.05 \times 4419.54$$

$$= 220.98 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	391.75	245.78
145.98	120.57	266.55

Maximum Net Horizontal Force = 266.55 kN

COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55 \text{ kN}$$

$$\mu R = 0.05 \times (2280.68 + 216.00 + 422.83 + 304.99)$$

$$= 0.05 \times 3224.50$$

$$= 161.23 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	332.00	186.02
145.98	180.32	326.30

Maximum Net Horizontal Force = 326.30 kN

COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55 \text{ kN}$$

$$\mu R = 0.05 \times (2280.68 + 216.00 + 422.83 + 1045.01)$$

$$= 0.05 \times 3964.52$$

$$= 198.23 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	369.00	223.03
145.98	143.32	289.30

Maximum Net Horizontal Force = 289.30 kN

COMB 6 : DL+SIDL -one Span Dislodge Normal case

Maximum Net Horizontal Force = 145.98 kN

COMB 7 : COMB 1 + WL -Normal Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to wind} = 26.916 \text{ kN} \times \text{Load factor } 0.9 = 24.225$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	158.09	12.11
145.98	-121.75	24.22

Maximum Net Horizontal Force = 24.22 kN

COMB 8: COMB 2 + WL -Wind Dry Case

Since Design Mean wind speed of 39.59 m/s exceed wind speed of 36 m/s, hence no Live load to be considered in wind load case

Maximum Net Horizontal Force = 24.22 kN

COMB 9 : DL+SIDL+WL -one Span Dislodge Wind case

From Right Span

$$F_h \text{ due to wind} = 31.057 \text{ kN} \times \text{Load factor } 0.9 = 27.952$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

$$F_h/2 + \mu R = 159.951 \text{ kN}$$

$$F_h - \mu R = -118.024 \text{ kN}$$

Maximum Net Horizontal Force = 159.95 kN

COMB 10: COMB 1 + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} F_h \text{ due to seismic} &= 0.12 \times (3378.79 + 320.00 + 483.23) \\ &= 0.12 \times 4182.02 = 501.84 \times \text{Load factor } 1.50 \\ &= 752.76 \end{aligned}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	522.36	376.38
145.98	606.79	752.76

Maximum Net Horizontal Force = 752.76 kN

COMB 11 : COMB 1 + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.3 \times 376.38 = 112.915 \text{ kN}$$

$$\mu R = 145.98$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	202.43	56.46
145.98	-33.06	112.91

$$\text{Maximum Net Horizontal Force} = 112.91 \text{ kN}$$

COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 0.00) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} \text{Fh due to seismic} &= 752.76 \text{ kN} \\ \text{Fh due to braking} &= 0.2 \times 200 + 0.05 \times 110.8 \\ &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total Fh} = 798.30 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 69.996) \\ &= 0.05 \times 2989.50 \\ &= 149.48 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	525.86	379.88
145.98	603.29	749.26

$$\text{Maximum Net Horizontal Force} = 798.30 \text{ kN}$$

COMB 13: COMB 3 (Min Reaction) + EQ-L -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\begin{aligned} \text{Fh due to seismic} &= 752.76 \text{ kN} \\ \text{Fh due to braking} &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total Fh} = 798.30 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 200.004) \\ &= 0.05 \times 3119.51 \\ &= 155.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	532.36	386.38
145.98	596.79	742.76

$$\text{Maximum Net Horizontal Force} = 798.30 \text{ kN}$$

COMB 14: COMB 4 (Max Transverse Moment)+ EQ-L -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\begin{aligned} \text{Fh due to seismic} &= 752.76 \text{ kN} \\ \text{Fh due to braking} &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total Fh} = 798.30 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 40.666) \\ &= 0.05 \times 2960.17 \\ &= 148.01 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	547.16	401.19
145.98	650.29	796.27

Maximum Net Horizontal Force = 798.30 kN

COMB 15: COMB 5 (Max Transverse Moment)+ EQ-L -Seismic HFL Case
From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 752.76 \text{ kN}$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 798.30 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL} \\ &= 0.05 \times (2280.68 + 216.00 + 422.83 + 139.334) \\ &= 152.94 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	552.09	406.12
145.98	645.36	791.34

Maximum Net Horizontal Force = 798.30 kN

COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 752.76 = 225.8 \text{ kN}$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 271.37 \text{ kN}$$

$$\mu R = 149.48 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	285.16	139.18
145.98	121.89	267.87

Maximum Net Horizontal Force = 271.37 kN

COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 752.76 = 225.8 \text{ kN}$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 271.37 \text{ kN}$$

$$\mu R = 155.98 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	291.66	145.68
145.98	115.39	261.37

Maximum Net Horizontal Force = 271.37 kN

COMB 18: COMB 4 (Max Transverse Moment) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 752.76 = 225.8 \text{ kN}$$

$$\begin{aligned}
 F_h \text{ due to braking} &= 45.54 \text{ kN} \\
 \text{Total } F_h &= 271.37 \text{ kN} \\
 \mu R &= 148.01 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	283.69	137.72
145.98	123.36	269.34

$$\text{Maximum Net Horizontal Force} = 271.37 \text{ kN}$$

COMB 19: COMB 5 (Max Transverse Moment) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 752.76 = 225.8 \text{ kN}$$

$$\begin{aligned}
 F_h \text{ due to braking} &= 45.54 \text{ kN} \\
 \text{Total } F_h &= 271.37 \text{ kN} \\
 \mu R &= 152.94 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	288.63	142.65
145.98	118.43	264.40

$$\text{Maximum Net Horizontal Force} = 271.37 \text{ kN}$$

COMB 20 : DL+SIDL+EQ-L -one Span Dislodge Seismic case

From Right Span

$$\begin{aligned}
 F_h \text{ due to seismic} &= 0.100 \times (3378.79 + 320.00 + 483.23) \\
 &= 0.10 \times 4182.02 = 418.20 \text{ kN} \\
 &= 313.65 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\
 &= 0.05 \times 2091.01 \\
 &= 104.55 \text{ kN}
 \end{aligned}$$

Load factor 0.75

$$\begin{aligned}
 F_h/2 + \mu R &= 261.376 \text{ kN} \\
 F_h - \mu R &= 209.101 \text{ kN} \\
 F_h &= 313.651 \text{ kN}
 \end{aligned}$$

$$\text{Maximum Net Horizontal Force} = 313.65 \text{ kN}$$

COMB 21 : DL+SIDL+EQ-T -one Span Dislodge Seismic case

From Right Span

$$\begin{aligned}
 F_h \text{ due to seismic} &= 0.3 \times 156.83 = 47.048 \text{ kN} \\
 \mu R &= 104.55 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 F_h/2 + \mu R &= 128.074 \text{ kN} \\
 F_h - \mu R &= -57.503 \text{ kN} \\
 F_h &= 47.048 \text{ kN}
 \end{aligned}$$

$$\text{Maximum Net Horizontal Force} = 128.07 \text{ kN}$$

Horizontal Force (HL) AT Bearings For Design of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - SPHERICAL BEARING

Left Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.35	1.35	2280.68	2280.68
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.62	1.75	1.75	422.83	422.83
FPLL	=	0.00	1.5	0.2	0.00	0.00
CWLLmax	=	0.00	1.5	0.2	0.00	0.00
CWLLmin	=	0.00	1.5	0.2	0.00	0.00

Right Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1.35	1.35	2280.68	2280.68
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	241.62	1.75	1.75	422.83	422.83
FPLL	=	0.00	1.5	0.2	0.00	0.00
CWLLmax-Max Reaction	=	1000.02	1.5	0.2	1500.03	200.00
CWLLmin-Max Reaction	=	349.98	1.5	0.2	524.97	70.00
CWLLmax-Max Transv. Moment	=	696.67	1.5	0.2	1045.01	139.33
CWLLmin-Max Transv. Moment	=	203.33	1.5	0.2	304.99	40.67

1-70RW +
2-Class A

1-70RW +
2-Class A

COMB 1: DL+SIDL-Normal Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} F_h &= 0 \text{ kN} \\ \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	145.98	0.00
145.98	-145.98	0.00

Maximum Net Horizontal Force = 0.00 kN

COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 0.00) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} \text{Fh due to braking} &= 0.2 \times (1500.00) + 0.05 \times (831.00) \\ &= 341.55 \text{ kN} \end{aligned}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 524.97) \\ &= 0.05 \times 3444.48 \\ &= 172.22 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	343.00	197.02
145.98	169.33	315.30

Maximum Net Horizontal Force = 315.30 kN

COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL CaseFrom Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 1500.03) \\ &= 0.05 \times 4419.54 \\ &= 220.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	391.75	245.78
145.98	120.57	266.55

Maximum Net Horizontal Force = 266.55 kN

COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry CaseFrom Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 304.99) \\ &= 0.05 \times 3224.50 \\ &= 161.23 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	332.00	186.02
145.98	180.32	326.30

Maximum Net Horizontal Force = 326.30 kN

COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL CaseFrom Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\text{Fh due to braking} = 341.55 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 1045.01) \\ &= 0.05 \times 3964.52 \\ &= 198.23 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	369.00	223.03
145.98	143.32	289.30

Maximum Net Horizontal Force = 289.30 kN

COMB 6 : DL+SIDL -one Span Dislodge Normal case

Maximum Net Horizontal Force = 145.98 kN

COMB 7 : COMB 1 + WL -Normal Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to wind} = 26.916 \text{ kN} \times \text{Load factor } 0.9 = 24.225$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	158.09	12.11
145.98	-121.75	24.22

Maximum Net Horizontal Force = 24.22 kN

COMB 8: COMB 2 + WL -Wind Dry Case

Maximum Net Horizontal Force = 24.22 kN

COMB 9 : DL+SIDL+WL -one Span Dislodge Wind case

From Right Span

$$F_h \text{ due to wind} = 31.057 \text{ kN} \times \text{Load factor } 0.90 = 27.952$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

$$F_h/2 + \mu R = 159.9513059 \text{ kN}$$

$$F_h - \mu R = -118.024 \text{ kN}$$

Maximum Net Horizontal Force = 159.95 kN

COMB 10: COMB 1 + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$F_h \text{ due to seismic} = 0.1620 \times (3378.79 + 320.00 + 483.23) = 677.49$$

$$\begin{aligned} &= 0.16 \times 4182.020 = 677.49 \times \text{Load factor } 1.50 \\ &= 1016.23 \end{aligned}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	654.09	508.12
145.98	870.26	1016.23

Maximum Net Horizontal Force = 1016.23 kN

COMB 11 : COMB 1 + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.3 \times 508.12 = 152.435 \text{ kN}$$

$$\mu R = 145.98$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	222.19	76.22
145.98	6.46	152.43

Maximum Net Horizontal Force = 152.43 kN

COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 0.00) \\ &= 0.05 \times 2919.51 \\ &= 145.98 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} F_h \text{ due to seismic} &= 1016.23 \\ F_h \text{ due to braking} &= 0.2 \times 200 + 0.05 \times 110.8 \\ &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total } F_h = 1061.77 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 69.996) \\ &= 0.05 \times 2989.50 \\ &= 149.48 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	657.59	511.62
145.98	866.76	1012.73

Maximum Net Horizontal Force = 1061.77 kN

COMB 13: COMB 3 (Min Reaction) + EQ-L -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\begin{aligned} F_h \text{ due to seismic} &= 1016.23 \\ F_h \text{ due to braking} &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total } F_h = 1061.77 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 200.004) \\ &= 0.05 \times 3119.51 \\ &= 155.98 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	664.09	518.12
145.98	860.26	1006.23

Maximum Net Horizontal Force = 1061.77 kN

COMB 14: COMB 4 (Max Transverse Moment)+ EQ-L -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$\begin{aligned} F_h \text{ due to seismic} &= 1016.23 \\ F_h \text{ due to braking} &= 45.54 \text{ kN} \end{aligned}$$

$$\text{Total } F_h = 1061.77 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 40.666) \\ &= 0.05 \times 2960.17 \\ &= 148.01 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	656.12	510.15
145.98	868.22	1014.20

Maximum Net Horizontal Force = 1061.77 kN

COMB 15: COMB 5 (Max Transverse Moment)+ EQ-L -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 1016.23$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 1061.77 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (2280.68 + 216.00 + 422.83 + 139.334) \\ &= 0.05 \times 3058.84 \\ &= 152.94 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	661.06	515.08
145.98	863.29	1009.26

Maximum Net Horizontal Force = 1061.77 kN

COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 1016.23 = 304.87$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 350.41 \text{ kN}$$

$$\mu R = 149.48 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	324.68	178.70
145.98	200.93	346.91

Maximum Net Horizontal Force = 350.41 kN

COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 1016.23 = 304.87$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 350.41 \text{ kN}$$

$$\mu R = 155.98 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	331.18	185.20
145.98	194.43	340.41

Maximum Net Horizontal Force = 350.41 kN

COMB 18: COMB 4 (Max Transverse Moment) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 1016.23 = 304.87$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 350.41 \text{ kN}$$

$$\mu R = 148.01 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force

-145.98	323.21	177.24
145.98	202.40	348.38

Maximum Net Horizontal Force = 350.41 kN

COMB 19: COMB 5 (Max Transverse Moment) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 145.98 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.30 \times 1016.23 = 304.87$$

$$F_h \text{ due to braking} = 45.54 \text{ kN}$$

$$\text{Total } F_h = 350.41 \text{ kN}$$

$$\mu R = 152.94 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-145.98	328.15	182.17
145.98	197.47	343.44

Maximum Net Horizontal Force = 350.41 kN

COMB 20 : DL+SIDL+EQ-L -one Span Dislodege Seismic case

From Right Span

$$F_h \text{ due to seismic} = 0.135 \times \left(\begin{array}{ccc} \text{DL} & \text{SIDL} & \text{WC} \\ 3378.79 & + & 320.00 & + & 483.23 \end{array} \right) \times \text{Load factor } 0.75$$

$$= 0.14 \times 4182.0196 = 564.57 \text{ kN}$$

$$= 423.43 \text{ kN}$$

$$\mu R = 0.05 \times \left(\begin{array}{ccc} \text{DL} & \text{SIDL} & \text{WC} \\ 1689.39 & + & 160.00 & + & 241.62 \end{array} \right)$$

$$= 0.05 \times 2091.01$$

$$= 104.55 \text{ kN}$$

$$F_h/2 + \mu R = 316.265 \text{ kN}$$

$$F_h - \mu R = 318.879 \text{ kN}$$

$$F_h = 423.43 \text{ kN}$$

Maximum Net Horizontal Force = 423.43 kN

COMB 21 : DL+SIDL+EQ-T -one Span Dislodege Seismic case

From Right Span

$$F_h \text{ due to seismic} = 0.3 \times 211.71 = 63.51 \text{ kN}$$

$$\mu R = 104.55 \text{ kN}$$

$$F_h/2 + \mu R = 136.308 \text{ kN}$$

$$F_h - \mu R = -41.036 \text{ kN}$$

$$F_h = 63.514 \text{ kN}$$

Maximum Net Horizontal Force = 136.31 kN

Horizontal Force (HL) AT Bearings For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - SPHERICAL BEARING

Left Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1	1	1689.39	1689.39
SIDL except wc	=	160.00	1	1	160.00	160.00
WC	=	241.62	1	1	241.62	241.62
FPLL	=	0.00	1	1	0.00	0.00
CWLLmax	=	0.00	1	1	0.00	0.00
CWLLmin	=	0.00	1	1	0.00	0.00

Right Span

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1689.39	1	1	1689.39	1689.39
SIDL except wc	=	160.00	1	1	160.00	160.00
WC	=	241.62	1	1	241.62	241.62
FPLL	=	0.00	1	1	0.00	0.00
CWLLmax-Max Reaction	=	1000.02	1	1	1000.02	1000.02
CWLLmin-Max Reaction	=	349.98	1	1	349.98	349.98
CWLLmax-Max Transv. Moment	=	696.67	1	1	696.67	696.67
CWLLmin-Max Transv. Moment	=	203.33	1	1	203.33	203.33

1-70RW +
2-Class A1-70RW +
2-Class A**COMB 1: DL+SIDL-Normal Dry Case**

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} F_h &= 0 \text{ kN} \\ \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	104.55	0.00
104.55	-104.55	0.00

Maximum Net Horizontal Force = 0.00 kN

COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned}
 F_h \text{ due to braking} &= 0.2 \times (1000.00) + 0.05 \times (554.00) \\
 &= 227.70 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 349.98) \\
 &= 0.05 \times 2440.99 \\
 &= 122.05 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	235.90	131.35
104.55	105.65	210.20

Maximum Net Horizontal Force = 210.20 kN

COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL CaseFrom Left Span

$$\begin{aligned}
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\
 &= 0.05 \times 2091.01 \\
 &= 104.55 \text{ kN}
 \end{aligned}$$

From Right Span

$$F_h \text{ due to braking} = 227.70 \text{ kN}$$

$$\begin{aligned}
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 1000.02) \\
 &= 0.05 \times 3091.03 \\
 &= 154.55 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	268.40	163.85
104.55	73.15	177.70

Maximum Net Horizontal Force = 177.70 kN

COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry CaseFrom Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$F_h \text{ due to braking} = 227.70 \text{ kN}$$

$$\begin{aligned}
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 203.33) \\
 &= 0.05 \times 2294.34 \\
 &= 114.72 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	228.57	124.02
104.55	112.98	217.53

Maximum Net Horizontal Force = 217.53 kN

COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL CaseFrom Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$F_h \text{ due to braking} = 227.70 \text{ kN}$$

$$\begin{aligned}
 \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62 + 696.67) \\
 &= 0.05 \times 2787.68 \\
 &= 139.38 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	253.23	148.68
104.55	88.32	192.87

Maximum Net Horizontal Force = 192.87 kN

COMB 6 : DL+SIDL -one Span Dislodge Normal case

Maximum Net Horizontal Force = 104.55 kN

COMB 7 : COMB 1 + WL -Normal Dry Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$F_h \text{ due to wind} = 26.916 \text{ kN} \times \text{Load factor } 1.00 = 26.916$$

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	118.01	13.46
104.55	-77.63	26.92

Maximum Net Horizontal Force = 26.92 kN

COMB 8: COMB 2 + WL -Wind Dry Case

Maximum Net Horizontal Force = 26.92 kN

COMB 9 : DL+SIDL+WL -one Span Dislodge Wind case

From Right Span

$$F_h \text{ due to wind} = 31.057 \text{ kN} \times \text{Load factor } 1 = 31.057$$

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

$$F_h/2 + \mu R = 120.0792062 \text{ kN}$$

$$F_h - \mu R = -73.493 \text{ kN}$$

Maximum Net Horizontal Force = 120.08 kN

COMB 10: COMB 1 + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned} F_h \text{ due to seismic} &= 0.162 \times (3378.79 + 320.00 + 483.23) \\ &= 0.16 \times 4182.0196 = 677.49 \times \text{Load factor } 1.00 \end{aligned}$$

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	443.29	338.74
104.55	572.94	677.49

Maximum Net Horizontal Force = 677.49 kN

COMB 11 : COMB 1 + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$F_h \text{ due to seismic} = 0.3 \times 338.74 = 101.623 \text{ kN}$$

$$\mu R = 104.55$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	155.36	50.81
104.55	-2.93	101.62

Maximum Net Horizontal Force = 101.62 kN

COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (min)} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\ &= 104.55 \text{ kN}\end{aligned}$$

From Right Span

$$\begin{aligned}\text{Fh due to seismic} &= 677.49 \\ \text{Fh due to braking} &= 0.2 \times 200.00 + 0.05 \times 110.80 \\ &= 45.54 \text{ kN} \\ \text{Total Fh} &= 723.03 \text{ kN}\end{aligned}$$

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (min)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 69.996) \\ &= 108.05 \text{ kN}\end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	469.56	365.01
104.55	614.98	719.53

Maximum Net Horizontal Force = 723.03 kN

COMB 13: COMB 3 (Min Reaction) + EQ-L -Seismic HFL Case

From Left Span

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (max)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\ &= 104.55 \text{ kN}\end{aligned}$$

From Right Span

$$\begin{aligned}\text{Fh due to seismic} &= 677.49 \text{ kN} \\ \text{Fh due to braking} &= 45.54 \text{ kN}\end{aligned}$$

$$\text{Total Fh} = 723.03 \text{ kN}$$

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (max)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 200.004) \\ &= 114.55 \text{ kN}\end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	476.06	371.51
104.55	608.48	713.03

Maximum Net Horizontal Force = 723.03 kN

COMB 14: COMB 4 (Max Transverse Moment)+ EQ-L -Seismic Dry Case

From Left Span

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (min)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\ &= 104.55 \text{ kN}\end{aligned}$$

From Right Span

$$\begin{aligned}\text{Fh due to seismic} &= 677.49 \text{ kN} \\ \text{Fh due to braking} &= 45.54 \text{ kN}\end{aligned}$$

$$\text{Total Fh} = 723.03 \text{ kN}$$

$$\begin{aligned}\mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (min)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 69.996) \\ &= 108.05 \text{ kN}\end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	469.56	365.01
104.55	614.98	719.53

Maximum Net Horizontal Force = 723.03 kN

COMB 15: COMB 5 (Max Transverse Moment)+ EQ-L -Seismic HFL Case

From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (max)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 0.00) \\ &= 104.55 \text{ kN} \end{aligned}$$

From Right Span

$$\text{Fh due to seismic} = 677.49 \text{ kN}$$

$$\text{Fh due to braking} = 45.54 \text{ kN}$$

$$\text{Total Fh} = 723.03 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL (max)/5} \\ &= 0.05 \times (1689.39 + 160.00 + 241.62 + 139.334) \\ &= 111.52 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	473.03	368.48
104.55	611.51	716.06

Maximum Net Horizontal Force = 723.03 kN

COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$\text{Fh due to seismic} = 0.30 \times 677.49 = 203.25$$

$$\text{Fh due to braking} = 45.54 \text{ kN}$$

$$\text{Total Fh} = 248.79 \text{ kN}$$

$$\mu R = 108.05 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	232.44	127.89
104.55	140.74	245.29

Maximum Net Horizontal Force = 248.79 kN

COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$\text{Fh due to seismic} = 0.30 \times 677.49 = 203.25$$

$$\text{Fh due to braking} = 45.54 \text{ kN}$$

$$\text{Total Fh} = 248.79 \text{ kN}$$

$$\mu R = 114.55 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	238.94	134.39
104.55	134.24	238.79

Maximum Net Horizontal Force = 248.79 kN

COMB 18: COMB 4 (Max Transverse Moment) + EQ-T -Seismic Dry Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$\text{Fh due to seismic} = 0.30 \times 677.49 = 203.25$$

$$\text{Fh due to braking} = 45.54 \text{ kN}$$

$$\text{Total Fh} = 248.79 \text{ kN}$$

$$\mu R = 108.05 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	232.44	127.89
104.55	140.74	245.29

$$\text{Maximum Net Horizontal Force} = 248.79 \text{ kN}$$

COMB 19: COMB 5 (Max Transverse Moment) + EQ-T -Seismic HFL Case

From Left Span

$$\mu R = 104.55 \text{ kN}$$

From Right Span

$$\text{Fh due to seismic} = 0.30 \times 677.49 = 203.25$$

$$\text{Fh due to braking} = 45.54 \text{ kN}$$

$$\text{Total Fh} = 248.79 \text{ kN}$$

$$\mu R = 111.52 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-104.55	235.91	131.36
104.55	137.27	241.82

$$\text{Maximum Net Horizontal Force} = 248.79 \text{ kN}$$

COMB 20 : DL+SIDL+EQ-L -one Span Dislodge Seismic case

From Right Span

$$\begin{aligned} \text{Fh due to seismic} &= 0.1350 \times (3378.79 + 320.00 + 483.23) \\ &= 0.14 \times 4182.0196 = 564.57 \text{ kN} \\ &= 282.29 \text{ kN} \end{aligned}$$

Load factor
0.50

$$\begin{aligned} \mu R &= 0.05 \times (1689.39 + 160.00 + 241.62) \\ &= 0.05 \times 2091.01 \\ &= 104.55 \text{ kN} \end{aligned}$$

$$\text{Fh}/2 + \mu R = 245.694 \text{ kN}$$

$$\text{Fh} - \mu R = 177.736 \text{ kN}$$

$$\text{Fh} = 282.29 \text{ kN}$$

$$\text{Maximum Net Horizontal Force} = 282.29 \text{ kN}$$

COMB 21 : DL+SIDL+EQ-T -one Span Dislodge Seismic case

From Right Span

$$\begin{aligned} \text{Fh due to seismic} &= 0.3 \times 141.14 = 42.343 \text{ kN} \\ \mu R &= 104.55 \text{ kN} \end{aligned}$$

$$\text{Fh}/2 + \mu R = 125.722 \text{ kN}$$

$$\text{Fh} - \mu R = -62.208 \text{ kN}$$

$$\text{Fh} = 42.343 \text{ kN}$$

$$\text{Maximum Net Horizontal Force} = 125.72 \text{ kN}$$

Horizontal Force (HL) AT Bearings For Servicibility Limit State

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - SPHERICAL BEARING

Left Span

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1689.39	1.00	1.00	1.00	1689.39	1689.39	1689.39
SIDL except wc	=	160.00	1.00	1.00	1.00	160.00	160.00	160.00
WC	=	241.62	1.20	1.20	1.20	289.94	289.94	289.94
FPLL	=	0.00	1.00	0.75	0.00	0.00	0.00	0.00
CWLLmax	=	0.00	1.00	0.75	0.00	0.00	0.00	0.00
CWLLmin	=	0.00	1.00	0.75	0.00	0.00	0.00	0.00

Right Span

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1689.39	1.00	1.00	1.00	1689.39	1689.39	1689.39
SIDL except wc	=	160.00	1.00	1.00	1.00	160.00	160.00	160.00
WC	=	241.62	1.20	1.20	1.20	289.94	289.94	289.94
FPLL	=	0.00	1.00	0.75	0.00	0.00	0.00	0.00
CWLLmax-Max Reaction	=	1000.02	1.00	0.75	0.00	1000.02	750.02	0.00
CWLLmin-Max Reaction	=	349.98	1.00	0.75	0.00	349.98	262.48	0.00
CWLLmax-Max Transv. Moment	=	696.67	1.00	0.75	0.00	696.67	522.50	0.00
CWLLmin-Max Transv. Moment	=	203.33	1.00	0.75	0.00	203.33	152.50	0.00

COMB 1: DL+SIDL-Normal Dry Case**-Rare Combination**From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} + \text{SIDL} + \text{Wearing Course}) \\ &= 0.05 \times (1689.39 + 160.00 + 289.94) \\ &= 106.97 \text{ kN} \end{aligned}$$

From Right Span

$$F_h = 0 \text{ kN}$$

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} + \text{SIDL} + \text{WC}) \\ &= 0.05 \times (1689.39 + 160.00 + 289.94) \\ &= 106.97 \text{ kN} \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	106.97	0.00
106.97	-106.97	0.00

Maximum Net Horizontal Force = 0.00 kN**COMB 2: DL+SIDL + LL(Max Reaction) -Normal -Rare Combination**From Left Span

$$\begin{aligned} \mu R &= 0.05 \times (\text{DL} + \text{SIDL} + \text{WC} + \text{LL}) \\ &= 0.05 \times (1689.39 + 160.00 + 289.94 + 0.00) \\ &= 106.97 \text{ kN} \end{aligned}$$

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 0.2 \times (1000.00) + 0.05 \times (554.00) \\
 &= 227.70 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 289.94 + 349.98) \\
 &= 0.05 \times 2489.31 \\
 &= 124.47 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	238.32	131.35
106.97	103.23	210.20

Maximum Net Horizontal Force = 210.20 kN

COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL Case -Rare Combination

From Left Span

$$\mu R = 106.97 \text{ kN}$$

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 227.70 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 289.94 + 1000.02) \\
 &= 0.05 \times 3139.35 \\
 &= 156.97 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	270.82	163.85
106.97	70.73	177.70

Maximum Net Horizontal Force = 177.70 kN

COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry Case -Rare Combination

From Left Span

$$\mu R = 106.97 \text{ kN}$$

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 227.70 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 289.94 + 203.33) \\
 &= 0.05 \times 2342.66 \\
 &= 117.13 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	230.98	124.02
106.97	110.57	217.53

Maximum Net Horizontal Force = 217.53 kN

COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL Case -Rare Combination

From Left Span

$$\mu R = 106.97 \text{ kN}$$

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 227.70 \text{ kN} \\
 \mu R &= 0.05 \times (1689.39 + 160.00 + 289.94 + 696.67) \\
 &= 0.05 \times 2836.00 \\
 &= 141.80 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	255.65	148.68
106.97	85.90	192.87

Maximum Net Horizontal Force = 192.87 kN

COMB 6 : DL+SIDL -one Span Dislodge Normal case -Rare Combination

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 0.00 \text{ kN} \\
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94 + 0.00) \\
 &= 106.97 \text{ kN}
 \end{aligned}$$

$$\text{Maximum Net Horizontal Force} = 106.97 \text{ kN}$$

COMB 7 : COMB 1 + WL -Normal Dry Case -Rare Combination

From Left Span

$$\mu R = 106.97 \text{ kN}$$

From Right Span

$$\begin{aligned}
 F_h \text{ due to wind} &= 26.916 \text{ kN} \times \text{Load factor } 1.00 = 26.916 \text{ kN} \\
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94) \\
 &= 106.97 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	120.42	13.46
106.97	-80.05	26.92

$$\text{Maximum Net Horizontal Force} = 26.92 \text{ kN}$$

COMB 8: COMB 2 + WL -Wind Dry Case -Rare Combination

$$\text{Maximum Net Horizontal Force} = 26.92 \text{ kN}$$

FREQUENT COMBINATION

COMB 1: DL+SIDL-Normal Dry Case

FREQUENT COMBINATION

From Left Span

$$\begin{aligned}
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{Wearing Course}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94) \\
 &= 106.97 \text{ kN}
 \end{aligned}$$

From Right Span

$$F_h = 0 \text{ kN}$$

$$\begin{aligned}
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94) \\
 &= 106.97 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	106.97	0.00
106.97	-106.97	0.00

$$\text{Maximum Net Horizontal Force} = 0.00 \text{ kN}$$

COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case

From Left Span

$$\begin{aligned}
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94 + 0.00) \\
 &= 106.97 \text{ kN}
 \end{aligned}$$

From Right Span

$$\begin{aligned}
 h \text{ due to braking} &= 0.2 \times (750.00) + 0.05 \times (415.50) \\
 &= 170.78 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 \mu R &= 0.05 \times (\text{DL} \quad \text{SIDL} \quad \text{WC} \quad \text{LL}) \\
 &= 0.05 \times (1689.39 + 160.00 + 289.94 + 262.48) \\
 &= 120.09 \text{ kN}
 \end{aligned}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	205.48	98.51
106.97	50.68	157.65

Maximum Net Horizontal Force = 157.65 kN

COMB 7 : COMB 1 + WL -Normal Dry Case

From Left Span

$$\mu R = 106.97 \text{ kN}$$

From Right Span

$$F_h \text{ due to wind} = 26.916 \text{ kN} \times \text{Load factor } 0.50 = 13.458 \text{ kN}$$

DL SIDL WC

$$\mu R = 106.97 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	113.70	6.73
106.97	-93.51	13.46

Maximum Net Horizontal Force = 13.46 kN

COMB 8: COMB 2 + WL -Wind Dry Case

Maximum Net Horizontal Force = 13.46 kN

1. Normal Case - (DL+SIDL)-Quasi-Permanent Combination

From Left Span

$$\mu R = 0.05 \times (\text{DL } 1689.39 + \text{SIDL } 160.00 + \text{Wearing Course } 289.94)$$

$$= 0.05 \times 2139.33$$

$$= 106.97 \text{ kN}$$

From Right Span

$$F_h = 0 \text{ kN}$$

$$\mu R = 0.05 \times (\text{DL } 1689.39 + \text{SIDL } 160.00 + \text{WC } 289.94)$$

$$= 0.05 \times 2139.33$$

$$= 106.97 \text{ kN}$$

Free Brg	Fix Brg	Net Horizontal Force
-106.97	106.97	0.00
106.97	-106.97	0.00

Maximum Net Horizontal Force = 0.00 kN

2. Normal One Span Dislodged Case - (DL+SIDL)-Quasi-Permanent Combination

Left Span Dislodge

Maximum Horizontal Force

$$\mu R = 106.97 \text{ kN}$$

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case

Seismic Case

Design Speed	V	=	60.00	kmph	60.00	kmph
Live Load	W	=	1350.00	kN	1350.00	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Calculation of Vertical Reaction Due to Braking Force

Braking Force	=	0.2 x (	1000	) +	0.05 x (	554	)
	=	227.7 kN					
				Deck Lvl. At Centre of Bridge		Soffit Level	
C.g. Of Force	=	1.2 + (	1458.446	-	1456.561	)	
	=	3.085 m					
Moment due to Braking Force	=	702.454 kNm					
Span	=	17.700 m					
Vertical Reaction	=	39.687 kN	Upward				
	=	-39.687 kN	Downward				

Wind Calculation (As Per Clause 209 of IRC:6-2014)

Basic wind Speed for Bridge = 47 m/s (As per IRC 6:2014 , Fig. 6)

Maximum Height of Bridge from Ground Surface = 5.915 m

As per Table 4 of IRC 6:2014

Basic wind speed = 33 m/s
 $V_z = 27.8$ m/s
 $P_z = 463.7$ N/m²

modified value of Hourly mean wind speed and wind pressure for this bridge are as per below

$V_{z,design} = V_z \times 1.424 = 39.59$ m/s
 $P_{z,design} = P_z \times 2.028 = 940.60$ N/m²

AS per Clause 209.3.7 of IRC 6:2014

Since Design Mean wind speed of 39.59 m/s exceed wind speed of 36 m/s, hence no Live load to be considered in wind load case

Design Wind Force on Superstructure

$F_T = P_z \times A_1 \times G \times C_d$ as per IRC 6:2014, Caluse 209.3.3

Where , $P_z = 940.60$ N/m²

Area (A1) in Elevation

Left Side Span

Span Length = 20.000 m
 Depth of superstructure = 1.820 m
 Crash Barrier = 1.115 m
 Area A1 = 58.700 m²

Right Side Span

Span Length = 20.000 m
 Depth of superstructure = 1.820 m
 Anticrash Railing = 1.115 m
 Area A1 = 58.700 m²

Gust factor , G = 2.0 as per Clause 209.3.3

Left Span RCC GIRDER

Width of Girder b = 12500 mm
 Depth of Girder d = 1820 mm
 Ratio b/d = 6.868

Drag Coeff. $C_d = 1.950$

RightSpan RCC Girder

Width of Girder b = 800 mm
 Depth of Girder d = 1820 mm
 Ratio b/d = 0.440

Drag Coeff. $C_d = 2.250$

$FT = 107665.768$ N from Left span
 N acting at Level = 1458.029 m
 Parallel to Span $FT \sin(\theta) = 0.000$ N acting at Level = 1456.323 m
 Perpendicular to span $FT \cos(\theta) = 107665.768$ N acting at Level = 1458.029 m

$FT = 124229.733$ N from Right Span
 acting at 1458.029 m
 Parallel to Span $FT \sin(\theta) = 0.000$ N acting at Level = 1456.141 m
 Perpendicular to span $FT \cos(\theta) = 124229.733$ N acting at Level = 1458.029 m

AS per clause 209.3.4

Longitudinal Wind Force

FL = 25% $FT = 26916.442$ N from Left span
 N acting at Level = 1456.323 m
 Parallel to Span $FL \cos(\theta) = 26916.442$ N acting at Level = 1456.323 m
 Perpendicular to span $FL \sin(\theta) = 0.000$ N acting at Level = 1458.029 m

FL = 25% $FT = 31057.433$ N from Right Span
 N acting at Level = 1456.141 m
 Parallel to Span $FL \cos(\theta) = 31057.433$ N acting at Level = 1456.141 m
 Perpendicular to span $FL \sin(\theta) = 0.000$ N acting at Level = 1458.029 m

Total Wind Force in Longitudinal Direction	FL =	FT sin (θ)	+	FL cos (θ)	from Left span
	=	0.00	+	26916.44	
	=	26916.44 N acting at Level = 1456.323 m			
	FL =	FT sin (θ)	+	FL cos (θ)	from Right Span
	=	0.00	+	31057.43	
	=	31057.43 N acting at Level = 1456.14 m			
Total Wind Force in Transverse Direction	FT =	FT cos (θ)	+	-FL sin (θ)	from Left span
	=	107665.77	+	0.00	
	=	107665.77 N acting at Level = 1458.029 m			
	FT =	FT cos (θ)	+	-FL sin (θ)	from Right Span
	=	124229.73	+	0.00	
	=	124229.73 N acting at Level = 1458.029 m			
Total FT =		231895.50 N acting at Level = 1458.029 m			

Upward / Downward Wind Vertical Load (Fv)

Fv = Pz x A3 x G x CL as per IRC 6:2014, Clause 209.3.5

Where ,

A3	=	Area in Plan (m2)	=	12.5 x 20
			=	250 m ² from Left span
A3	=		=	12.5 x 20
			=	250 m ² from Right Span
CL	=		=	0.75
Fv	=		=	176362.48 N from Left span
			=	-176362.48 N Upward
Fv	=		=	352724.97 N from Right Span
			=	-352724.97 N upwad

Design Wind Force on Substructure

AS per clause 209.4 , IRC 6:2014

Pier Cap

F _T	=	Pz x A2 x G x Cd		
Projected area of cap in elevation	A2	=	6.400 m ²	
	H/b	=	0.400	& d/b= 1.973
Correspo to H/b and d/b ratio fro Table 5	Cd	=	0.800	
	FT	=	9631.743 N	
Parallel to Span	FT sin (θ)	=	0.000 N acting at Level =	1455.123 m
Perpendicular to span	FT cos (θ)	=	9631.743 N acting at Level =	1455.123 m

Pier Shaft

Projected area of shaft in elevation	A2	=	1.792 m ²	
	H/b	=	1.792	& d/b= 4.785
	Cd	=	0.8 for rectangular pier	
	FT	=	2696.888 N	
Parallel to Span	FT sin (θ)	=	0.000 N acting at Level =	1453.427 m
Perpendicular to span	FT cos (θ)	=	2696.888 N acting at Level =	1453.427 m

Forces and Moments due to Water Current Force (HFL Case)

HFL	=	1453.531 m	Foundation lvl	=	1450.650 m
Pier shaft bottom level	=	1451.7 m	MSL	=	1452.531 m
Pier Cap bottom Level	=	1454.3 m	Pier Cap Top Level	=	1455.923 m

As per cl. 210.2 of IRC: 6 - 2014

Intensity of water current pressure	=	52 k V ² kg / m ²

where, θ = 0 degree
 $\cos (20 \pm \theta) = 0.9397$
 $\sin (20 \pm \theta) = 0.3420$

Assuming 20 degree variation in water current direction

Velocity in Transverse direction = $V_{\max} * \cos 20$
 Velocity in Longitudnal direction = $V_{\max} * \sin 20$

Where $V_{\max}$ = 1.414 x V

Mean velocity of flow = 2.100 m / s
 Therefore, $V_{\max}$ = 2.970 m / s

Water current pressure in Transverse direction

$$P = 52 k (V_{\max} \cos 20)^2$$

Component	RL	Pressure (kg / m ²)
HFL	1453.5	404.99 k
Pier Cap bottom Level	1454.3	725.74 k
MSL	1452.5	0.00 k

Water current force in Transverse direction

Component	k	Width (m)	Height (m)	Force (kN)	RL of force (m)	Moment @ RL (m)	Moment @ RL (m)
						1451.65	1450.65
Pier shaft	0.66	1.000	1.00	1.336	1453.198	2.068	3.40
Pier Cap	0.00	4.000	-0.79	0.000	1453.964	0.000	0.00
Total				1.336		2.068	3.405

Water current pressure in Longitudnal direction

$$P = 52 k (V_{\max} \sin 20)^2$$

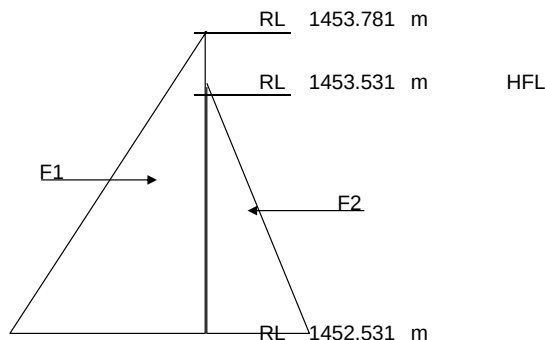
Component	RL	Pressure (kg / m ²)
HFL	1453.53	53.651 k
Pier Cap bottom Level	1454.32	96.142 k
MSL	1452.53	0.000 k

Water current force in Longitudinal direction

Component	k	Width (m)	Height (m)	Force (kN)	RL of force (m)	Moment @ RL (m) 1451.65	Moment @ RL (m) 1450.65
Pier shaft	1.5	4.785	1	1.926	1453.198	2.980	4.91
Pier Cap	0	7.893	-0.79	0.000	1453.964	0.000	0.00
Total				1.93		2.98	4.91

STATIC EFFECT OF CROSS WATER CURRENT IN LONGITUDINAL (L - L) DIRECTION

As per clause 210.6 of IRC:6-2010, for bridges with pucca floor, the effect of cross current in no case be taken as less than that of a static force due to a difference of head of 250 mm between the opposite faces of pier.



$$F1 = \frac{1}{2} \times \text{Unit wt. of Water} \times 10 \times 1.250 \times 1.250 \times 4.785 = 37.4 \text{ kN}$$

$$\text{Moment due to F1 @ base of foundation} = 37.4 \times \left(\frac{1.250}{3} + 1.000 \right) = 53 \text{ kNm}$$

$$\text{Moment due to F1 @ Pier Shaft Bottom} = 37.4 \times \left(\frac{1.250}{3} + 0.000 \right) = 16 \text{ kNm}$$

$$F2 = \frac{1}{2} \times \text{Unit wt. of Water} \times 10 \times 1.000 \times 1.000 \times 4.785 = 23.9 \text{ kN}$$

$$\text{Moment due to F2 @ base of foundation} = 23.9 \times \left(\frac{1.000}{3} + 1.000 \right) = 32 \text{ kNm}$$

$$\text{Moment due to F2 @ Pier Shaft Bottom} = 23.9 \times \left(\frac{1.000}{3} + 0.000 \right) = 8 \text{ kNm}$$

Net Moment due to Cross Current @ Foundation Level	=	53.0	-31.9	=	21.1 kNm
Net Moment due to Cross Current @ Pier Shaft Bottom	=	15.6	-8.0	=	7.6 kNm

Summary Moment for design due to water current shall be as follows:

		Force	Moment	
Transverse	Pier Base	1.34	2.07	kNm
	Foundation Lvl	1.34	3.40	kNm
Longitudinal	Pier Base	13.46	7.6	kNm
	Foundation Lvl	13.46	21.1	kNm

Hydrodynamic Force on Substructure:

Hydradynamic force, F	=	$C_e \times A_h \times W_e$	
A_h	=	Design of horizontal seismic coeff.	
W_e	=	weight of water in enveloping cylinder	
C_e	=	Constant	
HFL	=	1453.531 m	
Pier Shaft Bottom Level	=	1451.650 m	
Founding Level	=	1450.650 m	
Submerged height of Pier Shaft, H	=	1.881 m	
Radius of enveloping cylinder, a	=	2.393 m	
Submerged volume of Pier shaft	=	1.881 x 17.99	
	=	33.831 m ³	
Density of water	=	10 kN/m ³	
weight of water in the enveloping cylinder , W	=	338.310 kN.	
C_e	=	0.350 Corresponding to H/a	= 0.786

Hydrodynamic Force at Pier Shaft Bottom level :

		C_e		A_h		W_e
Total horizontal force , F	=	0.350	x	0.120	x	338.3
	=	14.227 kN.				
	=					
Resultant c.g.of force from bottom of pier shaft	=	0.429 H		=	0.806	m

Longitudinal Seismic Case:

FL	=	14.227 kN.
FT	=	4.268 kN.
FV	=	2.845 kN.
ML	=	11.470 kNm
MT	=	3.441 kNm

Transverse Seismic Case:

FL	=	4.268 kN.
FT	=	14.227 kN.
FV	=	2.845 kN.
ML	=	3.441 kNm
MT	=	11.470 kNm

Hydrodynamic Force at founding level :

		C_e		A_h		W_e
Total horizontal force , F	=	0.350	x	0.162	x	338.31
	=	19.206 kN.				
	=					
Resultant c.g.of force from founding level	=	0.429 H	+	1.000	=	1.806m

Longitudinal Seismic Case:

FL	=	19.206 kN.
FT	=	5.762 kN.
FV	=	3.841 kN.
ML	=	34.690 kNm
MT	=	10.407 kNm

Transverse Seismic Case:

FL	=	5.762 kN.
FT	=	19.206 kN.
FV	=	3.841 kN.
ML	=	10.407 kNm
MT	=	34.690 kNm

Calculation of Forces For Stability of Foundation

Foundation Lvl = 1450.65 m Soffit Level = 1456.561 m

COMB 1: DL+SIDL-Normal Dry Case

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Left Span										
Dead Load	1			1689.395			-1.150	0.031	-1942.804	53.138
SIDL except Wearing Course	1			160.000			-1.150	3.597	-184.000	575.502
Wearing Course	1			241.615			-1.150	-0.065	-277.857	-15.758
Superstructure-Right Span										
Dead Load	1			1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course	1			160.000			1.150	3.597	184.000	575.502
Wearing Course	1			241.615			1.150	-0.065	277.857	-15.758
Bearing Pedestal : Left Span	1	25	1.830	45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span	1	25	1.830	45.760			1.150	-0.881	52.624	-40.320
Seismic Arrestor	1	25	1.808	45.200			0.000	0.000	0.000	0.000
Substructure										
Pier Cap	1	25	60.457	1511.416			0.000	0.000	0.000	0.000
Additional weight of cap	1	25	0.000	0.000			0.000	0.000	0.000	0.000
Pier Shaft	1	25	12.791	319.784			0.000	0.000	0.000	0.000
Footing	1	25	50.515	1262.866			0.000	0.000	0.000	0.000
Earth fill on Footing	1	20	76.939	1538.789			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					0.000	0.000			0.000	0.000
Total				8951.594	0.000	0.000			0.000	1145.126

COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case (Maximum CWLL at Fixed End)

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Left Span										
Dead Load				1689.395			-1.150	0.031	-1942.804	53.138
SIDL except Wearing Course				160.000			-1.150	3.597	-184.000	575.502
Wearing Course				241.615			-1.150	-0.065	-277.857	-15.758
FPLL				0.000			-1.150	0.000	0.000	0.000
CWLL(Max)				0.000			-1.150	0.000	0.000	0.000
Superstructure-Right Span										
Dead Load				1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course				160.000			1.150	3.597	184.000	575.502
Wearing Course				241.615			1.150	-0.065	277.857	-15.758
FPLL				0.000			1.150	0.000	0.000	0.000
CWLL(Max)+vertical braking reaction				1000.022			1.150		1150.025	950.021
Bearing Pedestal : Left Span				45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span				45.760			1.150	-0.881	52.624	-40.320
Seismic Arrestor				45.200			0.000	0.000	0.000	0.000

Substructure										
Pier Cap				1511.416			0.000	0.000	0.000	0.000
Additional weight of cap				0.000			0.000	0.000	0.000	0.000
Pier Shaft				319.784			0.000	0.000	0.000	0.000
Footing				1262.866			0.000	0.000	0.000	0.000
Earth fill on Footing				1538.789			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					210.201	0.000			1242.499	0.000
Total				9951.616	210.201	0.000			2392.524	2095.147

COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL Case (Minimum CWLL at Fixed End)

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Left Span										
Dead Load				1689.395			-1.150	0.031	-1942.804	53.138
SIDL except Wearing Course				160.000			-1.150	3.597	-184.000	575.502
Wearing Course				241.615			-1.150	-0.065	-277.857	-15.758
FPLL				0.000			-1.150	0.000	0.000	0.000
CWLL(Max)				0.000			-1.150	0.000	0.000	0.000
Superstructure-Right Span										
Dead Load				1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course				160.000			1.150	3.597	184.000	575.502
Wearing Course				241.615			1.150	-0.065	277.857	-15.758
FPLL				0.000			1.150	0.000	0.000	0.000
CWLL(Max)+vertical braking reaction				349.978			1.150		402.475	332.479
Bearing Pedestal : Left Span				45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span				45.760			1.150	-0.881	52.624	-40.320
Seismic Arrestor				45.200			0.000	0.000	0.000	0.000
Substructure										
Pier Cap				1511.416			0.000	0.000	0.000	0.000
Additional weight of cap				0.000			0.000	0.000	0.000	0.000
Pier Shaft above HFL	1	25	3.790	94.751			0.000	0.000	0.000	0.000
Pier Shaft below HFL	1	15	9.001	135.020			0.000	0.000	0.000	0.000
Water Current	1				13.459	1.336			21.061	3.405
Footing	1	15	50.515	757.719			0.000	0.000	0.000	0.000
Earth fill on Footing	1	10	76.939	769.394			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					177.699	0.000			1050.378	0.000
Total				7937.018	191.158	1.336			1473.914	1481.010

COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry Case

(Maximum CWLL at Fixed End)

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Left Span	1									
Dead Load				1689.395			-1.150	0.031	-1942.804	53.138
SIDL except Wearing Course				160.000			-1.150	3.597	-184.000	575.502
Wearing Course				241.615			-1.150	-0.065	-277.857	-15.758
FPLL				0.000			-1.150	0.000	0.000	0.000
CWLL(Max)				0.000			-1.150	0.000	0.000	0.000
Superstructure-Right Span										
Dead Load				1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course				160.000			1.150	3.597	184.000	575.502
Wearing Course				241.615			1.150	-0.065	277.857	-15.758
FPLL				0.000			1.150	0.000	0.000	0.000
CWLL(Max)+vertical braking r				736.358			1.150		846.812	2197.998
Bearing Pedestal : Left Span				45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span				45.760			1.150	-0.881	52.624	-40.320
Seismic Arrestor				45.200			0.000	0.000	0.000	0.000
Substructure										
Pier Cap				1511.416			0.000	0.000	0.000	0.000
Additional weight of cap				0.000			0.000	0.000	0.000	0.000
Pier Shaft				319.784			0.000	0.000	0.000	0.000
Footing				1262.866			0.000	0.000	0.000	0.000
Earth fill on Footing				1538.789			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					217.534	0.000			1285.841	0.000
Total				9687.952	217.534	0.000			2132.652	3343.123

COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL Case

(Minimum CWLL at Fixed End)

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Left Span	1									
Dead Load				1689.395			-1.150	0.031	-1942.804	53.138
SIDL except Wearing Course				160.000			-1.150	3.597	-184.000	575.502
Wearing Course				241.615			-1.150	-0.065	-277.857	-15.758
FPLL				0.000			-1.150	0.000	0.000	0.000
CWLL(Max)				0.000			-1.150	0.000	0.000	0.000
Superstructure-Right Span										
Dead Load				1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course				160.000			1.150	3.597	184.000	575.502
Wearing Course				241.615			1.150	-0.065	277.857	-15.758
FPLL				0.000			1.150	0.000	0.000	0.000
CWLL(Max)+vertical braking r				243.016			0.000		0.000	641.502
Bearing Pedestal : Left Span				45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span				45.760			1.150	-0.881	52.624	-40.320

Seismic Arrestor				45.200			0.000	0.000	0.000	0.000
Substructure										
Pier Cap				1511.416			0.000	0.000	0.000	0.000
Additional weight of cap				0.000			0.000	0.000	0.000	0.000
Pier Shaft above HFL				94.751			0.000	0.000	0.000	0.000
Pier Shaft below HFL				135.020			0.000	0.000	0.000	0.000
Water Current					13.459	1.336			21.061	3.405
Footing				757.719			0.000	0.000	0.000	0.000
Earth fill on Footing				769.394			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					192.866	0.000			1140.034	0.000
Total				7830.056	206.325	1.336			1161.094	1790.033

COMB 6 : DL+SIDL -one Span Dislodge Normal case

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MT = PxeT
Superstructure-Right Span										
Dead Load	1			1689.395			1.150	0.031	1942.804	53.138
SIDL except Wearing Course	1			160.000			1.150	3.597	184.000	575.502
Wearing Course	1			241.615			1.150	-0.065	277.857	-15.758
Bearing Pedestal : Left Span				45.760			-1.150	-0.881	-52.624	-40.320
Bearing Pedestal : Right Span				45.760			1.150	-0.881	52.624	-40.320
Seismic Arrestor				45.200			0.000	0.000	0.000	0.000
Substructure										
Pier Cap	1	25	60.457	1511.416			0.000	0.000	0.000	0.000
Additional weight of cap				0.000			0.000	0.000	0.000	0.000
Pier Shaft	1	25	12.791	319.784			0.000	0.000	0.000	0.000
Footing	1	25	50.515	1262.866			0.000	0.000	0.000	0.000
Earth fill on footing	1	20	76.939	1538.789			0.000	0.000	0.000	0.000
Net Horizontal Force at Bearing					104.550	0.000			617.998	0.000
Total				6860.584	104.550	0.000			3022.659	532.243

COMB 7 : COMB 1 + WL -Normal Dry Case

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	C.g. of Force (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	Long. Moment		Trans. Moment	
										ML = PxeL	ML due to FL	MT = PxeT	MT due to FT
Superstructure-Left Span													
Dead Load				1689.395				-1.150	0.031	-1942.804		53.138	
SIDL except Wearing Course				160.000				-1.150	3.597	-184.000		575.502	
Wearing Course				241.615				-1.150	-0.065	-277.857		-15.758	
Superstructure-Right Span													
Dead Load				1689.395				1.150	0.031	1942.804		53.138	
SIDL except Wearing Course				160.000				1.150	3.597	184.000		575.502	
Wearing Course				241.615				1.150	-0.065	277.857		-15.758	
Bearing Pedestal : Left Span				45.760				-1.150	-0.881	-52.624		-40.320	
Bearing Pedestal : Right Span				45.760				1.150	-0.881	52.624		-40.320	
Seismic Arrestor				45.200				0.000	0.000	0.000		0.000	

Substructure													
Pier Cap				1511.416	0.000	9.632	1455.123	0.000	0.000	0.000	0.00	0.000	43.08
Additional weight of cap				0.000				0.000	0.000	0.000		0.000	
Pier Shaft				319.784	0.000	2.697	1453.427	0.000	0.000	0.000	0.00	0.000	12.06
Footing				1262.866				0.000	0.000	0.000		0.000	
Earth fill on footing				1538.789				0.000	0.000	0.000		0.000	
Wind Load					26.916	231.896	1458.0285				159.103		1711.041
Wind Down from Left Span				176.362				-1.150		-202.817			
Wind down from Right Span				352.725				1.150		405.634			
Wind UP from Left Span				-176.362				-1.150		202.817			
Wind UP from Right Span				-352.725				1.150		-405.634			
Total				Down	9480.682	26.916	244.224			202.817	159.103	1145.126	1766.187
				Up	8422.507					-202.817			

COMB 8: COMB 2 + WL -Wind Dry Case

Since Design Mean wind speed of 39.59 m/s exceed wind speed of 36 m/s, hence no Live load to be considered in wind load case

Hence Design forces will same as Load Combination 5

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	C.g. of Force (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	Long. Moment		Trans. Moment	
										ML = PxL	ML due to FL	MT = PxT	MT due to FT
Total			down	9480.682	26.916	244.224				202.817	159.103	1145.126	1766.187
			UP	8422.507						-202.817			

COMB 9 : DL+SIDL+WL -one Span Dislodge Wind case Left Span Dislodge

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL (kN)	FT (kN)	C.g. of Force (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	Long. Moment		Trans. Moment	
										ML = PxL	ML due to FL	MT = PxT	MT due to FT
Superstructure-Right Span													
Dead Load	1			1689.395				1.150	0.031	1942.804		53.138	
SIDL except Wearing Course	1			160.000				1.150	3.597	184.000		575.502	
Wearing Course	1			241.615				1.150	-0.065	277.857		-15.758	
Bearing Pedestal : Left Span				45.760				-1.150	-0.881	-52.624		-40.320	
Bearing Pedestal : Right Span				45.760				1.150	-0.881	52.624		-40.320	
Seismic Arrestor				45.200				0.000	0.000	0.000		0.000	
Substructure													
Pier Cap	1	25	60.457	1511.416	0.000	9.632	1455.123	0.000	0.000	0.000	0.00	0.000	43.08
Additional weight of cap				0.000				0.000	0.000	0.000		0.000	
Pier Shaft	1	25	12.791	319.784	0.000	2.697	1453.427	0.000	0.000	0.000	0.00	0.000	7.49
Footing	1	25	50.515	1262.866				0.000	0.000	0.000		0.000	
Earth fill on footing	1	20	76.939	1538.789				0.000	0.000	0.000		0.000	
Wind Load	1				120.079	231.896	1458.0285				709.788		1711.041
Wind down from Right Span				352.725				1.150		405.634			
Wind UP from Right Span				-352.725				1.150		-405.634			
Total			Down	7213.309	120.079	244.224				2810.295	709.788	532.243	1761.613
			UP	6507.859						1999.028			

COMB 10: COMB 1 + EQ-L -Seismic Dry CaseSeismic Effect Factor = **1**Volume of pier shaft below GL = **0.000** m³ $\alpha_h = 0.162$ In Longitudinal direction $\alpha_h = 0.140$ In Transverse direction $\alpha_v = 0.108$ In Vertical direction

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = PxeL	MLs due to FL	MLs due to Fv	MT = PxeT	MTs due to FT
Superstructure-Left Span															
Dead Load	1			1689.39		71.21	54.74	1457.77	-1.15	0.03	-1942.80		-62.95	53.14	507.31
SIDL except Wearing Course	1			160.00		6.74	5.18	1458.60	-1.15	3.60	-184.00		-5.96	575.50	53.59
Wearing Course	1			241.62		10.18	7.83	1458.45	-1.15	-0.07	-277.86		-9.00	-15.76	79.39
Superstructure-Right Span															
Dead Load	1			1689.39		71.21	54.74	1457.77	1.15	0.03	1942.80		62.95	53.14	507.31
SIDL except Wearing Course	1			160.00		6.74	5.18	1458.60	1.15	3.60	184.00		5.96	575.50	53.59
Wearing Course	1			241.62		10.18	7.83	1458.45	1.15	-0.07	277.86		9.00	-15.76	79.39
Bearing Pedestal : Left Span	1	25	1.8304	45.76					-1.15	-0.88	-52.62			-40.32	
Bearing Pedestal : Right Span	1	25	1.8304	45.76					1.15	-0.88	52.62			-40.32	
Seismic Arrestor	1	25	1.808	45.20					0.00	0.00	0.00			0.00	
Substructure															
Pier Cap	1	25	60.457	1511.42	244.85	63.71	48.97	1455.12	0.00	0.00	0.00	1095.21	0.00	0.00	284.96
Additional weight of cap	1	25	0.000	0.00					0.00	0.00	0.00			0.00	
Pier Shaft	1	25	12.791	319.78	51.81	13.48	10.36	1453.43	0.00	0.00	0.00	143.86	0.00	0.00	37.43
Footing	1	25	50.515	1262.87	204.58	53.23	40.92	1451.15	0.00	0.00	0.00	102.29		0.00	26.61
Earth fill on footing	1	20	76.939	1538.79					0.00	0.00	0.00			0.00	
Net Horizontal Force at Bearing					677.49	0.00						4004.63			0.00
TOTAL				8951.59	1178.73	306.69	235.75				0.00	5345.99	0.00	1145.13	1629.58
-235.745															
0.000															

COMB 11 : COMB 1 + EQ-T -Seismic Dry Case

COMB 11 : COMB 1 + EQ-T -Seismic Dry Case											Long. Moment			Trans. Moment	
Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = 0.3x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = Px eL	MLs due to FL	MLs due to Fv	MT = Px eT	MTs due to FT
Superstructure-Left Span															
Dead Load	1			1689.39		237.36	54.74	1457.77	-1.15	0.03	-1942.80		-62.95	53.14	1691.03
SIDL except Wearing Course	1			160.00		22.48	5.18	1458.60	-1.15	3.60	-184.00		-5.96	575.50	178.63
Wearing Course	1			241.62		33.95	7.83	1458.45	-1.15	-0.07	-277.86		-9.00	-15.76	264.65
Superstructure-Right Span															
Dead Load	1			1689.39		237.36	54.74	1457.77	1.15	0.03	1942.80		62.95	53.14	1691.03
SIDL except Wearing Course	1			160.00		22.48	5.18	1458.60	1.15	3.60	184.00		5.96	575.50	178.63
Wearing Course	1			241.62		33.95	7.83	1458.45	1.15	-0.07	277.86		9.00	-15.76	264.65

Bearing Pedestal : Left Span				45.76					-1.15	-0.88	-52.62			-40.32	
Bearing Pedestal : Right Span				45.76					1.15	-0.88	52.62			-40.32	
Seismic Arrestor				45.20					0.00	0.00	0.00			0.00	
Substructure															
Pier Cap	1	25	60.457	1511.42	73.45	212.35	48.97	1455.12	0.00	0.00	0.00	328.56	0.00	0.00	949.86
Additional weight of cap				0.00					0.00	0.00	0.00			0.00	
Pier Shaft	1	25	12.791	319.78	15.54	44.93	10.36	1453.43	0.00	0.00	0.00	43.16	0.00	0.00	124.77
Footing	1	25	50.515	1262.87	61.38	177.43	40.92	1451.15	0.00	0.00	0.00	30.69		0.00	88.72
Earth fill on footing	1	20	76.939	1538.79					0.00	0.00	0.00			0.00	
Net Horizontal Force at Bearing					101.62	0.00						600.69			0.00
TOTAL				8951.59	251.99	1022.29	235.75				0.00	1003.10	0.00	1145.13	5431.94
-235.745															
0.000															

COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case**(Maximum CWLL at Fixed End)**

(Maximum SWL at 1 m/s² Int.)											Long. Moment			Trans. Moment	
Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = Px eL	MLs due to FL	MLs due to Fv	MT = Px eT	MTs due to FT
Superstructure-Left Span															
Dead Load	1			1689.39		71.21	54.74	1457.77	-1.15	0.03	-1942.80		-62.95	53.14	507.31
SIDL except Wearing Course	1			160.00		6.74	5.18	1458.60	-1.15	3.60	-184.00		-5.96	575.50	53.59
Wearing Course	1			241.62		10.18	7.83	1458.45	-1.15	-0.07	-277.86		-9.00	-15.76	79.39
FPLL	1			0.00		0.00	0.00	1459.65	-1.15	0.03	0.00		0.00	0.00	0.00
CWLL(Max)	1			0.00		0.00	0.00	1459.65	-1.15	0.00	0.00		0.00	0.00	0.00
Superstructure-Right Span															
Dead Load	1			1689.39		71.21	54.74	1457.77	1.15	0.03	1942.80		62.95	53.14	507.31
SIDL except Wearing Course	1			160.00		6.74	5.18	1458.60	1.15	3.60	184.00		5.96	575.50	53.59
Wearing Course	1			241.62		10.18	7.83	1458.45	1.15	-0.07	277.86		9.00	-15.76	79.39
FPLL	1			0.00		0.00	0.00	1459.65	1.15	0.00	0.00		0.00	0.00	0.00
CWLL(Max)+Vertical Reaction	1			207.94		8.76	6.74	1459.65	1.15		239.13		7.75	190.00	78.85
Bearing Pedestal : Left Span				45.76					-1.15	-0.88	-52.62			-40.32	
Bearing Pedestal : Right Span				45.76					1.15	-0.88	52.62			-40.32	
Seismic Arrestor				45.20					0.00	0.00	0.00			0.00	
Substructure															
Pier Cap	1	25	60.457	1511.42	244.85	63.71	48.97	1455.12	0.00	0.00	0.00	1095.21	0.00	0.00	284.96
Additional weight of cap				0.00					0.00	0.00	0.00			0.00	
Pier Shaft	1	25	12.791	319.78	51.81	13.48	10.36	1452.99	0.00	0.00	0.00	121.04	0.00	0.00	31.49
Footing	1	25	50.515	1262.87	204.58	53.23	40.92	1451.15	0.00	0.00	0.00	102.29		0.00	26.61
Earth fill on footing	1	20	76.939	1538.79					0.00	0.00	0.00			0.00	
Net Horizontal Force at Bearing					723.03	0.00						4273.81			0.00
TOTAL				9159.54	1224.27	315.45	242.48				239.13	5592.36	7.75	1335.13	1702.49
-242.482															
-7.748															

(Minimum CWLL at Fixed End)

(Maximum CWLL at Fixed End)

(Minimum CWLL at Fixed End)

Total Reactions from COMB 13			7665.0	1160.5	292.0	222.8				89.6	5578.9	2.9	1211.6	1649.0
Minus CWLL from COMB 13			-77.9		-3.3	-2.5				-89.6		-2.9	-66.5	-29.6
CWLL for this COMB	1		48.6		2.0	1.6	1459.6	1.2		55.9		1.8	128.3	18.4
TOTAL			7635.6	1160.5	290.7	221.9				55.9	5578.9	1.8	1273.4	1637.9
-221.878 -1.811														

COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case

COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case											Long. Moment			Trans. Moment	
Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = 0.3x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = Px eL	MLs due to FL	MLs due to Fv	MT = Px eT	MTs due to FT
Superstructure-Left Span															
Dead Load	1			1689.4		237.4	54.7	1457.8	-1.2	0.0	-1942.8		-62.9	53.1	1691.0
SIDL except Wearing Course	1			160.0		22.5	5.2	1458.6	-1.2	3.6	-184.0		-6.0	575.5	178.6
Wearing Course	1			241.6		33.9	7.8	1458.4	-1.2	-0.1	-277.9		-9.0	-15.8	264.6
FPLL	1			0.0		0.0	0.0	1459.6	-1.2	0.0	0.0		0.0	0.0	0.0
CWLL(Max)	1			0.0		0.0	0.0	1459.6	-1.2	0.0	0.0		0.0	0.0	0.0
Superstructure-Right Span															
Dead Load	1			1689.4		237.4	54.7	1457.8	1.2	0.0	1942.8		62.9	53.1	1691.0
SIDL except Wearing Course	1			160.0		22.5	5.2	1458.6	1.2	3.6	184.0		6.0	575.5	178.6
Wearing Course	1			241.6		33.9	7.8	1458.4	1.2	-0.1	277.9		9.0	-15.8	264.6
FPLL	1			0.0		0.0	0.0	1459.6	1.2	0.0	0.0		0.0	0.0	0.0
CWLL(Max)	1			207.9		29.2	6.7	1459.6	1.2		239.1		7.7	190.0	262.8
Bearing Pedestal : Left Span				45.8					-1.2	-0.9	-52.6			-40.3	
Bearing Pedestal : Right Span				45.8					1.2	-0.9	52.6			-40.3	
Seismic Arrestor				45.2					0.0	0.0	0.0			0.0	
Substructure															
Pier Cap	1	25	60.457	1511.4	73.5	212.4	49.0	1455.1	0.0	0.0	0.0	328.6	0.0	0.0	949.9
Additional weight of cap				0.0					0.0	0.0	0.0			0.0	
Pier Shaft	1	25	12.791	319.8	15.5	44.9	10.4	1453.0	0.0	0.0	0.0	36.3	0.0	0.0	105.0
Footing	1	25	50.515	1262.9	61.4	177.4	40.9	1451.2	0.0	0.0	0.0	30.7		0.0	88.7
Earth fill on footing				76.939	1538.8				0.0	0.0	0.0			0.0	
Net Horizontal Force at Bearing					248.8	0.0						1470.6			0.0
TOTAL9159.5399.21051.5242.5239.11866.17.71335.15675.0															
-242.5															
-7.7															

COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case

COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case											Long. Moment			Trans. Moment	
Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = 0.3x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = Px eL	MLs due to FL	MLs due to Fv	MT = Px eT	MTs due to FT
Superstructure-Left Span															
Dead Load	1			1689.4		237.4	54.7	1457.8	-1.2	0.0	-1942.8		-62.9	53.1	1691.0
SIDL except Wearing Course	1			160.0		22.5	5.2	1458.6	-1.2	3.6	-184.0		-6.0	575.5	178.6
Wearing Course	1			241.6		33.9	7.8	1458.4	-1.2	-0.1	-277.9		-9.0	-15.8	264.6
FPLL	1			0.0		0.0	0.0	1459.6	-1.2	-0.1	0.0		0.0	0.0	0.0
CWLL(Max)	1			0.0		0.0	0.0	1459.6	-1.2	0.0	0.0		0.0	0.0	0.0
Superstructure-Right Span															
Dead Load	1			1689.4		237.4	54.7	1457.8	1.2	0.0	1942.8		62.9	53.1	1691.0
SIDL except Wearing Course	1			160.0		22.5	5.2	1458.6	1.2	3.6	184.0		6.0	575.5	178.6
Wearing Course	1			241.6		33.9	7.8	1458.4	1.2	-0.1	277.9		9.0	-15.8	264.6
FPLL	1			0.0		0.0	0.0	1459.6	1.2	-0.1	0.0		0.0	0.0	0.0
CWLL(Max)	1			77.9		10.9	2.5	1459.6	1.2		89.6		2.9	66.5	98.5

PIER -P1 @ Ch. 17+785

COMB 21 : DL+SIDL+EQ-T -one Span Dislodge Seismic case											Left Span Dislodge			Long. Moment			Trans. Moment	
Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	C.g. of Force in Trans. (m)	Long. Ecc. (eL) (m)	Transv. Ecc. (eT) (m)	ML = Px eL	MLs due to FL	MLs due to Fv	MT = Px eT	MTs due to FT			
Superstructure-Right Span																		
Dead Load	1			1689.4		164.8	33.0	1457.8	1.2	0.0	1942.8		37.9	53.1	1174.4			
SIDL except Wearing Course	1			160.0		15.6	3.1	1458.6	1.2	3.6	184.0		3.6	575.5	124.1			
Wearing Course	1			241.6		23.6	4.7	1458.4	1.2	-0.1	277.9		5.4	-15.8	183.8			
Bearing Pedestal : Left Span				45.8					-1.2	-0.9	-52.6			-40.3				
Bearing Pedestal : Right Span				45.8					1.2	-0.9	52.6			-40.3				
Seismic Arrestor				45.2					0.0	0.0	0.0			0.0				
Substructure																		
Pier Cap	1	25	60.457	1511.4	61.2	147.5	29.5	1455.1	0.0	0.0	0.0	273.8	0.0	0.0	659.7			
Additional weight of cap				0.0					0.0	0.0	0.0			0.0				
Pier Shaft	1	25	12.791	319.8	13.0	31.2	6.2	1453.0	0.0	0.0	0.0	30.3	0.0	0.0	72.9			
Footing	1	25	50.515	1262.9	51.1	123.2	24.6	1451.2	0.0	0.0	0.0	25.6		0.0	61.6			
Earth fill on footing	1	20	76.939	1538.8					0.0	0.0	0.0			0.0				
Net Horizontal Force at Bearing					125.7	0.0						743.1			0.0			
TOTAL				6860.6	251.0	505.9	101.2				2404.7	1072.8	46.9	532.2	2276.4			
-101.188																		
-46.927																		

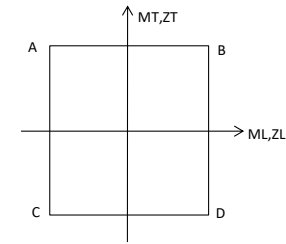
Centrifugal Force (C.F.) =	1.00	x	0.00	=	0.000 KN
Transverse Moment due to C.F. =	0.000	x (	1459.646 - 1450.650)	=	0.000 kNm

Centrifugal Force (C.F.) =	0.20	x	0.00	=	0.000 KN
Transverse Moment due to C.F. =	0.000	x (	1459.646 - 1450.650)	=	0.000 kNm

Properties of Footing Base:

		B		L		
A	=	7.000	x	10.000	=	70.000 m ²
ZL	=	10.000	x	8.167	=	81.667 m ³
ZT	=	7.000	x	16.667	=	116.667 m ³

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

Summary of Forces For Stability of Foundation

			Summary of Forces			Safe Bearing Capacity check						CHECK
CASE	LOAD COMBINATION		P (kN)	ML (kNm)	MT (kNm)	σ_A (kN/m ²)	σ_B (kN/m ²)	σ_C (kN/m ²)	σ_D (kN/m ²)	Max. Base Pressure	Min. Base Pressure	
Normal Case	COMB 1: DL+SIDL-Normal Dry Case		8951.59	0.00	1145.13	137.70	137.70	118.06	118.06	137.70	118.06	SAFE
	COMB 2: DL+SIDL + LL(Max Reaction) -Normal Dry Case		9951.62	2392.52	2095.15	130.83	189.42	94.91	153.50	189.42	94.91	
	COMB 3: DL+SIDL + LL(Min Reaction) -Normal HFL Case		7937.02	1473.91	1481.01	108.03	144.13	82.64	118.74	144.13	82.64	
	COMB 4: DL+SIDL + LL(Max Transverse Moment) -Normal Dry Case		9687.95	2132.65	3343.12	140.94	193.17	83.63	135.86	193.17	83.63	
	COMB 5: DL+SIDL + LL(Max Transverse Moment) -Normal HFL Case		7830.06	1161.09	1790.03	112.98	141.42	82.30	110.73	141.42	82.30	
	COMB 6 : DL+SIDL -one Span Dislodege Normal case		6860.58	3022.66	532.24	65.56	139.58	56.43	130.46	139.58	56.43	
										193.169	56.434	SAFE
Wind Case	COMB 7 : COMB 1 + WL -Normal Dry Case	Down	9480.68	361.92	2911.31	155.96	164.82	106.05	114.92	164.82	106.05	
		Up	8422.51	-43.71	2911.31	145.81	144.74	95.90	94.83	145.81	94.83	
	COMB 8: COMB 2 + WL -Wind Dry Case	Down	9480.68	361.92	2911.31	155.96	164.82	106.05	114.92	164.82	106.05	
		Up	8422.51	-43.71	2911.31	145.81	144.74	95.90	94.83	145.81	94.83	
	COMB 9 : DL+SIDL+WL -one Span Dislodege Wind case	Down	7213.31	3520.08	2293.86	79.61	165.81	40.28	126.49	165.81	40.28	
		Up	6507.86	2708.82	2293.86	79.46	145.80	40.14	106.48	145.80	40.14	
										165.812	40.139	

Seismic Case	COMB 10: COMB 1 + EQ-L -Seismic Dry Case	Down	9187	5346	2775	90	220	42	173	220	42	SAFE
		Up	8716	5346	2775	83	214	35	166	214	35	
	COMB 11 : COMB 1 + EQ-T -Seismic Dry Case	Down	9187	1003	6577	175	200	63	87	200	63	
		Up	8716	1003	6577	169	193	56	80	193	56	
	COMB 12: COMB 2 (Max Reaction) + EQ-L -Seismic Dry Case	Down	9402	5839	3038	89	232	37	180	232	37	
		Up	8917	5824	3038	82	225	30	173	225	30	
	COMB 13: COMB 3 (Min Reaction) + EQ-L -Seismic HFL Case	Down	7888	5671	2861	68	207	19	158	207	19	
		Up	7442	5666	2861	61	200	12	151	200	12	
	COMB 14: COMB 4 (Max Transverse Moment)+ EQ-L -Seismic D	Down	9339	5767	3264	91	232	35	176	232	35	
		Up	8858	5756	3264	84	225	28	169	225	28	
	COMB 15: COMB 5 (Max Transverse Moment)+ EQ-L -Seismic H	Down	7858	5637	2911	68	206	18	156	206	18	
		Up	7414	5633	2911	62	200	12	150	200	12	
	COMB 16: COMB 2 (Max Reaction) + EQ-T -Seismic Dry Case	Down	9402	2113	7010	169	220	48	100	220	48	
		Up	8917	2098	7010	162	213	42	93	213	42	
	COMB 17: COMB 3 (Min Reaction) + EQ-T -Seismic HFL Case	Down	9262	1969	6781	166	215	50	98	215	50	
		Up	8817	1964	6781	160	208	44	92	208	44	
	COMB 18: COMB 4 (Max Transverse Moment) + EQ-T -Seismic	Down	9339	1964	6781	167	216	51	99	216	51	
		Up	8858	2030	6781	160	210	44	93	210	44	
	COMB 19: COMB 5 (Max Transverse Moment) + EQ-T -Seismic	Down	9232	1935	6763	166	214	50	98	214	50	
		Up	8788	1931	6763	160	207	44	91	207	44	
	COMB 20 : DL+SIDL+EQ-L -one Span Dislodege Seismic case	Down	6962	5219	1215	46	174	25	153	174	25	
		Up	6759	5125	1215	44	170	23	149	170	23	
	COMB 21 : DL+SIDL+EQ-T -one Span Dislodege Seismic case	Down	6962	3524	2809	80	167	32	119	167	32	
		Up	6759	3431	2809	79	163	30	114	163	30	
										232.018	11.982	

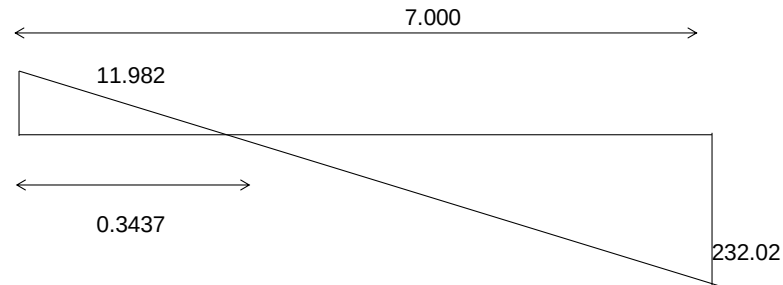
Tension at the base of foundation

As the tension is observed at the base of foundation, Area of base can be reduced to 67%

Minimum Base Pressure = 12.0 kN/m²
Maximum base Pressure = 232.0 kN/m²

67% of actual area = 46.9 m²
Reduced width of footing = 6.656 m
Reduce plan dimensions of footing = 6.656 x 10.00
Reduce area of footing = 66.563 m²
Actual area of footing = 70.000 m²

SAFE

**SECTIONAL PROPERTIES**

Width of Footing (B) = 6.66 m
 Length of Footing (L) = 10.00 m

A = 6.656 x 10.000 = 66.56 m²
 ZL = 10.000 x 7.384 = 73.84 m³
 ZT = IT1 + IT2

distance of extreme point from centre

IT1 = 6.656 x 83.333 = 554.7 m⁴
 IT2 (moment of inertia of triangle) = 6.656 x 0.000 = 0.000 m⁴
 + 0.500 x 6.656 x 0.000 x 25.000 = 0.000 m⁴
 from centre of footing = 0.000 m⁴

Moment of inertia of two triangle = 0.000 m⁴

Total moment of inertia = 554.69 m⁴

Distance of extreme point from centre of foc = 5.000 + 0.000 = 5.000 m

Total Section modulus (ZT) = 110.938 m³

Reduced Properties of Footing Base:

A	=	66.56 m ²
ZL	=	73.84 m ³
ZT	=	110.94 m ³

SAFE BEARING CAPACITY CHECK BASED ON REVISED PLAN AREA

LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure
	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²
COMB 1: DL+SIDL-Normal Dr	8952	0	1145	144.806	144.806	124.162	124.162	144.806
	9952	2393	2095	135.993	200.793	98.222	163.022	200.793
	7937	1474	1481	112.631	152.551	85.931	125.852	152.551
	9688	2133	3343	146.801	204.562	86.530	144.292	204.562
	7830	1161	1790	118.046	149.494	85.775	117.223	149.494
	6861	3023	532	66.934	148.801	57.338	139.205	148.801
								204.562
	9481	362	2911	163.774	173.576	111.289	121.091	173.576
	8423	-44	2911	153.370	152.186	100.884	99.700	153.370
	9481	362	2911	163.774	173.576	111.289	121.091	173.576
	8423	-44	2911	153.370	152.186	100.884	99.700	153.370
	7213	3520	2294	81.376	176.716	40.022	135.362	176.716
	6508	2709	2294	81.764	155.131	40.410	113.777	155.131
	0	0	0					176.716
	9187	5346	2775	90.640	235.434	40.617	185.411	235.434
	8716	5346	2775	83.557	228.350	33.534	178.327	228.350
	9187	1003	6577	183.727	210.896	65.155	92.324	210.896
	8716	1003	6577	176.644	203.812	58.072	85.240	203.812
	9402	5839	3038	89.556	247.708	34.793	192.946	247.708
	8917	5824	3038	82.480	240.213	27.717	185.450	240.213

7888	5671	2861	67.484	221.092	15.912	169.520	221.092
7442	5666	2861	60.867	214.318	9.296	162.746	214.318
9339	5767	3264	91.633	247.834	32.785	188.987	247.834
8858	5756	3264	84.554	240.459	25.707	181.611	240.459
7858	5637	2911	67.958	220.622	15.472	168.137	220.622
7414	5633	2911	61.340	213.906	8.854	161.421	213.906
9402	2113	7010	175.825	233.055	49.446	106.676	233.055
8917	2098	7010	168.749	225.560	42.370	99.180	225.560
9262	1969	6781	173.609	226.948	51.361	104.700	226.948
8817	1964	6781	166.993	220.175	44.744	97.926	220.175
9339	1964	6781	174.843	228.025	52.594	105.776	228.025
8858	2030	6781	166.716	221.698	44.468	99.450	221.698
9232	1935	6763	173.461	225.857	51.542	103.938	225.857
8788	1931	6763	166.843	219.142	44.924	97.222	219.142
6962	5219	1215	44.867	186.220	22.960	164.313	186.220
6759	5125	1215	43.098	181.909	21.190	160.001	181.909
6962	3524	2809	82.180	177.635	31.544	127.000	177.635
6759	3431	2809	80.410	173.324	29.775	122.689	173.324
							247.834

Stability Check Summary

Load Case		Base Pressure required	Permissible Base Pressure	Remarks
Normal Case	P_{max} (kN/m ²)	193	349	SAFE
	P_{min} (kN/m ²)	56	0	SAFE
Wind Case	P_{max} (kN/m ²)	166	436	SAFE
	P_{min} (kN/m ²)	40	0	SAFE
Seismic Case	P_{max} (kN/m ²)	232	436	SAFE
	P_{min} (kN/m ²)	12	0	SAFE

Footing Reinforcement

Type of reinforcement	Area of steel required	Area of steel provided						
Along Traffic Direction								
Main steel at bottom face	3522	25	mm bar @	200	mm c/c (i.e.)	4025	mm ²	(O.K)
		20	mm bar @	200	mm c/c (i.e.)			
secondary Steel at top face	805	16	mm bar @	200	mm c/c (i.e.)	1005	mm ²	(O.K)
Shear Reinforcement	Provide Shear Reinf.	12	mm 18 Legged bar @	200	mm c/c (i.e.)			SAFE
Across Traffic Direction								
Main steel at bottom face	2960	20	mm bar @	200	mm c/c (i.e.)	3142	mm ²	(O.K)
		20	mm bar @	200	mm c/c (i.e.)			
secondary Steel at top face	592	12	mm bar @	150	mm c/c (i.e.)	754	mm ²	(O.K)
Shear Reinforcement	Provide Shear Reinf.	12	mm 12 Legged bar @	200	mm c/c (i.e.)			SAFE

DESIGN OF ABUTMENT(A1) WITH
OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
18+016

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 50 m
Design speed of vehicle = 0 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 18.750m	18.750m
Thickness of Expansion Joint	= 0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side) =	0.600m	0.600m
Slab projection Beyond C/L of Bearing (Span Side) =	0.600m	0.600m
Span -c/c of E.J.	= 20.000m	20.00m
Type of Superstructure	= Precast RCC Girder	
Width of Crash barrier (Both Side)	= 0.450m	
Width of Carriageway	= 11.000m	
Projection beyond crash barrier	= 0.050m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on one side	= 0.000m	
Railing/kerb on footpath edge	= 0.250m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 4
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 3.000m (Right) 3.000 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 320 kN/m² (as per geotechnical report)
SBC of soil-Normal Case = 353 kN/m²
SBC of soil-Seismic Case = 441 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1.5 Normal Case
= 1.25 Seismic Case

Permissible FOS against Overturning = 2 Normal Case
= 1.5 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 2.120m	
Height of Trapering portion	= 0.337m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 1.850m	1.850m
Width of Abutmentcap at bottom of Tapering Portion	= 1.000m	1.000m
Projection of Abutment Cap (Span Side)	= 0.400m	0.400m
Projection of Abutment Cap Back Side	= 0.450m	0.450m
Abutmentcap thickness (Uniform portion)	= 0.300m	

Abutment cap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	1.127m	1.127m
Thickness of Abutment shaft at Bottom	=	1.200m	1.200m

Solid Return Wall

Length of Return wall	=	5.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	9.000m
abutment pedestal width	=	1.200m
abutment pedestal Height	=	1.000m
Width of Toe Slab	=	2.800m
Width of Heel Slab	=	5.000m
Thickness of Toe slab at tip	=	0.500m
Thickness of Toe slab near shaft	=	1.200m
Thickness of heel slab at tip	=	0.500m
Thickness of heel slab near shaft	=	1.200m
Width of backfill on heel slab	=	5.000m
Thickness of heel slab at back fill edge	=	1.200m
Height of back fill at bottom edge of heel slab	=	9.572m
Height of back fill at back fill edge of heel slab	=	8.872m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1453.920m	Cross Slope (Bi-directional)	=	2.500%	
Deck level at Outer Edge =	1454.064m	Height of Superstructure	=	1.820m	
Deck level at center line =	1453.920m	Min. Height of Footpath Side Pedestal (1)	=	0.300m	
Soffit Level at center of bridge =	1452.035m	Height of Pedestal (2)	=	0.375m	
Abutment cap top level =	1451.535m	Height of Pedestal (3)	=	0.450m	
Abutment cap bottom lvl (uniform portion ends)	1451.235m	Height of Pedestal (4)	=	0.525m	
Abutment cap bottom lvl (corbel portion ends)	1450.935m	Height of Pedestal (5)	=	0.000m	
Abutment shaft top level =	1450.935m	Height of Pedestal (6)	=	0.000m	
Ground level/LBL =	1447.237m	Distance of nearest girder to c.l. of deck	=	1.500m	
Abutment shaft bottom level =	1445.120m	Height (Avg.) of Dirt Wall	=	2.457m	
Foundation level =	1443.920m	Abutment shaft Above G.L	=	3.698m	
HFL	=	1447.237m	Abutment Shaft below G.L	=	2.117m
			Height of abutment shaft	=	5.815m
			MSL	=	1447.237m
			Wedge over girder flange	=	0.0250m

Material Specification

Concrete Grade	=	M 35
Characteristic compressive strength of concrete, f _{ck}	=	35.00 Mpa at 28 days
Design Compressive strength of Concrete, f _{cd}	=	15.63 Mpa at 28 (0.67/1.5 * f _{ck})
Tensile strength of concrete , f _{ctm}	=	2.77 MPa

Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E_{cm}	=	32308.250	N/mm ²
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f_y or f_{yk}	=	500	Mpa
Design Yield Strength of Reinforcement, f_{yd}	=	434.78	Mpa (1/1.15 * f_y)
Modulus of Elasticity of Steel (E_s)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m ³
Dry unit weight of soil	=	20	kN/m ³
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48	f_{ck}
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm
<u>Seismic Data:</u>			
Seismic Zone	=	4	
Z = Zone factor	=	0.24	
I = Importance factor	=	1.2	
R = Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction
<u>Properties of backfill material :</u>			
c	=	0	
ϕ	=	35	
θ	=	90	
β	=	0	
δ	=	22.5	

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.000m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

		eT
DL Reaction on Bearing No.-1	=	450.94 4.50
DL Reaction on Bearing No.-2	=	432.46 1.50
DL Reaction on Bearing No.-3	=	432.46 -1.50
DL Reaction on Bearing No.-4	=	450.94 -4.50
DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		1766.80 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	135.917 4.50
SIDL Reaction on Bearing No.-2	=	-55.917 1.50
SIDL Reaction on Bearing No.-3	=	-55.917 -1.50
SIDL Reaction on Bearing No.-4	=	135.917 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		160.00 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	60.954 4.50
SIDL Reaction on Bearing No.-2	=	66.495 1.50
SIDL Reaction on Bearing No.-3	=	66.495 -1.50
SIDL Reaction on Bearing No.-4	=	60.954 -4.50
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		254.90 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:
(1-70RW + 2-CLASS A)**

Reduction Factor = 0.9 (for 3 Lane)

max	=	981.82	KN	Corr. Transv	=	932.730 kNm	ecc.	0.95
min	=	368.18	KN	Corr. Transv	=	349.770 kNm	ecc.	0.95

SV Loading

max	=	2003.04	KN	Corr. Transv	=	600.912 kNm	ecc.	0.30
min	=	1416.96	KN	Corr. Transv	=	425.088 kNm	ecc.	0.30

**MAXIMUM TRASVERSE MOMENT CASE:
(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	758.73	KN	Corr. Transv	=	2393.795 kNm	ecc.	3.16
min	=	241.27	KN	Corr. Transv	=	761.205 kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.091
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.300 x	0.800 x	0.800 =	0.1920 m ³	4.500
Pedestal 2	=	0.375 x	0.800 x	0.800 =	0.2400 m ³	1.500
Pedestal 3	=	0.450 x	0.800 x	0.800 =	0.2880 m ³	-1.500
Pedestal 4	=	0.525 x	0.800 x	0.800 =	0.3360 m ³	-4.500
Pedestal 5	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					1.0560 m ³	-0.682

VOLUME CALCULATION

C.G. Of Footing	=	4.500 m
C.G. Of shaft from toe tip	=	3.400 m
Distance between c.g. of shaft and footing	=	1.100 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	2.120	7.950	-0.800	0.300	-4.200	0.000
-Trapering portion	1	12.50	0.300	0.337	1.263	-0.800	0.300	-4.200	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.100	0.000	-4.500	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-1.050	0.050	-4.450	0.000
Cap (uniform portion)	1	12.50	1.850	0.300	6.938	-0.025	1.075	-3.425	0.000
Cap (Corbel Portion)	1	12.50	1.850	0.300	5.263	-0.025	1.075	-3.425	0.000
		12.50	1.000						
Shaft above HFL	1	12.50	1.064	3.698	49.165	0.068	1.168	-3.332	0.000
Shaft below HFL	1	12.50	1.164	1.617	23.519	0.018	1.118	-3.382	0.000
Solid Return Wall	1	5.00	0.500	9.294	23.234	-3.100	-2.000	-6.500	6.000
Cantilever Return wall(Rectangular portion)	1	0.00	0.500	0.000	0.000	-0.600	0.500	-4.000	6.000
Cantilever Return wall(Traingular portion)	1	0.00	0.500	0.000	0.000	-0.600	0.500	-4.000	6.000
Footing									
Heel Slab	1	12.50	5.000	0.850	53.125		-1.657	-6.157	0.000
Toe Slab	1	12.50	2.800	0.850	29.750		2.908	-1.592	0.000
Portion between Heel and Toe	1	12.50	1.200	1.200	18.000		1.100	-3.400	0.000
Back filling above HFL over Heel Slab	1	12.50	5.000	6.683	417.688		-2.000	-6.500	0.000
Back filling below HFL over Heel Slab	1	12.50	5.000	2.467	154.188		-2.118	-6.618	0.000
Backfill above Heel slab	1	12.50	5.000	9.222	576.367		-2.032	-6.532	0.000
Front Filling over Toe Slab	1	12.50	2.800	2.467	86.345		3.034	-1.466	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	2.800	2.467	86.345		1.934	-2.566	0.000
Side filling between heel and toe	1	0.00	1.200	2.467	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-1.100	0.000	-4.500	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.127	6.683	10.625		0.573	-3.927	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.073	2.467	2.245		0.524	-3.976	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barr	2	8 kN/m	1.750	28.00kN		-0.800	0.300	-4.200

SECTIONAL PROPERTIES

Width of Footing (B)	=	9.000 m
Length of Footing (L)	=	12.500 m

A	=	9.000	x	12.500	=	112.500 m ²
ZL	=	12.500	x	13.500	=	168.750 m ³
ZT	=	IT1	+	IT2		
		distance of extreme point from centre				
IT1	=	9.000	x	162.760	=	1464.84 m ⁴
IT2 (moment of inertia of triangle)	=	9.000	x	0.000	+	0.500 x 9.000 x 0.000 x 39.063
from centre of footing	=	0.000	m ⁴			
Moment of inertia of two triangle	=	0.000	m ⁴			
Total moment of inertia	=	1464.84	m ⁴			
Distance of extreme point from centre of footing	=	6.250	+	0.000	=	6.250 m
Total Section modulus (ZT)	=	234.375	m ³			

Load Factors (As per IRC:6-2017)**Table B.1 Partial Safety Factor For Verification of Equilibrium**

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.100	0.900	1.100	0.900
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.000	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table B.3 Partial Safety Factor For Verification of Serviceability Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.00	1.00	1.00
wearing course	1.20	1.20	1.20
Back Filling Weight	1.00	1.00	1.00
Shrinkage Creep Effect	1.00	1.00	1.00
Earth Pressure due to back filling	1.00	1.00	1.00
CWLL and Associate load and FPLL	1.00	0.75	0.00
Live Load Surcharge	0.80	0.00	0.00

Table B.4 Partial Safety Factor For Design of Foundation

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Seismic Coefficient Calculation**(As Per IRC:6-2017 , Clause 219)**

Horizontal Seismic Force For Zone 4.0

F_{eq} = Seismic forces to be resisted
 F_{eq} = $A_h \times (\text{Dead load} + \text{Appropriate Live load})$
 A_h = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \frac{S_a}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.) or horizontal vibrations.

$$= 2.0 \frac{D^{1/2}}{1000F}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction

= 7.915 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction

= 9.505 m

Abutment Cap Top Level - Foundation Level

= 7.615 m

Dimensions of Abutment Shaft

Length = 12.50 m

Width = 1.10 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 1.040 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 179.036 m^4

E_{cm} = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 215.506 KN
 D = 2181.70 KN
 T = 0.2012 sec

Transverse Direction

Force = 28636.508 KN
 D = 2451.697 KN
 T = 0.0185 sec

For Hard or Rocky Strata, with $N > 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.278

For Medium Strata, with $N > 10 \& N < 30$

$\frac{S_a}{g}$ = 2.500

$\frac{S_a}{g}$ = 1.278

For Soft Soil Strata, with $N < 10$

$\frac{S_a}{g}$ = 2.5

$\frac{S_a}{g}$ = 1.278

Medium Soil Strata

S_a/g = 2.5

S_a/g = 1.278

Seismic Coeff. In Longitudinal Direction = 0.12

Seismic Coeff. In Transverse Direction = 0.184

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.120	In Longitudinal direction
Ah	=	0.184	In Transverse direction
Av	=	0.123	In Vertical direction

For Design of Foundation

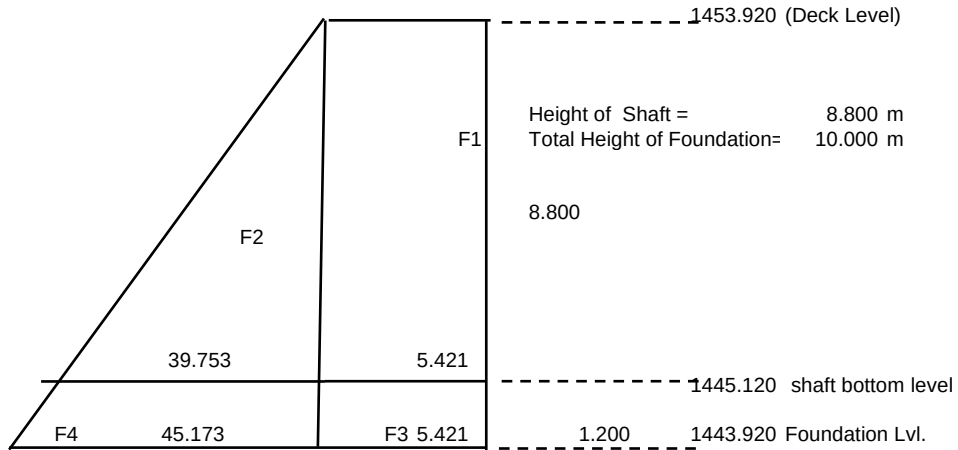
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.162	In Longitudinal direction
Ah	=	0.184	In Transverse direction
Av	=	0.123	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	22.5 degree	0.393 radians
	Kah	=	0.226 active component	
	Kph	=	5.249 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = 1.2 m
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1445.120 m (at Shaft Base)			
@ RL					1443.920 m (at Foundation Level)			
<u>Due to Live Load Surcharge</u>								
Intensity for rectangular portion	=	0.226	x	20	x	1.2	=	5.421 kN/m^2
F1	=	5.421	x	8.800	x	12.500	=	596.289 kN
M1	=	596.29	x	4.40	=	2623.671 kN.m		at Shaft Bottom
F3	=	5.421	x	10.000	x	12.500	=	677.601 kN
M3	=	677.601	x	5.000	=	3388.004 kN.m		at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	8.800	= 39.753 kN/m ²
F2	=	0.5	x	39.75	x	8.800	x 12.50
	=	2186.392 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2017)

M2	=	2186.39	x	3.70	=	8080.905 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	10.000	= 45.173 kN/m ²
F4	=	0.5	x	45.17	x	10.000	x 12.50
	=	2823.337 kN					
M4	=	2823.34	x	4.20	=	11858.015 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m ³	
Intensity for triangular portion (At Shaft bottom level)							
	=	4.800	x	8.800	=	42.240 kN/m ²	
F	=	0.5	x	42.240	x	8.800	x 12.500
	=	2323.200 kN					

$$M = 2323.20 \times 2.933 = 6814.720 \text{ kN.m at Shaft Bottom}$$

Intensity for triangular portion (At Foundation level)

$$= 4.800 \times 10.000 = 48.00 \text{ kN/m}^2$$

$$F = 0.5 \times 48.000 \times 10.00 \times 12.500 = 3000.000 \text{ kN}$$

$$M = 3000.00 \times 3.333 = 10000.000 \text{ kN.m at Foundation}$$

Intensity of Passive pressure

$$= 5.249 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

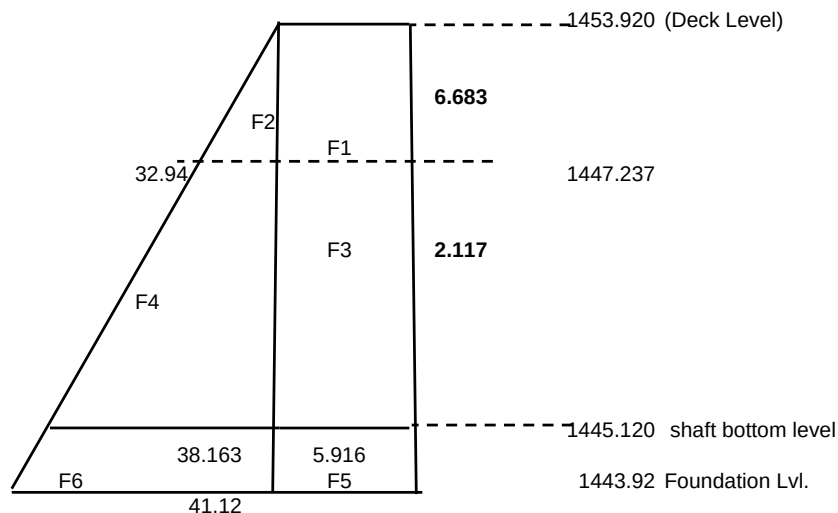
$$= 0.000 \times 0.000 = 0.000 \text{ kN.m at Foundation}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Lvl	At Shaft Bottom Lvl	At Foundation Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure	8080.905	11858.015	2186.392	2823.337
Due to Minimum Fluid Pressure	6814.720	10000.000	2323.200	3000.000
Governing of Two	8080.905	11858.015	2323.200	3000.000
Due to Live Load Surcharge	2623.671	3388.004	596.289	677.601
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	35 degree	0.611 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	11.3 degree	0.196 radians
	Ka'	=	0.246 active component	
	Kp'	=	5.249 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	
Assuming				

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1445.1 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m ²
F1	=	5.916	x	8.800	x	12.500	=	650.708 kN
M1	=	650.71	x	4.40	=	2863.115 kN.m		at Shaft Bottom
F3	=	5.916	x	10.000	x	12.500	=	739.441 kN
M3	=	739.44	x	5.00	=	3697.204 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	6.683	=	32.945 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	2.117	=	5.218 kN/m ²
F2	=	0.5	x	32.94	x	6.683	x	12.50
	=	1376.053 kN						
F4	=	$\left(\frac{32.94 + 38.16}{2} \right) \times$			2.12	x	12.50	
	=	940.836 kN						
Total Force	=	2316.888 kN						
M2	=	1376.05	x	4.92	=	6775.491 kN.m		
M4	=	940.84	x	1.03	=	971.515 kN.m		
Total Moment	=	7747.01 kN.m		at Shaft Bottom				

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 6.683 = 32.945 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 3.317 = 8.176 \text{ kN/m}^2$$

$$F2 = 0.5 \times 32.94 \times 6.683 \times 12.50 = 1376.053 \text{ kN}$$

$$F6 = \left(\frac{32.94 + 41.12}{2} \right) \times 3.32 \times 12.50 = 1535.457 \text{ kN}$$

$$\text{Total Force} = 2911.510 \text{ kN}$$

$$M2 = 1376.05 \times 6.12 = 8426.754 \text{ kN.m}$$

$$M6 = 1535.46 \times 1.60 = 2452.854 \text{ kN.m}$$

$$\text{Total Moment} = 10879.61 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

$$= 5.249 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2$$

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 12.50 = 0.000 \text{ kN}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = 0.000 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	7747.006	10879.608	2316.888	2911.510
Due to Minimum Fluid Pressure	6814.720	10000.000	2323.200	3000.000
Governing of Two	7747.006	10879.608	2323.200	3000.000
Due to Live Load Surcharge	2863.115	3697.204	650.708	739.441
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

αh	=	0.120	
αv	=	0.123	
ϕ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

αh	=	HORIZONTAL SEISMIC COEFFICIENT
αv	=	VERTICAL SEISMIC COEFFICIENT
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.106 \\ 0.136 \end{matrix}$$

$$C_a = \begin{matrix} 1 \\ 0.322 \end{matrix} \begin{matrix} 2 \\ 0.269 \end{matrix}$$

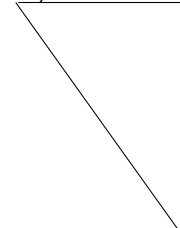
Ca	=	0.322	
Ka	=	0.226	
Dynamic Increment	=	0.322	-0.226 0.096

3 Earth Pressure : DRY CASE (Seismic case)

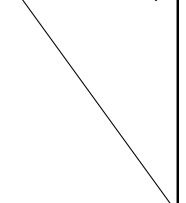
Equivalent Live Load Surcharge height	=	1.2 m
Assuming	γ_{dry}	= 20 kN/m ³
	γ_{water}	= 10.00 kN/m ³

Active Earth Pressure

$$3 (Ca - Ka) = 0.289$$

**Surcharge Earth Pressure**

$$2 (Ca - Ka) \frac{\cos a}{\cos(a-b)} = 0.19$$

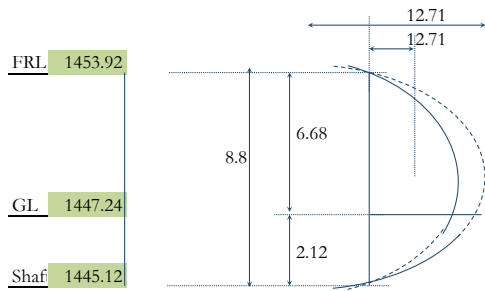


----- 1445.120 (Shaft bottom level)

----- 1443.920 (Foundation Bottom level)

Earth Pressure Diagram for Dynamic Increment**Due to Dynamic Live Load Surcharge**

	=	0.193	x	20.00	x	1.2	=	4.620 kN/m ²
at Shaft Bottom Level								
E1	=	0.50	x	4.620	x	8.800	x	12.500
	=	254.110	kN					
M1	=	254.110	x	5.896			=	1498.233 kN.m
at Foundation Bottom Level								
E2	=	0.50	x	4.620	x	8.800	x	12.500
	=	254.110	kN					
M2	=	254.110	x	7.096			=	1803.165 kN.m

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	12.71 t/m ²
h	=	8.8 m
y	=	2.283 m
L	=	12.500 m

Parabola below Water Level

p_mid_height	=	12.71 t/m ²
h	=	8.8 m
y	=	-2.283 m
L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	795.9	4.9
Parabola below Water Level	135.8	1.4
Total Dynamic Earth Pressure	931.7	4.4

$$E3 = 931.737 \text{ kN}$$

$$E4 = 931.737 \text{ kN}$$

$$M3 = 931.74 \times 4.40 = 4099.642 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 931.74 \times 5.60 = 5217.726 \text{ kN.m (Foundation Bottom level)}$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	8080.905	11858.015	2186.392	2823.337
Due to active Earth Pressure (dynamic Increment)	4099.642	5217.726	931.737	931.737
Total Earth Pressure	12180.547	17075.741	3118.129	3755.074
Due to Minimum Fluid Pressure	6814.720	10000.000	2323.200	3000.000
Governing of Two	8080.905	17075.741	3118.129	3755.074
Due to Live Load Surcharge (Static)	2623.671	3388.004	596.289	677.601
Due to Live Load Surcharge(Dynamic)	1498.233	1803.165	254.110	254.110
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

αh	=	0.120			
αv	=	0.123			
ϕ	=	35.00		0.611	
δ	=	11.25		0.196	
α	=	0.00		0.000	
β	=	0.00		0.000	
Ws	=	20.00	Kn/m3	2.00	gm/cm3
λ	=	$\tan^{-1} \frac{Ws}{Ws-1}$		$\frac{\alpha h}{(1 \pm \alpha v)}$	=
					0.211
					0.267
C_a'	=	1	2		
		0.431	0.382		
Ca'	=		0.431		
Ka'	=		0.246		
Dynamic Increment	=	0.431	-0.246	0.184	
Dynamic Increment	=			0.184	
γ_d	=			20	kN/m ³
γ_{sat}	=			10	kN/m ³
γ_{water}	=			10	kN/m ³
Equivalent Live Load Surcharge height	=			1.2	m
Foundation	=			9.00	m
Dynamic Active earth pressure Variation					
$3 (ca'-ka')$					
Dynamic Surcharge earth pressure Variation					
$2 (Ca'-Ka') \cos a$					1453.920 (Deck Level)
$\cos(a-b)$					
				6.683	
					1447.237 HFL
				2.117	
					1445.120 shaft bottom level
					1443.92 Foundation Bottom Lvl.

Earth Pressure Diagram

h'	=	2.117
h	=	8.800

Dynamic active earth pressure

$3 (ca'-ka')$	=	0.289	at deck level
$3 (ca'-ka')$	=	0.553	at deck level
$3 (ca'-ka')h'/h$	=	0.133	at HFL level

Dynamic Surcharge earth pressure

$2 (Ca'-Ka') \frac{\cos a}{\cos(a-b)}$	=	0.369	at deck level	Pressure	=	8.85	kN/m ²
	=	0.089	at HFL level	Pressure	=	2.13	kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	4.620	kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	2.13	kN/m ²

at Shaft Bottom Level

F1	=	3.374	x	6.683	x	12.500	=	281.897	kN
M1	=	281.90	x	5.46	=	1538.735	kN.m		
F2	=	0.50	x	2.129	x	2.117	x	12.500	
	=	28.167	kN						

$$M2 = 28.17 \times 1.42 = 39.952 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 310.064 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 1578.686 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

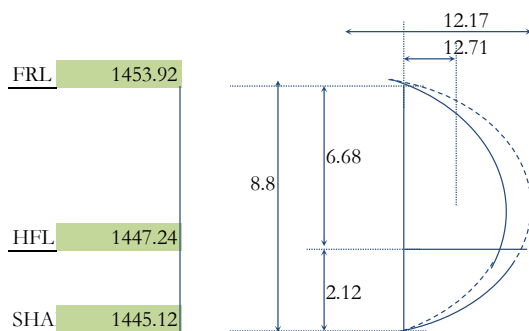
$$F3 = 3.374 \times 6.683 \times 12.500 = 281.897 \text{ kN}$$

$$M3 = 281.90 \times 6.66 = 1877.011 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 2.129 \times 12.500 \\ &= 28.167 \text{ kN} \end{aligned}$$

$$M4 = 28.17 \times 2.62 = 73.752 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 310.064 \text{ kN} \\ \text{Total Moment at Founding level} &= 1950.763 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 12.71 \text{ t/m}^2 \\ h &= 8.8 \text{ m} \\ y &= 2.283 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 12.17 \text{ t/m}^2 \\ h &= 8.8 \text{ m} \\ y &= -2.283 \text{ m} \\ L &= 12.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	795.9	4.9
Parabola below Water Level	130.1	1.4
Total Dynamic Earth Pressure	926.0	4.4

Total Force (F1 + F2)	=	925.986 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	925.986 kN	at Foundation Bottom Level

$$M1 = 925.99 \times 4.42 = 4091.719 \text{ kN.m}$$

$$\text{Total Mome} = 4091.719 \text{ kN.m} \text{ at Shaft Bottom}$$

$$M1 = 925.99 \times 5.62 = 5202.901 \text{ kN.m}$$

$$\text{Total Mome} = 5202.901 \text{ kN.m} \text{ at Foundation Bottom Level}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom	At Foundation Bottom	At Shaft Bottom Lvl	At Foundation Bottom Lvl
	kN-m	kN-m	kN	kN
Due to active Earth Pressure(Static)	7747.006	10879.608	2316.888	2911.510
Due to active Earth Pressure (Dynamic Increment)	4091.719	5202.901	925.986	925.986
Total Earth Pressure	11838.725	16082.509	3242.874	3837.495
Due to Minimum Fluid Pressure	6814.720	10000.000	2323.200	3000.000
Governing of Two	11838.725	16082.509	3242.874	3837.495
Due to Live Load Surcharge(Static)	2863.115	3697.204	650.708	739.441
Due to Live Load Surcharge (Dynamic Increment)	1578.686	1950.763	310.064	310.064
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2017)

Type of bearing - SPHERICAL BEARING

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1766.80	1.35	1.35	2385.18	2385.18	
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00	
WC	=	254.90	1.75	1.75	446.07	446.07	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	981.82	1.5	0.20	1472.73	196.36	(1-70RW + 2-CLASS A)
CWLLmax-Reaction case	=	2003.04	1	0.20	2003.04	400.61	SV Loading
CWLLmin	=	368.18	1.5	0.20	552.27	73.64	(1-70RW + 2-CLASS A)
CWLLmin	=	1416.96	1	0.20	1416.96	283.39	SV Loading
CWLLmax-Transv. Moment Case		758.73	1.5	0.20	1138.10	151.75	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 1108 = 255.4 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	3047.25	0	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.1	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.1	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.1	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	785.41	152.362	545.068	633.048	785.411	
DL+SIDL+LL-Max Reaction case		3120.89	836.49	156.044	574.290	680.447	836.491	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	836.49	162.181	580.426	674.310	836.491	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3199.00	836.49	159.950	578.195	676.541	836.491	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	235.623	152.362	270.174	83.261	270.174	
DL+SIDL+LL-Max Reaction case		3120.89	286.703	156.044	299.396	130.659	299.396	Dry Case
DL+SIDL+LL-Min Reaction case		3243.61	286.703	162.181	305.532	124.523	305.532	HFL Case

DL+SIDL+LL-Max Transv. Moment case		3199.00	286.703	159.950	303.301	126.753	303.301

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.35	1.35	2385.18	2385.18
SIDL except wc	=	160.00	1.35	1.35	216.00	216.00
WC	=	254.90	1.75	1.75	446.07	446.07
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	981.82	1.5	0.75	1472.73	736.37
CWLLmax-Transv. Moment Case	=	758.73	1.5	0.75	1138.10	569.05
CWLLmin	=	368.18	1.5	0.75	552.27	276.13

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	3047.25	0.000	152.362	152.362	-152.362	152.362	
DL+SIDL+LL-Max Reaction case	3599.52	383.100	179.976	371.526	203.124	371.526	(1-70RW + 2-CLASS A)
	5016.48	0.000	250.824	250.824	-250.824	250.824	SV Loading
DL+SIDL+LL-Min Reaction case	4519.98	383.100	225.999	417.549	157.101	417.549	(1-70RW + 2-CLASS A)
	5050.29	0.000	252.514	252.514	-252.514	252.514	SV Loading
DL+SIDL+LL-Max Transv. Moment case	4185.35	383.100	209.267	400.817	173.833	400.817	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	1060.30	152.362	682.515	907.942	1060.305	
DL+SIDL+LL-Max Reaction case		3323.38	1111.38	166.169	721.862	945.216	1111.385	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	1111.38	189.181	744.873	922.204	1111.385	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	1111.38	180.815	736.507	930.570	1111.385	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2181.70	3047.25	318.091	152.362	311.408	165.729	318.091	
DL+SIDL+LL-Max Reaction case		3323.38	369.171	166.169	350.755	203.002	369.171	Dry Case
DL+SIDL+LL-Min Reaction case		3783.62	369.171	189.181	373.766	179.991	373.766	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3616.30	369.171	180.815	365.401	188.357	369.171	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1	1.00	1766.80	1766.80
SIDL except wc	=	160.00	1	1.00	160.00	160.00
WC	=	254.90	1	1.00	254.90	254.90
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1	0.20	981.82	196.36
CWLLmax- Transv. Moment Case		758.73	1	0.20	758.73	151.75
CWLLmin	=	368.18	1	0.20	368.18	73.64

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	0.000	109.085	109.085	-109.085	109.085	
DL+SIDL+LL-Max Reaction case	2549.88	255.400	127.494	255.194	127.906	255.194	(1-70RW + 2-CLASS A)
	3598.66	0.000	179.933	179.933	-179.933	179.933	SV Loading
DL+SIDL+LL-Min Reaction case	3163.52	255.400	158.176	285.876	97.224	285.876	(1-70RW + 2-CLASS A)
	4184.74	0.000	209.237	209.237	-209.237	209.237	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2940.43	255.400	147.021	274.721	108.379	274.721	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	706.870	109.085	462.520	597.785	706.870	
DL+SIDL+LL-Max Reaction case	2255.33	757.950	112.767	491.742	645.183	757.950	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	757.950	118.903	497.878	639.047	757.950	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	757.950	116.672	495.647	641.278	757.950	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	212.061	109.085	215.115	102.976	215.115	
DL+SIDL+LL-Max Reaction case	2255.33	263.141	112.767	244.337	150.374	263.141	Dry Case
DL+SIDL+LL-Min Reaction case	2378.06	263.141	118.903	250.474	144.238	263.141	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2333.44	263.141	116.672	248.243	146.469	263.141	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - POT-PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1766.80	1.1	1.10	1943.48	1943.48
SIDL except wc	=	160.00	1.1	1.10	176.00	176.00
WC	=	254.90	1.35	1.35	344.11	344.11
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1.5	0.00	1472.73	0.00
CWLLmax- Transv. Moment Case		758.73	1.5	0.00	1138.10	0.00
CWLLmin	=	368.18	1.5	0.00	552.27	0.00

Braking Force = 255.4 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2463.59	0.000	123.180	123.180	-123.180	123.180	
DL+SIDL+LL-Max Reaction case	3015.86	383.100	150.793	342.343	232.307	342.343	(1-70RW + 2-CLASS A)
	3880.55	0.000	194.028	194.028	-194.028	194.028	SV Loading
DL+SIDL+LL-Min Reaction case	3936.32	383.100	196.816	388.366	186.284	388.366	(1-70RW + 2-CLASS A)
	4466.63	0.000	223.332	223.332	-223.332	223.332	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3601.69	383.100	180.084	371.634	203.016	371.634	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	1060.30	123.180	653.332	937.125	1060.305	
DL+SIDL+LL-Max Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	1060.30	123.180	653.332	937.125	1060.305	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	1060.30	123.180	653.332	937.125	1060.305	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2181.70	2463.59	318.091	123.180	282.225	194.912	318.091	
DL+SIDL+LL-Max Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	Dry Case
DL+SIDL+LL-Min Reaction case		2463.59	318.091	123.180	282.225	194.912	318.091	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2463.59	318.091	123.180	282.225	194.912	318.091	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1766.80	1	1	1	1766.80	1766.80	1766.80
SIDL except wc	=	160.00	1	1	1	160.00	160.00	160.00
WC	=	254.90	1.20	1.20	1.20	305.88	305.88	305.88
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax- Reaction case	=	981.82	1	0.75	0	981.82	736.37	0.00
CWLLmax- Transv. Moment Case	=	758.73	1	0.75	0	758.73	569.05	0.00
CWLLmin	=	368.18	1	0.75	0	368.18	276.13	0.00

Braking Force = 255.4 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2600.86	255.400	130.043	257.743	125.357	257.743	(1-70RW + 2- CLASS A)
	3649.64	0.000	182.482	182.482	-182.482	182.482	SV Loading
DL+SIDL+LL-Min Reaction case	3214.50	255.400	160.725	288.425	94.675	288.425	(1-70RW + 2- CLASS A)
	4235.72	0.000	211.786	211.786	-211.786	211.786	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2991.41	255.400	149.570	277.270	105.830	277.270	

Dry Case

HFL Case

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2508.81	191.550	125.441	221.216	66.109	221.216	Dry Case
DL+SIDL+LL-Min Reaction case	2969.04	191.550	148.452	244.227	43.098	244.227	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2801.72	191.550	140.086	235.861	51.464	235.861	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2232.68	0.000	111.634	111.634	-111.634	111.634	
DL+SIDL+LL-Max Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	Dry Case
DL+SIDL+LL-Min Reaction case	2232.68	0.000	111.634	111.634	-111.634	111.634	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2232.68	0.000	111.634	111.634	-111.634	111.634	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

$$\text{CENTRIFUGAL FORCE } C = \frac{W V^2}{127 R}$$

Normal Case**Seismic Case**

Design Speed	V	=	0.00	kmph	0.00	kmph
Live Load	W	=	981.82	kN	981.82	kN
Radius of Curvature	R	=	50.00	m	50.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

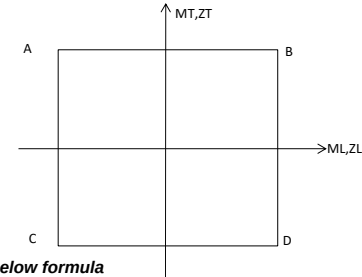
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1443.920 m

Properties of Footing Base:

A	=	112.500	m ²
ZL	=	168.750	m ³
ZT	=	234.375	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	$ML = ML \cos \theta + MT \sin \theta$
Moment along transverse axis	$MT = MT \cos \theta - ML \sin \theta$

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1766.799	1.100	1943.479	0.000	0.000
SIDL except Wearing Course	1			160.000	1.100	176.000	0.000	0.000
Wearing Course	1			254.898	1.100	280.387	0.000	0.000
Bearing Pedestal	1	25	1.056	26.400	1.100	29.040	-0.682	-18.000
				2208.097		2428.907		-18.000
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	7.950	198.750	0.300	59.625	0.000	0.000
Dirt Wall-Tapered portion	1	25	1.263	31.582	0.300	9.475	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	0.000	0.000	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	0.050	0.703	0.000	0.000
Cap - (uniform portion)	1	25	6.938	173.438	1.075	186.445	0.000	0.000
Cap - (corbel portion)	1	25	5.263	131.567	1.075	141.435	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.500	0.000	6.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.500	0.000	6.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.300	8.400	0.000	0.000
Approach Slab	1	25	6.563	164.063	0.000	0.000	0.000	0.000
				769.587		406.083		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	23.234	580.859	-2.000	-1161.719	6.000	3485.156
Abutment Shaft	1	25	72.684	1817.095	1.118	2031.536	0.000	0.000
Back filling over heel slab	1	20	576.367	11527.344	-2.032	-23419.271	0.000	0.000
Front Filling over toe slab	1	20	86.345	1726.900	3.034	5239.057	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	53.125	1328.125	-1.657	-2200.521	0.000	0.000
Toe slab	1	25	29.750	743.750	2.908	2162.708	0.000	0.000
portion between heel & toe	1	25	18.000	450.000	1.100	495.000	0.000	0.000
Vertical Components of active earth pressure	1			1242.641	-4.500	-5591.883	0.000	0.000
				19674.120		-22303.852		3485.156
Total				22651.804		-19468.863		3467.156

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.900	1590.120	-3.400	-5406.406
0.900	144.000	-3.400	-489.600
1.000	254.898	-3.400	-866.652
0.900	23.760	-3.400	-80.784
	2012.777		-6843.442
0.900	178.875	-4.200	-751.275
0.900	28.424	-4.200	-119.380
0.900	25.313	-4.500	-113.906
0.900	12.656	-4.450	-56.320
0.900	156.094	-3.425	-534.621
0.900	118.410	-3.425	-405.556
0.900	0.000	-4.000	0.000
0.900	0.000	-4.000	0.000
0.900	25.200	-4.200	-105.840
0.900	147.656	-4.500	-664.453
	692.628		-2751.351
0.900	522.773	-6.500	-3398.027
0.900	1635.385	-3.382	-5530.851
0.900	10374.609	-6.532	-67763.086
0.900	1554.210	-1.466	-2278.794
0.900	0.000	0.000	0.000
0.900	1195.313	-6.157	-7359.375
0.900	669.375	-1.592	-1065.750
0.900	405.000	-3.400	-1377.000
0.900	1118.377	-9.000	-10065.390
	17706.708		-99747.991
	20412.113		-109342.785

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1452.035	885.224
due to Earth pressure	1	3000.000		11858.015

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1452.035	999.602
due to Earth pressure	1.5	4500.000		17787.023
		4623.180		18786.625

Summary of Forces For SBC

P	22651.804	kN
ML	-6725.624	kNm
MT	3467.156	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		22651.804		-19468.863		3467.156
CWLL-Max. Reaction case	1	981.821	1.100	1080.003	0.950	932.730
Vertical Components of LL Surcharge	1	280.671	-4.500	-1263.022	0.000	0.000
Total		23914.296		-19651.881		4399.886

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	255.194	1452.035	2070.898
due to Earth pressure	1	3000.000		11858.015
due to Live load surcharge	1	677.601		3388.004

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		342.343	1452.035	2778.113
due to Earth pressure	1.5	4500.000		17787.023
due to Live load surcharge	1.2	813.121		4065.605
		5655.464		24630.741

Summary of Forces For SBC

P	23914.296	kN
ML	-2334.964	kNm
MT	4399.886	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	885.22	0.00	0.00
3000.00	11858.02	0.00	0.00
3109.085	12743.239	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	999.60	0.00	0.00
4500.00	17787.02	0.00	0.00
4623.180	18786.625	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	20412.11295		-109342.78
0.000		-3.400	0.00
0.900		-9.000	-2273.44
	20412.113		-111616.22

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
255.19	2070.90	0.00	0.00
3000.00	11858.02	0.00	0.00
677.601	3388.004	0.000	0.000
3932.79	17316.92	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
342.34	2778.11	0.00	0.00
4500.00	17787.02	0.00	0.00
813.12	4065.61	0.00	0.00
5655.464	24630.741	0.000	0.000

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				2208.097		2428.907		-18.000
Substructure & Foundation -Portion 1				769.587		406.083		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	23.234	580.859	-2.000	-1161.719	6.000	3485.156
Shaft above HFL	1	25	49.165	1229.116	1.168	1435.078	0.000	0.000
Shaft below HFL	1	15	23.519	352.787	1.118	394.421	0.000	0.000
Back filling above HFL over heel slab	1	20	417.688	8353.750	-2.000	-16707.500	0.000	0.000
Back filling below HFL over heel slab	1	10	154.188	1541.875	-2.118	-3266.042	0.000	0.000
Front Filling over toe slab	1	10	86.345	863.450	1.934	1669.733	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	53.125	796.875	-1.657	-1320.313	0.000	0.000
Toe slab	1	15	29.750	446.250	2.908	1297.625	0.000	0.000
Portion between Heel & Toe	1	15	18.000	270.000	1.100	297.000	0.000	0.000
Vertical Components of active earth pressure	1			1242.641	-4.500	-5591.883	0.000	0.000
				15912.556		-22820.034		3485.156
Total				18890.240		-19985.045		3467.156

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	2012.777		-6843.442
	692.628		-2751.351
0.900	522.773	-6.500	-3398.027
0.900	1106.204	-3.332	-3686.350
0.900	317.508	-3.382	-1073.810
0.900	7518.375	-6.500	-48869.437
0.900	1387.688	-6.618	-9184.031
0.900	777.105	-2.566	-1994.213
0.900	0.000	0.000	0.000
0.900	717.188	-6.157	-4415.625
0.900	401.625	-1.592	-639.450
0.900	243.000	-3.400	-826.200
0.900	1118.377	-9.000	-10065.390
	14321.301		-84983.884
	17026.706		-94578.678

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	109.085	1452.035	885.224
due to Earth pressure	1	3000.000		10879.608

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
109.08	885.22	0.00	0.00
3000.00	10879.61	0.00	0.00
3109.08	11764.83	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		123.180	1452.035	999.602
due to Earth pressure	1.5	4500.000		16319.412
		4623.180		17319.014

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
123.18	999.60	0.00	0.00
4500.00	16319.41	0.00	0.00
4623.18	17319.01	0.00	0.00

Summary of Forces For SBC		
P	18890.240	kN
ML	-8220.213	kNm
MT	3467.156	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		18890.240		-19985.045		3467.156
CWLL-Min. Reaction case	1	368.179	1.100	404.997	0.950	349.770
Vertical Components of LL Surcharge	1	306.286	-4.500	-1378.289	0.000	0.000
Total		19564.706		-20958.337		3816.926

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	17026.706		-94578.678
0.000	0.000	-3.400	0
0.900	275.658	-9.000	-2480.920
	17302.364		-97059.598

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	285.876	1452.035	2319.883
due to Earth pressure	1	3000.000		10879.608
due to Live load surcharge	1	739.441		3697.204

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
285.88	2319.88	0.00	0.00
3000.00	10879.61	0.00	0.00
739.441	3697.204	0.000	0.000
4025.32	16896.69	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		388.366	1452.035	3151.591
due to Earth pressure	1.5	4500.000		16319.412
due to live load surcharge	1.2	887.329		4436.645
		5775.695		23907.648

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
388.37	3151.59	0.00	0.00
4500.00	16319.41	0.00	0.00
887.329	4436.645	0.000	0.000
5775.70	23907.65	0.00	0.00

Summary of Forces For SBC

P	19564.706	kN
ML	-4061.642	kNm
MT	3816.926	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor = 1

ah= 0.162 In Longitudinal direction
ah= 0.184 In Transverse direction
av= 0.123 In Vertical direction

Weight of shaft below Ground level

= 723.761 KN

Weight of back fill below Ground level

= 2646.250 KN

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1766.799		97.513	65.008	1.100	1943.479	71.509	1453.248		0.000	0.000	909.630
SIDL except Wearing Course	1			160.000		8.831	5.887	1.100	176.000	6.476	1454.369		0.000	0.000	92.271
Wearing Course	1			254.898		14.068	9.379	1.100	280.387	10.317	1453.920		0.000	0.000	140.682
Bearing Pedestal	1	25	1.056	26.400		1.457	0.971	1.100	29.040	1.069			-0.682	-18.000	
				2208.097		121.868	81.246		2428.907	89.370				-18.000	1142.583
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	7.950	198.750	32.198	10.969	7.313	0.300	59.625	2.194	1452.860	287.846	0.000	0.000	98.066
Dirt Wall-Tapered portion	1	25	1.263	31.582	5.116	1.743	1.162	0.300	9.475	0.349	1451.632	39.455	0.000	0.000	13.442
Bracket - Uniform portion	1	25	1.125	28.125				0.000	0.000						
Bracket - Tapered portion	1	25	0.563	14.063				0.050	0.703						
Cap - (uniform portion)	1	25	6.938	173.438	28.097	9.572	6.382	1.075	186.445	6.860	1451.385	209.743	0.000	0.000	71.457
Cap - (corbel portion)	1	25	5.263	131.567	21.314	7.261	4.841	1.075	141.435	5.204	1451.085	152.714	0.000	0.000	52.028
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1454.064	0.000	6.000	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1454.064	0.000	6.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.300	8.400				0.000	0.000	
Approach Slab	1	25	6.563	164.063				0.000	0.000				0.000	0.000	

				769.587	86.725	29.546	19.697		406.083	14.607		689.757		0.000	234.993
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	23.234	580.859	94.099	32.059	21.372	-2.000	-1161.719	-42.745	1449.309	507.107	6.000	3485.156	172.766
Abutment Shaft	1	25	72.684	1817.095	177.120	60.343	40.229	1.118	2031.536	44.976	1449.086	915.002	0.000	0.000	311.731
Back filling over heel slab	1	20	576.367	11527.344	0.000	0.000	0.000	-2.032	-23419.271	0.000	1449.309	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	86.345	1726.900				3.034	5239.057				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	53.125	1328.125				-1.657	-2200.521				0.000	0.000	
Toe slab	1	25	29.750	743.750				2.908	2162.708				0.000	0.000	
portion between heel & toe	1	25	18.000	450.000				1.100	495.000				0.000	0.000	
Vertical component of active earth pressure	1			1169.464				-4.500	-5262.590				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			385.938				-4.500	-1736.721				0.000	0.000	
				19986.882	271.219	92.401	61.601		-23711.280	2.231		1422.109		3485.156	484.497
Total =				22964.565	357.944	243.816	162.544		-20876.291	106.208		2111.866		3467.156	1862.072
								-162.544		-106.208					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure								
Dead Load	0.9			1590.120	58.508	-3.400	-5406.406	-198.926
SIDL except Wearing Course	0.9			144.000	5.298	-3.400	-489.600	-18.015
Wearing Course	1.00			254.898	9.379	-3.400	-866.652	-31.888
Bearing Pedestal	0.9	25	1.056	23.760		-3.400	-80.784	
				2012.777	73.185		-6843.442	-248.828
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	7.950	178.875	6.582	-4.200	-751.275	-27.643
Dirt Wall-Tapered portion	0.9	25	1.263	28.424	1.046	-4.200	-119.380	-4.393
Bracket - Uniform portion	0.9	25	1.125	25.313				
Bracket - Tapered portion	0.9	25	0.563	12.656				
Cap - (uniform portion)	0.9	25	6.938	156.094	5.743	-3.425	-534.621	-19.671
Cap - (corbel portion)	0.9	25	5.263	118.410	4.357	-3.425	-405.556	-14.922
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-4.000	0.000	0.000
Cantilever Return Wall-Traingle portion	0.9	25	0.000	0.000	0.000	-4.000	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-4.200	-105.840	
Approach Slab	0.9	25	6.563	147.656		-4.500	-664.453	
				692.628	17.728		-2581.125	-66.629
Substructure & Foundation -Portion 2								
Abutment Shaft	0.9	25	72.684	1635.385	33.543	-3.382	-5530.851	-113.441
Solid Return wall	0.9	25	23.234	522.773	19.235	-6.500	-3398.027	-125.028
Back filling over heel slab	0.9	20	576.367	10374.609	0.000	-6.532	-67763.1	0.000
Front Filling over toe	0.9	20	86.345	1554.210		-1.466	-2278.794	
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000	
Heel slab	0.9	25	53.125	1195.313		-6.157	-7359.375	
Toe slab	0.9	25	29.750	669.375		-1.592	-1065.750	
portion between heel & toe	0.9	25	18.000	405.000		-3.400	-1377.000	
Vertical component of active earth pressure	0.9			1118.377		-9.000	-10065.390	
Vertical component of dynamic increment of earth pressure	0.9			347.344		-9.000	-3126.098	
				18054.052	52.778		-102874.09	-238.469
Total =				20759.457	143.690		-112298.66	-553.926
				-143.690		553.926		

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	32.198	1452.860	287.846
1.0	5.116	1451.632	39.455
1.0	28.097	1451.385	209.743
1.0	21.314	1451.085	152.714
1.0	0.000	1454.064	0.000
1.0	0.000	1454.064	0.000
	86.725		689.757
1.0	164.092	1449.086	847.701
1.0	94.099	1449.309	507.107
1.0	0.000	1449.309	0.000
	258.192		1354.808
	344.916		2044.565

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	121.868	1452.035	5736.249	1142.583
due to Substructure	1	357.944	121.948		2111.866	719.490
due to Earth pressure	1	3755.074			17075.741	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1452.035	8604.37
due to Substructure	1.5	517.374		3066.85
due to Active Earth pressure	1	2823.337		11858.02
due to dynamic Earth pressure	1.5	1397.605		7826.59
		5798.621		31355.825

Summary of Forces For SBC

	Downward	Upward	
P	23127.109	22802.021	kN
ML	4153.773	3941.357	kNm
MT	5329.229	5329.229	kNm

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	5736.25	0.00	0.00	0.00	0.00	121.87	1142.58
357.94	2111.87	0.00	0.00	0.00	0.00	121.95	719.49
3755.07	17075.74						
4819.89	24923.86	0.00	0.00	0.00	0.00	243.82	1862.07

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	8604.37	0.00	0.00
517.37	3066.85	0.00	0.00
2823.337	11858.015	0.000	0.000
1397.605	7826.589	0.000	0.000
5798.62	31355.82	0.00	0.00

Summary of Restoring Forces

Vertical Load	20615.767	kN
Moment	-112852.582	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				2208.097			81.246		2428.907	89.370				-18.000	1142.583
Forces from Substructure				20756.468	357.944	121.868	81.298		-23305.197	16.838		2111.866		3485.156	719.490
CWLL-Max. Reaction case	0.20			196.36		10.838	7.225	1.100	216.001	7.948	1455.120		0.950	186.546	121.382
Vertical component of LL Surcharge	0.20			56.13				-4.500	-252.604				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			21.05				-4.500	-94.730				0.000	0.000	
Total =				23238.115	357.944	254.654	169.769		-21007.625	114.156		2111.866		3653.702	1983.454
							-169.769			-114.156					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Forces from Superstructure				2012.777	73.185	0.000	-6843.44	-248.828
Forces from Substructure				18746.680	70.505	0.000	-105455.21	-305.098
CWLL-Max. Reaction case	0.00			0.00	0.000	-3.400	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-9.000	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-9.000	0.00	
Total =				20759.457	143.690		-112298.66	-553.926
					-143.690			553.926

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	757.950	132.706	1452.035	6150.763	1263.964
due to Substructure	1	357.944	121.948		2111.866	719.490
due to Earth pressure	1	3755.074			17075.741	
due to Live load surcharge	0.20	186.342			1038.234	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1452.035	8604.373
due to Substructure	1.5	517.374		3066.848
due to Active Earth pressure	1	2823.337		11858.015
due to dynamic Earth pressure	1.5	1397.605		7826.58941
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		5798.621		31355.825

Summary of Forces For SBC

	Downward	Upward	
P	23407.884	23068.346	kN
ML	5483.135	5254.824	kNm
MT	5637.156	5637.156	kNm

Summary of Restoring Forces

Vertical Load	20615.767	kN
Moment	-112852.582	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				2208.097		121.868	81.246		2428.907	89.370				-18.000	1142.583
Substructure & Foundation -Portion 1				769.587	86.725	29.546	19.697		406.083	14.607		689.757		0.000	234.993
Substructure & Foundation -Portion 2															
Solid Return wall	1		23.23	580.86	94.099	32.059	21.372	-2.00	-1161.719	-42.745	1449.31	507.107	6.00	3485.156	172.766
Shaft above HFL	1	25	49.165	1229.116	199.117	67.837	45.225	1.168	1435.078	52.803	1449.086	1028.637	0.000	0.000	350.446
Shaft below HFL	1	15	23.519	352.787	-21.997	-7.494	-4.996	1.118	394.421	-5.586	1447.237	-72.963	0.000	0.000	-24.858
Back filling above HFL over heel slab	1	20	417.688	8353.750	0.000	0.000	0.000	-2.000	-16707.500	0.000	1450.579	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	154.188	1541.875	0.000	0.000	0.000	-2.118	-3266.042	0.000	1445.879	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	86.345	863.450				1.934	1669.733				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	53.125	796.875				-1.657	-1320.313				0.000	0.000	
Toe slab	1	15	29.750	446.250				2.908	1297.625				0.000	0.000	
portion between heel & toe	1	15	18.000	270.000				1.100	297.000				0.000	0.000	
Vertical component of active earth pressure	1			1205.987				-4.500	-5426.941				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			383.556				-4.500	-1726.001				0.000	0.000	
				16259.458	309.282	105.369	70.246		-24514.657	4.472		1647.744		3485.156	561.368
Total =				19237.142	396.006	256.783	171.189		-21679.668	108.449		2337.501		3467.156	1938.944

-171.189

-108.449

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxL2	MLs due to Fv
Superstructure				2012.777	73.185		-6843.442	-248.828
Substructure & Foundation -Portion 1				692.628	17.728		-2581.125	-66.629
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	23.234375	522.773	19.235	-6.50	-3398.027	-125.028
Shaft above HFL	0.9	25	49.165	1106.204	40.702	-3.332	-3686.350	-135.637
Shaft below HFL	0.9	15	23.519	317.508	-7.159	-3.382	-1073.810	24.213
Back filling above HFL over heel slab	0.9	20	417.688	7518.375	0.000	-6.500	-48869.437	0.000
Back filling below HFL over heel slab	0.9	10	154.188	1387.688	0.000	-6.618	-9184.031	0.000
Front Filling ovetoe	0.9	10	86.345	777.105		-2.566	-1994.213	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	53.125	717.188		-6.157	-4415.625	
Toe slab	0.9	15	29.750	401.625		-1.592	-639.450	
portion between heel & toe	0.9	15	18.000	243.000		-3.400	-826.200	
Vertical component of active earth pressure	0.9			1085.388		-9.000	-9768.493	
Vertical component of dynamic increment of earth pressure	0.9			345.200		-9.000	-3106.802	
				14633.512	60.558		-87793.79	-267.041
Total =				17338.917	151.471		-97218.36	-582.498

-151.471

582.498

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
	86.725		689.757
1.0	94.099	1449.309	507.107
1.0	199.117	1449.086	1028.637
1.0	-35.024	1447.237	-116.176
1.0	0.000	1450.579	0.000
1.0	0.000	1445.879	0.000
	258.192		1419.568
	344.916		2109.325

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	706.870	121.868	1452.035	5736.249	1142.583
due to Substructure	1	396.006	134.915		2337.501	796.361
due to Earth pressure	1	3837.495			16082.509	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1452.035	8604.373
due to Substructure	1.5	517.374		3163.988
due to Active Earth pressure	1	2911.510		10879.608
due to dynamic Earth pressure	1.5	1388.978		7804.352
		5878.167		30452.321

Summary of Forces For SBC

	Downward	Upward	
P	19408.331	19065.953	kN
ML	2585.041	2368.142	kNm
MT	5406.100	5406.100	kNm

Forces along Long. Axis

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
706.87	5736.25	0.00	0.00	0.00	0.00	121.87	1142.58
396.01	2337.50	0.00	0.00	0.00	0.00	134.92	796.36
3837.50	16082.51						
4940.37	24156.26	0.00	0.00	0.00	0.00	256.78	1938.94

Forces along Long. Axis

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1060.30	8604.37	0.00	0.00
517.37	3163.99	0.00	0.00
2911.510	10879.608		
1388.978	7804.352		
5878.17	30452.32	0.00	0.00

Summary of Restoring Forces

Vertical Load	17187.447	kN
Moment	-97800.854	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha h \times P$ (kN)	FT = $0.3 \times \alpha h \times P$ (kN)	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				2208.097		121.868	81.246		2428.907	89.370				-18.000	1142.583
Forces from Substructure				17029.045	396.006	134.915	89.943		-24108.575	19.079		2337.501		3485.156	796.361
CWLL-Max. Reaction case	0.20			73.64		4.064	2.709	1.100	80.999	2.980	1455.120		0.950	69.954	
Vertical component of LL Surcharge	0.20			61.26				-4.500	-275.658				0.000	0.000	45.518
Vertical component of dynamic increment LL Surcharge	0.20			25.69				-4.500	-115.589				0.000	0.000	
Total =				19397.722	396.006	260.848	173.898		-21989.916	111.430		2337.501		3537.110	1984.461
							-173.898			-111.430					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				2012.777	73.185		-6843.442	-248.828
Forces from Substructure				15326.140	78.286		-90374.91	-333.670
CWLL-Max. Reaction case	0.00			0.00	0.00		-3.40	0.00
Vertical component of LL Surcharge	0.00			0.00		-3.400	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-9.000	0.000	0.000
Total =				17338.917	151.471		-97221.76	-582.498
							-151.471	582.498

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	757.950	125.933	1452.035	6150.763	1188.100
due to Substructure	1	396.006	134.915		2337.501	796.361
due to Earth pressure	1	3837.495			16082.509	
due to Live load surcharge	0.20	209.901			1129.593	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
757.95	6150.76	0.00	0.00	0.00	0.00	125.93	1188.10
396.01	2337.50	0.00	0.00	0.00	0.00	134.92	796.36
3837.50	16082.51						
209.90	1129.59						
5201.35	25700.37	0.00	0.00	0.00	0.00	260.85	1984.46

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1060.305	1452.035	8604.373
due to Substructure	1.5	517.374		3163.98776
due to Active Earth pressure	1	2911.510		10879.608
due to dynamic Earth pressure	1.5	1388.978		7804.35218
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		5878.167		30452.321

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
1060.30	8604.37	0.00	0.00
517.37	3163.99	0.00	0.00
2911.510	10879.608		
1388.978	7804.352		
0.000	0.000		
0.000	0.000		
5878.17	30452.32	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	19571.620	19223.823	kN
ML	3821.881	3599.021	kNm

Summary of Restoring Forces

Vertical Load	17187.447	kN
Moment	-97804.254	kNm

MT	5521.572	5521.572	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x $\alpha_h \times P$ (kN)	FT = $\alpha_h \times P$ (kN)	Fv = 0.3 x α_v $\times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				2208.097		406.228	81.246		2428.907	89.370				-18.000	3808.608
Substructure & Foundation -Portion 1				769.587	26.017	98.487	19.697		406.083	14.607		206.927		0.000	783.309
Substructure & Foundation -Portion 2				19986.882	81.366	308.005	61.601		-23711.280	2.231		426.633		3485.156	1614.990
Total =				22964.565	107.383	812.720	162.544		-20876.291	106.208		633.560		3467.156	6206.908
							-162.544			-106.208					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x $\alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				2012.777	73.185		-6843.44	-248.828
Substructure & Foundation -Portion 1				692.628	17.728		-2581.12	-66.629
Substructure & Foundation -Portion 2				18054.052	52.778		-102874.09	-238.469
Total =				20759.457	143.690		-112298.66	-553.926
							-143.690	553.926

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x α_h $\times P$ (kN)	C.g. of Force (m)	MLs due to FL
	86.725		689.757
	258.192		1354.808
	344.916		2044.565

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.115	406.228	1452.035	1745.661	3808.608
due to Substructure	1	107.383	406.492		633.560	2398.299
due to Earth pressure	1	3755.074			17075.741	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	1745.66	0.00	0.00	0.00	0.00	406.23	3808.61
107.38	633.56	0.00	0.00	0.00	0.00	406.49	2398.30
3755.07	17075.74						
4077.57	19454.96	0.00	0.00	0.00	0.00	812.72	6206.91

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1452.035	2581.312
due to Substructure	1.5	517.374		3066.84752
due to Active Earth pressure	1	2823.337		11858.0151
due to dynamic Earth pressure	1.5	419.282		2347.97682
		4078.084		19854.151

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2581.31	0.00	0.00
517.37	3066.85	0.00	0.00
2823.337	11858.015		
419.282	2347.977		
4078.08	19854.15	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	23127.109	22802.021	kN
ML	-1315.121	-1527.537	kNm
MT	9674.064	9674.064	kNm

Summary of Restoring Forces

Vertical Load	20615.767	kN
Moment	-112852.582	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxL1	MLs due to Fv	MT = PxT
Total =	23238.115	169.769	-21007.62	114.156	3653.702
		-169.769		-114.156	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	442.354	1452.035	2135.389	4213.214
due to Substructure	1	107.383	406.492		633.560	2398.299
due to Earth pressure	1	3755.074			17075.741	
due to Live load surcharge	0.2	186.342			1038.234	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1452.035	2581.312
due to Substructure	1.5	155.21		920.054
due to Active Earth pressure	1	2823.34		11858.015
due to dynamic Earth pressure	1.5	419.28		2347.977
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		3715.92		17707.358

Summary of Forces For SBC

	Downward	Upward	
P	23407.884	23068.346	kN
ML	-10.545	-238.857	kNm
MT	10265.216	10265.216	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Lloads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxL1	MLs due to Fv	MT = PxT
Superstructure	2208.097	81.246	2428.907	89.370	-18.000
Substructure & Foundation -Portion 1	769.587	19.697	406.083	14.607	0.000
Substructure & Foundation -Portion 2	16259.458	70.246	-24514.657	4.472	3485.156
Total =	19237.142	171.189	-21679.668	108.449	3467.156
		-171.189		-108.449	

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxL2	MLs due to Fv
Total =	20759.457	143.690	-112298.66	-553.926
		-143.690		553.926

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
263.14	2135.39	0.00	0.00	0.00	0.00	442.35	4213.21
107.38	633.56	0.00	0.00	0.00	0.00	406.49	2398.30
3755.07	17075.74						
186.34	1038.23						
4311.94	20882.92	0.00	0.00	0.00	0.00	848.85	6611.51

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
318.09	2581.31	0.00	0.00
155.21	920.05	0.00	0.00
2823.337	11858.015		
419.282	2347.977		
0.000	0.000		
0.000	0.000		
3715.92	17707.36	0.00	0.00

Summary of Restoring Forces

Vertical Load	20615.767	kN
Moment	-112852.582	kNm

Vertical Forces For Restoring or Resisting Effect

Lloads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxL2	MLs due to Fv
Superstructure	2012.777	73.185		-6843.442	-248.828
Substructure & Foundation	692.628	17.728		-2581.125	-66.629
Substructure & Foundation	14633.512	60.558		-87793.790	-267.041
Total =	17338.917	151.471		-97218.356	-582.498
		-151.471			582.498

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	215.12	406.23	1452.04	1745.66	3808.61
due to Substructure	1	118.80	449.72		701.25	2654.54
due to Earth pressure	1	3837.50			16082.51	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.091	1452.035	2581.31
due to Substructure	1.5	155.212		949.20
due to Active Earth pressure	1	2911.510		10879.61
due to dynamic Earth pressure	1.5	416.693		2341.31
		3801.507		16751.42

Summary of Forces For SBC

	Downward	Upward	
P	19408.331	19065.953	kN
ML	-3041.798	-3258.697	kNm
MT	9930.302	9930.302	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	ML = $P \times eL1$	MLs due to F_v	MT = $P \times eT$
Total =	19397.722	173.898	-21989.92	111.430	3537.110
		-173.898		-111.430	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	263.141	419.77521	1452.035	2135.389	3960.33412
due to Substructure	1	118.802	449.71668		701.250	2654.53749
due to Earth pressure	1	3837.495			16082.509	
due to Live load surcharge	0.2	209.901			1129.593	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		318.09	1452.035	2581.31
due to Substructure	1.5	155.21		949.20
due to Active Earth pressure	1	2911.51		10879.61
due to dynamic Earth pressure	1.5	416.69		2341.31
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		3801.51		16751.42

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
215.12	1745.66	0.00	0.00	0.00	0.00	406.23	3808.61
118.80	701.25	0.00	0.00	0.00	0.00	449.72	2654.54
3837.50	16082.51						
4171.41	18529.42	0.00	0.00	0.00	0.00	855.94	6463.15

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2581.31	0.00	0.00
155.21	949.20	0.00	0.00
2911.510	10879.608		
416.693	2341.306		
3801.51	16751.42	0.00	0.00

Summary of Restoring Forces

Vertical Load	17187.447	kN
Moment	-97800.854	kNm

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = $P \times eL2$	MLs due to F_v
Total =	17338.917	151.471		-97221.76	-582.498
		-151.471			582.498

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
263.14	2135.39	0.00	0.00	0.00	0.00	419.78	3960.33
118.80	701.25	0.00	0.00	0.00	0.00	449.72	2654.54
3837.50	16082.51						
209.90	1129.59						
4429.34	20048.74	0.00	0.00	0.00	0.00	869.49	6614.87

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
318.09	2581.31	0.00	0.00
155.21	949.20	0.00	0.00
2911.510	10879.608		
416.693	2341.306		
0.000	0.000		
0.000	0.000		
3801.51	16751.42	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	19571.620	19223.823	kN
ML	-1829.744	-2052.604	kNm
MT	10151.982	10151.982	kNm

Summary of Restoring Forces

Vertical Load	17187.447	kN
Moment	-97804.254	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1455.264 -	1443.920)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1455.264 -	1443.920)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe} + M_n$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	22651.804	-6725.624	3467.156	255.998	176.287	226.412	146.701	255.998	146.701	4623.180	14288.479	3.09	18786.625	109342.78	5.82
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	23914.296	-2334.964	4399.886	245.181	217.508	207.635	179.962	245.181	179.962	5655.464	14288.479	2.53	24630.741	111616.22	4.53
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	18890.240	-8220.213	3467.156	231.419	133.994	201.832	104.408	231.419	104.408	4623.180	11918.694	2.58	17319.014	94578.68	5.46
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	19564.706	-4061.642	3816.926	214.263	166.125	181.692	133.554	214.263	133.554	5775.695	12111.654	2.10	23907.648	97059.60	4.06
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	23127.109	4153.773	5329.229	203.697	252.927	158.221	207.451	252.927	158.221	5798.621	14431.037	2.49	31355.825	112852.58	3.60
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	23407.884	5483.135	5637.156	199.629	264.615	151.526	216.511	264.615	151.526	5798.621	14431.037	2.49	31355.825	112852.58	3.60
								SAFE	SAFE			SAFE			SAFE

Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	19408.331	2585.041	5406.100	180.266	210.903	134.134	164.771	210.903	134.134	5878.167	12031.213	2.05	30452.321	97800.85	3.21
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	19571.620	3821.881	5521.572	174.880	220.177	127.763	173.059	220.177	127.763	5878.167	12031.213	2.05	30452.321	97804.25	3.21
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	23127.109	-1315.121	9674.064	254.644	239.057	172.092	156.505	254.644	156.505	4078.084	14431.037	3.54	19854.151	112852.58	5.68
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	23407.884	-10.545	10265.216	251.931	251.806	164.334	164.209	251.931	164.209	3715.922	14431.037	3.88	17707.358	112852.58	6.37
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	19408.331	-3041.798	9930.302	232.913	196.862	148.175	112.124	232.913	112.124	3801.507	12031.213	3.16	16751.422	97800.85	5.84
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	19571.620	-1829.744	10151.982	228.128	206.442	141.498	119.812	228.128	119.812	3801.507	12031.213	3.16	16751.422	97804.25	5.84
								SAFE	SAFE			SAFE			SAFE

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

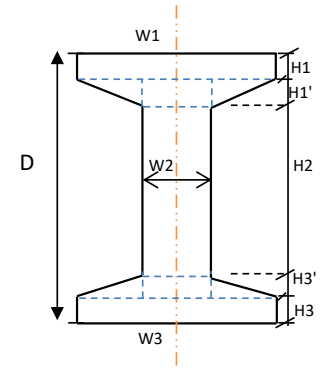
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	950
H3'	=	150
H3	=	250
W1	=	1000
W2	=	325
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	675	100	33750
1' (Rect.)	325	100	32500
2 (Rect.)	325	950	308750
3' (Tri.)	300	150	22500
3' (Rect.)	325	150	48750
3 (Rect.)	625	250	156250
			752500



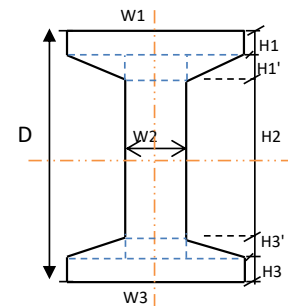
Total Area of Section A = 0.753 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	78
H2	=	1122
H3'	=	0
H3	=	250
W1	=	1000
W2	=	475
W3	=	475

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	525	78	20417
1' (Rect.)	475	78	36944
2 (Rect.)	475	1122	533056
3' (Tri.)	0	0	0
3' (Rect.)	475	0	0
3 (Rect.)	475	250	118750
			859167



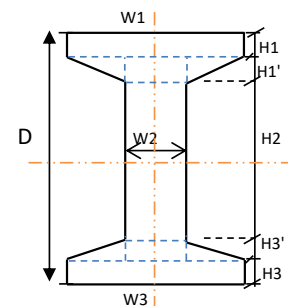
Total Area of Section A = 0.859 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

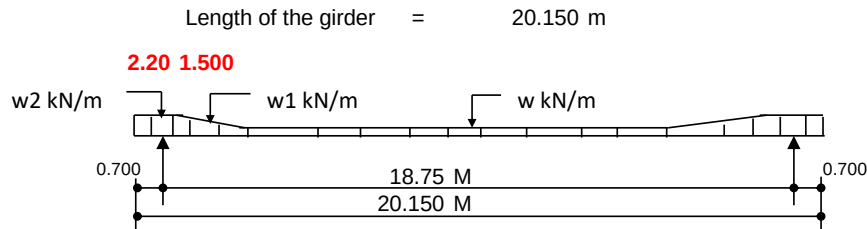
D	=	1600
H1	=	150
H1'	=	56
H2	=	1394
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	625
W3	=	625

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	375	56	10417
1' (Rect.)	625	56	34722
2 (Rect.)	625	1394	871528
3' (Tri.)	0	0	0
3' (Rect.)	625	0	0
3 (Rect.)	625	0	0
			1066667



Total Area of Section A = 1.067 m²

1.0 Self weight of Inner Girder & Outer Girder



UDL , w	=	18.81 kN/m
Avg. , w ₁	=	21.48 kN/m
UDL , w ₂	=	26.67 kN/m
Supp. Reaction , R	=	210.82 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	3.250 m	For Outer Girder
Effective Width of Slab	=	3.00 m	For Inner Girder
Thickness of Slab	=	0.22 m	
Extra average slab thickness for camber	=	0.00 m	
Weight of green concrete	=	25 kN/m ³	
UDL due to deck slab weight	=	17.875 kN/m	For Outer Girder
on entire length of girder	=	16.500 kN/m	For Inner Girder
Supp. Reaction , R	=	180.091 kN	For Outer Girder
	=	166.238 kN	For Inner Girder
weight of deck slab has been Increase by 5 %			
Supp. Reaction , R	=	189.095 kN	For Outer Girder
	=	174.549 kN	For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam	=	1.57 m	
Thickness of cross beam	=	0.4 m	
Support Reaction , R	=	51.025 kN	on outer bearings
	=	47.100 kN	on Inner bearings

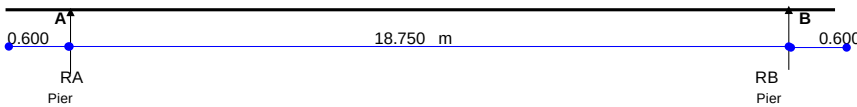
4.0 Weight due to wearing coat

Wearing coat width	=	2.750 m	For Outer Girder
Wearing coat width	=	3.000 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	
UDL due to Wearing coat	=	6.050 kN/m	For Outer Girder
on entire length of girder	=	6.600 kN/m	For Inner Girder
Supp. Reaction , R	=	60.954 kN	For Outer Girder
	=	66.495 kN	For Inner Girder

ANNEXURE-II (LOAD CALCULATION-CARRIAGEWAY LIVE LOAD)

Maximum Live Load Reaction Case**Due to Class A 1-Lane , 2-Lanes, 3-Lanes & 4-Lanes**Considering **114.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**

Nos. of Lane for design purpose = **3 LANE**

Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

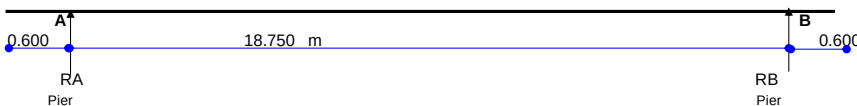
1st Train	Total Load	500.0kN				
					363.637	136.363
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	363.637	136.363

Due to Class 70R WheeledConsidering **170.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

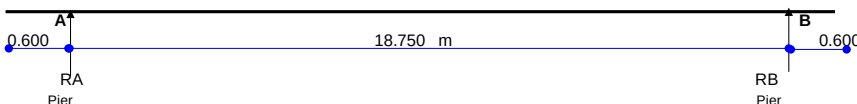
1st Train	Total Load	1000.0kN				
					758.731	241.269
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	758.731	241.269

Due to Class A 1-Lane/2-Lane (along with Class 70R)Considering **68.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = **18.75 m**
 Projection beyond c/L of bearing at Abut. end = **0.600 m**
 Projection beyond c/L of bearing at far end = **0.600 m**
 Total length including Overhangs = **19.95 m**



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN

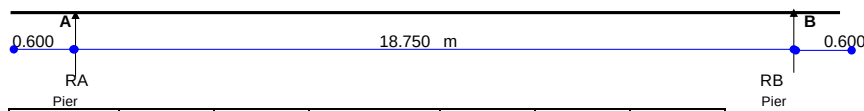
1st Train	Total Load	554.0kN				
					284.864	269.136
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
27.000	1.100	18.800	18.200	491.400		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	284.864	269.136

Due to Class SV LoadingConsidering **180.00** kN(I.e. **0.000m** from extreme Left End)

Length of the Span = 18.75 m
 Projection beyond c.L. of bearing at Abut. end = 0.600 m
 Projection beyond c.L. of bearing at far end = 0.600 m
 Total length including Overhangs = 19.95 m



Axle Load in kN	Spacing between two successive axles in metres	Distance of the axles from Extreme Left Node in metres	Distance of the axles from the Support A in metres	Total Moment of all Loads @ Support A in kN-m	Reaction at Support A in kN	Reaction at Support B in kN
Ist Train	Total Load	3420.0kN				
180.000		0.000	-0.600	-108.000	2003.040	1416.960
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		
180.000	1.500	9.000	8.400	1512.000		
180.000	1.500	10.500	9.900	1782.000		
180.000	1.500	12.000	11.400	2052.000		
180.000	1.500	13.500	12.900	2322.000		
180.000	1.500	15.000	14.400	2592.000		
180.000	1.500	16.500	15.900	2862.000		
180.000	1.500	18.000	17.400	3132.000		
180.000	1.500	19.500	18.900	3402.000		
0.000	1.500	0.000	-0.600	0.000		
180.000	1.500	1.500	0.900	162.000		
180.000	1.500	3.000	2.400	432.000		
180.000	1.500	4.500	3.900	702.000		
180.000	1.500	6.000	5.400	972.000		
180.000	1.500	7.500	6.900	1242.000		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	2003.040	1416.960

Deck Width = 12.500 m

transverse eccentricity of superstructure about c.g of abutment and foundation = 0 m

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	2003.040 KN	Corr. MT	= 600.912 kNm
Minimum Reaction (RB)	=	1416.960 KN	Corr. MT	= 425.088 kNm

1-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-0.9	0	0	=	4.45 m
Maximum Reaction (RA)	=	363.637 KN		Corr. MT	=	1618.186 kNm			
Minimum Reaction (RB)	=	136.363 KN		Corr. MT	=	606.814 kNm			

2-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-0.85	0	=	2.70 m
Maximum Reaction (RA)	=	727.275	KN	Corr. MT	=	1963.642	kNm		
Minimum Reaction (RB)	=	272.725	KN	Corr. MT	=	736.358	kNm		

3-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	0.95 m
Maximum Reaction (RA)	=	1090.912 KN	Corr. MT	=	1036.366 kNm				
Minimum Reaction (RB)	=	409.088 KN	Corr. MT	=	388.634 kNm				

4-CLASS A

transverse ecc.	=	6.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-0.8 m
Maximum Reaction (RA)	=	1454.549 KN		Corr. MT	=	-1163.639 kNm				
Minimum Reaction (RB)	=	545.451 KN		Corr. MT	=	-436.361 kNm				

1- 70RW

Transverse Ecc.	=	for 70RW	=	6.25	-0.5	-1.2	-1.395	=	3.155 m
Maximum Reaction (RA)	=	758.731 KN		Corr. MT	=	2394			
Minimum Reaction (RB)	=	241.269 KN		Corr. MT	=	761			

1-CLASS A + 1-70RW

Transverse Ecc.	for Class A	=	6.25	-0.5	-0.4	-0.9	=	4.45 m		
	for 70RW	=	6.25	-0.5	-0.4	-1.8	-0.25	-1.8	-1.395	0.105
Maximum Reaction (RA)	=	1043.595 KN	Corr. MT	=	284.86 x	4.45 +	758.73 x	0.105	=	1347
Minimum Reaction (RB)	=	510.405 KN	Corr. MT	=	269.136	4.45 +	241.269	0.105	=	1223

2-CLASS A + 1-70RW

<u>CORNER AT 70RW</u>												
Transverse Ecc.	=	for 1st Class A for 2nd Class A for 70RW	= = =	4.45 m 4.45 2.7	-0.90 -0.85	-0.85	0 -1.8	= -0.25	2.7 m -1.2	-1.395	=	-2.795
Maximum Reaction (RA)	=	1328.459 KN		Corr. MT	=	284.86 x	4.45 + +	284.86 x 758.73 x	2.700 -2.795		=	-84
Minimum Reaction (RB)	=	779.541 KN		Corr. MT	=	269.136	4.45 + +	269.14 x 241.27 x	2.700 -2.795		=	1250

1- 70RW + 1-CLASS A

for 70RW		=	6.25	-0.5	-1.2	-1.395	=	3.155 m	
for Class A		=	6.25	-0.5	-7.25	-0.25	-0.9 =	-2.650 m	
Maximum Reaction (RA)	=	1043.595 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650	= 1639
Minimum Reaction (RB)	=	510.405 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650	= 48

1- 70RW + 2-CLASS A

Transverse Ecc.	for 70RW	=	3.155 m							
	for 1st Class	=	-2.650 m							
	for 2nd Class	=	-2.650	-0.9	-0.85	=		-4.400 m		
Maximum Reaction (RA)	=	1328.459 KN	Corr. MT	=	758.73 x	3.155 +	284.86 x	-2.650	=	386
					+		284.86 x	-4.400		
Minimum Reaction (RB)	=	779.541 KN	Corr. MT	=	241.269	3.155 +	269.136	-2.650	=	-1136
					+		269.14 x	-4.400		

2- 70RW

Transverse Ecc.	for 1-70RW for 2-70RW	= =	3.16 m 3.155	-1.395	-1.2	-1.40	=	-0.835	m	
Maximum Reaction (RA)	=	1517.461 KN	Corr. MT	=	758.73 x	3.155 +	758.73 x	-0.835	=	1760
Minimum Reaction (RB)	=	482.539 KN	Corr. MT	=	241.269	3.155 +	241.269	-0.835	=	560

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	363.637	1618.186
2-CLASS A	727.275	1963.642
3-CLASS A	1090.912	1036.366
4-CLASS A	0.000	0.000
1- 70RW	758.731	2393.795
1-CLASS A + 1-70RW	1043.595	1347.312
1- 70RW + 1-CLASS A	1043.595	1638.906
2-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 2-CLASS A	0.000	0.000
2- 70RW	0.000	0.000

Reaction RB, , (kN)	Corr. MT (kNm)
136.363	606.814
272.725	736.358
409.088	388.634
0.000	0.000
241.269	761.205
510.405	1222.988
510.405	47.994
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 1090.912 KN
 Reaction RB = 409.088 KN

& Corresponding moment = 1036.366 kNm
 & Corresponding moment = 388.634 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 758.731 KN
 Reaction RB = 241.269 KN

& Max moment = 2393.795 kNm
 & Max moment = 761.205 kNm

Stability Check Summary

Description	P _{max} (kN/m ²)	P _{min} (kN/m ²)	Sliding	Overturnin g	Shear (Heel slab)	Shear (Toe slab)
Normal Dry Case						
Case 1 : DL+SIDL-Normal Dry Case	256.00	146.70	3.09	5.82	0.103	0.597
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	245.18	179.96	2.53	4.53		
Permissible	353	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Normal HFLCase						
Case 3 : DL+SIDL-Normal HFL Case	231.42	104.41	2.58	5.46		
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	214.26	133.55	2.10	4.06		
Permissible	353	0	1.5	2		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic Dry Case						
Case 5 : DL+SIDL-Long. Seismic Dry Case	252.93	158.22	2.49	3.60		
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	264.61	151.53	2.49	3.60		
Permissible	441	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Longitudinal Seismic HFL Case						
Case 7 : DL+SIDL-Long. Seismic HFL Case	210.903	134.134	2.05	3.21		
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	220.177	127.763	2.05	3.21		
Permissible	441	0	1.25	1.50		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic Dry Case						
Case 9 : DL+SIDL-Trans. Seismic Dry Case	254.644	156.505	3.54	5.68		
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	251.931	164.209	3.88	6.37		
Permissible	441	0	1.25	1.5		
Remarks	O.K	O.K	O.K	O.K		
Transverse Seismic HFL Case						
Case 11 : DL+SIDL-Trans. Seismic HFL Case	232.913	112.124	3.16	5.84		
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	228.128	119.812	3.16	5.84		
Permissible	441	0	1.25	1.50	4.192	4.192
Remarks	O.K	O.K	O.K	O.K	O.K	O.K

Reinforcement summary

Type of reinforcement	Area of steel required	Area of steel provided						
Heel Slab								
Main steel at bottom face	1610	16	mm bar @	200	mm c/c (i.e.)	2011	mm ²	(O.K)
		16	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	402	10	mm bar @	150	mm c/c (i.e.)	524	mm ²	(O.K)
Shear Reinforcement	OK, No shear reinf. Req.	10	mm 20 Legged bar @	150	mm c/c (i.e.)			
Toe Slab								
Main steel at bottom face	3010	20	mm bar @	200	mm c/c (i.e.)	3142	mm ²	(O.K)
		20	mm bar @	200	mm c/c (i.e.)			
Distribution reinforcement	628	12	mm bar @	150	mm c/c (i.e.)	754	mm ²	(O.K)
Shear Reinforcement	Provide Shear Reinf.	10	mm 22 Legged bar @	150	mm c/c (i.e.)			

Abutment Shaft								
Vertical steel at earth face	4650	25	mm bar @	200	mm c/c (i.e.)	4909	mm ²	(O.K)
		25	mm bar @	200	mm c/c (i.e.)			
Vertical steel at non-earth face	1440	20	mm bar @	200	mm c/c (i.e.)	1571	mm ²	(O.K)
Distribution steel	1620	16	mm 2 Legged bar @	200	mm c/c (i.e.)	2011	mm ²	(O.K)
Abutment Cap								
Top Longitudinal Reinf.	1049	12	mm dia bar	12	Nos.	1357	mm ²	(O.K)
Bottom Longitudinal Reinf.	1049	12	mm dia bar	12	Nos.	1357	mm ²	(O.K)
Side Face Longitudinal Reinf.		12	mm dia bar	200	mm c/c (i.e.)			
Transverse Reinf.	2.08E+06	12	mm 2 Legged bar @	150	mm c/c (i.e.)	2.62E+06	mm ³ /m	(O.K)
Corbel Reinforcement								
Primary tensile reinf.	1214	16	mm bar @	150	mm c/c (i.e.)	1340	mm ²	(O.K)
Horizontal Stirrups	287.75	12	mm 2 Legged bar @	in 2 Vertical Layers		452	mm ² /m	(O.K)
Vertical Stirrups	303.52	10	mm 2 Legged bar @	200	mm c/c (i.e.)	785	mm ³ /m	(O.K)

DESIGN OF WALLTYPE
ABUTMENT WITH OPEN
FOUNDATION

FOR MINOR BRIDGE AT CH:-
19+194

Details of Superstructure:

Skew Angle of Bridge = 0 Degree = 0.000 Radians COS θ = 1.000
SIN θ = 0.000

Radius of Curvature of Superstructure = 0 m
Design speed of vehicle = 100 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 9.580m	9.580m
Thickness of Expansion Joint	= 0.020m	0.020m
Slab projection Beyond C/L of Bearing (Back Side) =	0.200m	0.200m
Slab projection Beyond C/L of Bearing (Span Side) =	0.200m	0.200m
Span -c/c of E.J.	= 10.000m	10.00m
Type of Superstructure	= RCC SOLID SLAB	
Width of Crash barrier (Both Side)	= 0.500m	
Width of Carriageway	= 11.500m	
Projection beyond crash barrier	= 0.000m	
Thickness of Wearing coat	= 0.065m	
Length of Approach Slab (Right)	= 3.500m	3.500m
Width of Footpath on both side	= 0.000m	
Railing/kerb on footpath edge	= 0.000m	
Total Width of Superstructure	= 12.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

Type of Bearing = Tar Paper Bearing
Coeff. Of Friction for POT-PTFE Bearing = 0.5

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 300 kN/m² (as per geotechnical report with ground improvement)
SBC of soil-Normal Case = 335 kN/m²
SBC of soil-Seismic Case = 418 kN/m²

Coeff. of friction between concrete and soil = 0.7 for weathered rock

Permissible FOS against Sliding = 1.5 Normal Case
= 1.25 Seismic Case
Permissible FOS against Overturning = 2 Normal Case
= 1.5 Seismic Case

Dirt Wall

	Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	= 0.300m	0.300m
Width of Dirt wall at Bottom	= 0.300m	0.300m
Height of Uniform portion	= 0.865m	
Height of Tapering portion	= -0.143m	
Length of Dirt Wall at top (Uniform portion)	= 12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	= 0.720m	0.720m
Width of Abutmentcap at bottom of Tapering Portion	= 0.720m	0.720m
Projection of Abutment Cap (Span Side)	= 0.000m	0.000m
Projection of Abutment Cap Back Side	= 0.000m	0.000m
Abutmentcap thickness (Uniform portion)	= 0.300m	
Abutmentcap thickness (Tapering Portion)	= 0.000m	
Length of Abutment Cap at top (Uniform portion)	= 12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	= 12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	= 0.720m	
Width of abutment shaft	= 12.500m	12.500m
Thickness of Abutment shaft at Top	= 0.720m	0.720m

Design Calculation

RODIC

INPUT

Thickness of Abutment shaft at HFL	=	0.720m	0.720m
Thickness of Abutment shaft at Bottom	=	0.720m	0.720m

Solid Return Wall

Length of Return wall	=	3.700m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Cantilever Return Wall

Height of Return Wall-Free edge	=	0.000m
Height of wall at abutment	=	0.000m
Length of Return wall	=	0.000m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	6.720m
abutment pedestal width	=	0.720m
abutment pedestal Height	=	1.000m
Width of Toe Slab	=	2.300m
Width of Heel Slab	=	3.700m
Thickness of Toe slab at tip	=	0.500m
Thickness of Toe slab near shaft	=	1.000m
Thickness of heel slab at tip	=	0.500m
Thickness of heel slab near shaft	=	1.000m
Width of backfill on heel slab	=	3.700m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	6.544m
Height of back fill at back fill edge of heel slab	=	6.044m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.500m (skew dimension)
Width of foundation -Tapering portion	=	12.500m (skew dimension)

Levels

Deck Level at Median Edge=	1436.310m
Deck level at Outer Edge =	1436.023m
Deck level at center line =	1436.310m
Soffit Level at center of bridge =	1435.445m
Abutment cap top level =	1435.444m
Abutment cap bottom lvl (uniform portion ends)	1435.144m
Abutment cap bottom lvl (corbel portion ends)	1435.144m
Abutment shaft top level =	1435.144m
Ground level/LBL =	1432.575m
Abutment shaft bottom level =	1430.122m
Foundation level =	1429.122m
HFL	1433.575m

Cross Slope (Bi-directional)	=	2.500%
Height of Superstructure	=	0.800m
Min. Height of Footpath Side Pedestal (1)	=	0.000m
Height of Pedestal (2)	=	0.000m
Height of Pedestal (3)	=	0.000m
Height of Pedestal (4)	=	0.000m
Distance of nearest girder to c.l. of deck	=	0.000m
Height (Avg.) of Dirt Wall	=	0.722m
Abutment shaft Above G.L	=	2.569m
Abutment Shaft below G.L	=	2.453m
Height of abutment shaft	=	5.022m
MSL	=	1432.575m
Wedge over girder flange	=	0.0020m

Material Specification

Concrete Grade	=	M 35	
Characteristic compressive strength of concrete, f_{ck}	=	35.00 Mpa at 28 days	
Design Compressive strength of Concrete, f_{cd}	=	15.63 Mpa at 28 , (0.67/1.5 * f_{ck})	
Tensile strength of concrete , f_{ctm}	=	2.77 MPa	
Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02	
Ultimate Strain, ϵ_{cu2}	=	0.035	
E_{cm}	=	32308.250 N/mm ²	
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, f_y or f_{yk}	=	500 Mpa	
Design Yield Strength of Reinforcement, f_{yd}	=	434.78 Mpa	(1/1.15 * f_y)
Modulus of Elasticity of Steel (E_s)	=	200000.00 Mpa	
Dry weight of Concrete	=	25 kN/m ³	
Dry unit weight of soil	=	20 kN/m ³	
Permissible Crack Width	=	0.3 mm - For Moderrate/ severe Exposure Condition	
Maximum compressive stress in concrete under rare combination	=	0.48 f_{ck}	
	=	16.8	N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²
<u>Creep Coefficient</u>			
For Abutment Shaft	=	1.2	for 365 days
For Footing	=	1.2	for 365 days
<u>Clear Cover to Reinforcement</u>			
Earth Face	=	75	mm
Non-Earth Face	=	50	mm

Seismic Data:

NO NEED TO CHECK FOR SEISMIC EFFECT

Seismic Zone	=	4	
Z =Zone factor	=	0.24	
I =Importance factor	=	1.2	
R =Response Reduction factor	=	3	in Longitudinal direction
	=	1	In Transverse direction

Properties of backfill material :

c	=	0
ϕ	=	30
θ	=	90
β	=	0
δ	=	20.0

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.160m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.000m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

Self weight of Slab	=	0.80	x	10.00	x	12.50	x	25.00
	=	2500.00	KN					
Reaction at one end	=	1250.00	KN					
Transverse Eccentricity	=	0.000	m					

Super Imposed Dead Load Reactions (Excluding Wearing Course)

Weight of Crash barrier	=	2	x	8.00	x	10.00
	=	160.00	KN			
Reaction at one end	=	80.00	KN			
Transverse Eccentricity	=	0.00	m			

Reaction Due to Wearing Course only

Weight due to Wearing Coat	=	2.2	x	10	x	12.5
	=	275	KN			
Reaction at one end	=	137.5	KN			
Transverse Eccentricity	=	0.00	m			

Carriageway Live Load Reactions

Reduction Factor =

0.9 (for 3 Lane)

MAXIMUM REACTION CASE:

Congestion factor =

1 (As per Table 3 of IRC :112-2014)

1- 70RW + 2-CLASS A**Max CWLL**

Vertical	Transverse ecc
932.33	2.41

Min CWLL

Vertical	Transverse ecc
466.27	3.11

SV Loading**Max CWLL**

Vertical	Transverse ecc
2561.34	0.30

Min CWLL

Vertical	Transverse ecc
858.66	0.30

MAXIMUM TRASVERSE MOMENT CASE:**1- 70RW + 2-CLASS A****Max CWLL**

Vertical	Transverse ecc
932.33	2.41

Min CWLL

Vertical	Transverse ecc
466.27	3.11

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

VOLUME CALCULATION

C.G. Of Footing = 3.360 m
 C.G. Of shaft from toe tip = 2.660 m
 Distance between c.g. of shaft and footing = 0.700 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m ³	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	0.865	3.244	-0.210	0.490	-2.870	0.000
-Trapering portion	1	12.50	0.300	-0.143	-0.535	-0.210	0.490	-2.870	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-0.510	0.190	-3.170	0.000
(Corbel)	0.5	1	12.50	0.300	0.563	-0.460	0.240	-3.120	0.000
Cap (uniform portion)	1	12.50	0.720	0.300	2.700	0.000	0.700	-2.660	0.000
Cap (Corbel Portion)	1	12.50	0.720	0.000	0.000	0.000	0.700	-2.660	0.000
Shaft above HFL	1	12.50	0.720	1.569	14.121	0.000	0.700	-2.660	0.000
Shaft below HFL	1	12.50	0.720	2.953	26.577	0.000	0.700	-2.660	0.000
Solid Return Wall	2	3.70	0.500	6.438	23.821	-2.210	-1.510	-4.870	0.000
Cantilever Return wall(Rectangular portion)	2	0.00	0.500	0.000	0.000	-0.360	0.340	-3.020	0.000
Cantilever Return wall(Traingular portion)	2	0.00	0.500	0.000	0.000	-0.360	0.340	-3.020	0.000
Footing									
Heel Slab	1	12.50	3.700	0.750	34.688		-1.304	-4.664	0.000
Toe Slab	1	12.50	2.300	0.750	21.563		2.082	-1.278	0.000
Portion between Heel and Toe	1	12.50	0.720	1.000	9.000		0.700	-2.660	0.000
Back filling above HFL over Heel Slab	1	12.50	3.700	2.735	126.494		-1.510	-4.870	0.000
Back filling below HFL over Heel Slab	1	12.50	3.700	3.703	171.264		-1.552	-4.912	0.000
Backfill above Heel slab	1	12.50	3.700	6.294	291.109		-1.534	-4.894	0.000
Front Filling over Toe Slab	1	12.50	2.300	2.703	77.711		2.175	-1.185	0.000
Side filling between heel and toe	1	0.00	0.720	2.703	0.000		0.000	0.000	0.000
Approach Slab	1	12.500	1.750	0.300	6.563	-0.510	0.190	-3.170	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.000	2.735	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.000	3.703	0.000		0.000	0.000	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barri	2	8 kN/m	1.750	28.00kN		-0.210	0.490	-2.870

SECTIONAL PROPERTIES

Width of Footing (B) = 6.72 m
 Length of Footing (L) = 12.500 m

A = 6.720 x 12.500 = 84.000 m²
 ZL = 12.500 x 7.526 = 94.080 m³
 ZT = $\frac{IT1}{+} + \frac{IT2}{+}$
 distance of extreme point from centre

IT1 = 6.720 x 162.760 = 1093.75 m⁴
 IT2 (moment of inertia of triangle) = 6.720 x 0.000 + 0.500 x 6.720 x 0.000 x 39.063
 from centre of footing = 0.000 m⁴

Moment of inertia of two triangle = 0.000 m⁴
 Total moment of inertia = 1093.75 m⁴

Distance of extreme point from centre of footing = 6.250 + 0.000 = 6.250 m

Total Section modulus (ZT) = 175.000 m³

Load Factors (As per IRC:6-2014)**Table 3.1 Partial Safety Factor For Verification of Equilibrium**

-Refer Table 3.1 of IRC:6-2014

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Load, SIDL & Backfill except wearing course	1.050	0.950	1.050	0.950
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.500	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table 3.3 Partial Safety Factor For Verification of Serviceability Limit State

-Refer Table 3.3 of IRC:6-2014

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Load+SIDL including wearing course	1.000	1.00	1.00
wearing course	1.200	1.20	1.20
Back Filling Weight	1.000	1.00	1.00
Shrinkage Creep Effect	1.000	1.00	1.00
Earth Pressure due to back filling	1.000	1.000	1.000
CWLL and Associate load and FPLL	1.000	0.750	0.000
Live Load Surcharge	0.800	0.00	0.00

Table 3.4 Partial Safety Factor For Design of Foundation

-Refer Table 3.4 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Possible Load Combination

Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case
Normal HFL Case	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case
Longitudinal Seismic HFL Case	Case 7 : DL+SIDL-Long. Seismic HFL Case Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case
Transverse Seismic Dry Case	Case 9 : DL+SIDL-Trans. Seismic Dry Case Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case
Transverse Seismic HFL Case	Case 11 : DL+SIDL-Trans. Seismic HFL Case Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Seismic Coefficient Calculation

(As Per IRC:6-2014 , Clause 219)

Horizontal Seismic Force For Zone	4.0

Feq	=	Seismic forces to be resisted
Feq	=	Ah x (Dead load + Appropriate Live load)
Ah	=	horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \quad \frac{Sa}{g}}{R}$$

Z	=	Zone factor	=	0.24	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	3.0	in Longitudinal direction
			=	1.0	In Transverse direction

T = Fundamental period of the bridge member (in sec.) or horizontal vibrations.

$$= 2.0 \frac{D}{1000F}^{1/2}$$

D = Appropriate dead load of the superstructure , and live load in KN

F = Horizontal force in KN required to be applied at the center of mass of the superstructure for one mm horizontal deflection at the top of the pier/abutment along the considered direction of horizontal force.

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction
= 6.322 m

C.g. of Horizontal Force acting at a height from Foundation Level in Transverse direction
= 6.899 m

Abutment Cap Top Level - Foundation Level
= 6.322 m

Dimensions of Abutment Shaft

Length	=	12.50	m
Width	=	0.72	m

Moment of Inertia , $I_{\text{longitudinal}}$	=	0.389	m^4
Moment of Inertia , $I_{\text{transverse}}$	=	117.188	m^4

$$E_{cm} = 3.231E+07 \text{ kN/m}^2$$

Longitudinal Direction

Force	=	74.571	KN
D	=	1467.50	KN
T	=	0.2806	sec

Transverse Direction

Force	=	19768.282	KN
D	=	1747.220	KN
T	=	0.0188	sec

Medium Soil Strata

Sa/g = 2.5

Sa/g = 2.5

Seismic Coeff. In Longitudinal Direction = 0.12

Seismic Coeff. In Transverse Direction = 0.36

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

A _h	=	0.120	In Longitudinal direction
A _h	=	0.360	In Transverse direction
A _v	=	0.240	In Vertical direction

For Design of Foundation

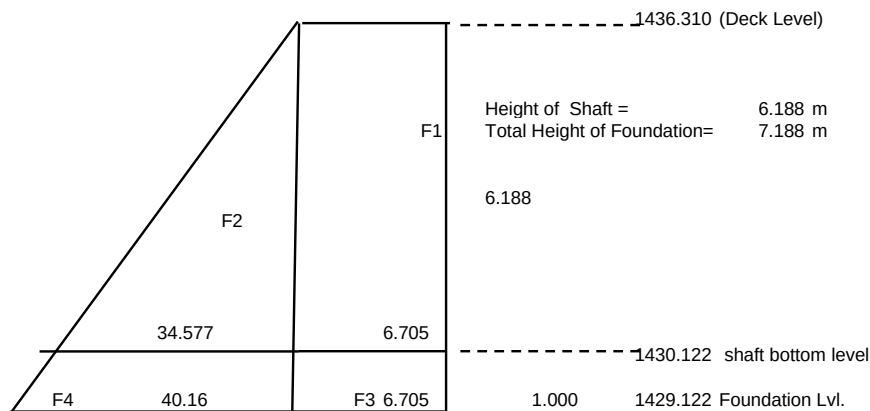
(35% increment in Seismic Coeff for Foundation as per IRC:6-2014, Clause No. 219.8)

A _h	=	0.162	In Longitudinal direction
A _h	=	0.360	In Transverse direction
A _v	=	0.240	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0	
	ϕ	=	30 degree	0.524 radians
	θ	=	90.00 degree	1.571 radians
	θ_1	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	20.0 degree	0.349 radians
	Kah	=	0.279 active component	
	Kph	=	3.766 Passive component	
	γ	=	20 kN/m ³	

Equivalent Live Load Surcharge height = 1.2 m
Assuming

**Earth Pressure Diagram**

Horizontal Forces and Moments @ RL					1430.122 m (at Shaft Base)		
@ RL					1429.122 m (at Foundation Level)		
<u>Due to Live Load Surcharge</u>							
Intensity for rectangular portion	=	0.279	x	20	x	1.2	= 6.705 kN/m ²
F1	=	6.705	x	6.188	x	12.500	= 518.648 kN
M1	=	518.65	x	3.09	=	1604.696 kN.m	at Shaft Bottom
F3	=	6.705	x	7.188	x	12.500	= 602.463 kN
M3	=	602.463	x	3.594	=	2165.252 kN.m	at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)	=	0.279	x	20	x	6.188	= 34.577 kN/m ²
F2	=	0.5	x	34.58	x	6.188	x 12.50
	=	1337.247 kN					

(Centre of pressure considered at an elevation of 0.42m of the height of the shaft as per cl. 217.1 of IRC:6-2014)

M2	=	1337.25	x	2.60	=	3475.451 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.279	x	20	x	7.188	= 40.164 kN/m ²
F4	=	0.5	x	40.16	x	7.188	x 12.50
	=	1804.376 kN					
M4	=	1804.38	x	3.02	=	5447.340 kN.m	at Foundation

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014				γ fluid	=	4.8 kN/m ³	
Intensity for triangular portion (At Shaft bottom level)	=	4.800	x	6.188	=	29.702 kN/m ²	
F	=	0.5	x	29.702	x	6.188	x 12.500
	=	1148.740 kN					
M	=	1148.74	x	2.063	=	2369.468 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	4.800	x	7.188	=	34.50 kN/m ²	
F	=	0.5	x	34.502	x	7.19	x 12.500
	=	1550.020 kN					
M	=	1550.02	x	2.396	=	3713.849 kN.m	at Foundation

Intensity of Passive pressure

$$\begin{aligned}
 &= 3.766 \times 20 \times 0.000 = 0.000 \text{ kN/m}^2 \\
 \text{Force due to passive @ Foundation, F} &= 0.5 \times 0.000 \times 12.50 \\
 &= \mathbf{0.000 \text{ kN}}
 \end{aligned}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = \mathbf{0.000 \text{ kN.m}} \quad \text{at Foundation}$$

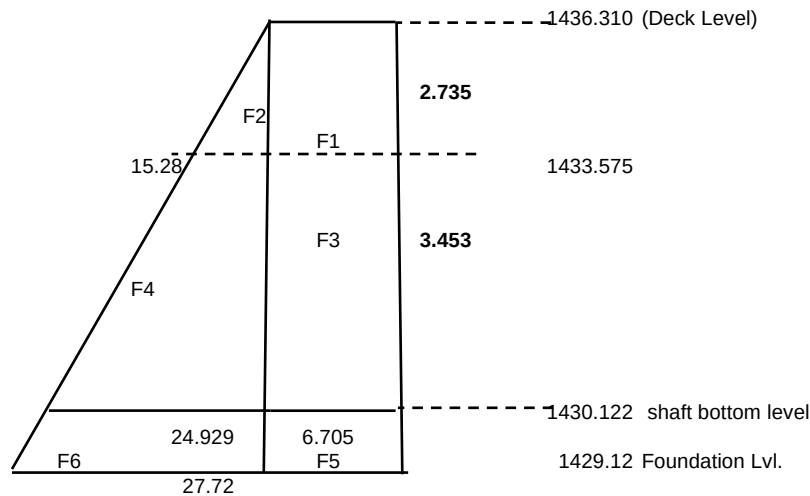
Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Lvl kN-m	At Shaft Bottom Lvl kN	At Foundation Lvl kN
Due to active Earth Pressure	3475.451	5447.340	1337.247	1804.376
Due to Minimum Fluid Pressure	2369.468	3713.849	1148.740	1550.020
Governing of Two	3475.451	5447.340	1337.247	1804.376
Due to Live Load Surcharge	1604.696	2165.252	518.648	602.463
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	30 degree	0.524 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	20.0 degree	0.349 radians
	Kah	=	0.279 active component	
	Kph	=	3.766 passive component	
	γ_d	=	20 kN/m ³	
	γ_{water}	=	10 kN/m ³	
Equivalent Live Load Surcharge height		=	1.2 m	

Assuming

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL****1430.1 m (at Shaft Base)****Due to Live Load Surcharge**

Intensity for rectangular portion	=	0.279	x	20	x	1.200	=	6.705 kN/m ²
F1	=	6.705	x	6.188	x	12.500	=	518.648 kN
M1	=	518.65	x	3.09	=	1604.696 kN.m		at Shaft Bottom
F3	=	6.705	x	7.188	x	12.500	=	602.463 kN
M3	=	602.46	x	3.59	=	2165.252 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.279	x	20	x	2.735	=	15.282 kN/m ²
(At Shaft bottom level) Below HFL	=	0.279	x	10	x	3.453	=	9.647 kN/m ²
F2	=	0.5	x	15.28	x	2.735	x	12.50
	=	261.232 kN						
F4	=	(15.28 + 24.93)	x	3.45	x	12.50		
	=	867.818 kN						
Total Force =		1129.050 kN						
M2	=	261.23	x	4.60	=	1202.109 kN.m		
M4	=	867.82	x	1.59	=	1378.471 kN.m		
Total Mome =		2580.58 kN.m		at Shaft Bottom				

Intensity for triangular portion

Upto HFL	=	0.279	x	20	x	2.735	=	15.282 kN/m ²
at	=	0.279	x	10	x	4.453	=	12.441 kN/m ²

Foundation

$$\begin{aligned}
 F2 &= 0.5 \times 15.28 \times 2.735 \times 12.50 \\
 &= \mathbf{261.232 \text{ kN}} \\
 F6 &= \left(\frac{15.28 + 27.72}{2} \right) \times 4.45 \times 12.50 \\
 &= \mathbf{1196.897 \text{ kN}}
 \end{aligned}$$

Total Force = 1458.129 kN

$$M2 = 261.23 \times 5.60 = \mathbf{1463.341 \text{ kN.m}}$$

$$M6 = 1196.90 \times 2.01 = \mathbf{2407.919 \text{ kN.m}}$$

Total Mome = 3871.26 kN.m Foundation Lvl.

Intensity of Passive pressure:

$$\begin{aligned}
 &= 3.766 \times 10 \times 0.00 = 0.000 \text{ kN/m}^2 \\
 \text{Force due to passive @ Foundation, F} &= 0.5 \times 0.000 \times 12.50 \\
 &= \mathbf{0.000 \text{ kN}}
 \end{aligned}$$

Moment due to passive @ Foundation, M

$$= 0.000 \times 0.000 = \mathbf{0.000 \text{ kN.m Foundation Lvl.}}$$

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	2580.581	3871.260	1129.050	1458.129
Due to Minimum Fluid Pressure	2369.468	3713.849	1148.740	1550.020
Governing of Two	2580.581	3871.260	1148.740	1550.020
Due to Live Load Surcharge	1604.696	2165.252	518.648	602.463
Due to Passive pressure		0.000		0.000

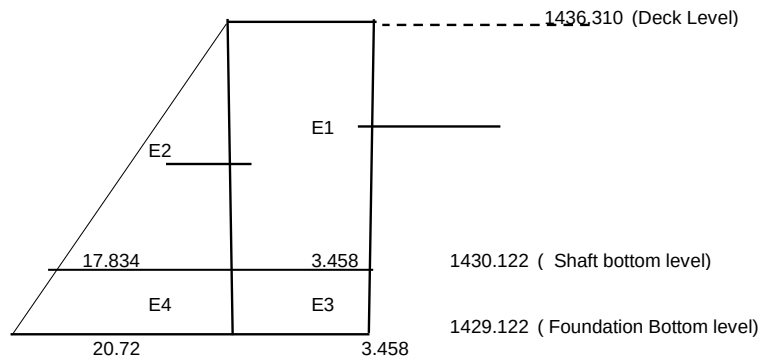
Earth Pressure : Seismic Dry Case**As per Clause 219.5.4, IRC:6-2014****Seismic Zone = 4.0****Dynamic increment due to seismic force**

$$C_a = \frac{\cos^2(\phi - \lambda - \alpha) \cos \delta (1 \pm \alpha v)}{\cos^2 \alpha \cos(\alpha + \delta + \lambda) \cos \lambda [1 + \sqrt{\sin(\phi + \delta) \sin(\phi - \beta - \lambda) / (\cos(\alpha + \delta + \lambda) \cos(\alpha - \beta))}]^2}$$

αh	=	0.120		
αv	=	0.240		
ϕ	=	30.00	0.524	
δ	=	20.00	0.349	
α	=	0.00	0.000	
β	=	0.00	0.000	
αh	=	HORIZONTAL SEISMIC COEFFICIENT		
αv	=	VERTICAL SEISMIC COEFFICIENT		
ϕ	=	ANGLE OF INTERNAL FRICTION OF SOIL		
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL		
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,		
β	=	SLOPE OF EARTH FILL		
λ	=	$\tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)}$		
				0.096
				0.157
C_a	=	$\frac{1}{0.423}$	$\frac{2}{0.296}$	
Ca	=	0.423	0.296	
Ka	=	0.423	0.279	
Dynamic Increment	=	0.423	-0.279	0.144

3 Earth Pressure : DRY CASE (Seismic case)

Equivalent Live Load Surcharge height = 1.2 m
 Assuming $\gamma_{dry} = 20 \text{ kN/m}^3$
 $\gamma_{water} = 10.00 \text{ kN/m}^3$

**Earth Pressure Diagram for Dynamic Increment****Horizontal Forces and Moments @ RL**

			1430.1 m (at Shaft Base)			
			1429.1 m (at Foundation Bottom Level)			
Due to Dynamic Live Load Surcharge						
=	0.144	x	20	x	1.2	= 3.458 kN/m ²
at Shaft Bottom Level						
E1	=	3.458	x	6.188	x	12.500 = 267.510 kN
M1	=	267.510	x	4.146		= 1109.087 kN.m
at Foundation Bottom Level						
E3	=	3.458	x	7.188	x	12.500 = 310.741 kN
M3	=	310.741	x	4.816		= 1496.515 kN.m

Due to Dynamic Active Earth Pressure

(At Shaft bottom level)

$$= 0.144 \times 20 \times 6.188 = 17.834 \text{ kN/m}^2$$

(at Foundation Bottom Level)

$$= 0.144 \times 20 \times 7.188 = 20.716 \text{ kN/m}^2$$

$$E2 = 0.50 \times 17.83 \times 6.19 \times 12.500$$

$$= 689.731 \text{ kN}$$

$$E4 = 0.50 \times 20.72 \times 7.19 \times 12.500$$

$$= 930.669 \text{ kN}$$

$$M2 = 689.73 \times 3.09 = 2134.027 \text{ kN.m} \quad (\text{ Shaft bottom level})$$

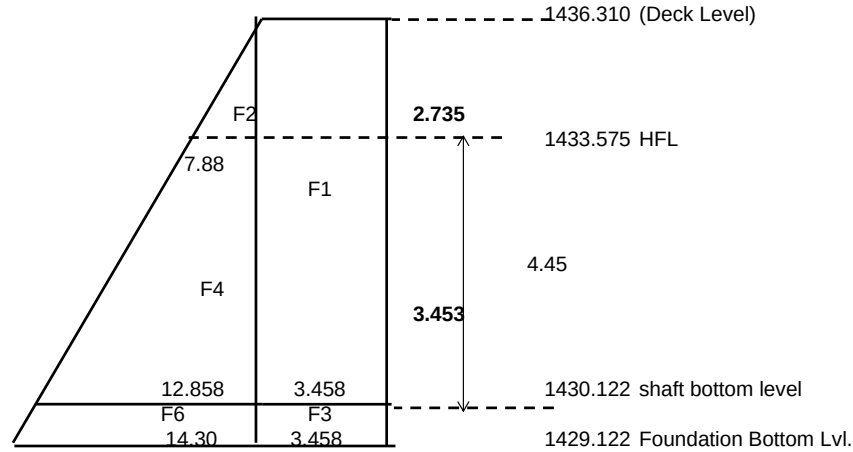
$$M4 = 930.67 \times 3.59 = 3344.823 \text{ kN.m} \quad (\text{ Foundation Bottom level})$$

Summary of Moment and Horizontal Force**Dry Seismic Case**

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom kN	At Foundation Bottom kN
Due to active Earth Pressure(Static)	3475.451	5447.340	1337.247	1804.376
Due to active Earth Pressure (dynamic Increment)	2134.027	3344.823	689.731	930.669
Total Earth Pressure	5609.478	8792.163	2026.978	2735.045
Due to Minimum Fluid Pressure	2369.468	3713.849	1148.740	1550.020
Governing of Two	3475.451	8792.163	2026.978	2735.045
Due to Live Load Surcharge (Static)	1604.696	2165.252	518.648	602.463
Due to Live Load Surcharge(Dynamic)	1109.087	1496.515	267.510	310.741
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

Dynamic Increment	=	0.144
γ_d	=	20 kN/m ³
γ_{water}	=	10 kN/m ³
Equivalent Live Load Surcharge height	=	1.2 m
Assuming		

**Earth Pressure Diagram****Horizontal Forces and Moments @ RL**

1430.122 m (at Shaft Base)
1429.122 m (at Foundation Bottom Level)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.144	x	20	x	1.200	=	3.458 kN/m ²

at Shaft Bottom Level

F1	=	3.458	x	6.188	x	12.500	=	267.510 kN
M1	=	267.51	x	4.08	=	1092.533 kN.m		

at Foundation Bottom Level

F3	=	3.458	x	7.188	x	12.500	=	310.741 kN
M3	=	310.74	x	4.74	=	1474.179 kN.m		

Due to Dynamic Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.144	x	20	x	2.735	=	7.882 kN/m ²

(At Shaft bottom level) Below HFL	=	0.144	x	10	x	3.453	=	4.976 kN/m ²

(At Foundation bottom level) Below HFL	=	0.144	x	10	x	4.453	=	6.417 kN/m ²

F2	=	0.5	x	7.88	x	2.73	x	12.50
	=	134.739 kN						
F4	=	$\left(\frac{7.88 + 12.86}{2} \right) \times 3.45$				x	12.50	
	=	447.607 kN						
F6	=	$\left(\frac{7.88 + 14.30}{2} \right) \times 4.45$				x	12.50	
	=	617.341 kN						

Total Force (F2 + F4)	=	582.346 kN	at Shaft Bottom Level
Total Force (F2 + F6)	=	752.080 kN	at Foundation Bottom Level

M2	=	134.74	x	4.82	=	649.510 kN.m

M4	=	447.61	x	1.59	=	710.994 kN.m

Total Mome = 1360.503 kN.m at Shaft Bottom

M2	=	134.74	x	5.82	=	784.249 kN.m

M6	=	617.34	x	2.01	=	1241.966 kN.m

Total Mome = 2026.215 kN.m at Foundation Bottom Level

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom Lvl kN	At Foundatio n Bottom Lvl kN
Due to active Earth Pressure(Static)	2580.581	3871.260	1129.050	1458.129
Due to active Earth Pressure (Dynamic Increment)	1360.503	2026.215	582.346	752.080
Total Earth Pressure	3941.084	5897.475	1711.396	2210.209
Due to Minimum Fluid Pressure	2369.468	3713.849	1148.740	1550.020
Governing of Two	3941.084	5897.475	1711.396	2210.209
Due to Live Load Surcharge(Static)	1604.696	2165.252	518.648	602.463
Due to Live Load Surcharge (Dynamic Increment)	1092.533	1474.179	267.510	310.741
Due to passive pressure		0.000		0.000

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1250.00	1.35	1.35	1687.50	1687.50	
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00	
WC	=	137.50	1.75	1.75	240.63	240.63	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmin	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmin	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmax-Transv. Moment Case		0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A

Braking Force = 0.2 x 1000 + 0.05 x 554
= 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	2036.13	0	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case	2036.13	341.55	1018.063	1018.063	1- 70RW + 2-CLASS A
	2036.13	0	1018.063	1018.063	SV Loading
DL+SIDL+LL-Min Reaction case	2036.13	341.55	1018.063	1018.063	1- 70RW + 2-CLASS A
	2036.13	0	1018.063	1018.063	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2036.13	341.55	1018.063	1018.063	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	528.30	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	573.84	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	573.84	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	573.84	1018.063	1018.063	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	158.490	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	204.030	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	204.030	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	204.030	1018.063	1018.063	

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1.35	1.35	1687.50	1687.50
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00
WC	=	137.50	1.75	1.75	240.63	240.63
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax- Reaction case	=	0.00	1.5	0.75	0.00	0.00
CWLLmax- Transv. Moment Case	=	0.00	1.5	0.75	0.00	0.00
CWLLmin	=	0.00	1.5	0.75	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	2036.13	0.000	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case	2036.13	341.550	1018.063	1018.063	1- 70RW + 2- CLASS A
	2036.13	0.000	1018.063	1018.063	SV Loading
DL+SIDL+LL-Min Reaction case	2036.13	341.550	1018.063	1018.063	1- 70RW + 2- CLASS A
	2036.13	0.000	1018.063	1018.063	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2036.13	341.550	1018.063	1018.063	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	713.21	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	758.75	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	758.75	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	758.75	1018.063	1018.063	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	213.962	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	259.502	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	259.502	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	259.502	1018.063	1018.063	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1	1.00	1250.00	1250.00
SIDL except wc	=	80.00	1	1.00	80.00	80.00
WC	=	137.50	1	1.00	137.50	137.50
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1	0.20	0.00	0.00
CWLLmin	=	0.00	1	0.20	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	0.000	733.750	733.750	
DL+SIDL+LL-Max Reaction case	1467.50	227.700	733.750	733.750	1- 70RW + 2- CLASS A
	1467.50	0.000	733.750	733.750	SV Loading
DL+SIDL+LL-Min Reaction case	1467.50	227.700	733.750	733.750	1- 70RW + 2- CLASS A
	1467.50	0.000	733.750	733.750	SV Loading
DL+SIDL+LL-Max Transv. Moment case	1467.50	227.700	733.750	733.750	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	475.470	733.750	733.750	
DL+SIDL+LL-Max Reaction case	1467.50	521.010	733.750	733.750	Dry Case
DL+SIDL+LL-Min Reaction case	1467.50	521.010	733.750	733.750	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1467.50	521.010	733.750	733.750	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	142.641	733.750	733.750	
DL+SIDL+LL-Max Reaction case	1467.50	188.181	733.750	733.750	Dry Case
DL+SIDL+LL-Min Reaction case	1467.50	188.181	733.750	733.750	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1467.50	188.181	733.750	733.750	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1.05	1.05	1312.50	1312.50
SIDL except wc	=	80.00	1.05	1.05	84.00	84.00
WC	=	137.50	1.35	1.35	185.63	185.63
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1.5	0.00	0.00	0.00
CWLLmin	=	0.00	1.5	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1582.13	0.000	791.063	791.063	
DL+SIDL+LL-Max Reaction case	1582.13	341.550	791.063	791.063	1- 70RW + 2- CLASS A
	1582.13	0.000	791.063	791.063	SV Loading
DL+SIDL+LL-Min Reaction case	1582.13	341.550	791.063	791.063	1- 70RW + 2- CLASS A
	1582.13	0.000	791.063	791.063	SV Loading
DL+SIDL+LL-Max Transv. Moment case	1582.13	341.550	791.063	791.063	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	1582.13	713.21	791.063	791.063	
DL+SIDL+LL-Max Reaction case		1582.13	713.21	791.063	791.063	Dry Case
DL+SIDL+LL-Min Reaction case		1582.13	713.21	791.063	791.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1582.13	713.21	791.063	791.063	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1467.50	1582.13	213.962	791.063	791.063	
DL+SIDL+LL-Max Reaction case		1582.13	213.962	791.063	791.063	Dry Case
DL+SIDL+LL-Min Reaction case		1582.13	213.962	791.063	791.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1582.13	213.962	791.063	791.063	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1250.00	1	1	1	1250.00	1250.00	1250.00
SIDL except wc	=	80.00	1	1	1	80.00	80.00	80.00
WC	=	137.50	1.20	1.20	1.20	165.00	165.00	165.00
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax- Reaction case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax- Transv. Moment Case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmin	=	0.00	1	0.75	0	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1495.00	0.000	747.500	747.500	
DL+SIDL+LL-Max Reaction case	1495.00	227.700	747.500	747.500	1- 70RW + 2- CLASS A Dry Case
	1495.00	0.000	747.500	747.500	SV Loading
DL+SIDL+LL-Min Reaction case	1495.00	227.700	747.500	747.500	1- 70RW + 2- CLASS A HFL Case
	1495.00	0.000	747.500	747.500	SV Loading
DL+SIDL+LL-Max Transv. Moment case	1495.00	227.700	747.500	747.500	

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1495.00	0.000	747.500	747.500	
DL+SIDL+LL-Max Reaction case	1495.00	170.775	747.500	747.500	Dry Case
DL+SIDL+LL-Min Reaction case	1495.00	170.775	747.500	747.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1495.00	170.775	747.500	747.500	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	Max (Fh/2 or μR)	
DL+SIDL	1495.00	0.000	747.500	747.500	
DL+SIDL+LL-Max Reaction case	1495.00	0.000	747.500	747.500	Dry Case
DL+SIDL+LL-Min Reaction case	1495.00	0.000	747.500	747.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1495.00	0.000	747.500	747.500	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

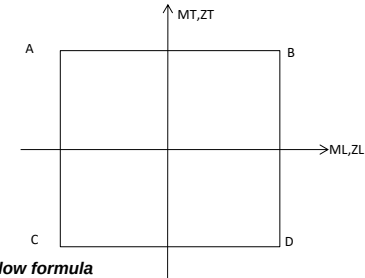
CENTRIFUGAL FORCE	C	=	$\frac{W V^2}{127 R}$			
			Normal Case		Seismic Case	
Design Speed	V	=	100.00	kmph	100.00	kmph
Live Load	W	=	932.33	kN	932.33	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1429.122 m

Properties of Footing Base:

A	=	84.000	m ²
ZL	=	94.080	m ³
ZT	=	175.000	m ³



For Skew bridges, Resolve the moment due to braking force, Seismic force due to superstructure & substructure in both major and minor principal axis using below formula

Moment along longitudinal axis	ML = ML Cos θ + MT Sin θ
Moment along transverse axis	MT = MT Cos θ - ML Sin θ

Case 1 : DL+SIDL-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load (P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1250.000	0.860	1075.000	0.000	0.000
SIDL except Wearing Course	1			80.000	0.860	68.800	0.000	0.000
Wearing Course	1			137.500	0.860	118.250	0.000	0.000
				1467.500		1262.050		0.000
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	3.244	81.094	0.490	39.736	0.000	0.000
Dirt Wall-Tapered portion	1	25	-0.535	-13.383	0.490	-6.558	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	0.190	5.344	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	0.240	3.375	0.000	0.000
Cap - (uniform portion)	1	25	2.700	67.500	0.700	47.250	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.700	0.000	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.340	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.340	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.490	13.720	0.000	0.000
Approach Slab	1	25	6.563	164.063	0.190	31.172	0.000	0.000
				369.461		134.039		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	23.821	595.515	-1.510	-899.228	0.000	0.000
Abutment Shaft	1	25	40.698	1017.450	0.700	712.215	0.000	0.000
Back filling over heel slab	1	20	291.109	5822.181	-1.534	-8934.098	0.000	0.000
Front Filling over toe slab	1	20	77.711	1554.225	2.175	3379.733	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	25	34.688	867.188	-1.304	-1131.198	0.000	0.000
Toe slab	1	25	21.563	539.063	2.082	1122.448	0.000	0.000
portion between heel & toe	1	25	9.000	225.000	0.700	157.500	0.000	0.000
Vertical Components of active earth pressure	1			656.739	-3.360	-2206.644	0.000	0.000
				11277.361		-7799.271		0.000
Total				13114.321		-6403.182		0.000

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501
due to Earth pressure	1	1804.376		5447.340

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load (P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.950	1187.500	-2.500	-2968.750
0.950	76.000	-2.500	-190.000
1.000	137.500	-2.500	-343.750
	1401.000		-3502.500
0.950	77.039	-2.870	-221.102
0.950	-12.714	-2.870	36.488
0.950	26.719	-3.170	-84.698
0.950	13.359	-3.120	-41.681
0.950	64.125	-2.660	-170.573
0.950	0.000	-2.660	0.000
0.950	0.000	-3.020	0.000
0.950	0.000	-3.020	0.000
0.950	26.600	-2.870	-76.342
0.950	155.859	-3.170	-494.074
	350.988		-1051.982
0.950	565.739	-4.870	-2755.150
0.950	966.577	-2.660	-2571.096
0.950	5531.072	-4.894	-27071.796
0.950	1476.514	-1.185	-1750.340
0.950	0.000	0.000	0.000
0.950	823.828	-4.664	-3842.701
0.950	512.109	-1.278	-654.362
0.950	213.750	-2.660	-568.575
0.950	623.902	-6.720	-4192.623
	10713.492		-43406.643
	12465.480		-47961.125

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00
1804.38	5447.34	0.00	0.00
2538.126	10086.841	0.000	0.000

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2706.564		8171.010
		3497.627		13172.898

Summary of Forces For SBC

P	13114.321	kN
ML	3683.659	kNm
MT	0.000	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		13114.321		-6403.182		0.000
CWLL-Max. Reaction case	1	932.327	0.860	801.801	2.411	2248.220
Vertical Components of LL Surcharge	1	219.279	-3.360	-736.776	0.000	0.000
Total		14265.927		-6338.157		2248.220

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501
due to Earth pressure	1	1804.376		5447.340
due to Live load surcharge	1	602.463		2165.252

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2706.564		8171.010
due to Live load surcharge	1.2	722.955		2598.302
		4220.582		15771.200

Summary of Forces For SBC

P	14265.927	kN
ML	5913.935	kNm
MT	2248.220	kNm

Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		13114.321		-6403.182		0.000
CWLL-Max. Reaction case	1	2561.336	0.860	2202.749	0.300	768.401
Vertical Components of LL Surcharge	1	219.279	-3.360	-736.776	0.000	0.000
Total		15894.936		-4937.209		768.401

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2706.56	8171.01	0.00	0.00
3497.627	13172.898	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	12465.480		-47961.12
0.000		-2.500	0.00
0.950		-6.720	-1399.87
	12465.480		-49361.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00
1804.38	5447.34	0.00	0.00
602.463	2165.252	0.000	0.000
3140.59	12252.09	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2706.56	8171.01	0.00	0.00
722.96	2598.30	0.00	0.00
4220.582	15771.200	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	12465.48038		-47961.1249
0.000		-2.500	0.00
0.950		-6.720	-1399.87
	12465.480		-49361.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00

Design Calculation

RODIC

Stability of Foundation

due to Earth pressure	1	1804.376	5447.340
due to Live load surcharge	1	602.463	2165.252

1804.38	5447.34	0.00	0.00
602.463	2165.252	0.000	0.000
3140.59	12252.09	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2706.564		8171.010
due to Live load surcharge	1.2	722.955		2598.302
		4220.582		15771.200

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2706.56	8171.01	0.00	0.00
722.96	2598.30	0.00	0.00
4220.582	15771.200	0.000	0.000

Summary of Forces For SBC

P	15894.936	KN
ML	7314.883	kNm
MT	768.401	kNm

Case 3 : DL+SIDL-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				1467.500		1262.050		0.000
Substructure & Foundation -Portion 1				369.461		134.039		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	23.821	595.515	-1.510	-899.228	0.000	0.000
Shaft above HFL	1	25	14.121	353.025	0.700	247.117	0.000	0.000
Shaft below HFL	1	15	26.577	398.655	0.700	279.058	0.000	0.000
Back filling above HFL over heel slab	1	20	126.494	2529.875	-1.510	-3820.111	0.000	0.000
Back filling below HFL over heel slab	1	10	171.264	1712.637	-1.552	-2657.385	0.000	0.000
Front Filling over toe slab	1	10	77.711	777.112	2.175	1689.867	0.000	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1	15	34.688	520.313	-1.304	-678.719	0.000	0.000
Toe slab	1	15	21.563	323.438	2.082	673.469	0.000	0.000
Portion between Heel & Toe	1	15	9.000	135.000	0.700	94.500	0.000	0.000
Vertical Components of active earth pressure	1			564.161	-3.360	-1895.582	0.000	0.000
				7909.731		-6967.013		0.000
Total				9746.692		-5570.924		0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	1401.000		-3502.500
	350.988		-1051.982
0.950	565.739	-4.870	-2755.150
0.950	335.374	-2.660	-892.094
0.950	378.722	-2.660	-1007.401
0.950	2403.381	-4.870	-11704.467
0.950	1627.006	-4.912	-7991.254
0.950	738.257	-1.185	-875.170
0.950	0.000	0.000	0.000
0.950	494.297	-4.664	-2305.620
0.950	307.266	-1.278	-392.617
0.950	128.250	-2.660	-341.145
0.950	535.953	-6.720	-3601.605
	7514.245		-31866.524
	9266.233		-36421.007

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501
due to Earth pressure	1	1550.020		3871.260

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00
1550.02	3871.26	0.00	0.00
2283.77	8510.76	0.00	0.00

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2325.030		5806.890
		3116.093		10808.778

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2325.03	5806.89	0.00	0.00
3116.09	10808.78	0.00	0.00

Summary of Forces For SBC

P	9746.692	KN
ML	2939.837	kNm
MT	0.000	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		9746.692		-5570.924		0.000
CWLL-Min. Reaction case	1	466.273	0.860	400.995	3.109	1449.594
Vertical Components of LL Surcharge	1	219.279	-3.360	-736.776	0.000	0.000
Total		10432.244		-5906.705		1449.594

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501
due to Earth pressure	1	1550.020		3871.260
due to Live load surcharge	1	602.463		2165.252

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2325.030		5806.890
due to live load surcharge	1.2	722.955		2598.302
		3839.048		13407.080

Summary of Forces For SBC

P	10432.244	kN
ML	4769.308	kNm
MT	1449.594	kNm

Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		9746.692		-5570.924		0.000
CWLL-Min. Reaction case	1	858.664	0.860	738.451	0.300	257.599
Vertical Components of LL Surcharge	1	219.279	-3.360	-736.776	0.000	0.000
Total		10824.635		-5569.249		257.599

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1435.445	4639.501
due to Earth pressure	1	1550.020		3871.260
due to Live load surcharge	1	602.463		2165.252

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Earth pressure	1.5	2325.030		5806.890
due to live load surcharge	1.2	722.955		2598.302
		3839.048		13407.080

Summary of Forces For SBC**Vertical Forces For Restoring or Resisting Effect**

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	9266.233		-36421.007
0.000	0.000	-2.500	0
0.950	208.315	-6.720	-1399.874
	9474.547		-37820.881

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00
1550.02	3871.26	0.00	0.00
602.463	2165.252	0.000	0.000
2886.23	10676.01	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2325.03	5806.89	0.00	0.00
722.955	2598.302	0.000	0.000
3839.05	13407.08	0.00	0.00

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	9266.233		-36421.007
0.000	0.000	-2.500	0.00
0.950	208.315	-6.720	-1399.874
	9474.547		-37820.881

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4639.50	0.00	0.00
1550.02	3871.26	0.00	0.00
602.463	2165.252	0.000	0.000
2886.23	10676.01	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
2325.03	5806.89	0.00	0.00
722.955	2598.302	0.000	0.000
3839.05	13407.08	0.00	0.00

P	10824.635	kN
ML	5106.764	kNm
MT	257.599	kNm

Case 5 : DL+SIDL-Long. Seismic Dry Case

Seismic Effect Factor =	1	$\alpha_h =$	0.162	In Longitudinal direction	Weight of shaft below Ground level	=	551.925 KN
		$\alpha_h =$	0.360	In Transverse direction	Weight of back fill below Ground level	=	2269.025 KN
		$\alpha_v =$	0.240	In Vertical direction			

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha_h \times P$ (kN)	FT = $0.3 \times \alpha_h \times P$ (kN)	Fv = $0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1250.000		135.000	90.000	0.860	1075.000	77.400	1435.978		0.000	0.000	925.605
SIDL except Wearing Course	1			80.000		8.640	5.760	0.860	68.800	4.954	1436.759		0.000	0.000	65.983
Wearing Course	1			137.500		14.850	9.900	0.860	118.250	8.514	1436.310		0.000	0.000	106.742
				1467.500		158.490	105.660		1262.050	90.868				0.000	1098.330
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	3.244	81.094	13.137	8.758	5.839	0.490	39.736	2.861	1435.878	88.748	0.000	0.000	59.166
Dirt Wall-Tapered portion	1	25	-0.535	-13.383	-2.168	-1.445	-0.964	0.490	-6.558	-0.472	1435.516	-13.863	0.000	0.000	-9.242
Bracket - Uniform portion	1	25	1.125	28.125				0.190	5.344						
Bracket - Tapered portion	1	25	0.563	14.063				0.240	3.375						
Cap - (uniform portion)	1	25	2.700	67.500	10.935	7.290	4.860	0.700	47.250	3.402	1435.294	67.491	0.000	0.000	44.994
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.700	0.000	0.000	1435.144	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.340	0.000	0.000	1436.310	0.000	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.340	0.000	0.000	1436.310	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.490	13.720				0.000	0.000	0.000
Approach Slab	1	25	6.563	164.063				0.190	31.172				0.000	0.000	0.000
				369.461	21.904	14.603	9.735		134.039	5.791		142.376		0.000	94.917
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	23.821	595.515	96.473	64.316	42.877	-1.510	-899.228	-64.744	1433.163	389.837	0.000	0.000	259.891
Abutment Shaft	1	25	40.698	1017.450	75.415	50.277	33.518	0.700	712.215	23.462	1433.860	357.279	0.000	0.000	238.186
Back filling over heel slab	1	20	291.109	5822.181	0.000	0.000	0.000	-1.534	-8934.098	0.000	1433.163	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1	20	77.711	1554.225				2.175	3379.733				0.000	0.000	0.000
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	0.000
Heel slab	1	25	34.688	867.188				-1.304	-1131.198				0.000	0.000	0.000
Toe slab	1	25	21.563	539.063				2.082	1122.448				0.000	0.000	0.000
portion between heel & toe	1	25	9.000	225.000				0.700	157.500				0.000	0.000	0.000
Vertical component of active earth pressure	1			656.739				-3.360	-2206.644				0.000	0.000	0.000
Vertical component of dynamic increment of earth pressure	1			338.736				-3.360	-1138.152				0.000	0.000	0.000
				11616.096	171.888	114.592	76.395		-8937.423	-41.282		747.116		0.000	498.077
Total =				13453.057	193.793	287.685	191.790		-7541.334	55.377		889.492		0.000	1691.324

-191.790

-55.377

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha_v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure								
Dead Load	0.95			1187.500	85.500	-2.500	-2968.750	-213.750
SIDL except Wearing Course	0.95			76.000	5.472	-2.500	-190.000	-13.680
Wearing Course	1.00			137.500	9.900	-2.500	-343.750	-24.750
				1401.000	100.872		-3502.500	-252.180
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.95	25	3.244	77.039	5.547	-2.870	-221.102	-15.919
Dirt Wall-Tapered portion	0.95	25	-0.535	-12.714	-0.915	-2.870	36.488	2.627
Bracket - Uniform portion	0.95	25	1.125	26.719				
Bracket - Tapered portion	0.95	25	0.563	13.359				

For Overturning or Sliding Effect

Load Factor	FL = $\alpha_h \times P$ (kN)	C.g. of Force (m)	MLs due to FL
1.0	13.137	1435.878	88.748
1.0	-2.168	1435.516	-13.863

Design Calculation

RODIC

Stability of Foundation

Cap - (uniform portion)	0.95	25	2.700	64.125	4.617	-2.660	-170.573	-12.281
Cap - (corbel portion)	0.95	25	0.000	0.000	0.000	-2.660	0.000	0.000
Cantilever Return Wall-Rectangle portion	0.95	25	0.000	0.000	0.000	-3.020	0.000	0.000
Cantilever Return Wall-Triangle portion	0.95	25	0.000	0.000	0.000	-3.020	0.000	0.000
RCC Railing or Crash Barrier	0.95			26.600		-2.870	-76.342	
Approach Slab	0.95	25	6.563	155.859		-3.170	-494.074	
				350.988	9.248		-925.603	-25.573
Substructure & Foundation -Portion 2								
Abutment Shaft	0.95	25	40.698	966.577	29.855	-2.660	-2571.096	-79.414
Solid Return wall	0.95	25	23.821	565.739	40.733	-4.870	-2755.150	-198.371
Back filling over heel slab	0.95	20	291.109	5531.072	0.000	-4.894	-27071.796	0.000
Front Filling over Pile Cap	0.95	20	77.711	1476.514		-1.185	-1750.340	
Side filling between heel and toe	0.95	20	0.000	0.000		0.000	0.000	
Heel slab	0.95	25	34.688	823.828		-4.664	-3842.701	
Toe slab	0.95	25	21.563	512.109		-1.278	-654.362	
portion between heel & toe	0.95	25	9.000	213.750		-2.660	-568.575	
Vertical component of active earth pressure	0.95			623.902		-6.720	-4192.623	
Vertical component of dynamic increment of earth pressure	0.95			321.799		-6.720	-2162.489	
				11035.291	70.588		-45569.13	-277.785
Total =				12787.279	180.709		-49997.23	-555.538

1.0	10.935	1435.294	67.491
1.0	0.000	1435.144	0.000
1.0	0.000	1436.310	0.000
1.0	0.000	1436.310	0.000
	21.904		142.376
1.0	70.709	1433.860	334.985
1.0	96.473	1433.163	389.837
1.0	0.000	1433.163	0.000
	167.183		724.822
	189.087		867.198

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	158.490	1435.445	4639.501	1098.330
due to Substructure	1	193.793	129.195		889.492	592.995
due to Earth pressure	1	2735.045			8792.163	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	158.49	1098.33
193.79	889.49	0.00	0.00	0.00	0.00	129.20	592.99
2735.04	8792.16						
3662.59	14321.16	0.00	0.00	0.00	0.00	287.69	1691.32

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.89
due to Substructure	1.5	283.630		1300.80
due to Active Earth pressure	1.5	2706.564		8171.01
due to dynamic Earth pressure	1.5	1396.003		5017.23
		5177.260		19490.929

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
283.63	1300.80	0.00	0.00
2706.564	8171.010	0.000	0.000
1396.003	5017.235	0.000	0.000
5177.26	19490.93	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	13644.847	13261.267	kN
ML	6835.198	6724.445	kNm
MT	1691.324	1691.324	kNm

Summary of Restoring Forces

Vertical Load	12606.571	kN
Moment	-50552.772	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Forces from Superstructure				1467.500		158.490	105.660		1262.050	90.868				0.000	1098.330
Forces from Substructure				11985.557	193.793	129.195	86.130		-8803.384	-35.491		889.492		0.000	592.995
CWLL-Max. Reaction case	0.20			186.47		20.138	13.426	0.860	160.360	11.546	1437.510		2.411	449.644	168.920
Vertical component of LL Surcharge	0.20			43.86				-3.360	-147.355				0.000	0.000	

Design Calculation

RODIC

Stability of Foundation

Vertical component of dynamic increment LL Surcharge	0.20			22.62				-3.360	-76.003				0.000	0.000	
Total =				13705.998	193.793	307.823	205.216		-7604.333	66.922		889.492		449.644	1860.244
								-205.216		-66.922					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				1401.000	100.872	0.000	-3502.50	-252.180
Forces from Substructure				11386.279	79.837	0.000	-46494.73	-303.358
CWLL-Max. Reaction case	0.00			0.00	0.000	-2.500	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-6.720	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-6.720	0.00	
Total =				12787.279	180.709		-49997.23	-555.538
				-180.709		555.538		

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	178.628	1435.445	4639.501	1267.249
due to Substructure	1	193.793	129.195		889.492	592.995
due to Earth pressure	1	2735.045			8792.163	
due to Live load surcharge	0.20	182.641			732.353	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	178.63	1267.25
193.79	889.49	0.00	0.00	0.00	0.00	129.20	592.99
2735.04	8792.16						
182.64	732.35						
3845.23	15053.51	0.00	0.00	0.00	0.00	307.82	1860.24

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Substructure	1.5	283.630		1300.797
due to Active Earth pressure	1.5	2706.564		8171.010
due to dynamic Earth pressure	1.5	1396.003		5017.23475
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		5177.260	19490.929	

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
283.63	1300.80	0.00	0.00
2706.564	8171.010		
1396.003	5017.235		
0.000	0.000		
0.000	0.000		
5177.26	19490.93	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	13911.214	13500.783	kN
ML	7516.099	7382.254	kNm
MT	2309.888	2309.888	kNm

Summary of Restoring Forces

Vertical Load	12606.571	kN
Moment	-50552.772	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = ah x P (kN)	FT = 0.3 x ah x P (kN)	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				1467.500		158.490	105.660		1262.050	90.868				0.000	1098.330
Substructure & Foundation -Portion 1				369.461	21.904	14.603	9.735		134.039	5.791		142.376		0.000	94.917
Substructure & Foundation -Portion 2															
Solid Return wall	1		23.82	595.51	96.473	64.316	42.877	-1.51	-899.228	-64.744	1433.16	389.837	0.00	0.000	259.891
Shaft above HFL	1	25	14.121	353.025	57.190	38.127	25.418	0.700	247.117	17.792	1434.360	299.533	0.000	0.000	199.689

Design Calculation

RODIC

Stability of Foundation

Shaft below HFL	1	15	26.577	398.655	-24.830	-16.553	-11.035	0.700	279.058	-7.725	1433.075	-98.152	0.000	0.000	-65.435
Back filling above HFL over heel slab	1	20	126.494	2529.875	409.840	273.226	182.151	-1.510	-3820.111	-275.048	1434.943	2385.47	0.000	0.000	1590.31
Back filling below HFL over heel slab	1	10	171.264	1712.637	-90.135	-60.090	-40.060	-1.552	-2657.385	62.158	1431.599	-223.22	0.000	0.000	-148.81
Front Filling over Pile Cap	1	10	77.711	777.112				2.175	1689.867				0.000	0.000	
Side filling between heel and toe	1	10	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	15	34.688	520.313				-1.304	-678.719				0.000	0.000	
Toe slab	1	15	21.563	323.438				2.082	673.469				0.000	0.000	
portion between heel & toe	1	15	9.000	135.000				0.700	94.500				0.000	0.000	
Vertical component of active earth pressure	1			530.716				-3.360	-1783.204				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			273.735				-3.360	-919.748				0.000	0.000	
				8150.020	448.539	299.026	199.351		-7774.384	-267.566		2753.471		0.000	1835.648
Total =				9986.981	470.443	472.119	314.746		-6378.295	-170.908		2895.847		0.000	3028.895
-314.746170.908															

Summary of Forces For SBC

	Downward	Upward	
P	10301.727	9672.235	kN
ML	6883.621	7225.437	kNm
MT	3028.895	3028.895	kNm

Summary of Restoring Forces

Vertical Load	9331.977	kN
Moment	-38335.815	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = $\alpha h \times P$ (kN)	FT = $0.3 \times \alpha h \times P$ (kN)	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				1467.500		158.490	105.660		1262.050	90.868				0.000	1098.330
Forces from Substructure				8519.481	470.443	313.629	209.086		-7640.345	-261.776		2895.847		0.000	1930.565
CWLL-Max. Reaction case	0.20			93.25		10.072	6.714	0.860	80.199	5.774	1437.510		3.109	289.919	84.480
Vertical component of LL Surcharge	0.20			43.86				-3.360	-147.355				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			22.62				-3.360	-76.003				0.000	0.000	
Total =				10146.712	470.443	482.190	321.460		-6521.455	-165.134		2895.847		289.919	3113.375

-321.460

165.134

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = $0.3 \times \alpha v \times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				1401.000	100.872		-3502.500	-252.180
Forces from Substructure				8093.507	61.658		-34326.132	-255.003
CWLL-Max. Reaction case	0.00			0.00	0.00		-2.50	0.00
Vertical component of LL Surcharge	0.00			0.00		-2.500	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-6.720	0.000	0.000
Total =				9494.507	162.530		-37831.13	-507.183

-162.530

507.183

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	168.562	1435.445	4639.501	1182.810
due to Substructure	1	470.443	313.629		2895.847	1930.565
due to Earth pressure	1	2210.209			5897.475	
due to Live load surcharge	0.20	182.641			727.886	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Substructure	1.5	219.048		1072.48742
due to Active Earth pressure	1.5	2187.194		5806.890
due to dynamic Earth pressure	1.5	1128.119713		3039.32294
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0

4325.424

14920.588

Forces along Long. Axis				Forces along Trans. Axis			
FL Cos θ	ML Cos θ	FT Sin θ	MT Sin θ	FL Sin θ	ML Sin θ	FT Cos θ	MT Cos θ
733.75	4639.50	0.00	0.00	0.00	0.00	168.56	1182.81
470.44	2895.85	0.00	0.00	0.00	0.00	313.63	1930.56
2210.21	5897.48						
182.64	727.89						
3597.04	14160.71	0.00	0.00	0.00	0.00	482.19	3113.37

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
791.06	5001.89	0.00	0.00
219.05	1072.49	0.00	0.00
2187.194	5806.890		
1128.120	3039.323		
0.000	0.000		
0.000	0.000		
4325.42	14920.59	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	10468.172	9825.252	kN
ML	7474.122	7804.389	kNm
MT	3403.293	3403.293	kNm

Summary of Restoring Forces

Vertical Load	9331.977	kN
Moment	-38338.315	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = Px eL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = Px eT	MTs due to FT
Superstructure				1467.500		528.300	105.660		1262.050	90.868				0.000	3661.099
Substructure & Foundation -Portion 1				369.461	6.571	48.676	9.735		134.039	5.791		42.713		0.000	316.391
Substructure & Foundation -Portion 2				11616.096	51.567	381.974	76.395		-8937.423	-41.282		224.135		0.000	1660.257
Total =				13453.057	58.138	958.950	191.790		-7541.334	55.377		266.848		0.000	5637.748
							-191.790			-55.377					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = Px eL2	MLs due to Fv
Superstructure				1401.000	100.872		-3502.50	-252.180
Substructure & Foundation -Portion 1				350.988	9.248		-925.60	-25.573
Substructure & Foundation -Portion 2				11035.291	70.588		-45569.13	-277.785
Total =				12787.279	180.709		-49997.23	-555.538
					-180.709			555.538

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x αh x P (kN)	C.g. of Force (m)	MLs due to FL
	21.904		142.376
	167.183		724.822
	189.087		867.198

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	528.300	1435.445	4639.501	3661.099
due to Substructure	1	58.138	430.650		266.848	1976.649
due to Earth pressure	1	2735.045			8792.163	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	528.30	3661.10
58.14	266.85	0.00	0.00	0.00	0.00	430.65	1976.65
2735.04	8792.16						
3526.93	13698.51	0.00	0.00	0.00	0.00	958.95	5637.75

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1435.445	5001.888
due to Substructure	1.5	283.630		1300.79658
due to Active Earth pressure	1.5	2706.564		8171.00988
due to dynamic Earth pressure	1.5	418.801		1505.17043
		4200.058		15978.865

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
283.63	1300.80	0.00	0.00
2706.564	8171.010		
418.801	1505.170		
4200.06	15978.87	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	13644.847	13261.267	kN
ML	6212.554	6101.801	kNm
MT	5637.748	5637.748	kNm

Summary of Restoring Forces

Vertical Load	12606.571	kN
Moment	-50552.772	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Vertical Forces For SBC Calculation****Vertical Forces For Restoring or Resisting Effect**

Design Calculation

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \Delta v$ $\times P$ (kN)	$M_L = P \times e_L1$	MLs due to F_v	$M_T = P \times e_T$
Total =	13705.998	205.216	-7604.33	66.922	449.644
		-205.216		-66.922	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	595.428	1435.445	4639.501	4224.165
due to Substructure	1	58.138	430.650		266.848	1976.649
due to Earth pressure	1	2735.045			8792.163	
due to Live load surcharge	0.2	182.641			732.353	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.06	1435.445	5001.888
due to Substructure	1.5	85.09		390.239
due to Active Earth pressure	1.5	2706.56		8171.010
due to dynamic Earth pressure	1.5	418.80		1505.170
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		4001.52		15068.307

Summary of Forces For SBC

	Downward	Upward	
P	13911.214	13500.783	kN
ML	6893.455	6759.610	kNm
MT	6650.457	6650.457	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads		Vertical Load(P) kN.	$F_v = 0.3 \times \Delta v$ $\times P$ (kN)	$M_L = P \times e_L1$	MLs due to F_v	$M_T = P \times e_T$
Superstructure		1467.500	105.660	1262.050	90.868	0.000
Substructure & Foundation -Portion 1		369.461	9.735	134.039	5.791	0.000
Substructure & Foundation -Portion 2		8150.020	199.351	-7774.384	-267.566	0.000
Total =		9986.981	314.746	-6378.295	-170.908	0.000
			-314.746		170.908	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.75	528.30	1435.45	4639.50	3661.10
due to Substructure	1	141.13	1045.43		868.75	6435.22
due to Earth pressure	1	2210.21			5897.48	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)

RODIC

Stability of Foundation

Loads	Vertical Load(P) kN.	$F_v = 0.3 \times \Delta v$ $\times P$ (kN)	$M_L = P \times e_L2$	MLs due to F_v
Total =	12787.279	180.709	-49997.23	-555.538
		-180.709		555.538

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	595.43	4224.16
58.14	266.85	0.00	0.00	0.00	0.00	430.65	1976.65
2735.04	8792.16						
182.64	732.35						
3709.57	14430.87	0.00	0.00	0.00	0.00	1026.08	6200.81

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
85.09	390.24	0.00	0.00
2706.564	8171.010		
418.801	1505.170		
0.000	0.000		
0.000	0.000		
4001.52	15068.31	0.00	0.00

Summary of Restoring Forces

Vertical Load	12606.571	kN
Moment	-50552.772	kNm

Vertical Forces For Restoring or Resisting Effect

Loads		Vertical Load(P) kN.	$F_v = 0.3 \times \Delta v$ $\times P$ (kN)	Long. Ecc. (eL2) @ Toe (m)	$M_L = P \times e_L2$	MLs due to F_v
Superstructure		1401.000	100.872		-3502.500	-252.180
Substructure & Foundation		350.988	9.248		-925.603	-25.573
Substructure & Foundation		7742.519	52.410		-33400.529	-229.430
Total =		9494.507	162.530		-37828.632	-507.183
			-162.530			507.183

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	528.30	3661.10
141.13	868.75	0.00	0.00	0.00	0.00	1045.43	6435.22
2210.21	5897.48						
3085.09	11405.73	0.00	0.00	0.00	0.00	1573.73	10096.32

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ

due to Superstructure		791.063	1435.445	5001.89
due to Substructure	1.5	65.714		321.75
due to Active Earth pressure	1.5	2187.194		5806.89
due to dynamic Earth pressure	1.5	338.436		911.80
		3382.406		12042.32

791.06	5001.89	0.00	0.00
65.71	321.75	0.00	0.00
2187.194	5806.890		
338.436	911.797		
3382.41	12042.32	0.00	0.00

Summary of Forces For SBC			
	Downward	Upward	
P	10301.727	9672.235	kN
ML	4856.528	5198.344	kNm
MT	10096.316	10096.316	kNm

Summary of Restoring Forces

Vertical Load	9331.977	kN
Moment	-38335.815	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	10146.712	321.460	-6521.45	-165.134	289.919
		-321.460		165.134	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Total =	9494.507	162.530		-37831.13	-507.183
		-162.530			507.183

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	561.871676	1435.445	4639.501	3942.69841
due to Substructure	1	141.133	1045.428637		868.754	6435.21661
due to Earth pressure	1	2210.209			5897.475	
due to Live load surcharge	0.2	182.641			727.886	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4639.50	0.00	0.00	0.00	0.00	561.87	3942.70
141.13	868.75	0.00	0.00	0.00	0.00	1045.43	6435.22
2210.21	5897.48						
182.64	727.89						
3267.73	12133.62	0.00	0.00	0.00	0.00	1607.30	10377.92

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.06	1435.445	5001.89
due to Substructure	1.5	65.71		321.75
due to Active Earth pressure	1.5	2187.19		5806.89
due to dynamic Earth pressure	1.5	338.44		911.80
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		3382.41		12042.32

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5001.89	0.00	0.00
65.71	321.75	0.00	0.00
2187.194	5806.890		
338.436	911.797		
0.000	0.000		
0.000	0.000		
3382.41	12042.32	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	10468.172	9825.252	kN
ML	5447.029	5777.296	kNm
MT	10667.834	10667.834	kNm

Summary of Restoring Forces

Vertical Load	9331.977	kN
Moment	-38338.315	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 kN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1437.510 -	1429.122)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00	=	0.000 kN	Seismic
Transverse Moment due to C.F.	=	0.000	x (	1437.510 -	1429.122)	=	0.000 kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

Base pressure on corner A	=	σ_A	=	$P/A - ML/ZL + MT/ZT$
Base pressure on corner B	=	σ_B	=	$P/A + ML/ZL + MT/ZT$
Base pressure on corner C	=	σ_C	=	$P/A - ML/ZL - MT/ZT$
Base pressure on corner D	=	σ_D	=	$P/A + ML/ZL - MT/ZT$

LOAD CASES	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= $\mu P + c.A + F_p$	FOS	Overturning moment	Restoring Moment = $\Sigma P.e_{Toe+M_p}$	FOS
	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN/m ²	kN	kN		kNm	kNm	
Normal Dry Case															
Case 1 : DL+SIDL-Normal Dry Case	13114.321	3683.659	0.000	116.968	195.277	116.968	195.277	195.277	116.968	3497.627	8725.836	2.49	13172.898	47961.12	3.64
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	14265.927	5913.935	2248.220	119.819	245.540	94.125	219.846	245.540	94.125	4220.582	8725.836	2.07	15771.200	49361.00	3.13
Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case	15894.936	7314.883	768.401	115.865	271.368	107.083	262.586	271.368	107.083	4220.582	8725.836	2.07	15771.200	49361.00	3.13
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	9746.692	2939.837	0.000	84.784	147.280	84.784	147.280	147.280	84.784	3116.093	6486.363	2.08	10808.778	36421.01	3.37
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	10432.244	4769.308	1449.594	81.783	183.171	65.216	166.604	183.171	65.216	3839.048	6632.183	1.73	13407.080	37820.88	2.82
Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case	10824.635	5106.764	257.599	76.056	184.618	73.112	181.674	184.618	73.112	3839.048	6632.183	1.73	13407.080	37820.88	2.82
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	13644.847	6835.198	1691.324	99.450	244.756	80.121	225.427	244.756	80.121	5177.260	8824.599	1.70	19490.929	50552.77	2.59
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	13911.214	7516.099	2309.888	98.919	258.700	72.520	232.301	258.700	72.520	5177.260	8824.599	1.70	19490.929	50552.77	2.59
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	10301.727	7225.437	3028.895	63.147	216.749	28.531	182.133	216.749	28.531	4325.424	6532.384	1.51	14920.588	38335.82	2.57
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	10468.172	7804.389	3403.293	61.114	227.023	22.219	188.129	227.023	22.219	4325.424	6532.384	1.51	14920.588	38338.32	2.57
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															

Case 9 : DL+SIDL-Trans. Seismic Dry Case	13644.847	6212.554	5637.748	128.620	260.689	64.188	196.258	260.689	64.188	4200.058	8824.599	2.10	15978.865	50552.77	3.16
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	13911.214	6893.455	6650.457	130.340	276.885	54.335	200.879	276.885	54.335	4001.517	8824.599	2.21	15068.307	50552.77	3.35
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	10301.727	5198.344	10096.316	125.078	235.587	9.692	120.201	235.587	9.692	3382.406	6532.384	1.93	12042.321	38335.82	3.18
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	10468.172	5777.296	10667.834	124.172	246.988	2.254	125.070	246.988	2.254	3382.406	6532.384	1.93	12042.321	38338.32	3.18
								SAFE	SAFE			SAFE			SAFE

DESIGN OF RCC SOLID SLAB

SPAN - 10m

OVERALL DECK WIDTH – 12.5 m

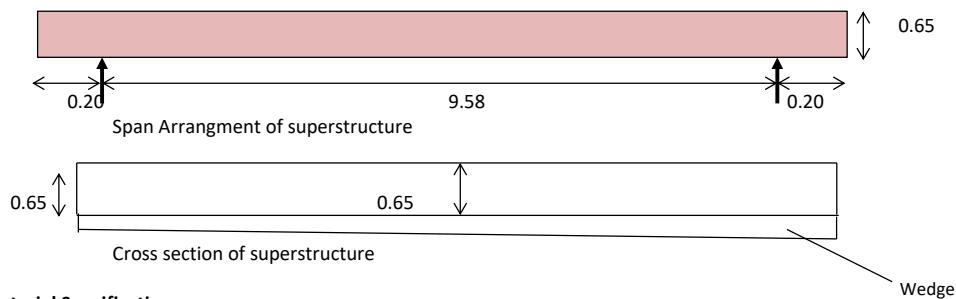
1.0 Design Standard:

The following points has been considered in designing the slab.

- 1 Structure has been analysed by effective width method
- 2 Structure is designed for per metre width.
- 3 Slab need not to be checked for shear in effective width method as per clause no. Annexure B-3, Clause B-3.2 of IRC:112-2011
- 4 On top slab 65mm thick wearing coat is assumed for SIDL.
- 5 Deck width taken-12.5 m and Carriageway width- 11 m
- 6 0.3mm permissible Crack width has been considered for Moderate/Severe condition of exposure.
- 7 The following 3 lane carriageway live load has been conisered
3-Lane Class A
1-Class A + 1-70RWheel
1-70R Track
Class SV
- 8 Slab has been designed by ultimate limit state method and stresses and crack width has been calculated by rare combination and quasi-permanent combination respectively.
- 9 The temparature loading for uniform rise or fall has been considered.
- 10 The following IRC Codes has been used for the design of slab
IRC: 6-2014
IRC:112-2011

2.0 Input Data :

Clear span of the slab	=	9.18	m		
Bearing over support	=	0.40	m		
Span between Centre to centre of bearing	=	9.58	m		
Thickness of filler type exp. Gap	=	0.02	m		
Overall span of slab	=	10.00	m		
Overall width of slab	=	12.50	m		
Carriageway width	=	11.00	m		
Width of footpath	=	0.00	m		
Camber	=	2.50	% (uni directional)		
Depth of Slab at Carriageway edge	=	0.650	m		
Depth of Slab at Carriageway centre	=	0.650	m		
Wearing Coat thickness	=	0.065	m		
Clear cover to steel	=	0.050	m		
Diameter of main reinforcement bar	=	0.020	m		
Avg. Depth of Solid Slab	=	0.65	m		
Effective Depth of Slab	=	0.65	m	-	0.05
		0.590	m		0.010
Effective Span	Min(l+ws , l+d)				
	L	=	9.58	m	

**3.0 Material Specification**

Concrete Grade	=	M 30		
Characteristic Compressive Strength of Concrete, f_{ck}	=	30.00	Mpa at 28 days	
Design Compressive strength of Concrete, f_{cd}	=	13.40	Mpa at 28 days	$(0.67/1.5 * f_{ck})$
Tensile strength of concrete , f_{ctm}	=	2.50	MPa	
Strain at reaching Characteristic Strength, ϵ_{c2}	=	0.02		
Ultimate Strain, ϵ_{cu2}	=	0.035		
Modulus of Elasticity of Concrete (E_c)	=	2.74E+04	N/mm ²	$(5000 * \sqrt{f_{ck}})$
E_{cm}	=	3.12E+04	N/mm ²	
Steel Grade	=	Fe 500		(HYSD Steel)
Yield Strength of Reinforcement, f_y or f_{yk}	=	500	Mpa	
Design Yield Strength of Reinforcement, f_{yd}	=	434.78	Mpa	$(1/1.15 * f_y)$
Modulus of Elasticity of Steel (E_s)	=	2.00E+05	Mpa	
Dry weight of Concrete	=	25	kN/m ³	
Dry unit weight of soil	=	20	kN/m ³	
Permissible Crack Width	=	0.3	mm - For Severe/Moderate Exposure Condition	
Maximum compressive stress in concrete under rare combination	=	0.48 f_{ck}	=	14.4 N/mm ²
Maximum tensile stress in steel under rare combination	=	300	N/mm ²	

4.0 Calculation of Load ,Bending Moment**4.1 Self-weight of Deck Slab**

Average thickness of deck slab	=	0.65 m
U.D.L. Due to deck slab weight	=	16.25 kN/m ²
Bending Moment Deck Slab weight	=	186.42 kNm/m

4.2 SIDL1 Due to Crash Barrier & Railing

Due to crash barrier	=	2.00	x 8.00 =	16.00 kN/m
U.D.L due to crash barrier	=	16.00	/ 12.50 =	1.28 kN/m ²
Due to RCC Railing	=	2.00	x 0.00 =	0.00 kN/m
	=	0.00	/ 12.50 =	0.00 kN/m ²

Total SIDL1 due to crash barrier & railing	=	1.28 kN/m ²
Bending Moment due to SIDL1	=	14.68 kNm/m

4.3 SIDL2 Due to Wearing Course

due to wearing course and additional ove	=	2.00 kN/m ²
Bending Moment due to SIDL2	=	22.94 kNm/m

4.4 Footpath Load

Refer clause 206.3 of IRC:6-2014

P1	=	0 kg/m ²
Bending Moment due to FPLL	=	0.00 kNm/m

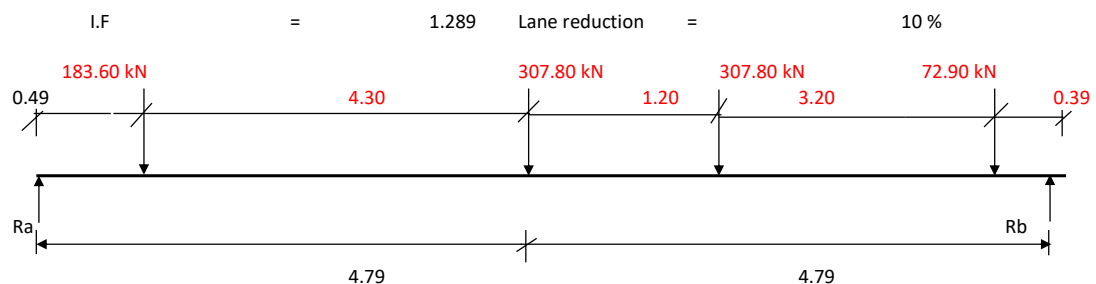
4.5 CWLL Bending Moment

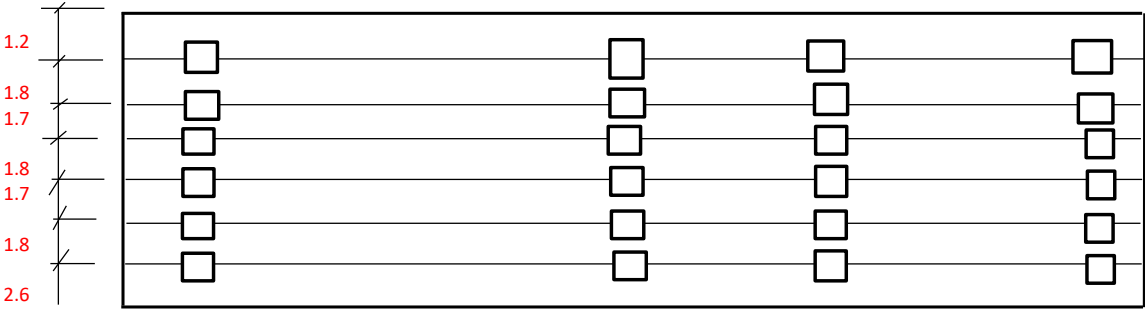
b/L	=	1.30	interpolation	1.30	2.72
α	=	2.72 For simply supported slab		1.40	2.80
b_{eff}	=	$\alpha * a * (1 - a / L) + b_1$			
A	=	$\alpha * a * (1 - a / L)$			
b_1	=	Breadth of the concentration area of the load			

IRC- Class A Loading

Axle Load (kN)	Breadth of tyre across span (m) W	Breadth of tyre along span (m) B
114.00	0.50	0.25
68.00	0.38	0.20
27.00	0.20	0.15

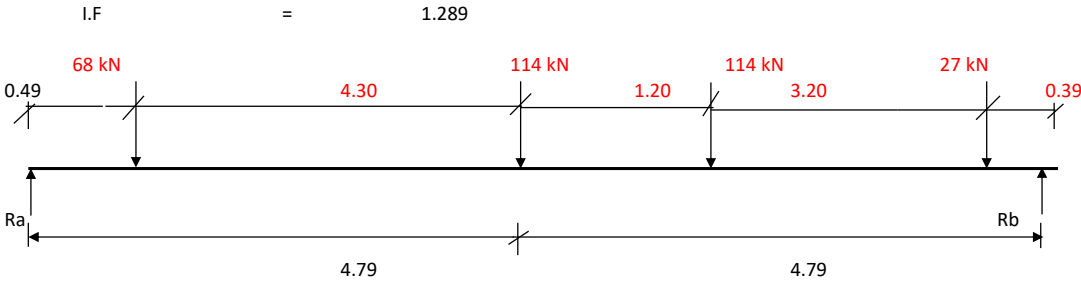
b_1	=	0.63	114 kN
b_1	=	0.51	68 kN
b_1	=	0.33	27 kN

For Class A - 3 Lanes



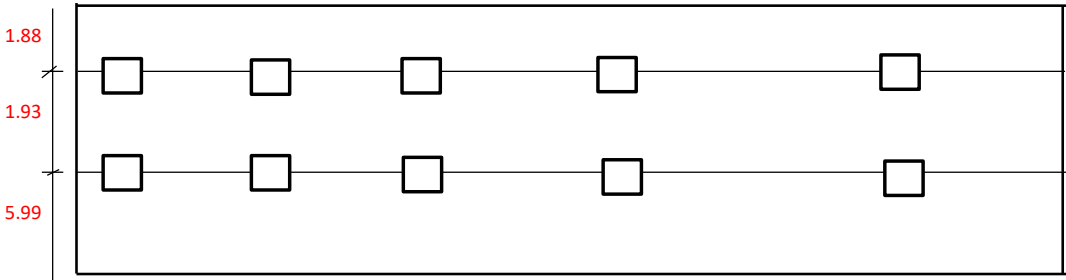
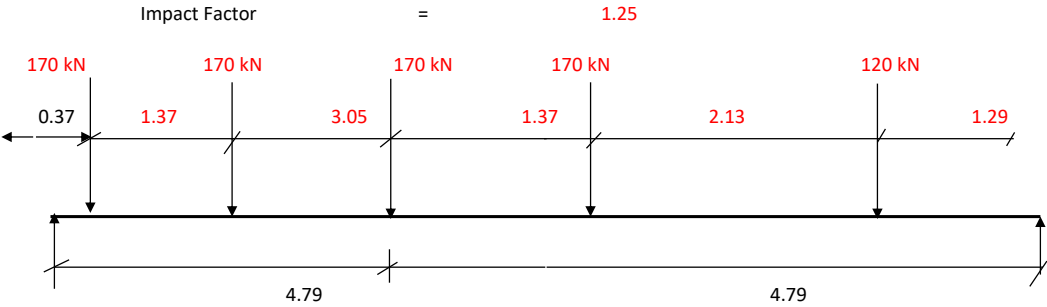
Axle Load (kN)	a (m)	$\alpha \cdot a (1 - a / L), (m)$	b1 (m)	beff (m)	beff for all axles (m)	Bending Moment (kNm/m)
183.60	0.49	1.27	0.51	1.78	10.58	5.48
307.80	4.79	6.52	0.63	7.15	12.50	76.01
307.80	3.59	6.11	0.63	6.74	12.50	56.97
72.90	0.39	1.02	0.33	1.35	8.09	2.26
						140.72

Class A - 1 Lane



CLASS 70 RW Loading

placed along with Class A Loading



Dimensions of 70RWheel

Tyre pressure =

5.273 kg/cm²

527.30 kN/m²

Axle Load (kN)	Breadth of tyre across span (m) W	Breadth of tyre along span (m) B
170.00	0.81	0.40
120.00	0.81	0.28

Axle Load (kN)	a (m)	$\alpha * a (1 - a / L), (m)$	b1 (m)	beff (m)	beff for all axles (m)	Bending Moment (kNm/m)
170	0.37	0.97	0.94	1.91	3.82	10.30
170	1.74	3.88	0.94	4.82	6.22	29.73
170	4.79	6.52	0.94	7.46	7.54	67.48
170	3.42	5.99	0.94	6.93	7.27	49.95
120	1.29	3.04	0.94	3.98	5.80	16.68
B . M =						174.13

1-Class A + 1- 70RW

I.F

=

1.289

Lane reduction =

10 %

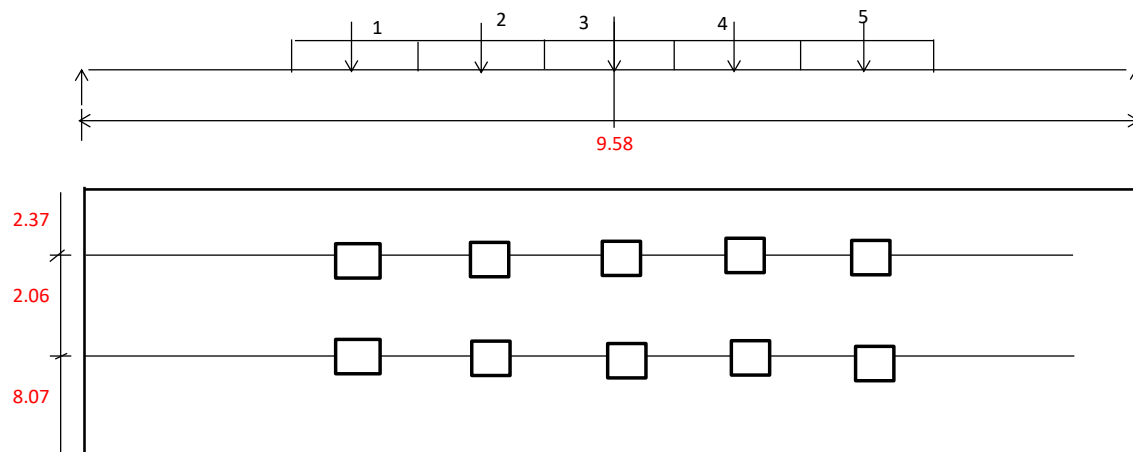
Axle Load (kN)	a (m)	beff for all axles (m)	Bending Moment (kNm/m)
170	0.37	3.82	10.62
238	1.74	8.92	26.93
284	4.79	10.24	77.03
284	3.42	9.97	56.47
147	1.29	8.50	12.94
B . M =			183.99

CLASS 70 R(T) Loading

Impact Factor

=

1.10



Class 70R (T)

=

700.00

KN

Length of track load

=

4.57

m

Axle Load (kN)	Breadth of tyre across span (m) W	Breadth of tyre along span (m) B
140.00	0.840	0.914

Divide the track load into 5 equal wheel load

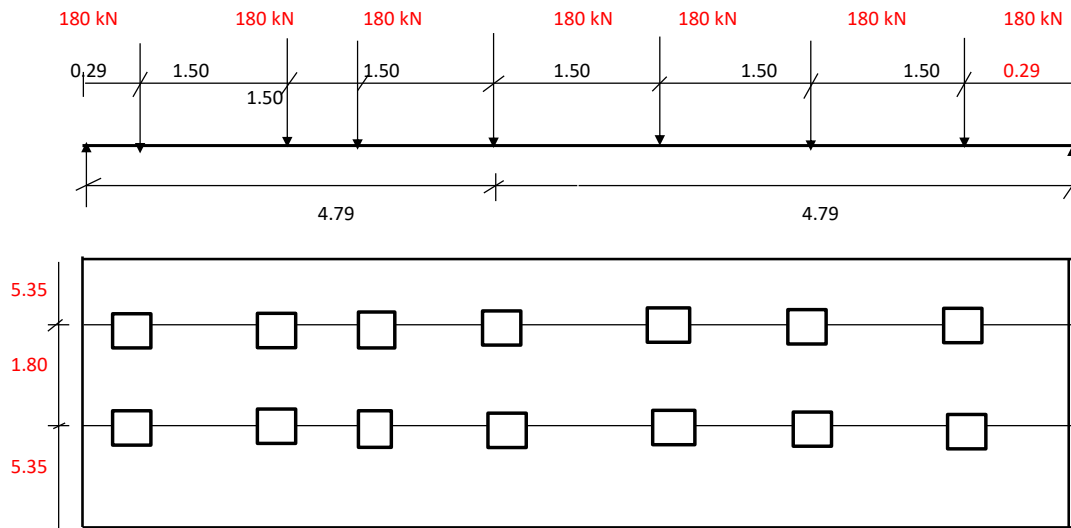
Axle Load (kN)	a (m)	$\alpha * a (1 - a / L), (m)$	b1 (m)	beff (m)	beff for all axles (m)	Bending Moment (kNm/m)
140	2.96	5.57	0.97	6.54	7.70	29.61
140	3.88	6.29	0.97	7.26	8.06	37.04
140	4.79	6.52	0.97	7.49	8.18	45.11
140	3.88	6.29	0.97	7.26	8.06	37.04
140	2.96	5.57	0.97	6.54	7.70	29.61
B . M =						178.41

CLASS SV Loading

Impact Factor

=

1.00



Dimensions of SV Loading

Tyre pressure =

2.250 kg/cm²225.00 kN/m²

Axle Load (kN)	Breadth of tyre across span (m) W	Breadth of tyre along span (m) B
180.00	0.450	0.274

Axle Load (kN)	a (m)	$\alpha * a (1 - a / L), (m)$	b1 (m)	beff (m)	beff for all axles (m)	Bending Moment (kNm/m)
180	0.29	0.77	0.58	1.35	2.69	9.70
180	1.79	3.96	0.58	4.54	6.34	25.39
180	3.29	5.88	0.58	6.46	8.26	35.83
180	4.79	6.52	0.58	7.10	8.90	48.42
180	3.29	5.88	0.58	6.46	8.26	35.83
180	1.79	3.96	0.58	4.54	6.34	25.39
180	0.290	0.77	0.58	1.35	2.69	9.70
B . M =						190.25

Unfactored CWLL Bending Moment For

1.) CLASS A - 3 Lane	=	140.72	kN-m/m
2.) 1-CLASS A + 1- 70 R Wheel	=	183.99	kN-m/m
3.) CLASS 70 R Track	=	178.41	kN-m/m
4.) SV Loading	=	190.25	kN-m/m

Total Bending Moment at Mid Span

Loads	Unfactored B.M. (kNm/m)	ULS Factor	ULS Moment (kNm/m)	Rare Combination factor	Rare Combination Moment (kNm/m)	Quasi permanent combination Factor	Quasi permanent combination Moment (kNm/m)
DL	186.42	1.35	251.67	1.00	186.42	1.00	186.42
SIDL1	14.68	1.35	19.82	1.00	14.68	1.00	14.68
SIDL2	22.94	1.75	40.15	1.00	22.94	1.00	22.94
FPLL	0.00	1.50	0.00	1.00	0.00	0.00	0.00
Case 1: Leading Live load with accompanied thermal load							
CWLL - (Class A /70R)	183.99	1.50	275.99	1.00	183.99	0.00	0.00
CWLL - SV Load	190.25	1.00	190.25	1.00	190.25	0.00	0.00
Total Moment (Case 1)			=		587.63		224.05
Case 2: Leading thermal load with accompanied live load							
CWLL - (Class A /70R)	183.99			0.75	137.99		
CWLL - SV Load	190.25			0.75	142.69		
Total Moment (Case 2)			=		366.74		

5.0 ULTIMATE DESIGN OF SOLID SLAB**DECK SLAB FOR ULS FLEXURAL MOMENT**

Min. Thickness of slab	=	650 mm
Clear Cover to outer steel	=	50 mm
Maximum Diameter of Reinforcement	=	20 mm
Effective Depth Provided (deff)	=	590 mm
Ultimate Design bending moment	=	587.63 kNm/m
Mu	=	0.36 . fck . b . X (1 - 0.416 X)
Xulim/d	=	0.0035 / (0.0035 + 0.87 fy/Es)
Xulim	=	0.617 d

$$M_{ulim} = 0.165 \times f_{ck} \times b \times d^2 = 587.63 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \sqrt[3]{\frac{587.63 \times 1000000}{0.165 \times 30.00 \times 1000}}$$

Effective Depth of Cap Required (dreq)	=	344.548 mm
Total Depth Required (Dreq)	=	404.55 mm
Total Depth Provided (Dprov)	=	650.00 mm

OK

$$R = M_{RD} / (b \cdot d^2) = 1.69$$

Ast Required:

$$\frac{p_t}{100} = \frac{A_{st_{req}}}{b \cdot d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$$

$$A_{st_{req}} = 2460.833 \text{ mm}^2/\text{m}$$

Minimum Longitudinal Reinforcement :

As. Min	=	0.26 x	$\frac{f_{ctm}}{f_{yk}}$	x	b . d	- Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011
Whichever is higher	OR	=	0.0015	x	b . d	-Refer Clause 16.9 of IRC:112-2011'
	b	=	1000.00	mm		
	d	=	590.00	mm		
	Ast min	=	885.00	mm ² /m		
Governing Reinf. Ast	=		2460.83	mm ² /m		

Provide 20 mm dia @ 110 mm c/c + 0 mm dia @ 200 mm c/c

Area provided=	2855.99 mm ² /m	>	2460.83	mm ² /m	OK
Percentage of Steel (pt%)		=	0.48	%	

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

Smax	=	2 h	=	1180.00	
	OR	=	250.00	mm	whichever is minimum

Provided Spacing is less than Smax, Hence OK

$$\text{Limiting Depth of Neutral Axis , } X_m = \frac{0.0035 \cdot d}{(0.0035 + f_{yd} / E_s)} = \frac{0.0035 \times 590.00}{0.0035 + 0.0022}$$

$$= 363.95 \text{ mm}$$

$$\text{Depth of Neutral Axis , } X = \frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b} = \frac{434.78 \times 2855.99}{0.36 \times 30.00 \times 1000}$$

$$= 114.98 \text{ mm}$$

UNDER REINFORCED DESIGN, OK

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$z = d - 0.416 \cdot X = 590.00 - 47.83$$

$$= 542.17 \text{ mm}$$

Moment of Resistance of Section w.r.t. Steel (MR)
MR

$$\begin{aligned}
 &= f_{yd} \cdot A_{st} \cdot z \\
 &= 434.78 \quad \times \quad 2855.99 \quad \times \quad 542.17 \\
 &= 6.73E+08 \quad \text{Nmm / m} \\
 &= 673.23 \quad \text{kNm / m} > 587.63 \quad \text{kNm / m} \quad \text{SAFE}
 \end{aligned}$$

Moment of Resistance of Slab is More than Design Bending Moment , HENCE SLAB IS SAFE IN BENDING

Distribution reinforcement:

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$$\frac{20.00}{100.00} \times 2855.99 = 571.199 \quad \text{mm}^2/\text{m}$$

Provide	12 dia bar	150 direction in top face. (Providing =	753.98 mm ²)	OK

6.0 STRESS CHECK IN SLS CASE

1. Early Age Check (before creep has taken place)
2. Long Term Check (after creep has taken place)

Maximum compressive stress in concrete under rare combination	=	14.4 N/mm ²
Maximum tensile stress in steel under rare combination	=	300 N/mm ²
Maximum Tensile stress in concrete	f _{ctm} =	2.50 N/mm ²
Permissible crack width	=	0.30 mm

Section Properties of Uncracked Section:

Width of Section , b	=	1000.00 mm
Depth of Section , D	=	650.00 mm
Gross Cross sectional area , A _g	=	650000 mm ²
Gross Moment of Inertia I _{gx}	=	2.29E+10 mm ⁴
Gross Moment of Inertia I _{gy}	=	5.42E+10 mm ⁴

Modular Ratio : for Early Age Check

Modulus of elasticity of concrete, E _{eff} = E _{cm}	=	31187 N/mm ²
Modulus of elasticity of steel, E _s	=	200000 N/mm ²
Modular Ratio (m) = E _s / E _{eff}	=	6.41

Modular Ratio: for Long Term Check

Perimeter of section (u)	=	3300 mm
2 A _c /u	=	393.939 mm
Modulus of elasticity of concrete, E _{cm}	=	31187 N/mm ²
Modulus of elasticity of steel, E _s	=	200000 N/mm ²
For Moist atmospheric condition		
Creep coeff.	ϕ =	1.34
E _{eff} = $\frac{E_{cm}}{(1 + \phi)}$	=	1.33E+04

Modular Ratio (m) = E _s / E _{eff}	=	14.99

Load factor

Leading thermal load	=	1
Accompanied thermal load	=	0.6

6.1 STRESS CHECK : RARE CASE

Depth of neutral axis =

$$d_c = \frac{-A_s E_s + \sqrt{(A_s E_s)^2 + 2bA_s E_s E_{c,eff} d}}{bE_{c,eff}}$$

Cracked Second Moment of Area I_{NA} = $A_s (d - d_c)^2 + \frac{1}{3} \frac{E_{c,eff}}{E_s} b d_c^3$
(in steel units)

	unit	Case 1: Leading Live load with accompanied thermal load		Case 2: Leading thermal load with accompanied live load		Remarks
		For Early Age	For Long Term Check	For Early Age	For Long Term Check	
DL+SIDL+LL Moment	kNm/m	414.303	414.303	366.74	366.74	
Width of Section, b	mm	1000.00	1000.00	1000.00	1000.00	
Depth of section , D	mm	650.00	650.00	650.00	650.00	
Effective cover , C _{eff}	mm	60.00	60.00	60.00	60.00	
Effective depth , d	mm	590.00	590.00	590.00	590.00	
E _{eff}	N/mm ²	31186.57	13342.57	31186.57	13342.57	
E _s	N/mm ²	200000.00	200000.00	200000.00	200000.00	
Flexural Ast Provided , A _s	mm ² /m	2855.99	2855.99	2855.99	2855.99	

dc			129.83	185.99	129.83	185.99	
Cracked Second Moment of area , I_{NA}		mm ⁴	7.185E+08	6.092E+08	7.185E+08	6.092E+08	
section modulus, $Z_t = I_{NA} / dc$		mm ³	5.534E+06	3.276E+06	5.534E+06	3.276E+06	
section modulus, $Z_b = I_{NA} / (d-dc)$		mm ³	1.561E+06	1.508E+06	1.561E+06	1.508E+06	
Normal Bending Stresses	Top	N/mm2	11.67	8.44	10.33	7.47	
	Bottom	N/mm2	265.33	274.74	234.87	243.20	
Thermal stresses : Temp. Rise case	Top	N/mm2	-1.88	-1.88	-3.13	-3.13	
	Bottom	N/mm2	-0.64	-0.64	-1.07	-1.07	
Thermal stresses : Temp. Fall case	Top	N/mm2	1.70	1.70	2.83	2.83	
	Bottom	N/mm2	0.60	0.60	0.99	0.99	
Final stresses with Temp. Rise case	Top	N/mm2	13.55	10.32	13.47	10.60	< 14.4 SAFE
	Bottom	N/mm2	264.69	274.10	233.81	242.13	< 300 SAFE
Final stresses with Temp. Fall case	Top	N/mm2	9.97	6.74	7.50	4.63	< 14.4 SAFE
	Bottom	N/mm2	265.93	275.34	235.87	244.19	< 300 SAFE

6.2 STRESS CHECK : QUASI-PERMANENT CASE

	unit	
Span Moment	kNm/m	224.049
Width of Section, b	mm	1000.00
Depth of section , D	mm	650.00
Effective cover , C_{eff}	mm	60.00
Effective depth , d	mm	590.00
E _{eff}	N/mm ²	31186.57
E _s	N/mm ²	200000.00
Flexural Ast Provided , A _s	mm ² /m	2855.99
dc		129.83
Cracked Second Moment of area , I_{NA}	mm ⁴	7.185E+08
section modulus, $Z_t = I_{NA} / dc$	mm ³	5.534E+06
section modulus, $Z_b = I_{NA} / (d-dc)$	mm ³	1.561E+06
Maximum compressive stress in concrete= $M/Z_t \times E_{eff} / E_s$	N/mm2	6.31
Maximum Tensile stress in steel = M/Z_b	N/mm2	143.49

6.3 CRACK WIDTH CHECK

Refer Clause 12.3.4 of IRC:112-2011

$$\text{Crack Width} = W_k = S_{r \max} (\epsilon_{sm} - \epsilon_{cm})$$

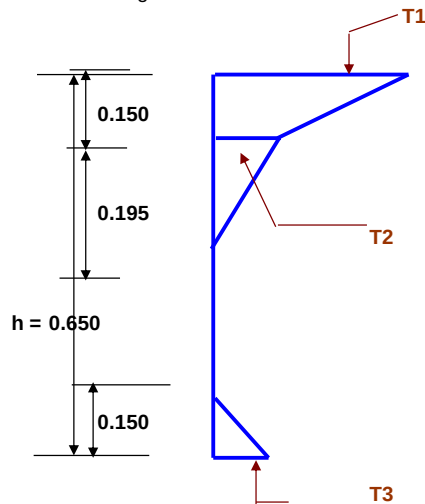
Where, $S_{r \max}$ = maximum crack spacing
 ϵ_{sm} = mean strain in the reinforcement
 ϵ_{cm} = mean strain in concrete between cracks

Spacing between reinf. = $5*(c+\phi/2)$	mm	300
Spacing provided	mm	55
Check for spacing criteria		OK
$S_{r \max} = 3.4 c + \frac{0.425 k_1 k_2 \phi}{\rho_{p \text{ eff}}}$		
Clear Cover , c	mm	50
Diameter of Main Bar , ϕ	mm	20
Coefficient , k_1		0.8
Coefficient , k_2		0.5
Width of section , b	mm	1000
Depth of section , D	mm	650
Effective Depth of Section , d	mm	590
Depth of Neutral axis , $y_t=dc$	mm	129.832
$hc \text{ eff} = \text{Min of } 2.5 (D - d) , D - dc/3 , D/2$	mm	150.00
$A_{c, \text{ eff}} = b * hc, \text{ eff}$	mm ²	150000
$\rho_{p \text{ eff}} = A_s/A_{c \text{ eff}}$		0.019
$S_{r \max}$	mm	348.572
$(\epsilon_{sm} - \epsilon_{cm}) = \frac{\sigma_{sc} - k_1 f_{ct \text{ eff}} (1 + \alpha_e \rho_{p \text{ eff}})}{E_s}$		
Stress in tension steel , σ_{sc} (in Quasi-Permanent Case)	N/mm2	143.49
Kt		0.500
Tensile strength of concrete = $f_{ct \text{ eff}} = f_{ctm}$	N/mm2	2.501
$\alpha_e = E_s / E_{cm}$		6.413
$(\epsilon_{sm} - \epsilon_{cm})$		0.000349
$0.6 \sigma_{sc} / E_s$		0.000430
Crack Width , W_k	mm	0.15
Limited Crack width	mm	0.30
Check for Crack width		< 0.3 mm SAFE

7.0 CALCULATIONS FOR TEMPERATURE GRADIENT

7.1 TEMPERATURE RISE CASE

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



$h =$	Depth of Superstructure =	0.65	m

Parameters:

Grade of Concrete	M =	30	
Elasticity for Concrete	$E_i =$	3.119E+04	MPa
Modified Elasticity for Concrete	$E_c = E_i$	3.119E+04	MPa
Coefficient of Thermal Expansion	$a =$	1.200E-05	/ °C

Generalised Temperature & corresponding Force Diagram

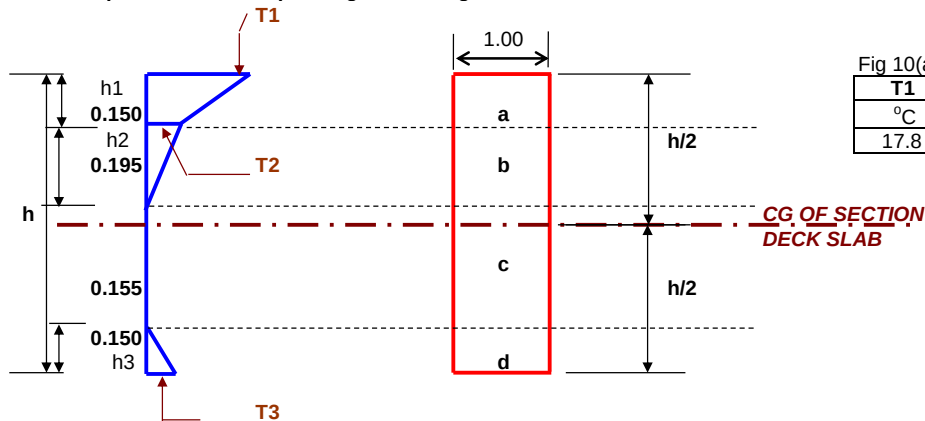


Fig 10(a) of IRC:6-2014

T1	T2	T3
°C	°C	°C
17.8	4	2.1

ZONE	A (m ²)	y (m)	Ay (m ³)	Ay ² (m ⁴)	t (°C)	At	Aty	θy	αt	f _{ei} Mpa	Nature of Stress
a	0.15	0.075	0.01125	8.4E-04	10.9	1.635	1.2E-01	1.68E-05	1.3E-04	-1.08	Compressive
b	0.195	0.2475	0.048263	1.2E-02	2.000	0.39	9.7E-02	5.5E-05	2.4E-05	1.05	Tensile
c	0.155	0.423	0.065488	2.8E-02	0.00	0	0.0E+00	9.5E-05	0.0E+00	0.58	Tensile
d	0.15	0.575	0.08625	5.0E-02	1.05	0.1575	9.1E-02	1.29E-04	1.26E-05	-0.88	Compressive
	0.65		0.21125	0.0901		2.1825	0.3097				

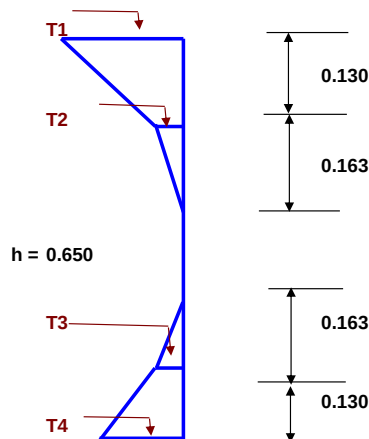
θ	2.24E-04	ε _o	1.13E-04

Temperature Stress Summary

Location	y m	t °C	θy	αt	f _{ei} Mpa	Nature of Stress
Stress at top fiber	0	17.80	0.00E+00	2.14E-04	-3.13	Compressive
Stress at CG of steel	0.590	1.26	1.32E-04	1.51E-05	-1.07	Compressive

7.2 TEMPERATURE FALL CASE

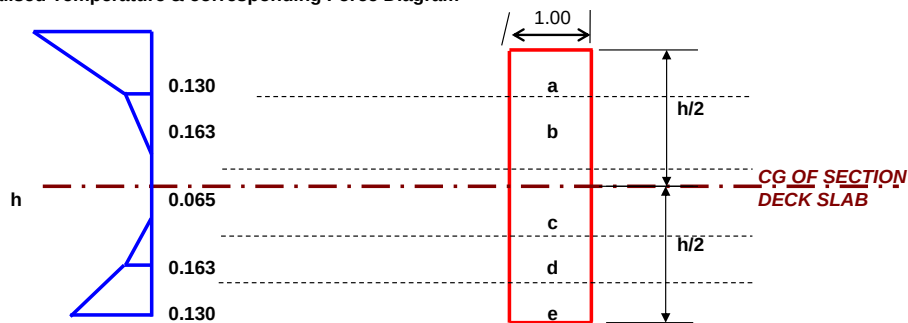
The solid slab is designed for the effects of the distribution of the temperature across the deck depth as given in the sketch below.



$h =$	Depth of Superstructure =	0.65	m

Parameters:			
Grade of Concrete	M =	30	
Elasticity for Concrete	$E_c =$	3.119E+04	MPa
Modified Elasticity for Concrete	$E_{cm} = E_c$	3.119E+04	MPa
Coefficient of Thermal Expansion	$\alpha =$	1.200E-05	/°C

Generalised Temperature & corresponding Force Diagram



T1	T2	T3	T4
°C	°C	°C	°C
-10.60	-0.70	-0.80	-6.60

Fig 10(a) of IRC:6-2014

ZONE	A	y	Ay	Ay ²	t	At	Aty	θy	αt	f_{ei}	Nature of Stress
	(m ²)	(m)	(m ³)	(m ⁴)	(°C)					Mpa	
a	0.13	0.065	0.00845	0.0005493	-5.65	-0.7345	-0.0477	-2.3E-06	-7E-05	1.05	Tensile
b	0.1625	0.2113	0.0343281	0.0072518	-0.350	-0.0569	-0.012	-7.6E-06	-4E-06	-0.77	Compressive
c	0.065	0.325	0.021125	0.0068656	0.00	0	0	-1.2E-05	0	-0.77	Compressive
d	0.1625	0.4388	0.0712969	0.0312815	-0.4	-0.065	-0.0285	-1.6E-05	-5E-06	-0.49	Compressive
e	0.13	0.585	0.07605	0.0444893	-3.7	-0.481	-0.2814	-2.1E-05	-4E-05	0.90	Tensile
	0.65		0.21125	0.090		-1.337	-0.370				

θ	-3.58E-05
ϵ_o	-3.63E-05

Temperature Stress Summary

Location	y	t	θy	αt	f_{ei}	Nature of Stress
	m	°C			Mpa	
Stress at top fiber	0	-10.60	0.00E+00	-1.27E-04	2.83	Tensile
Stress at CG of steel	0.590	-3.92	-2.11E-05	-4.71E-05	0.99	Tensile

8.0 DEFLECTION CHECK

Refer Clause 12.4.1 of IRC:112-2011

Allowable Deflection under Live load	=	Span/800	=	11.98 mm

Cracked Moment of inertia = $0.70 \cdot I_{\text{uncracked}}$	=	1.60E+10 mm ⁴	
E _{eff}	=	1.33E+04 N/mm ²	with long term effect
Span, l	=	9580 mm	

Deflection under Live load**3-Class A**

Axle Load , P (kN)	Effective width, B _{eff} ,(m)	Axle Load per effective width , P (kN)	Dist. (a) from Left support,m	Dist. (b) from Right support,m	Dist. X = span/2	Centre Deflection , y, (mm)
183.60	10.58	17.36	0.49	9.09	4.79	0.227
307.80	12.50	24.62	4.79	4.79	4.79	2.110
307.80	12.50	24.62	5.99	3.59	4.79	1.928
72.90	8.09	9.01	9.19	0.39	4.79	0.094
Σ						4.360

$$y = \frac{Pbx}{6EI} (l^2 - x^2 - b^2) \text{ for } 0 < x < a$$

$$y = \frac{Pb}{6EI} \left[\frac{l}{b} (x-a)^3 + (l^2 - b^2) x - x^3 \right] \text{ for } a < x < l$$

1-Class A + 1- 70RW

Axle Load , P (kN)	Effective width, B _{eff} ,(m)	Axle Load per effective width , P (kN)	Dist. (a) from Left support,m	Dist. (b) from Right support,m	Dist. X = span/2	Centre Deflection , y, (mm)
170	3.82	44.53	0.37	9.21	4.79	0.44
238	8.92	26.68	1.74	7.84	4.79	1.19
284	10.24	27.73	4.79	4.79	4.79	2.38
284	9.97	28.47	6.16	3.42	4.79	2.17
147	8.50	17.29	8.29	1.29	4.79	0.58
Σ						6.762

CLASS 70 R(T) Loading

Axle Load , P (kN)	Effective width, B _{eff} ,(m)	Axle Load per effective width , P (kN)	Dist. (a) from Left support,m	Dist. (b) from Right support,m	Dist. X = span/2	Centre Deflection , y, (mm)
140	7.70	18.18	2.96	6.62	4.79	1.26
140	8.06	17.37	3.88	5.70	4.79	1.41
140	8.18	17.12	4.79	4.79	4.79	1.47
140	8.06	17.37	5.70	3.88	4.79	1.41
140	7.70	18.18	6.62	2.96	4.79	1.26
Σ						6.814

CLASS SV Loading

Axle Load , P (kN)	Effective width, B _{eff} ,(m)	Axle Load per effective width , P (kN)	Dist. (a) from Left support,m	Dist. (b) from Right support,m	Dist. X = span/2	Centre Deflection , y, (mm)
180	2.69	66.86	0.29	9.29	4.79	0.52
180	6.34	28.37	1.79	7.79	4.79	1.30
180	8.26	21.78	3.29	6.29	4.79	1.62
180	8.90	20.22	4.79	4.79	4.79	1.73
180	8.26	21.78	6.29	3.29	4.79	1.62
180	6.34	28.37	7.79	1.79	4.79	1.30
180	2.69	66.86	9.29	0.29	4.79	0.52
Σ						8.612

Maximum Deflection under Live load	=	8.612 mm	SAFE